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DRYING SHRINKAGE CRACKING OF HIGH STRENGTH CONCRETE

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الخلاصة

هناك اهتمام كبير بالخرسانة عالية المقاومة من قبل تقنيات الأبنية الحديثة ، كرس مقدار كبير من الاهتمام لإمكانية أنتاج خرسانة عالية المقاومة من المواد المتوفرة وطرق الانجاز المتاحة . اهتمت هذه الدراسة بالانكماش لكونه المشكلة الرئيسية التي تواجه المهندسين في العراق والشرق الأوسط بسبب الظروف الجوية

انقسمت الدراسة في هذا البحث إلى قسمين , الأول إنتاج خرسانة عالية المقاومة تتراوح مقاومتها من (٥٠ - ٨٥) ميكا ودراسة خواصها الميكانيكية وتغيراتها الحجمية نتيجة لانكماش الجفاف ، أما الجزء الثاني فيهتم بدراسة تأثير إضافة رماد قشور الرز بنسبتين (٥- ٧,٥) % من وزن الأسمنت على الحالة الطرية , الخواص الميكانيكية والتغيرات الحجمية للخرسانة عالية المقاومة . الخواص الميكانيكية التي تضمنها البحث شملت ، مقاومة الانضغاط ، مقاومة الشد (الانشطار) ، معامل المرونة .

تراوحت القيم بالنسبة للخرسانة عالية المقاومة ، مقاومة الانضغاط (٣٤-١٠٢) ميكاباسكال ، مقاومة الشد (٢,٩ - ٥,١) ميكا باسكال ،ومعامل المرونة (٢٩,٦-٤٢) كيكا باسكال .

لوحظ أن إضافة رماد قشور الرز في كلا النسبتين (٥- ٧,٥) % يسبب نقصان في مقاومة الانضغاط حوالي (١٦- ٣٤)% ، مقاومة الشد(١١- ١٦)% و معامل المرونة (٤-١٠)%.

أن قيم الانكماش للخرسانة عالية المقاومة هي قيم متدنية وتراوحت من (٢٠- ٦٥) مايكروسترين في الأعمار المبكرة (٣ يوم) و (٣٧٥-٤٨٠) مايكروسترين في الأعمار المتأخرة(٧٠) يوم .

أن إضافة رماد قشور الرز في كلا النسبتين (٥- ٧,٥) % يسبب نقصان في انكماش الجفاف مقداره (٧ - ٤٣) %.

اعتمادا على نتائج البحث ، يمكن الاستنتاج بان انفعال انكماش الجفاف بالنسبة للخرسانة الاعتيادية في الأعمار المتأخرة (٧٠) يوم اعلى من انفعال انكماش الجفاف في الخرسانة عالية المقاومة بمقدار (٨-٢٨)% ، إضافة رماد قشور الرز سبب نقصان في انفعال انكماش الجفاف ، وزيادة في انفعال الشد الأقصى(١١-٤١)% والزحف للخرسانة (٢٠ -١٤٥) % (عند حدوث التشقق) .

بالإضافة إلى ذلك وجد أن إضافة رماد قشور الرز إلى الخرسانة يؤخر من حدوث تشققات الانكماش وان تطور عرض الشق يصبح أبطأ مع إضافة رماد قشور الرز ومع زيادة كمية الرماد المضافة صعودا إلى

٧,٥% .أن موقع حدوث الشق في كل الخلطات الخرسانية (عالية المقاومة ، الاعتيادية)حدث في منطقة التضييق وعلى مسافة تتراوح بين (٥- ٢١) ملم من منتصف القالب .

وأخيرا تم اقتراح وتطوير نموذجين رياضيين تعتمد على معادلات غير خطية متعددة المتغيرات للحصول على قيم تخمينية لانفعال الانكماش وعرض الشق وكانت قيمة معامل الارتباط بالنسبة لنموذج انفعال الانكماش (٠,٩٩٢) ،(٠,٨٧) لنموذج عرض الشق ، قيمة الخطأ القياسي كانت   (١٩,٦٣) لنموذج انكماش الجفاف ، (٠,٢٣٣) لنموذج عرض الشق .

ABSTRACT

Modern construction techniques have been the major interest in high strength concrete (HSC), a good attention has been devoted to achieve high strength using available materials and production methods. This study is concerned with the latter interest arises because the shrinkage is more important and major problems facing the engineers in Iraq and other countries in the Middle East due to weather condition.

This work divided into two parts, the first part was to design high strength concrete (HSC) of strength between (50 – 85) MPa , and to study its mechanical properties and volume changes, the second is to investigate the effect of addition Rice Husk Ash (RHA) in two percentages (5, 7.5) % by weight of cement on the fresh state of concrete, mechanical properties and volume changes. The properties investigated were, compressive strength, splitting tensile strength and static modulus of elasticity.

Compressive strength of the investigated concreted were ranged between (34 – 102) MPa, splitting tensile strength between (2.9 – 5.1) MPa, static modulus of elasticity (29.6 – 42) GPa for high strength concrete .

The addition of RHA to concrete causing a deficiency by about (16 - 34) % for compressive strength, (11 - 16) % for splitting tensile strength and (4 - 10) % for modulus of elasticity, this reduction was observed for two percentages of RHA (5, 7.5) %.

The shrinkage of high strength concrete had a few values ranged between (20 - 65) microstrain at early ages (3 day) and about (375 - 480) micro strain at later ages (70 days).

The addition of RHA to concrete for two percentages (5, 7.5) % led to reduction in drying shrinkage strain by about (7 - 43) %.

Based on the results of this work, it can be concluded that the drying shrinkage for normal concrete at later ages (70 days) is higher than that of high strength concrete by about (8 - 28) %, addition RHA to concrete causing a deficiency in drying shrinkage and an increasing in tensile strain capacity by about (11 - 41) % and (20 – 145) % for creep (at date of cracking).

In other hand, it was found that RHA addition retards the occurrence of shrinkage cracks. Furthermore, the crack width development become slower with addition RHA and with increases RHA content up to 7.5 %. But the crack width of concrete without RHA develops higher than concrete with RHA, development of crack width of normal concrete slower than these of high strength. For all mixes crack occurred within the narrower at the middle of the web at distance from the center ranged between (5 - 21) mm .

Finally mathematical models for the prediction of shrinkage strain and crack width were proposed and developed in this study. These models use multivariable non – linear regression equation to evaluate good correlation coefficient (R = 0.992) for drying shrinkage model, (R = 0.87) for crack width model. Standrder error (19.63) for shrinkage model and (0.233) for crack width model.

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NOTATIONS & SYMBOL

Most of common symbols are listed below; others are defined where they appear in the research.

Notations	*Description*
GGBFS	Ground granulated blast- furnace slag
HSC	High Strength Concrete
NSC	Normal Strength Concrete
OPC	Ordinary Portland cement
RH	Relative humidity of the ambient environment (%)
RHA	Rice Husk Ash
SF	Silica fume
SRPC	Sulphat resistance Portland cement
W/b	Water/ binder(cement +RHA) (by weight)
W/c	Water/ cement (by weight)

Symbol	***Description***
A_c	Cross- sectional area (mm^2)
Af	Fine aggregate content
Atotal	Total aggregate content
$\mathbf{C_a}$	Coarse aggregate content kg/m^3
$\mathbf{C_C}$	Cement content kg/m^3
$\boldsymbol{E_c}$	Static modulus of elasticity
f_{cm}	Mean compressive strength of concrete at the age of 28 days (cube).
$\boldsymbol{f_{sp}}$	Splitting tensile strength
S	Slump of fresh concrete
SP	Reference concrete mix with 1.25 % SP
SPH_1	Concrete mix with 1.25 % SPand 5% RHA
SPH_2	Concrete mix with 1.25% SP and 7.5 % RHA
t	Time (day)
u	Perimeter (mm)
v/s	Volume to surface ratio
$\varepsilon_c (t)$	Autogenous shrinkage of concrete
ε_{sh}	Free drying shrinkage
$(\varepsilon_{sh})_u$	Ultimate shrinkage strain

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CERTIFICATION

We certify that this thesis titled "***Drying Shrinkage Cracking of High Strength Concrete*** " was prepared by "***Nadia Moneem Abd – Alhussain Al - abdaly*** " under our supervision at University of Babylon in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering.

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We certify that we have read this thesis, titled "***Drying Shrinkage Cracking of High Strength Concrete*** ", and as an examining committee examined the student " ***Nadia Moneem Abd – Alhussain Al - abdaly*** " in its contents and in what is connected with it, and that in our opinion it meet the standard of thesis for the degree of Master of Science in Civil Engineering.

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CHAPTER ONE 1

INTRODUCTION

1.1 General :

In recent years, the construction industry has shown significant interest in the use of high strength concrete (HSC). This is due to the improvements in structural performance, such as high strength, that can provide comparison to traditional normal strength concrete (NSC). The use of high strength concrete has become common, because it offers a significant economic and architectural advantage over normal strength concrete in buildings.

Recently, the use of high strength concrete, which was perilously in application such as bridges, off - shore structures and infrastructure projects , has been extended to high rise buildings. One of the major uses of HSC in buildings is for columns. However, for a variety of reasons that include the need for a reduction in the cross – sectional areas of structural elements , the construction industry is moving towards higher grades of concrete ***(Kodur, 2003)***.

High strength concrete is a relatively recent development in concrete technology made possible by the introduction of efficient water – reducing admixtures and high strength cementitious materials***(John and Ban,2003)***.The results of many recent tests on HSC shown that there are significant differences between the performance of high strength concrete compared with normal strength concrete. These differences include, mechanical properties (compressive strength, splitting tensile strength and modulus of elasticity).

All cracks are a potential source of concrete deterioration, reduce its life cycle and steel corrosion by allowing ingress to deleterious substances. Even small cracks that pass through the full thickness of section will also allow water percolation resulting in damp patches and surface staining .

In present years , some attention was given to characteristics of such concretes, in particular with respect to their cracking sensitivity. It has been argued and demonstrated experimentally that such concrete may undergo internal shrinkage due to self – desiccation, and as a result, internal tensile stresses may develop, leading to micro cracking and macro cracking. Much of the study of this problem has been based on determination of free shrinkage strains, yet to assess the problem properly, stresses developed under restrained conditions should be evaluated ***(Bloom and Bentur,1995)***.

1-2 Research Significance

This work was aimed to produce High - Strength concrete from the available materials, with investigation of its mechanical properties, such as compressive strength, modulus of elasticity, splitting tensile strength and study the volume changes of concrete. Since production of this type of concrete is limited, and need to be investigated, particularly when it is used in special structures. Relatively little information is available on the drying shrinkage characteristics of HSC ***(ACI Committee363, 1997)***.

1-3 Objectives of this Work

This work planned to produce high strength concrete ranged between (50 – 85) MPa. The main aims of this work are as follows :

1- Studying the mechanical properties of high strength concrete produced and to be compared with normal concrete (eight concrete mixes were investigated, seven of these mixes are high strength, with a total of (90) concrete cubes of 100 mm, (42) concrete cylinders of (100 × 200) mm for splitting tensile strength, (42) concrete cylinders of (150 × 300) mm for modulus of elasticity
2- Studying the free and restrained drying shrinkage of high strength concrete (seven concrete mixes were investigated with a total of 21 concrete beams of size (3 × 0.125 × 0.12))m.
3- Studying the behavior of free and restrained drying shrinkage of normal concrete (3 concrete beams of size (3 × 0 .125 × 0.12) m) .
4- Studying the effect of addition Rice Husk Ash (RHA) in two percentage (5 , 7.5) % on free and restrained drying shrinkage .

1- 4 Research layout

This research consists of five chapters :

Chapter One present introduction about High Strength Concrete and research significance and layout .

Chapter Two presents a literature review , introduces a definition of the high strength concrete (HSC), HSC achievement, advance materials used in production of HSC, extensive studies performed on shrinkage, type of shrinkage and factors influencing it, creep, tensile strain capacity, the effect of supplementary materials such as (RHA) on drying shrinkage of high strength concrete, cracking time and other important properties of concrete were discussed in this chapter .

Chapter Three gives details of the studies which includes a description of the molds used to achieve shrinkage cracking is presented , mixing, casting, curing and testing the specimens for compressive strength drying shrinkage, tensile strength, and modulus of elasticity are explained .

Chapter four the analysis and discussion of the experimental work were presented. Properties of fresh HSC and mechanical properties (compressive strength, splitting tensile strength, modulus of elasticity), the effect of RHA on free shrinkage, elastic tensile strain capacity, tensile strain capacity, creep, cracking time, development of crack width of the effect of free shrinkage, crack width development with age, adopted and modified formula for the relationship between compressive strength and splitting tensile strength, another one between compressive strength and modulus of elasticity, developed model for prediction free drying shrinkage and adopted formula for the relation between maximum crack width were presented in this chapter .

Chapter Five is devoted to the main conclusions and recommendations for future work .

CHAPTER TWO 2

LITERATTURE REVIEW

2.1 INTRODUCTION

High strength concrete (HSC) has a simple definition would be " concrete with a compressive strength above the present existing limits in national code about 40 MPa and up to 130 MPa" ***(ACI Committee 363, 1997and CEB /FIP, 1990)***. In the UK this would include concrete with a characteristic compressive strength of 60 MPa or more. But in Norway the design code already includes concrete with characteristic cube strength up to 105 MPa ***(Helland , 1996)***. High strength concrete is often considered a relatively new material, its development has been gradual over many years. As the development has continued ,the definition of high - strength Concrete has changed. In 1950_sconcrete with compressive strength of (34 MPa) was considered high – strength. In 1960_s concretes with (42 and 52 MPa) compressive strength were used commercially. In 1970_s (62MPa) concrete was being produced more recently, compressive strength approaching (138 - 170 MPa). High strength concrete (HSC) which was widely used in applications such as bridges, offshore structures and infrastructure projects, has been extended to building columns, shear walls of high rise buildings, elevated structures, tunnels, mine shaftliners, precast, and prestressed products, multistory building ***(ACI Committee 363 , 1997)***. HSC is made possible by the development of high cementing materials but this material can be produced by judicious blending of Portland cement with silica fume, ground granulated blast furnace slag and superplasticizing admixtures. HSC can be used with most conventional construction techniques, but special attention must be given to avoiding delays during placing, finishing and curing ***(John and Ban, 2003).***

2.2 Materials Selection

High strength concrete is prepared through careful selection of each of ingredients and the mix proportioning are more critical processes than the design of normal strength concrete mixes. Each material namely (cement, sand, coarse aggregate, concrete admixtures, and pozzolans) must be evaluated as to type, strength characteristics, gradation, fineness and interaction in combination each other.

2.2.1 Cements

The selection of cement is an important parameter affecting the ultimate value of compressive strength for high – strength concrete. The range of cement content in HSC is (392 to 594) kg/m^3 ***(ACI Committee 363 , 1997)***. In other investigation the cement reach to (641.8 kg/m^3) ***(Nassif, 2003)*** and (740 kg/m^3) ***(Habeeb , 2000)***. HSC can be produced with most available cements. But those cements that are particularly coarsely ground are unsuitable ***(Mehta and Aitcin , 1990a)***. In Norway special cements have been developed for HSC which are more finely ground and with lower tricalcium aluminates ($C_3A \leq 4\%$) content ***(Helland , 1996)***.

2.2.2 Aggregates

2.2.2.1 Fine Aggregate

The particle shape , fineness modulus , grading of the fine aggregate are significant factors in the production of high – strength concrete. Fine aggregate with a rounded particle shape and smooth texture have been found to require less mixing water in concrete and for this reason are preferable in high – strength concrete, gradation of fine aggregate effect water requirement than on physical packing. The clay, silt, dust content should kept as low as possible ***(Johan and Ban, 2003)***. Sand with a fineness modulus (FM) below 2.5 gave the concrete a sticky consistency, making it difficult to compact, sand with a (FM) of about 3.0 gave the best workability and compressive strength ***(ACI Committee 363, 1997).***

2.2.2.2 Coarse Aggregate

The important parameters of coarse aggregate are its shape, texture and the maximum size . Since the aggregate is generally stronger than the paste, its strength is not a major factor for normal strength concrete, however, the aggregate strength becomes important in the case of high - strength concrete .Surface texture and mineralogy affect the bond between the aggregates and the paste as well as the stress level at which microcracking begins. The effect of different types of coarse aggregate on concrete strength has been reported by ***(Sarkar and Aitcin, 1990)*** which studied characterization of twelve different coarse aggregates that have performed with variable success in high strength concrete in Canada and the United States. They pointed out the intrinsic strength of coarse aggregate is not an important factor if water – cement or water – binder (W/C , W/b) ratio falls within the range of (0.5 - 0.7) , this due to the fact that the cement – aggregate bond or the hydrated cement paste fails long before aggregate do. It is not true for high strength concretes with very low (W/C or W/b ratio) of 0.2 to 0.3 .

(Aitcin and Mehta, 1990), discussed the importance of evaluating the characteristics of coarse aggregates to be used in HSC. They studied four different types of coarse aggregates available in California, with W/b ratio 0.275 to produce concrete strengths ranging from (85 to 105) MPa. The results showed that the compressive strength and elastic modulus were influenced by the mineralogical characteristics of the aggregates .

(Leming, 1990) compared mechanical properties of high strength concrete made with four different types of aggregates (crushed shell - limestone, crushed granite, partially crushed gravel, and diabase) available in North Carolina. The 28- days concrete strength ranged from (51 to 81) MPa with W/b ratio varying from (0.42 to 0.28). He observed that the mechanical properties varied significantly depending on the source and type of the coarse aggregates, he also found the W/b ratio alone is not an effective predictor of strength for HSC made with significantly different aggregates and paste composition .

A similar study ***(Giaccio et- al. 1992)*** with basalt , granite, and limestone aggregates and a constant W/b ratio of 0.3 plus 2.5 % naphthalene – based superplasticizer showed that the three different aggregates produced a compressive strength of 92 MPa, 80 MPa and 60 MPa respectively, the specimen used in that test was (100 × 200) mm cylinder .

Maximums aggregate size of (10 – 14) mm is usually selected in HSC, although aggregates up to 20 mm may be used if they are strong and free of internal flaws or fractures ***(Mehta and Aitcin, 1990a).*** The use of larger maximum size of aggregate affects the strength in several ways. First, since large aggregates have less specific surface area and the aggregate – paste bond strength is less, the compressive strength of concrete is reduced. Secondly, for a given volume of concrete, using large aggregate results in a smaller volume of paste required, thereby providing more restraint to volume changes of the paste.

In a similar study by ***(Larrard and Belloc, 1992)*** using crushed limestone aggregates, Portland cement, silica fume, and superplasticizer for eight different concrete mixtures, it was shown that better performances and economy could be achieved with (20 to 25) mm, maximum size aggregates even though previous researchers had suggested that (10 - 12) mm is the maximum size of aggregates preferable for making high – strength concrete. The effect of coarse aggregate size on concrete strength was investigated by ***(Cook, 1989)*** who used two different sizes of limestone (10 and 25) mm, superplasticizer was used in all mixes .

In general, for a given W/b ratio, the smallest size of the coarse aggregate produced the highest strength; however, it was feasible to produce compressive strengths in excess of 69 MPa using a 25 mm maximum size aggregate when the mixture was properly proportioned with a high – range water reducing admixture.

2.2.3 Pozzolan

Pozzolan by definition is a siliceous or siliceous and aluminous materials, which in the presence of moisture chemically reacts with $Ca(OH)_2$ to form strong cementing material (CSH – gel) hence pozzolans reduce the weak bonds in concrete, making the concrete stronger, decreasing permeability, and increasing its durability against chemical attack. These materials include Fly Ash, Silica Fume, Granulated blast - Furnace Slag and Rice Hush Ash ***(Nassif, 2003)***.

2.2.3.1 Fly ash

Fly ash is the finely divided residue resulting from the combustion of ground or powdered coal and is transported by flue gases. This waste by product is used extensively in the production of high - strength concrete . Due to its outstanding pozzolanic and cementitious properties, fly ash is used to improve durability and enhance strength gain. As concrete containing fly ash is cured, the products of the pozzolanic reaction fill in the spaces around cement particles. This results in a paste of lower permeability and greater resistance to chemical attack. Because the pozzolanic reaction is slower than the hydration of Portland cement, fly ash is often used to control the amount of early heat generation and the detrimental effects of early temperature rise commonly experienced in massive concrete structures ***(Adams , 1988)*** . Fly ash has fine particles with sizes ranging from (1 to 100) μm .

Fly ash could be divided into two categories. The main difference between them is the calcium contents or, more specifically, the lime (CaO) contents. The first category is referred to as high - calcium fly ash, (15 to 35) percent of CaO. This ash is generally obtained from the combustion of lignite and bituminous coals. The ***(ASTM C 618 , 1996)*** also refers to it as Class C fly ash. The second category is low - calcium fly ash, which usually contains less than 5 percent of CaO. Low - calcium fly ash is a product of the combustion of anthracite and bituminous coals ***(Roy , 1989)*** .

2.2.3.2 Ground granulated blast-furnace slag

This is a glassy material that is formed when blast-furnace slag is rapidly cooled, such as by immersion in water. It is composed essentially of silicates and aluminosilicates of calcium. When it is ground to cement fineness , it is referred to as ground granulated blast - furnace slag (GGBFS), and it is commonly used in HSC mixtures. The use of GGBFS reduces the permeability of the mature concrete. It is believed that this improvement is a result of the reaction of the GGBFS with the calcium hydroxide released during hydration of the Portland cement. The reaction products fill pore spaces in the paste and result in a denser microstructure. Like fly ash, GGBFS is also used to reduce temperature rise in mass concrete. Ground granulated blast-furnace slag also improves the workability of fresh concrete ***(ACI Committee 226, 1987)***. The effect of blast furnace slag on the workability are much less than those of fly ash due to lesser specific surface of 3250 ± 200 cm^2/gm ***(Roy , 1989).***

2.2.3.3 Silica Fume

Silica fume is a highly reactive pozzolanic material primarily composed of silicon dioxide (SiO_2) in no crystalline form. It is a light to dark gray or bluish green - gray powder produced by an electric arc furnace during the manufacture of silicon or ferrosilicon alloy. It has a spherical like fly ash, but it is 100 times smaller, with an average diameter size of 0.1 μm. Its specific surface area is very large compared to Portland cement ((20,000 m^2/kg (by nitrogen absorption method) versus 300 m^2/kg, respectively). Silica fume is a highly active pozzolan, having a large specific surface area. Like other pozzolans. Because of the small size and the large surface area of silica fume, it absorbs a lot of water, making the water - demand for silica fume concrete very high and the setting time of the concrete very fast. In general, concrete containing silica fume has either a superplasticizer or retarder added to it. Silica fume is known for its ability to increase the strength of concrete because of its chemistry and hydration. Silica fume, when added to Portland cement, reacts rapidly to form C-S-H ***(Adams, 1988 and Roy, 1989).*** The effect of silica fume on the strength of concrete has been discussed by ***(Collepardi et- al., 1990)*** who studied the effect of combined of silica fume and superplasticizer on concrete compressive strength by taking into account some parameters as :

1) type and dosage rate of superplasticizer ,
2) type and content of Portland cement , and
3) way of silica fume utilization (as addition or as cement replacement). They concluded that in the presence of silica fume, for both type I and type III Portland cement, the melamine sulphonated polymer superplasticizer performs better than the naphthalene sulphonated polymer, particularly when a high dosage such as 4% is used, change from 2 to 4% by weight of cement, superplaticizer dosage rate does not modify or reduce compressive strength in the absence of silica fume, whereas significantly increases compressive strength in the present of silica fume. Concrete with characteristic strengths up to 150 MPa can be produced with normal aggregates, chemical admixtures, and microsilica additions of 10 - 15 percent.

2.2.3.4 Rice Husk Ash (RHA)

Rice husks which are as agricultural waste, constitute about one – fifth of the 500 million tones of rice produced annually world wide***(Mahmud et - al, 2004)***. The Primary work on RHA was started at the Asian Institute of Technology by ***(Columana, 1974)***, Who found out that ' village burnt husks were converted to ash at temperatures less than 300°C. Rice husk ash prepared in this way is expected to have a considerable amount of carbon which has an adverse effect upon its pozzolanic action. ***(Mehta, 1975)*** found that field burning of rice husks produced crystalline silica ash. However, burning the husks in a controlled temperature furnace, the residual silica ash is in a highly reactive form, and when mixed with lime produced rich black cement . ***(Cook, et- al , 1976)***, used the RHA as a pozzolanic material. The material was prepared by burning the husks at 450º C for 4 hours and grinding the husks for 1.5 hours. According to ASTM C618 – 94 a, RHA can be classified as an artificial pozzolan, of siliceous material. ***(Zhang and Malhotra, 1996)*** compared between effect of two pozzolanic materials (silica fume, rice husk ash) on compressive strength of concrete mix with 10 % of (RHA, SF) as cement replacement by weight and (W/b = 0.4) ratio, the compressive strength was measured at (1, 7, 28) days after initial curing 1day in the mold and 6 days in lime saturated water, the results showed the compressive strength of SF concrete higher than compressive strength of RHA concrete and the later higher than control concrete. ***(Habeeb, 2000)*** studied the effect of addition RHA on high strength concrete, in his investigation twelve different concrete mixes were investigated, the compressive strength has three series (40, 60, 80) MPa, the RHA was used in two percents of RHA (7.5, 15) % by weight of cement, 14 mm nominal of coarse aggregate, 3% melamine L10 as superplasticizer and ordinary Portland cement was used (580 - 740) kg/m^3, he also studied the effect of addition of RHA on drying shrinkage, prisms (100 × 100 × 400) mm were cast and cured for 28days in water and after that exposed to drying laboratory conditions. The result showed that the 3% superpalsticizer led to a reduction of 23 % in water content and increased compressive strength (13 - 22) %, the values of drying shrinkage of high strength concrete were between (400 – 537) microstrains at 56 days. The use of superplasticizer led to increase the drying shrinkage values by about (10 - 20) % , while RHA led to reduce it by about 10 % .

(Mahmud et- al, 2004) studied the use of RHA to produce high strength concrete and they studied the effect of addition or replacement of RHA compared with silica fume on fresh and hardened mechanical properties. In this investigation OPC, RHA, SF and asulphonated naphthalene formaldehyde based superplasticizer was used as chemical admixture with varying amounts to maintain workability. The results showed that the addition of RHA up to 15% leads to increase the compressive strength, increasing the amount above these values decreases the strength of the concrete. The optimum addition level of RHA to produce HSC approximately 10% and the optimum cement replacement with RHA is about 5% .

2.2.4 Admixtures

(ACI 116R) defines the term admixture as " a material other than water, aggregates, hydraulic cement, and fiber reinforcement, used as an ingredient of a cementitious mixture to modify its freshly mixed, setting, or hardened properties and that is added to the batch before or during its mixing". The use of superplasticizer in concrete is an important in the advancement of concrete technology. Since their introduction in the early 1960_s in Japan in the early 1970_s in Germany, it is widely used all over the world ***(Shetty, 2004)*** .

The purpose of water-reducing admixtures is stated by ***(ACI Committee 212.2 , 1997)*** as: “ Water-reducing admixtures are used to reduce the water content by 5 to 12%, depending on the admixture, dosage, and other materials and proportions. The dosage rate of Type A water - reducers range from 130 to 390 ml /100 kg of cementitious materials . The primary difference between these admixtures and conventional water-reducing admixtures is that high - range water-reducing (HRWR) admixtures, often referred to as superplasticizers, may reduce the water requirement by more than 30%, without the side effect of excessive retardation. HRWR admixtures should meet the requirements of ***(ASTM C494, 1982)*** for classification as Type F, High - Range Water - Reducing, or Type G, High - Range water reducing. HRWR admixtures are organic products that typically fall into three families based on ingredients:

1. Sulfonated melamine - formaldehyde condensate;
2. Sulfonated naphthalene - formaldehyde condensate; and
3. Polyether - polycarboxylates.

HRWR admixtures act in a manner similar to conventional water-reducing admixtures, except that they are more efficient at dispersing fine-grained materials such as cement, fly ash, ground granulated blast-furnace slag, and silica fume. HRWR admixtures based on polyether - polycarboxylate technology are different chemically and more effective than those based on sulfonated melamine-formaldehyde and sulfonated naphthalene -formaldehyde condensates and, as a result, are typically added at the batch plant. Polyether-polycarboxylate HRWR also retard less and develop strength faster compared to the other HRWR formulations. The strength of hardened concrete containing HRWR admixtures is normally higher than that predicted by the lower W/b alone. As with conventional admixtures, shrinkage and permeability may also be reduced and the overall durability of the concrete may be increased ***(Suchorski and Frang , 2004)*** .

2.3 Materials Technology of HSC

In order to achieve high strength compressive strength, it is important to understand the factors that govern the strength of concrete ***(John and Ban, 2003).***

1- The properties of the cement paste.

2- The properties of the transition zone between the paste and the aggregate.

3- The properties of the aggregate.

4- The relative proportion of the constituent materials.

2.3.1 Paste Properties

It is generally accepted that the most important parameter affecting concrete strength is the W/C or W/b (binder) ratio. Even though the strength of concrete is dependent largely on the capillary porosity or gel / space ratio, these are not easy quantities to measure or predict. The capillary porosity of a properly compacted concrete is determined by the W/b ratio and degree of hydration. Most high strength concretes are produced with a W/b ratio of 0.40 or less. The practical use of very low W/b ratio concretes has been made possible by use of both conventional and high - range water reducers, which permit production of workable concrete with very low water contents ***(Burg and Ost. , 1992).***

2.3.2 Transition Zone Properties

Transition zone is the interface between the paste and the coarse aggregate particles. If the transition zone to the aggregate is weak, the strength of the concrete will not increase. In conventional concretes, this transition zone is quite large and is characterized by a high porosity and large crystalline hydration products (such as $Ca(OH)_2$).

Reducing the water / paste ratio and the incorporation of silica fume into the concrete both contribute to reduce the width and improve the strength of the transition zone ***(Mindess, 1994).***

2.3.3 Aggregate Properties

Since coarse aggregate in concrete occupy large percent of the volume of high - strength concrete their characteristics play a major role in determining the properties of hardened concrete. It may be appropriate to roughly divide the characteristics into two groups; exterior features (maximum size, particle cleanliness and texture) and interior quality (strength, density, porosity, hardness, elastic modulus, chemical mineral composition.. etc) ***(Chang and Koonsu, 1996)***.

When the transition zone between the paste and the aggregate is improved the transfer of stresses from the paste to the aggregate particles becomes more effective. Consequently the mechanical properties of the aggregate particles themselves may be the " weakest link " leading to limitation of achievable concrete strength. Fracture surfaces in high strength concrete often pass through aggregate particles rather than around them ***(John and Ban, 2003)***. Crushed rock aggregates are generally preferred to smooth gravels as there is some evidence that the strength of the transition zone is weakened by smooth aggregate ***(Mehta and Aitcin, 1990)***.

During the crushing process, aggregate particles may severely microcracked The number of micro cracks will be greater in large particle, consequently it is common practice to use smaller particles (10 - 14 mm nominal size) for high strength concrete ***(Mehta and Aitcin, 1990a).*** Smaller sizes of aggregate produce higher concrete strengths and particle shape and texture affect both workability of fresh concrete and the strength of hardened concrete ***(Chang and Koonsu, 1996).*** A source of aggregate is much more critical for high strength concrete than for conventional concretes ***(Johan and Bang, 2003).***

The ideal aggregate should be clean, cubical, angular, 100 percent crushed aggregate with a minimum of flat and elongated particles ***(ACI Committee 363, 1997)*** .

The elastic modulus of HSC is strongly influenced by the elastic properties of coarse aggregates, the good bond at the interface of the coarse aggregate and mortar results in composite material whose components are cement paste and coarse ***(Baaibaki et- al., 1991)***.

2.4 Shrinkage of Concrete

Shrinkage is the decrease of concrete volume with time ***(ACI Committee 224, 1972).*** This decrease is due to changes in the moisture content of the concrete and physicochemical changes, which occur without stress attributable to actions external to the concrete. Shrinkage of high strength concrete may be expected to differ from conventional concrete in three broad areas: plastic shrinkage, drying shrinkage, and autogenous shrinkage.

Plastic shrinkage occurs during the first few hours after fresh concrete is placed. During this period, moisture may evaporate faster from the concrete surface than it is replaced by bleed water from lower layers of the concrete mass. Paste - rich mixes, such as high strength concretes, will be more susceptible to plastic shrinkage than conventional concretes. Drying shrinkage occurs after the concrete has already attained its final set and a good portion of the chemical hydration process in the cement gel has been accomplished.

The drying shrinkage of high strength concretes, potentially larger due to higher paste volumes, do not, in fact, appear to be appreciably larger than conventional concretes. This is probably due to the increase in stiffness of the stronger mixes. Autogenous shrinkage due to self - desiccation is more likely in concretes with very low W/b ratio , self – desiccation may be especially harmful to the durability properties of high strength concrete since the microstructure of the paste is adversely affected . With inadequate hydration, the near – surface regions become more susceptible to the penetration of deleterious materials from the surrounding environment. ***(Alfes , 1992).***

Carbonation shrinkage is caused by the reaction between carbon dioxide (CO_2) present in the atmosphere and calcium hydroxide ($CaOH_2$) present in the cement paste. The amount of combined shrinkage varies according to the sequence of occurrence of carbonation and drying process if both phenomena take place simultaneousl, less shrinkage develop. The process of carbonation, however, is dramatically reduced at relative humilities below 50 percent ***(Zia et al. , 1993 b) .***

2.4.1 Drying Shrinkage Mechanisms

There are three main mechanisms of drying shrinkage : capillary stress, disjoining pressure, and surface tension. Each of these mechanisms is dominant in a different range of relative humidity. The most relevant range of relative humidity for field conditions is 45% to 90%. In this range the capillary stress mechanism is the most dominant. As water evaporates, the tensile stresses that were confined to the surface tension of the water are transferred to the walls of the capillary pores (< 50 µm). The tension in the capillary walls results in shrinkage of the concrete ***. (Neville and Brooks , 1987)*** Shrinkage is a function of the paste, but is significantly influenced by the stiffness of the coarse aggregate . The key factors affecting the magnitude of shrinkage ***(Zia et- al, 1993b).***

2.4.1.1 Aggregate

The aggregate acts to restrain the shrinkage of cement paste; hence concrete with higher aggregate content exhibits smaller shrinkage. In addition, concrete with aggregates of higher modulus of elasticity or of rougher surfaces is more resistant to the shrinkage process ***(Zia et- al, 1993b).*** Shrinkage reduces by about 20% when increasing aggregate content from 71% to 74 % ***(Neville and Brooks, 1987).*** The possible reason to explain the effects of coarse aggregate content on shrinkage strain of concrete is shown in Figure (2 - 1), for the lean concrete mixture with a high coarse aggregate content , the coarse aggregate particles will have point - to point contacts or even face - to face contacts with each other . Concrete with such an aggregate structure will be very effective in resisting stresses caused by cement paste shrinkage, the coarse aggregate particles be pushed more closely under the action of interior stress caused by shrinkage, for the rich concrete mixture, there are greater distances between the coarse aggregate particles. This state gives the concrete less resistance to movement caused by shrinkage ***(Mang et- al , 2005) .***

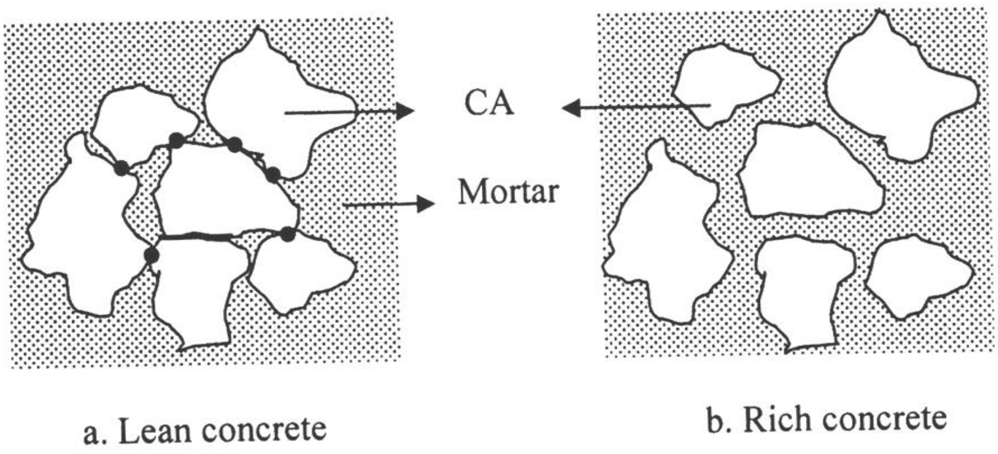


Figure (2 - 1): Effect of coarse aggregate content on the shrinkage of concrete ***(Mang et- al, 2005).***

(Alfes, 1992) examined how shrinkage was affected by the aggregate content, the aggregate modulus of elasticity, and the silica fume content. Using W/C ratio in the range of 0.25 to 0.3 with 20% silica fume by weight of cement and varying amount and type of aggregates (basalt, low density - slag, and iron granulate), he produced concretes with 28- day strength in the range of (102 to 182) MPa, the test results showed that there is a direct and linear relationship between the shrinkage value and the modulus of elasticity of the concrete .

The skeleton of coarse aggregate in a concrete can restrain the shrinkage of the cement matrix. If the skeleton of coarse aggregate in the concrete is stiffer, the shrinkage strain of concrete will be less; the shrinkage of concrete made with a steel aggregate will be lower than the one made with normal aggregate. Similarly, the shrinkage of a concrete made with expanded shale aggregate will be higher than the one made with a normal aggregate ***(Zhou and Barr, 2000) .***

(Han and Walarven, 1996) found the rate of shrinkage of the limestone mixture was less than that of the mixture containing the other aggregate types like (crushed gravel, granite) it was determined that the limestone mixture had the highest early elastic modulus which explains the reduced shrinkage behavior as shown in Figure (2 - 2).

Another types of aggregate such as quartz, feldspar and dolomite with a high modulus of elasticity produce a concrete with less shrinkage compared with other type of aggregate such as expanded shale aggregate ***(Georage, 1995and Gambhir, 1990)***.

The size and shape of aggregate also affect the shrinkage of hardened concrete. ***(Bisschop and Van, 2000)*** indicated that the total length and the depth of micro cracking caused by shrinkage of concrete will increase with large aggregate size. ***(Carlson et -al., 1979)*** reported that there is a consistent decrease in tensile strain capacity with increasing size and amount of aggregate; they also found the tensile strain capacity increases with decrease in modulus of elasticity of aggregate. ***(Troxel et- al, 1968)*** explains that smaller size aggregate experience more uniforms shrinkage .

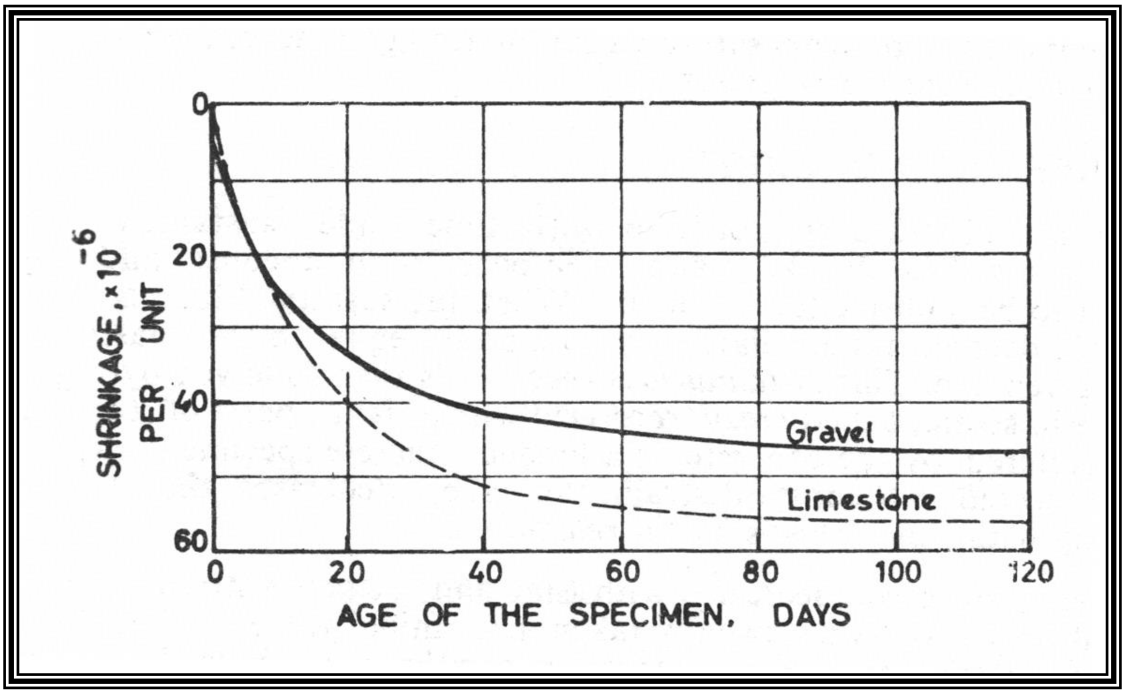


Figure (2 - 2) : Effect of the type of aggregate on the drying shrinkage of concrete ***(Gambhir , 1990).***

2.4.1.2 <u>Water-cementitious material ratio</u>

The higher the W/C ratio is, the higher the shrinkage. This occurs due to two interrelated effects. As W/C increases, paste strength and stiffness decrease, and as water content increases, shrinkage potential increases as shown in Figure (2 - 3) ***(Zia et- al, 1993b) .***

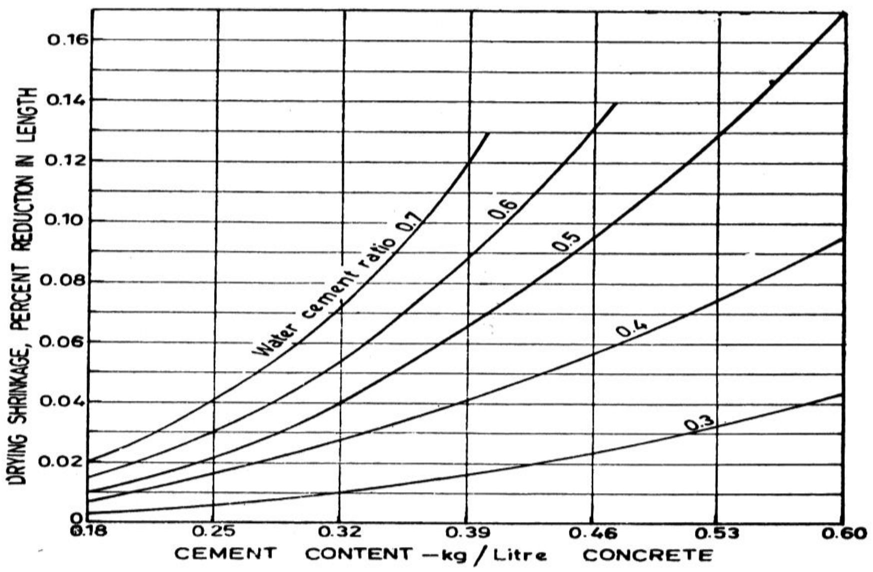


Figure (2- 3): Effect of W/c ratio on drying shrinkage of concrete***(Gambhir , 1990).***

2.4.1.3 Member Size

Both the rate and the total magnitude of shrinkage decrease with an increase in the volume of the concrete member. However, the duration of shrinkage is longer for larger members since more time is needed for shrinkage effects to reach the interior regions ***(Bazant and Wittman, 1982). (Hansen and Mattock, 1966)***, concluded that the changes in size and shape of the member affect, at all ages, both the rate and the final values of shrinkage, these values decreased as the member become large .

2.4.1.4 Medium Ambient Conditions

The relative humidity greatly affects the magnitude of shrinkage; the rate of shrinkage is lower at higher values of relative humidity. Shrinkage becomes stabilized at low temperatures ***(Zia et- al, 1993b).*** The drying shrinkage of concrete in an atmosphere of 70 percent relative humidity is about one third lower than in 50 percent relative humidity as shown in Figure (2 - 4) ***(Ghambhi , 1990 and Troxl, 1968).***

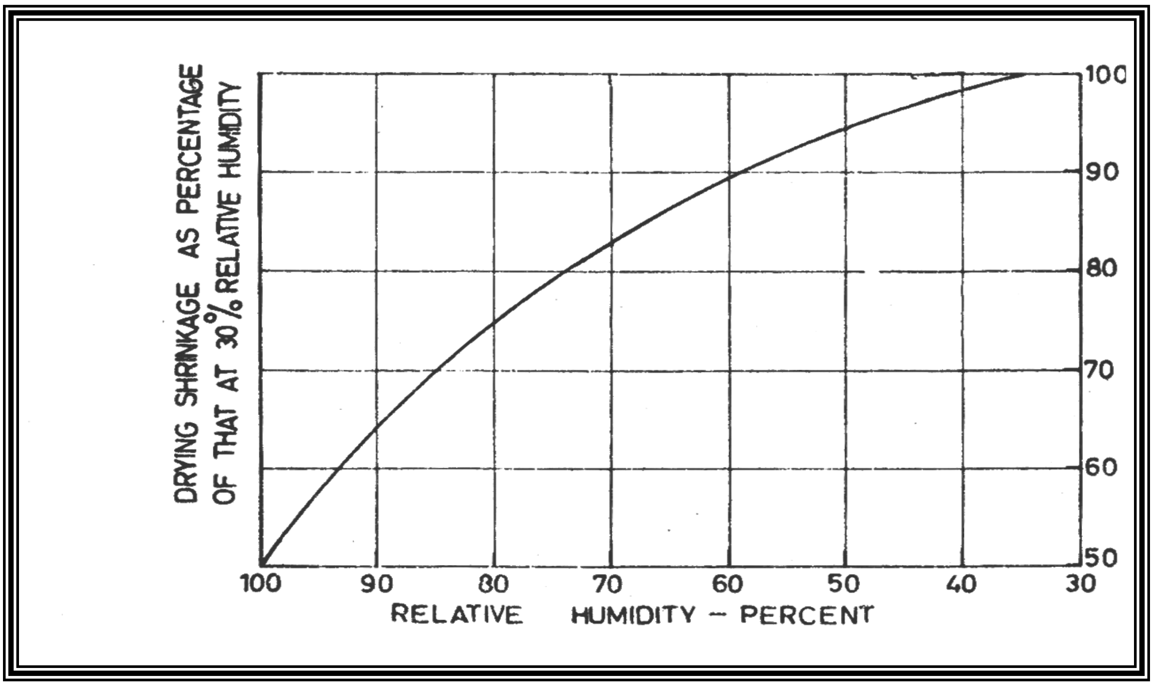


Figure (2- 4): Effect of relative humidity on drying shrinkage ***(Chambhir, 1990).***

2.4.1.5 Curing

Curing can also cause shrinkage cracking if proper procedures are not followed**. *(Nassif, 2003)*** studied the effect of using three different curing methods on the early age performance of high strength concrete. The curing condition consisted of air – dry curing , burlap or moist curing, and curing compound, he found the moist burlap should be applied within one hour, despite the higher drying shrinkage, increasing the curing time could reduce drying shrinkage . The effects of self – desiccation of low water – cement ratio can be largely controlled by careful attention to curing, especially during the initial 7 days after placement.

2.4.1.6 Admixtures

Admixture effect varies from admixture to admixture. Any material which substantially changes the pore structure of the paste will affect the shrinkage characteristics of the concrete. In general, as pore refinement is enhanced, shrinkage is increased. Pozzolans typically increase the drying shrinkage, due to several factors. With adequate curing, pozzolans generally increase pore refinement. Use of a pozzolan results in an increase in the relative paste volume due to two mechanisms; pozzolans have a lower specific gravity than Portland cement and, in practice, more slowly reacting pozzolans (such as Class F fly ash). Chemical admixtures will tend to increase shrinkage unless they are used in such a way as to reduce the evaporable water content of the mix, in which case the shrinkage will be reduced. Calcium chloride, used to accelerate the hardening and setting of concrete, increases the shrinkage. Air - entraining agents, however, seem to have little effect ***(Zia et- al, 1991) .***

(Zhang and Malhotra, 1996) compared between effect of two pozzolanic materials (silica fume, rice husk ash) on drying shrinkage, two prisms (102 × 76 × 390) mm were caste, a concrete mix with 10 % of (RHA, SF) as cement replacement by weight and (W/b = 0.4) ratio, the drying shrinkage was measured at (7, 14, 28, 56, 112, 224, 448) days after initial curing 1day in the mold and 6 days in lime saturated water, the results showed the RHA had drying shrinkage of 638×10^{-6} after 448 days, which was similar to the strains for the silica fume concrete.

(Mahmud et- al, 2004) showed that inclusion of pozzolainc admixtures (either RHA or SF) in concrete mix appear to increase the drying shrinkage compared to control mix, RHA addition displayed marginally lower shrinkage than its replacement.

2.4.1.7 Cement Type

The effects of cement type are generally negligible except as rate – of - strength - gain changes. Even here the interdependence of several factors makes it difficult to isolate causes. Rapid hardening cement gains strength more rapidly than ordinary cement but shrinks somewhat more than other types, primarily due to an increase in the water demand with increasing fineness ***(Krauss and Rogalla, 1996).*** Similar trend was given by ***(Troxell, 1968)*** found that the shrinkage of low heat cement (type IV) is greater than others like type (I). This is believed to be due to high (C_2S) content in (type IV) cement, which exhibits high shrinkage than (C_3S) because the (C_2S) has low percentage of $Ca(OH)_2$.

$$2C_2S + 4H \rightarrow C_3S_2H_3 + Ca(OH)_2$$

$$2C_3S + 6H \rightarrow C_3S_2H3 + 3Ca(OH)_2$$

The fineness of cement has probably two opposing influences on concrete shrinkage. Finer cement hydrates faster and thus produces a denser gel, which has a lower shrinkage. On other hand, the gel of the finer cement is stronger and therefore it has a higher effect against restraint of aggregate and this increases the shrinkage ***(Carlson et -al , 1979).***

C_3A producing of a family of non - stable compounds and with time these compounds loose water and this leads to high shrinkage ***(Troxel et- al, 1968).*** ***(ACI committee 224, 1972)*** , suggested that lower shrinkage with lower (C_3A/SO_3) ratio, lower Alkali content (Na_2O and K_2O, and higher (C_4AF) content .

2.4.2 Plastic Shrinkage Cracking

Plastic shrinkage cracks develop when the rate of evaporation exceeds the rate of bleed water migration to the surface. During placement, concrete is unable to withstand the tensile force resulting from the rapid evaporation of surface moisture eventually leading to plastic shrinkage crack formation. Some high strength concretes are prone to plastic shrinkage, which occurs in the wet concrete, and may result in significant cracking during the setting process. This cracking occurs due to capillary tension in the pore water ***(Kevin et- al , 2004)***

2.4.3 Shrinkage Induce Cracking

In structure all concrete members are always subjected to some kind of restraint which either at the boundaries or within the concrete element itself, the degree of restraint depends greatly on the stiffness ratio and relative dimensions between the concrete member and that of the restraining object, this restrained in concrete induce tensile stress and strain, especially when the concrete surface is allowed to dry condition rapidly (as can happen in summer) ***(Neville and Brooks, 1987)*** . When the tensile stress exceed its tensile strength the concrete will crack, as shown in Figure (2 – 5).

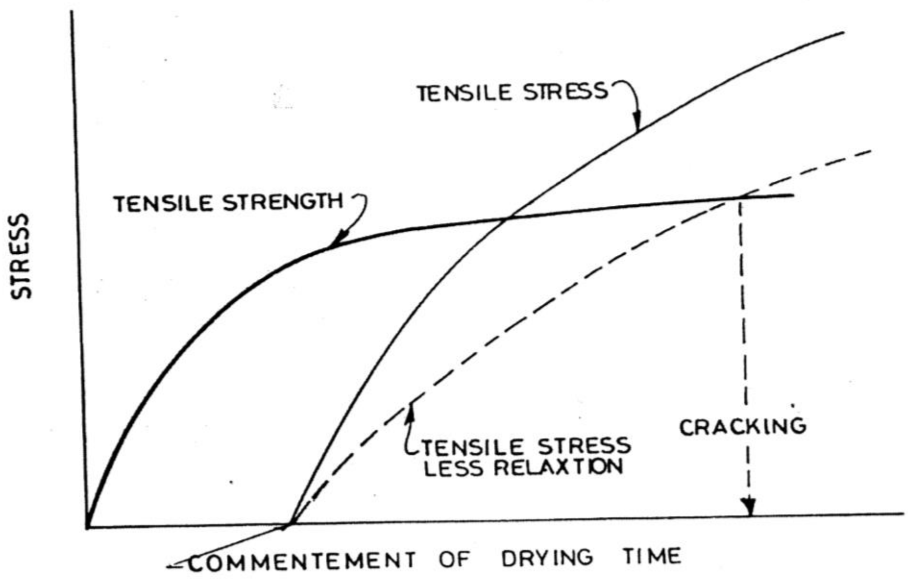


Figure (2 - 5) : Sketch of crack development (stress wise)***(AL-Rawi, 1985)***

These stresses increase with time, but the rate of this increase will decrease due to the decreased shrinkage rate and due to the relaxation of stresses in concrete. Relaxation takes place and may prevent the development of cracking if the shrinkage develops slowly.

The magnitude of tensile stress developed during drying of the concrete is influenced by combination of factors such as:

1- The amount of shrinkage.
2- The degree of restraint.
3- The modulus of elasticity of the concrete.
4- The creep relaxation of the concrete.

There are different types of restraint in a concrete member:

1- Internal restraint.

2- External base and end restraint.

The internal restraint takes place due to the non - uniform volume change through a member cross section. All the member at the first stage of drying would be under non - uniform moisture loss due to the faster rate of drying at the surface than the internal elements of the member ***(Neville, 1987and Gilbert, 2001)*** **.**

External base restraint acts along a contact surface between a member and its surrounding, in the case of wall restrained by the ground along its base. End restraint is typical for concrete whose restraint to volume change is concentrated at its ends, with no or without minimum base restraint .

2.4.4 Shrinkage Models

There are many models for prediction shrinkage in concrete, however, there are two base models :

1) ACI Committee 209 and

2) CEB - FIP Model Code 1990 . 3) others Models .

Numerous additions or modifications have been suggested according to the change in mix constituents and ambient conditions. More recently, an attempt has been made by ***(Miyazawa and Tazawa, 1999)*** to propose a model that can predict autogenous shrinkage.

2.4.4.1 ACI Committee 209 Model for Shrinkage

(ACI 209, 1993) model is based on amodel proposed by ***(Branson and Christianson, 2003)***. The model is a general - purpose model and did not set any limitation on the strength of concrete being predicted. However, one of the criteria is that the concrete must be moist cured for a minimum of 7 – days. Furthermore, the model is only applicable to type I and III Portland cement. The model takes into account relative humidity, specimen size, curing type and age at the end of curing. The concrete shrinkage prediction model recommended by ACI - 209 is shown by equation [2 - 1] :

$$(\varepsilon_{sh})_t = \frac{t}{35+t}(\varepsilon_{sh})_u \qquad \ldots\ldots\ldots\ldots[2 - 1]$$

Where:

$(\varepsilon_{sh})_t$ - time dependent shrinkage strain

$(\varepsilon_{sh})_u$ - ultimate shrinkage strain at RH = 40 % .

t - Time in days.

if there is no available shrinkage data from the concrete to be evaluated , the ultimate shrinkage strain, $(\varepsilon_{sh})_u$, can be assumed to be the following :

$$(\varepsilon_{sh})_u = 780 \times \gamma_{sh} \times 10^{-6} \qquad \ldots\ldots\ldots\ldots [2 - 2]$$

γ_{sh} - a product of all the applicable correction factors for the testing conditions other than the standard condition; γ_{sh} = 1 under standard testing condition.

γ_{sh} - is obtained by multiplying the ultimate shrinkage strain under the standard condition by the appropriate correction factors as described :

1- Correction factors for the effect of initial moist curing

1.0 For moist cured for 7 days, and

0.93 For that moist cured for 14 days .

0.86 For that moist cured for 28 days.

0.75 For moist cured for 90 days .

2- Correction factor for the effect of ambient relative humidity

$\gamma_\lambda = 1.4 - 0.0102\,\lambda$, **for** $40 \le \lambda \le 80$ …………. *[2 - 3]*

$\gamma_\lambda = 3.00 - 0.030\,\lambda$, **for** $80 \le \lambda \le 100$ …………. *[2 - 4]*

Where

γ_λ – Correction factor for the effect of relative humidity

λ – Relative humidity

3- Correction factor for the effects of specimen size effect (γ_{vs}) is given by equation (2-5)

$$\gamma_{VS} = 1.2\exp(-0.12.\frac{v}{s}) \qquad \text{…………….}[\,2 - 5\,]$$

Where: γ_{vs} – Correction factor for the effects of specimen size

$\frac{v}{s}$ – Volume – surface area ratio of the specimen in inches.

4- Correction factor for the slump effect (γ_s) is given by equation (2- 6)

$$\gamma_s = 0.89 + 0.0264\,S \quad \ldots\ldots\ldots\ldots\ldots\ldots [\,2-6\,]$$

5- Correction factor for the fine aggregate effect (γ_{Af}) is given by equation (2 – 7) , (2 – 8)

$$\gamma_{Af} = 0.3 + 0.014(\,Af\,/\,Atotal\,)\,100 \text{ if } (\,Af\,/\,Atotal\,) \le 50\,\% \quad \ldots\ldots [\,2\text{ -}7\,]$$

$$= 0.9 + 0.002(\,Af\,/Atotal\,)\,100 \text{ if } (Af\,/Atotal\,) \ge 50\% \quad \ldots\ldots\ldots\ldots [\,2\text{- }8\,]$$

6- Correction factor for the cement content effect (γ_{Cc}) is given by equation (2 – 9).

$$\gamma_{Cc} = 0.75 + 0.00061\,Cc \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots [\,2-9\,]$$

7- Correction factor for the air content % effect (γ_{Ac}) is given by equation (2- 10) .

$$\gamma_{Ac} = 0.95 + 0.008\,Ac \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots [\,2\text{ - }10\,]$$

2.4.4.2 CEB – FIP Model code 1990 for shrinkage :

The CEB - FIP model is proposed by ***(CEB - FIP, 1990)*** based on the work by ***(Muller and Hillsdorf, 1990)***. The model is only applicable for concrete with 28 days compressive strength between (20 - 90 MPa). The input parameters of this model differ from the ACI model in terms of compressive strength and type of curing method where the later dose not consider 28 day compressive strength while CEB 1990 model considers only curing. The strain due to shrinkage at normal temperature may be calculated as :

$$\varepsilon_{cs}(t, t_s) = \varepsilon_{cso} \cdot \beta_s(t - t_s) \qquad \text{.................. [2- 11]}$$

Where:

$\varepsilon_{cs}(t, t_s)$ - Time dependent total shrinkage strain

ε_{cso} - Notional shrinkage coefficient ;

$\beta_s(t - t_s)$ - Coefficient to describe development of shrinkage with time

ε_{cso} can be estimated by

$$\varepsilon_{cs0} = \left(160 + 10\beta_{sc}\left(9 - \left(\frac{f_{cm}}{f_{cmo}}\right)\right)\right) \times 10^{-6}\beta_{RH} \qquad \text{............[2 – 12]}$$

Where:

β_{sc} – Coefficient which depends on the type of cement : $\beta_{sc} = 4$ for slowly hardening cements, 5 for normal or rapid hardening cements, 8 for rapid hardening high strength cements .

f_{cm} – The mean compressive strength of concrete at the age of 28 days.

f_{cmo} =1 MPa

β_{RH} = -1.55 β_{SRH} for 40% ≤ RH < 99 %;

β_{RH} = 0.25 for RH ≥ 99%

$$\beta_{SRH} = 1 - \left(\frac{RH}{RH_0}\right)^3 \qquad \text{..............[2- 13]}$$

RH – The relative humidity of the ambient environment (%)

$RH_0 = 100\%$

$\beta_s (t - t_s)$ can be estimated by

$$\beta_S(t-t_S)=\left[\frac{\frac{(t-t_S)}{t_1}}{350.\left[\frac{h}{h_0}\right]^2+\frac{t-t_S}{t_1}}\right]^{0.5} \qquad \text{.......... [2- 14]}$$

$h = \frac{2A_c}{u}$ - The notational size of member (in mm) , where A_c is the cross-sectional area (mm^2) and u is the perimeter (mm) of the member with atmosphere.

$h_0 = 100$ mm

t_1 – day

2.4.4.3 GL 2001 model (Gardner,2001)

This model is based on ***(CEB-FIP, 1990)*** model where the earlier includes the type of cement as one of the input parameter.

The shrinkage strain is calculated using the following equation:

$$\varepsilon_{sh} = \varepsilon_{shu}\, \beta(h)\ \beta(t) \qquad \text{............... [2- 15]}$$

Where : ε_{shu} =ultimate shrinkage strain= $1000\times10^{-6}.K\left(\frac{30}{f_{cm_{28}}}\right)^{\frac{1}{2}}$ ***[2-16]***

where fcm_{28} is the concrete mean compressive strength at 28 days in MPa, and K is a shrinkage constant that depends on the cement type ,K as function of cement type.

1.00 for Type I ,

0.75 for Type II,

1.15 for Type III .

β (h) = correction term for effect of humidity ;

$$\beta(h) = (1 - 1.18h^4) \qquad \text{............[2- 17]}$$

$$\beta_t = \left(\frac{t-t_c}{t-t_c+0.12\left(\frac{v}{s}\right)^2}\right)^{\frac{1}{2}} \qquad \text{............[2- 18]}$$

β (h) = correction term for effect of humidity ;

β(t) = correction term for effect of time ;

2.4.4.4 Model by Miyazawa and Tazawa :

Two models were purposed to predict autogenous and drying shrinkage, respectively. They are based on ***(CEB-FIP, 1990)***. These models could be applied to high strength concrete as well as normal strength concrete and concrete with w/c ratio as low as 0.2. For autogenous shrinkage, the model considers w/c ratio and cement types by applying correction coefficients. As for drying shrinkage the rate of humidity is also added through the correction coefficients. The equation for autogenous shrinkage is:

$$\varepsilon_c\,(t) = \gamma\,\varepsilon_{co}\,\frac{W}{C}\,\beta_a\,(t) \qquad \text{..........[2- 19]}$$

Where :

$\varepsilon_c\,(t)$ = autogenous shrinkage of concrete at age t ($\times 10^{-6}$) ;

γ = a coefficient to describe the effect of cement type;

$\varepsilon_{co}\,\frac{W}{C}$ = the ultimate autogenous shrinkage ;

β_a (t) = a coefficient to describe the development of autogenous shrinkage with time .

The equation for drying shrinkage is:

$$\varepsilon_d\,(t, t_d) = \varepsilon_{do}\,(RH)\,\beta_d\,(t) \qquad \text{...........[2 - 20]}$$

Where :

$\varepsilon_{do}\,(RH)$ = the ultimate drying shrinkage ($\times 10^{-6}$)

β_d =a coefficient to describe the development of autogenous shrinkage with time .

2.4.4.5 Model by (Gillbert, 2001)

This model considered by Standards Australia, and proposed by ***(Gillbert , 1998)*** involves the total shrinkage strain, ε_{cs} , being divided into two component, autogenous shrinkage, ε_{cse}, (which is develop relatively rapidly and increases with concrete strength) and drying shrinkage, ε_{csd} (which develops more slowly, but decreases with concrete strength). At any time t (in days) after pouring, the autogenous shrinkage is given by:

$$\varepsilon_{cse} = \varepsilon^{*}_{cse} \left(1.0 - e^{-0.1\, t} \right) \qquad \text{…………[2 - 21]}$$

Where ε^{*}_{cse} is the final antogenous shrinkage and may be taken as

$$\varepsilon^{*}_{cse} = (3\ f'_{c} - 50) \times 10^{-6} \qquad \text{……………..[2 - 22]}$$

, where $f'c$ is in MPa. The basic drying shrinkage ε^{*}_{csd} is given by

$$\varepsilon^{*}_{csd} = (1100 - 8f'_{c}) \times 10^{-6} \geq 250 \times 10^{-6} \qquad \text{………..[2 - 23]}$$

And at any time t (in days) after the commencement of drying , the drying shrinkage may be taken as :

$$\varepsilon_{csd} = K_{1}\, \varepsilon^{*}_{csd} \qquad \text{…........... [2 - 24]}$$

The variable k_1 is given by $\mathbf{K_1} = \dfrac{k_4 k_5 t^{0.8}}{t^{0.8} + \left(\frac{t_h}{7} \right)}$ …………… [2 - 25]

Where: $k_4 = 0.8 + 1.2e^{-0.005 t_h}$ …………[2 – 26] and K_5 is equal to

0.7 For an arid environment

0.6 For a temperate environment

0.5 For a tropical / coastal environment

0.65 For an interior environment

t_h -hypothetical thicknesses

The final model (Gillbert, 2001) much closer to the experimental work of this investigation.

2.5 Creep of Concrete

(ACI 116) defines creep as the time - dependent deformation due to sustained load. Creep deformation can be several times as large as the strain on loading ***(Neville and Brook, 1987).*** Creep deformation of structural concrete under sustained load continues for a very long time. Half of ultimate creep will usually take place after two to six months under the load depending on the size of the member, while 80 % will probably occur after the first two years or so ***(Avarm et-al, 1981).***

The effect of creep on concrete can be both harmful and beneficial . It can be detrimental to structural behavior causing increased deflections, spalling, buckling of long columns and loss of prestress, and it can be beneficial to ductility, reduction in stress concentrations .

Creep characteristics are influenced by water and cement content, aggregate characteristics, age at time loading, type of curing, and applied stress to strength ratio ***(Collins, 1989) .***

2.6 Tensile Strain Capacity

The tensile strain capacity can be defined as the maximum strain that concrete can sustain in tension before cracking occurs. ***(AL-Rawi, 1987)*** defined the elastic tensile strain capacity " amount of strain which is instantly relived due to elastic recovery of restraint concrete upon cracking." Tensile strain capacity of concrete has the greatest influence on cracking tendency of concrete since it is the limiting strain which concrete can withstand before cracking ***(Harrison, 1981)***. Tensile strain capacity effect by some factors such as moisture condition, riches of the mix and volume and type of aggregate, age, there is some contradiction regarding the effect of age on tensile strain capacity does not show the same agreement , some researchers found the tensile strain increase with increase in age whilst others do not ***(Mujbil , 1988)***. ***(Houk et.al, 1970)*** found that increase in tensile strain capacity from (100 to 160) $\times 10^{-6}$ with increase in age from (7days) to (180days). Also ***(AL-Rawi, 1987)*** found tensile strain capacity increase with increase in concrete age.

The tensile strain capacity increase with increase in cement content. ***(Carlson e t- al, 1979)*** they found that when the cement content increased from (130 to 550 Kg/m^3) the tensile strain capacity increased significantly. ***(AL- Ali, 1985)*** studied the effect of coarse aggregate, the results found the decrease in tensile strain capacity with increasing maximum size and amount of aggregate, when used crushed type aggregate the tensile increasing from (95×10^{-6}) for rounded to (139×10^{-6}) for the crushed aggregate type .

According to ***(Al – Rawi, 1985)*** the tensile strain capacity of mortar beams increase by (16 %) with increase in curing time from (3 – 7) days.

2.7 Cracking Time

Cracking time is the time required for first crack to occur. The first crack is dependent on the same factors that influence the cracking tendency of concrete (degree of restraint, shrinkage, creep, tensile strain capacity …..etc). ***(AL- Rawi, 1985)*** reported that the crack time depends on both shrinkage and tensile strain capacity, crack time increases with increased curing time or with used smaller size and amount of coarse aggregate and increase with the reduction in w/c ratio. ***(Blackey and Lewis, 1959)*** studied the effect of curing on crack age, they found the crack age increases with the increase in duration of curing ,when the continuous to 28 days in fog spray followed by air curing the specimens did not crack. ***(AL –Nassar, 2002)*** also studied the effect of admixtures (plasticizer, superplasticizer, BVD and waterproofing) on the shrinkage of concrete, BVD is the modified sodium lignosulphate local plasticizer, steel I shaped moulds having a channel section was cast to study restrained shrinkage cracking in concrete, the results showed, the development of shrinkage strain is affected by both type and amount of admixture, cracking time increase when using superplasticizer and plasticizer, while it decreases with BVD and there was no change with waterproofing admixture .

(Kadhum, 2003) studied the cracking behavior due to the restrained drying shrinkage of the reinforced concrete slabs, these slab were externally restrained by different ends (two end, three end and, four restrained end), the slab with dimensions (2250 × 2250 × 100) mm, mix proportions were (545 : 635 : 1195) kg/m^3 (cement : sand: gravel) with (W/c = 0.48) ratio, the reinforced slab concrete exposed to drying for two months, the results showed the first crack takes place at (7 , 15 , 15) days, and crack width were (0.24, 0.255 and 0.275) mm for two, three and four end restrained slabs respectively at the age of 60 days.

(Alwash, 2005) studied the effect of sulphates in sand on drying shrinkage , a restrained shrinkage models were cast and exposed to laboratory conditions, four levels of SO_3 content in were investigated (0.29, 0.5, 1, 1.5) % of weight of sand, two zone of sand (2, 4) and two type of cement (OPC, SRPC), The results showed that increasing in sulphate content up to optimum 1.0 % by weight of sand lead to increasing in compressive strength, cracking time, free shrinkage strain at early ages, tensile strain capacity and creep (at cracking), then crack time increasing with increase in sluphate content and decreases with the higher value of sulphate than the optimum .

Also ***(Alkhafaji, 2007)*** studied the effects of adding finely divided mineral admixtures on drying shrinkage strain of end restrained and free concrete beams, end restrained steel molds having a channel section were cast and exposed to laboratory conditions, local admixtures of finely divided mineral admixtures were used which are lime stone dust, silica flour and bentonite, each types was additions in three levels by weight of cement, for lime stone dust (1.5, 3, 4) %, (2.5, 5, 8) % for silica flour admixtures and (2.5, 4.5, 6) % for bentonite admixtures, the results obtained that the increase of content of adding admixtures lead to increase in compressive strength especially with bentonite at level (2.5) %, when the percentage up to 6% the compressive strength decrease, also the results illustrate that the free shrinkage is affect by the type and amount of admixture addition, free shrinkage strain decreased at early and later ages with different levels of lime stone dust, the reduction at level (3) %

was about (22) % at 130 days of drying period, there is no effect was observed at all ages with silica flour. Also the results showed the cracking time is decreased with the addition of different contents admixtures (lime stone dust, silica flour, bentonite) to concrete mixes, while its same of that of the control mix with lime stone dust addition at level (1.5 %) .

(Kanastad et - al, 2000) found the cracking for high - strength silica fume concrete develops much faster and is significantly wider than that of normal strength concrete .

2.8 Conclude Remark

1- Shrinkage of high strength concrete may be expected to differ from conventional concrete in three broad areas: plastic shrinkage, drying shrinkage, and autogenous shrinkage.

2- High Sstrength Concretes, will be more susceptible to plastic shrinkage than conventional concretes.

3- Autogenous shrinkage due to self - desiccation is more likely in concretes with very low W/b ratio .

4- The drying shrinkage of high strength concretes lower than conventional concretes. This is probably due to the increase in stiffness of the stronger matrix.

5- Higher aggregate content with ahigher modulus of elasticity exhibits smaller shrinkage.

6-The drying shrinkage in atmosphere of 70percent relative humidity is about one third lower than in 50 percent .

7- Increasing the curing time could reduce drying shrinkage .

8- Chemical admixtures will tend to increase shrinkage unless they are used in such a way as to reduce the evaporable water content of the mix, in which case the shrinkage will be reduced.

9- Inclusion of pozzolainc admixtures (either RHA or SF) in concrete mix appear to increase the drying shrinkage compared to control mix, RHA addition displayed marginally lower shrinkage than its replacement.

10- Creep characteristics are influenced by water and cement content, aggregate characteristics, age at time loading, type of curing, and applied stress to strength ratio.

11- Tensile strain capacity effect by some factors such as moisture condition, riches of the mix and volume and type of aggregate, age.

12- Crack time increases with increased curing time or with used smaller size and amount of coarse aggregate and increase with the reduction in w/c ratio.

13- Cracking of high strength concrete is significantly wider than of normal strength concrete.

CHAPTER THREE

3

EXPERIMENTAL WORK

2.1 INTRODUCTION

This chapter investigates the experimental work, which consists of two parts , the first part deals with the experiment work, which is necessary for achieving High Strength Concrete from available materials. The second part deals with the drying shrinkage and the effect of addition Rice Husk Ash on the dimensional changes, for free and end restrained drying shrinkage, first crack, crack width, tensile strain capacity, elastic tensile strain capacity in high strength concrete, the dimensional change in normal strength concrete also studied .

3.2 Materials

3.2.1 Cement

After a survey carried out on several types of ordinary Portland cement (OPC) available, samples were taken from each type, tested for chemical and physical requirements to suit those required for production of HSC, Saudia Arabia (Eastern province cement) was found to be the most suitable cement (C_3S = 54.0). Tables (3 – 1) and (3 – 2) show the chemical and physical properties of this cement respectively, the tests was made in the testing cement laboratory of Babylon University.

Table (3 - 1): *Chemical analysis of the cement test in the testing cement laboratory of Babylon University .*

Oxide	***%***	***I.O.S. 5: 1984 Limits***
CaO	62.04	_
SiO_2	20.8	_
Al_2O_3	4.8	_
Fe_2O_3	4.0	_
MgO	1.1	< 5.0
K_2O	0.75	
Na_2O	0.35	
SO_3	2.5	< 2.8
Loss On Ignition (L.O.I)	1.7	< 4.0
Lime Saturation Factor (L.S.F)	0.81	0.66 - 1.02
Insoluable residue (I.R)	0.4	< 1.5 %
Free lime (F.L)	0.67	_
Total	99.92	
Compound Composition	***%***	***I.O.S. 5: 1984 Limits***
C_3S	54.0	_
C_2S	20.12	_
C_3A	4.0	_
C_4AF	13.17	_

Table (3-2) : *Physical Properties of Cement test in the testing cement laboratory of Babylon University .*

Physical Properties	***Test Results***	***I.O.S.5: 1984 Limits***
Fineness , Blaine , cm^2/gm	3300	>2300
Setting Time : Initial hrs ; min Final hrs ; min	 2;00 3;00	 ≥45 min ≤10hrs
Compressive Strength MPa 3-days 7-days	 24 32	 ≥15 ≥23

3.2.2 Aggregate

3.2.2.1 Coarse Aggregate

To obtained (HSC) crushed gravel with a maximum size (14) mm was used throughout this work. This aggregate was washed, then stored in air to dry and sieving into difrrerent particle sizes, then these sizes were remixed again. Table (3 – 4) and Table (3 – 3) shows the grading , physical and chemical properties of coarse aggregate .

Table (3 - 3) : *Grading of Coarse Aggregate**

Sieve size (mm	***Passing %***	***I.O. S NO 45 : 1984 Limits***
14	100	90 – 100
10	65	50 – 85
5	10	0 – 10
2.36	0	0

***** The tests was made in the concrete laboratory of Kufa University.

Table (3 - 4) : *Physical and Chemical Properties of Coarse Aggregate**

Physical Properties	***Test Results***	***I.O. S NO 45 : 1984 Limits***
Specific gravity (S.G)	2.6	_
Absorption %	1.1	_
Sulfate Content (SO_3) %	0.06	≤0.1
Clay %	0.5	≤1.0

***** The tests was made in the concrete laboratory of Kufa University.

3.2.2.2 Fine Aggregate

Natural sand (from Al- Najaf's Sea) was used. Table (3 - 5) shows the grading of the fine aggregate and the limits of the ***(Iraqi specification No . 45 /1984)*** .Table (3 - 6) shows the physical and chemical properties of fine aggregate .

Table (3 - 5): *Grading of Fine Aggregate**

Sieve size (mm)	Passing %	I.O.S.45:1984 Limits Zone (2)
10	100	100
5	93	90-100
2.36	76	75-100
1.18	65	55-90
0.6	42	35-59
0.3	16	8-30
0.15	5	0-10

Fineness modulus of fine aggregate (FM) = 3.03

* The tests was made in the concrete laboratory of Kufa University

Table (3 - 6): *Physical and Chemical Properties of Fine Aggregate**

Physical Properties	***Test Results***	***L.O.S.45 1984 Limits***
Specific gravity	2.65	_
Absorption %	1.6	_
Sulfate content (SO_3) %	0.324	≤ 0.5
Clay %	2.3	≤3.0

* The tests was made in the concrete laboratory of Kufa University

3.2.3 Water

For all mixing and curing of concrete tap water was used in this investigation .

3.2.4 Superplasticizer

To produce High Strength Concrete, super plasticizer (high range water reducing) (HRWR) called Glenium 51(Polyether - PolyCarboxylic) was used in this work. The normal dosage for Glenium 51 is between 0.5 and 0.8 liters per 100 kg of cement (cementitious materials). According to ASTM C 494, it has accelerated effect on the HSC, the typical properties are shown in Table (3 - 7).

The superplasticizer was added to mix after addition of 70 % of water and then addition the remained water, superplasticizer must not be added to the dry materials(*).

Table (3 - 7) : *Typical Properties of Super plasticizer* (*)

Form	Viscous liquid
Colour	Light brown
Relative density	1.1 @ 20ºC
pH value	6.6
Viscosity	128+/-30 cps @20ºC
Transport	Not classified as dangerous
Labelling	No hazard label required

(*) From the catalog of manufacture (***Degussa Construction Chemicals***)

3.2.5 Preparation of RHA

Burning of rice husks according to ***(Habeeb, 2000 and Hana, 1984)*** was carried out in a furnace with controlled temperature in order to establish the optimum burning temperature and burning time.

The produced ash was burnt at a temperature of 500 ºC for two hours, to minimize its carbon content to about 3.03 percent. The burning was made in AL-Zahraa –Women Organization – Najaf –Ministry of Youth .

3.2.5.1 RHA Grinding

For 4.5 hours Los Angeles Machine was used to ground the ash, using twelve special steel balls (425 gm) in addition to two steel rod (2.25 kg) and a steel chain of 2 meters length and mass of 1.9 kg. For more finesse the ash was ground by small mill for another 4 min.

3.2.5.2 Chemical Analysis and Physical Properties of RHA

The chemical composition and physical properties of the RHA are given in Table (3 - 8). The test was made in the laboratory of New Cement Plant of Kufa.

Table (3 -8): *Chemical analysis and physical properties of the RHA*

Chemical analysis oxide (%)		***Physical Properties***	
SiO_2	86..2		
Al_2O_3	0.43	Specific gravity	2.06
FeO_3	0.28	Fineness Specific Surface Blain cm^2/gm	10800 cm^2/gm
CaO	1.21		
MgO	0.37		
Na_2O	1.15		
K_2O	3.34		
SO_3	1.7		
Carbon	3.03		
Loss On Ignition (L. O.I)	2.3		

3.2.5.3 Addition of the RHA to Concrete Mixes

In this work two values of RHA content of 5 and 7.5 % by weight of cement were used in addition to the amount of Portland cement. RHA was added to Portland cement and mixed in dry state by trowel for a period more than 10 minutes in order to break up the lumps of RHA and ensure uniform dispersion of the fine particles of the RHA throughout the cement particles.

3.2.6 Addition Superplasticizer

In order to assess the superplasticizer – cement addition , the percentage water reduction for the (SP) mix having the same workability as an admixture – free mix using the flow - table method BS1881: part 105 : 1984. Figure (3 - 1) is obtained from experimental work. This Figure shows the amount of SP addition by weight of cement and percentage of water reduction.

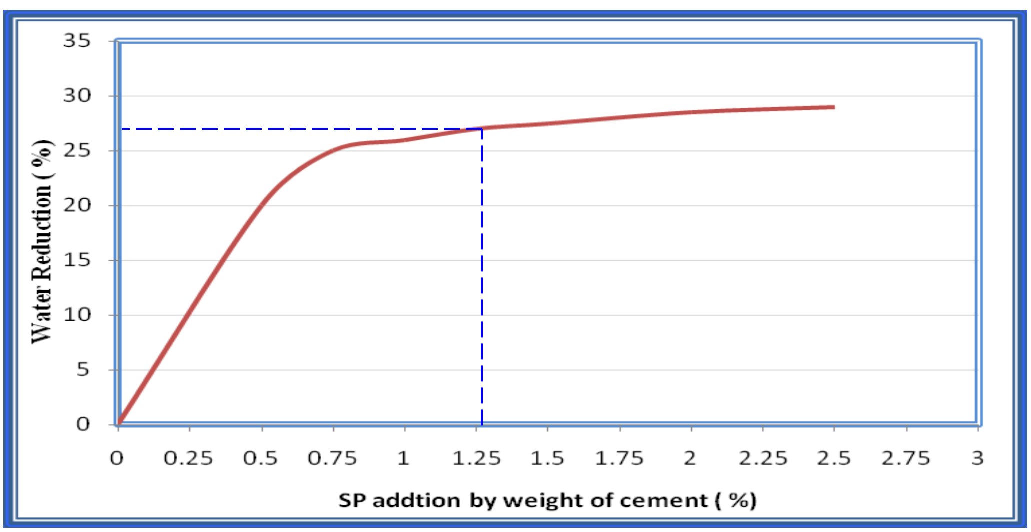


Figure (3 - 1) : Water reduction for superplasticizer mixes.

For this work a ratio of (1.25 % of SP addition was chosen, since it gave a good water reduction of (27 %) first and also for economical use of superplasticizer to limit the amount of SP added to concrete.

3.3 Mix Preparations

3.3.1 Mix Design

Three target design strengths of (50, 70, 85) MPa and a normal strength concrete of (25 MPa) were designed according to British mix design method BS5328. Part 2:1991, in this investigation two levels of RHA were added to concrete mixes (5 and 7.5%) were used as a percent of the cement content. Three mixes were prepared, the first mix, is a reference mix in which only super plasticizer was used, the second and the third mixes were super plasticizer and RHA were used .

3.3.2 Mix Proportions

Table (3 – 9) shows the mix proportions of the concrete mixes, an example of the notation of these mixes is shown below for the 50 MPa concrete mixes.

SP : reference concrete mix with 1.25% SP.

SPH_1 : concrete mix with 1.25% SP and 5% RHA.

SPH_2 : concrete mix with 1.25% SP and 7.5 % RHA

Normal : reference concrete

Table (3-9) Mix Proportions and Slump Values

Mix Notation	Super-Plasticizer %	Cement materials		Water kg/m³	Fine Agg. kg/m³	Coarse Agg. kg/m³	W/C or W/C+RHA	Target Strength MPa	Slump mm
		Cement kg/m3	RHA kg/m³						
SP 50	1.25	540	_	200	630	1020	0.37	50	150
SPH_150	1.25	540	27	200	620	1000	0.35	50	125
SPH_250	1.25	540	40.5	200	602	985	0.34	50	110
SP70	1.25	580	_	174	661	1015	0.3	70	62
SPH_170	1.25	580	29	174	635	999	0.28	70	30
SPH_270	1.25	580	43.5	174	615	982	0.27	70	18
SP 85	1.25	740	_	162	485	1100	0.22	85	10
NORMAL	_	385	_	220	700	1000	0.57	25	65

SP : mix with superplasticizer , SPH : mix with superplasticizer and rice husk ash

SP 85 : concrete mix with 1.25 % SP and high amount of cement (740) kg/m^3, this amount is not practical nor economical, its used to find the effect of maximum cement ratio on drying shrinkage cracking

3.3.3 Molds :

The molds used in this work were as follows :

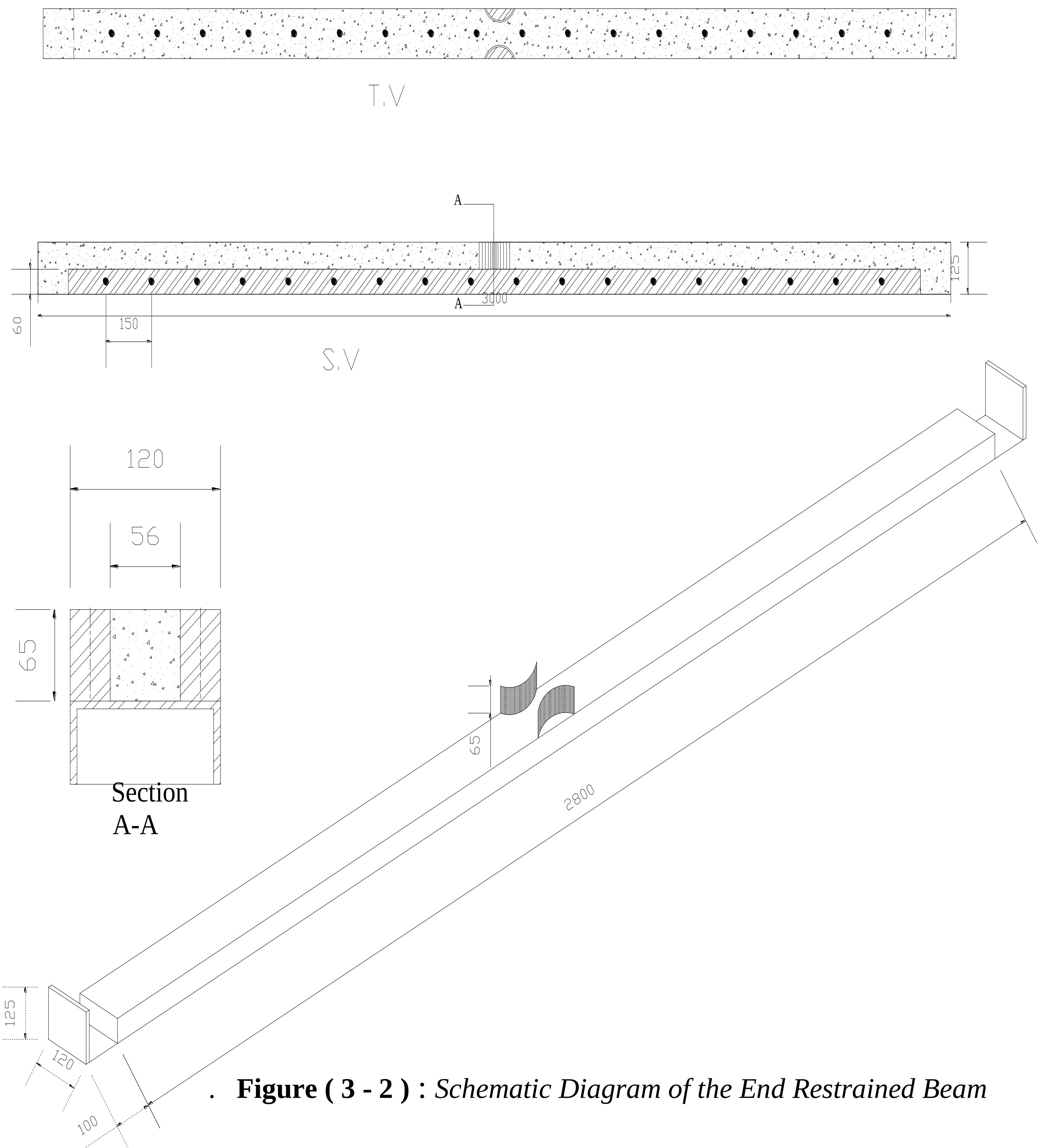
1-(100 ×100×100) mm cubes to obtain concrete specimens for compressive strength test .

2- (100 × 200) mm cylinders to obtain concrete specimens for splitting tensile strength test .

3- (150 × 300) mm cylinders to obtain concrete for static modulus of elasticity test .

4- Shape molds having a channel section with middle narrower as shown in Figure (3 - 2) used to study free shrinkage and cracking of end – restrained concrete members .

All the molds were cleaned and rigidly tightened they were oiled in order to facilitate the demolding process, for the shrinkage mold, layer of polyethylene sheets was put over the channel base and oiled to minimize base friction .



. **Figure (3 - 2) :** *Schematic Diagram of the End Restrained Beam*

* all dimensions in millimeters

3.4 Fresh concrete

3.4.1 Mixing of Concrete

A horizontal drum laboratory mixer with a capacity of 38 kg was used to mix the concrete, to begin the mixing process, the mix content were weighed (coarse and fine aggregate, cement), interior surface of the mixer cleaned and moist before placing the materials, cement added to half of coarse aggregate and half of fine aggregate, then the residual coarse and fine aggregate was added , the dry materials were well mixed for about 2 minutes to obtain uniform mix , 70 % of the mixing water was added and the whole mix constituents were mixed for another 2 minutes, the Super plasticizer was added to the concrete and add the remaining water then concrete constituents were further mixed for at least one minute. For concrete mixes with RHA , the RHA was blended section (3 . 2 .5.3) and then followed the same procedures.

3.4.2 Slump Test

Slump of each concrete batch was measured immediately after mixing. According to ***(ASTM C143).***

3.4.3 Casting and Compacting of Fresh Concrete

The concrete mix was cast in the molds in two layers for cubes ,three layers for cylinders and two layers for shrinkage mold, finally the molds were leveled by hand trawling. The concrete mixes were fully compacted on a vibrating table. The vibration time to reach full compacting was decided upon the stopping of air bubbles immigration from the surface of fresh concrete.

3.4.4 Curing and Drying Condition

In order to avoid plastic shrinkage cracks after casting of concrete, a polyethylene sheet was used to cover the beams after casting and when the curing start, wetted hessian sheet and polythene sheet were used to cover the upper surface of the beams, the beams was cured by covering them with hessian and polythene sheets and wetted once every day for first 7 – days, then air dried in uncontrolled laboratory conditions till the age of 70 – days, Plate (3 - 1) show shrinkage mold during 7days curing **.**

demecs point

Plate (3 - 1) *:shrinkage mold during 7 days curing*

The restrained shrinkage tests were conducted under the drying conditions in laboratory . Temperature and relative humidity variation in the laboratory were measured and recorded. Specimens of series (70, 85) MPa was casted in April. The measurements continued till June. Series 50 MPa and normal concrete was cast in June and measurements continued till September for drying shrinkage speciments. In this period the relative humidity and temperature are given in Table (4 – 6) .

Table (3 - 10):- Temperature and Relative Humidity During Measurement Period (2007) (*).

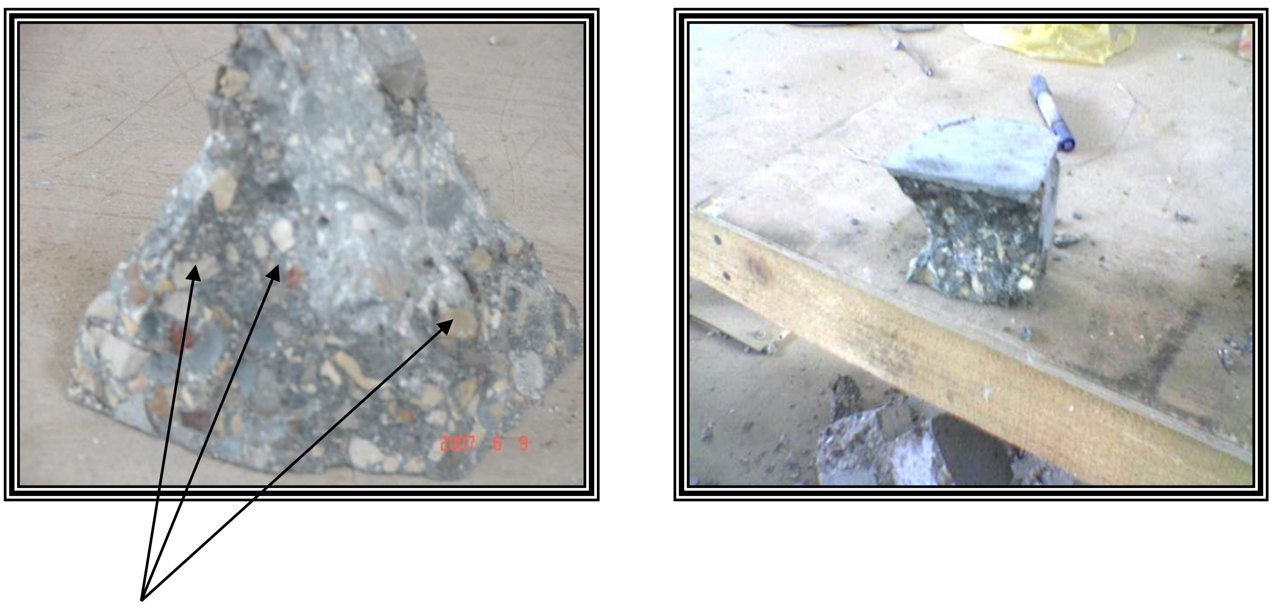
Month	Temperature (av.)	Relative humidity (av .) %
April	*32.1*	*48*
May	*36.7*	*39*
June	*39.9*	*34*
July	*44.9*	*36*
August	*42.5*	*39*
September	*38.8*	*43*

(*) From Center of Weather Foretast in Najaf

3.5 Testing of hardened concrete

3. 5.1 Compressive Strength Measurement

Use of smaller specimens is preferable, specifically; (100 mm) cubes are satisfactory to fit the capacity of testing machine. In this work all compressive strengths were measured on (100 mm) cubes, which were prepared according to ***(BS1881: part 116: 1983).*** After casting the specimens moist cured for 24 hours, then cured in water at (20 ± 2°C) until the age of 28 days (include the period in mold). The specimens were removed and left inside the laboratory until test at age of 60 days to measure compressive strength using a digital compression machine, with a capacity of (2000 kN). The compressive strength of cubes was determined by applying the load at a constant rate of (3 kN/min). For each age (3, 7, 28, 60) days the compressive strength was taken as the average of three cubes was taken. Plate (3 – 2) shows the cubes shape failure after testing.



aggregate fracture **Plate (3- 2)** *Cubes shape after failure*

3.5.2 <u>Splitting Tensile Strength</u>

The splitting tensile strength was determined according to***(ASTM C – 496)***. After casting the cylinders moist cured for 24 hours at a temperature (20 ± 2°C) and relative humidity of about 95 %, then cured by wetted every day for 7 – days, then air dried in uncontrolled laboratory condition until age of 60 days . Each splitting tensile strength value was the average of two specimens (100 × 200) mm, using the same machine which was used in measuring compressive strength.

3.5.3 Modulus of Elasticity Test

The static modulus of elasticity was determined according to ***(ASTM C – 469)*** specification. The cylinders were cast into three layers, then demolded and cured as for of splitting tensile strength . The top of cylinder was well – finished and smoothed by using electric grinding machine to avoid any loss of strength .The specimen is placed in compressive machine with the strain – measuring equipment attached. Two types of strain measuring equipments were used, one consists of a compressometre and the other linear voltage displacement transducer (LVDT), and the specimen was loaded three times, during the first load, which was primarily for the seating of the gauges, the average of second and three load was taken. The load was applied at a constant rate to 40% of the ultimate load, the load and deformation was recorded to determine the modulus of elasticity, the results are plotted on a graph, in which the slope is the modulus of elasticity. The test made in laboratory of AL-Najaf Technical Institute.

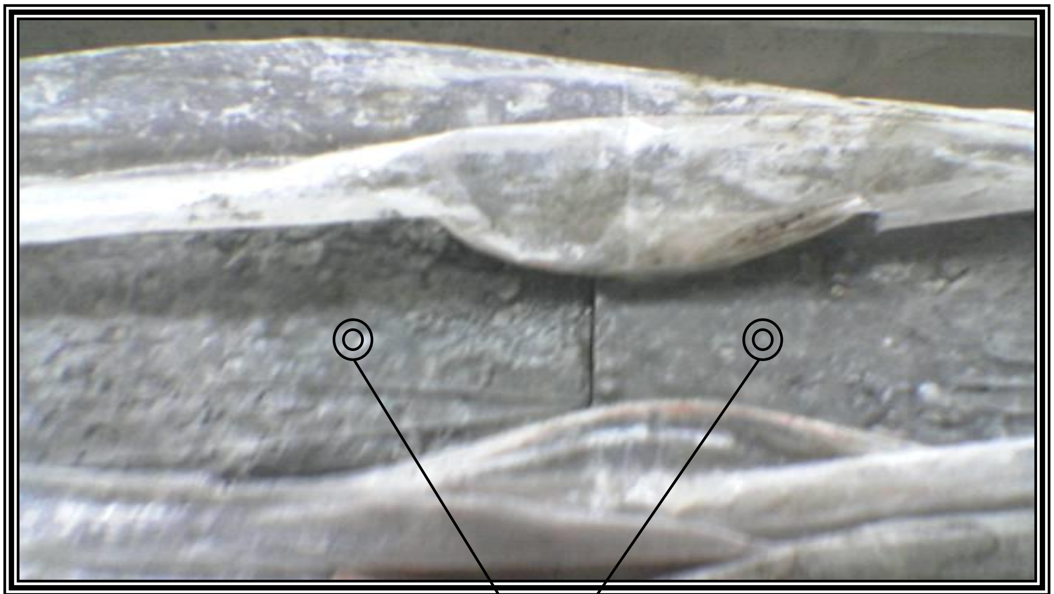
3.6 End Restrained –Shrinkage Test

Concrete beams were tested for restrained shrinkage. The shrinkage cracking model was based on the model devised by ***(Al-Rawi, 1985)*** with a modification by making it narrow at the middle of the web Figure (3 - 2) where the beams were left in the molds (to achieve end restraint by the mold). For each mix, three molds were used, two molds for restrained shrinkage test and the other was supplied with artificial crack (opining) in the web for free shrinkage determinations as can be seen in plate (3 - 3) the contraction at the surface of the drying concrete web was measured by the distance between the demec points, using a mechanical dial gauge with (15 cm) in length and an accuracy of (0.002 mm / division) the demec points were positioned on the surface of concrete in fresh state, after 24 hours to more fixed used adhesive epoxy resin.

After the occurrence of cracking, the measurement was repeated to record the recovery free contraction of concrete (the elastic tensile strain capacity) at the onset of cracking. Further demec points were fixed at the steel mold beam side in order to measure the amount of the loss of restraint, which is due to shortening of the steel mold prior to cracking as shown in Plate (3 - 4).

The free shrinkage of concrete was determined by fixing demec points at both side of the gap for beams with artificial crack (opining) in the web and daily measuring the widening of the artificial crack in the middle of the beam. Shrinkage reading were taken from the next day after leaving the mold in the laboratory, till little or no movement could be recorded. The crack width is measured at the moment of crack apparent by using a portable microscope.

The elastic tensile strain capacity was measured by using the same end restrained beam, after the occurrence of cracking, the measurement was repeated to record the recovery free contraction of concrete. Creep strain was calculated by subtraction the elastic tensile strain capacity from the tensile strain capacity.



Two demecs point

Plate (3- 3) :*artificial gap and two demecs on the sides of the gab*

Plate (3- 4): *fixed demec points in side the mold*

Plate (3 - 5): *Microscope (crack width)*

Plate (3 – 6): *Measured crack width measurement*

Plate (3- 7) : *Measurement Device(measured volume change)*

CHAPTER FOUR 4

RESULTS AND DISCUSSION

4.1 INTRODUCTION

In this chapter, the results obtained from the experimental work are presented and discussed. These results include the following characteristics of the fresh concrete (slump) and the hardened properties of the concrete (compressive strength, tensile strength, volume changes). To condense the data, only the average values of test results are provided in form of Tables and Figures .

4.2 Fresh Concrete Properties

4.2.1 <u>Slump</u>

Slump test was used to measure workability of the mixes according to ***(ASTM C143)***, the results of this test are shown in Table (3 – 9) and Figure (4 - 1) . It is clear that the slump of series SP50 was higher than (SP 70, SP 85), this may be due to higher water content and higher W/b ratio, and the lower slump for (SP 85) can attributed to very high cement content (740 kg /m ^{3}) and lower W/b ratio .

Slump of the mixes which include RHA in two percentages (5, 7.5) % is less than mixes without RHA by about (17, 27) %, (50, 71) % for series (50 , 70) MPa respectively, this reduction in slump may be attributed to higher cementituios materials and higher surface area of RHA (10800 cm^2/gm) .

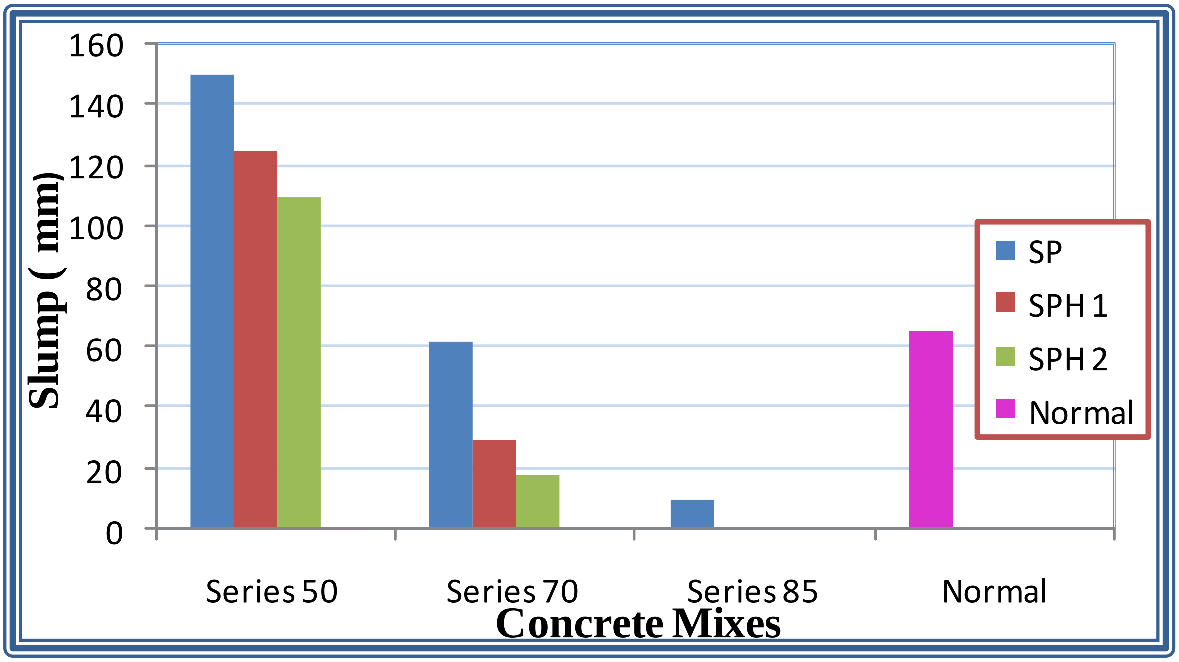


Figure (4 - 1) : results of slump test

4.3 Compressive Strength

The compressive strength test results of the concrete specimens (cubes 100 mm) were tested at age (3, 7, 28 and 60) days, the results of tests and development of compressive strength gain are shown in Table (4 - 1) and Figure (4 - 2) to Figure (4 - 6).

It is clear that the compressive strength of series SP concretes ranged between (50 – 102) MPa , series SPH_1 concretes ranged between (36 - 74) MPa , series SPH_2 concretes ranged between (34 - 72) MPa .

The increasing in series SP can be first attributed to the greater degree of hydration caused by the superplasticizers more over can contributed to improvement in the mechanical properties of concrete mixes, (lower initial volume of voids that are needed to be filled by the products of hydration due to the reduction in total volume of mixing water).

From the results of this work, it was found , the addition of (5% RHA) content lead to reduction in compressive strength at early ages by (28, 22) %, for series 50 MPa , (26, 24) % for series 70 MPa at ages (3, 7) days respectively .

Increasing RHA content up to (7.5 %), lead to a deficiency in compressive strength around (32, 29) % for series 50 MPa , (34, 27) % for series 70 MPa for (3, 7) days ages .

These results may be due to the fact that the quantity of RHA present in the mix is higher than that required to combine with the liberated lime during the process of hydration thus leading to excess silica leached out and causing a deficiency in strength, also may alters the hydration rate effectively delays the dissolution of the RHA particles.

On the other hand the rate of development of compressive strength between 28 and 60 days for series SP was increasing by about (24, 16, 15)%, for series (50, 70, 85) MPa respectively, while normal concrete rate was (36) %, from these results it can be concluded that series 50 MPa, and normal strength concrete experienced a major increase in compressive strength between 28 and 60 days

. **Table (4 – 1) :-** *Compressive Strength (MPa)of the concrete mixes with and without RHA content*

Mix Notation	***Compressive Strength (MPa) Age days***			
	3	***7***	***28***	***60***
SP 50	50	55	62	77
SP H_1 50	36	43	52	63
SP H_2 50	34	39	46	57
SP 70	62	71	76	88
SP H_1 70	46	54	63	74
SP H_2 70	41	52	60	72
SP 85	67	80	89	102
Normal			24.8	33.7

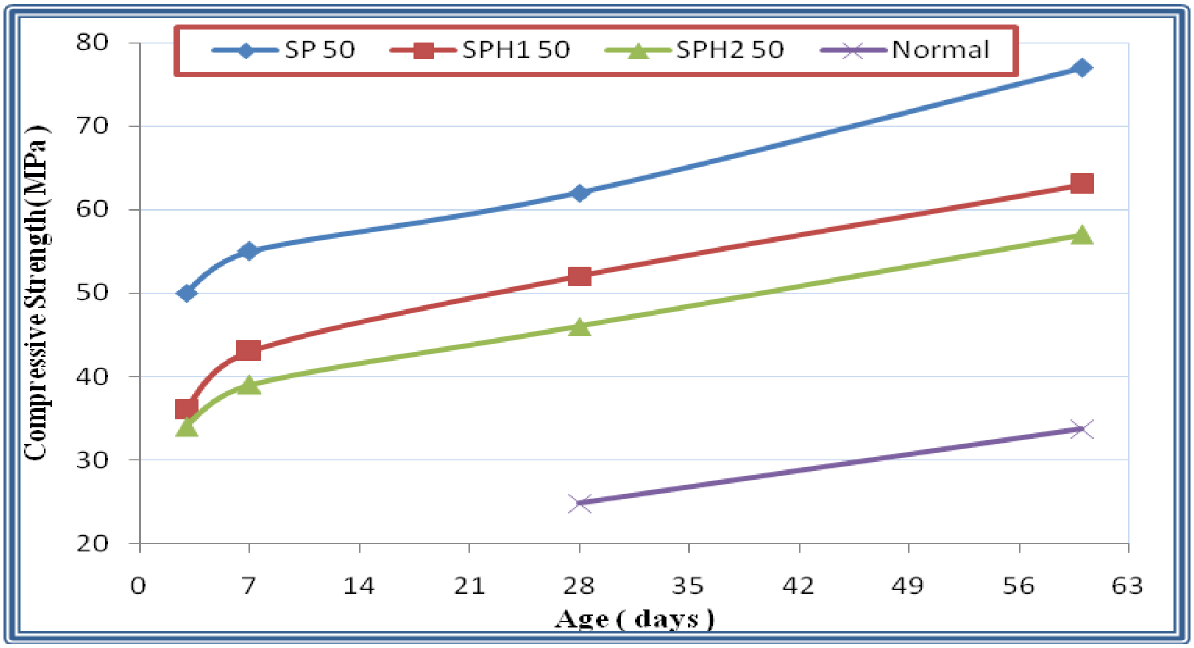


Figure (4 - 2): Development of Compressive Strength of series SP50 and Normal Concrete.

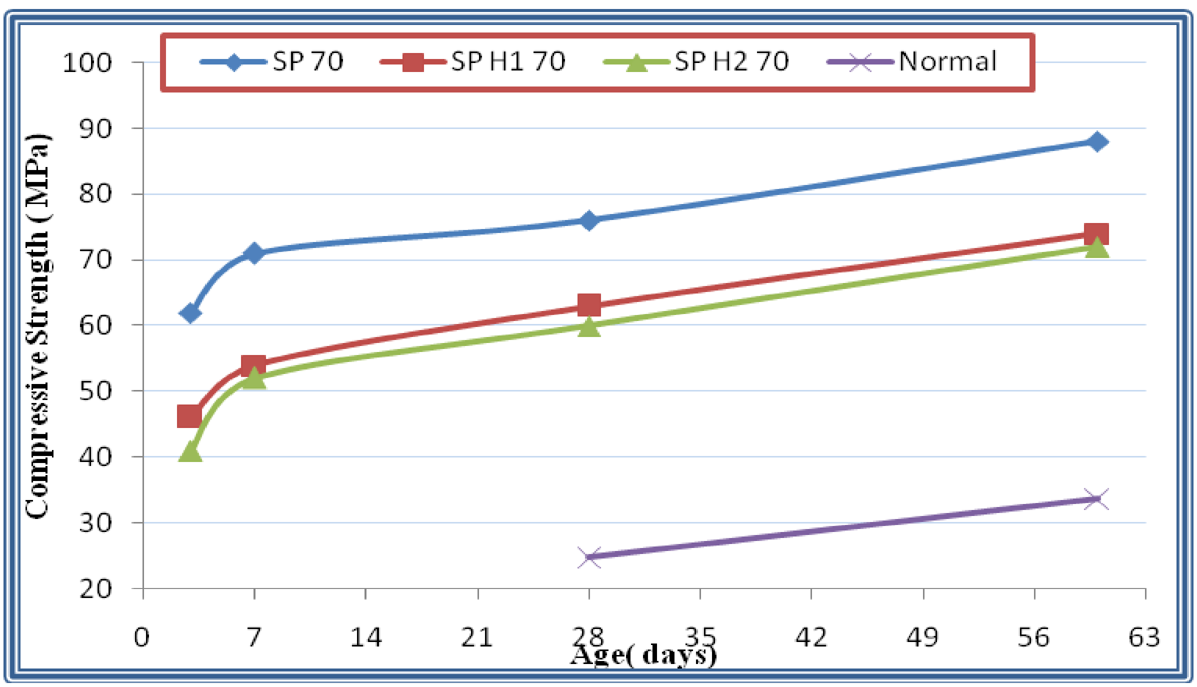


Figure (4 - 3) : Development of Compressive Strength of Series SP 70and Normal Concrete.

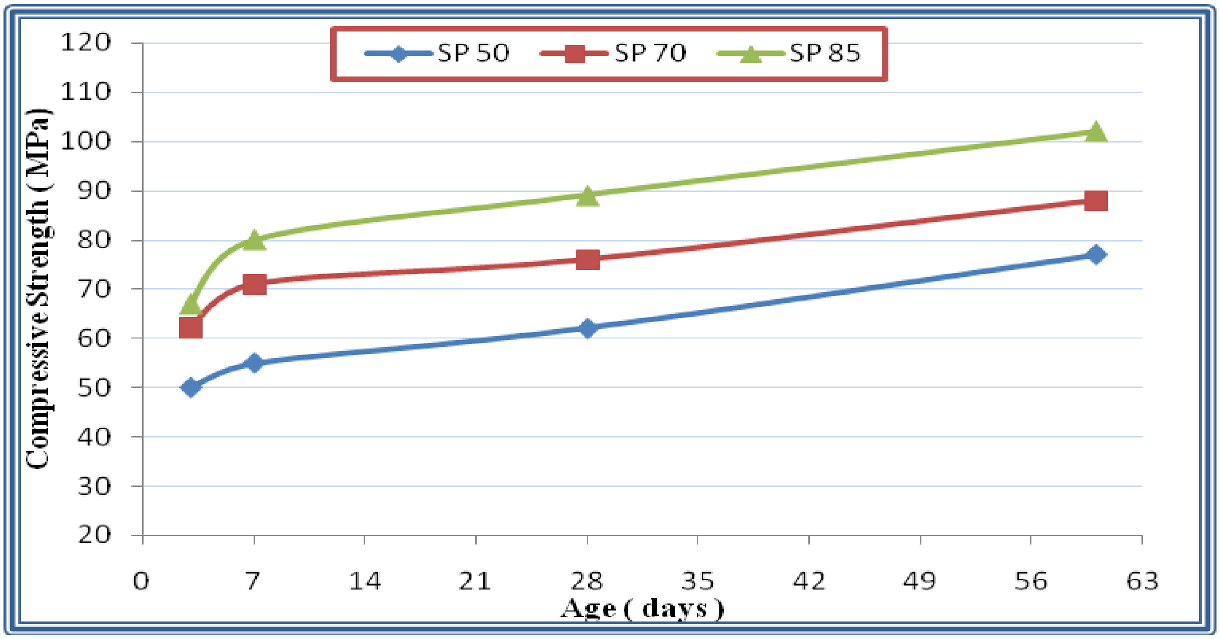


Figure (4 - 4): Development of Compressive Strength of Series SP (50 , 70 , 85) MPa

The rate development of compressive strength of series (SPH_1, SPH_2) MPa between 28 and 60 days were about (21, 24), (17, 20) % for series (50, 70) MPa respectively. From these results it be can observed that, (SPH_1 50, SPH_2 50) have development higher than these series (SPH_1 70, SPH_2 70) by about (19, 17) %, as shown in Figures (4 – 5), (4 – 6) .

The higher water to cementitious materials ratio and lower amount of cementitious material means larger volume of voids needed to be filled by solid products of hydration which require longer period of time, this is with complete agreement with studies carried by other researchers ***(Sarkar et -al , 1990)*** .

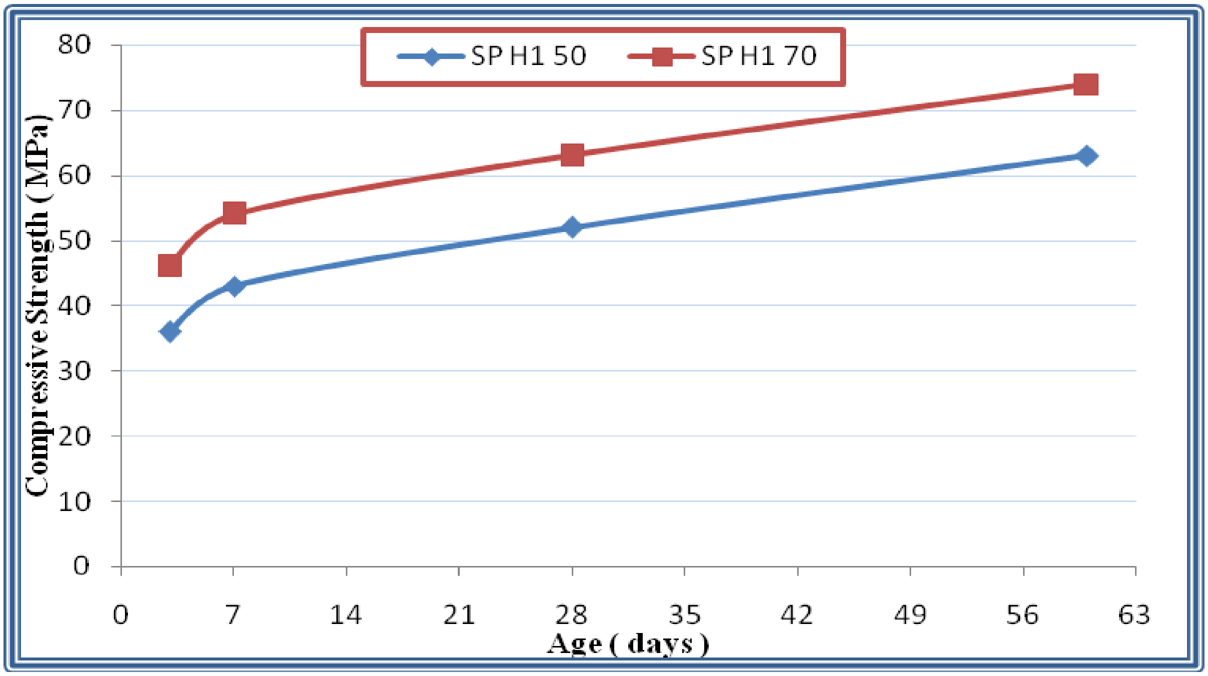


Figure (4 - 5): Development of Compressive Strength of Series SPH1 (50 , 70) MPa

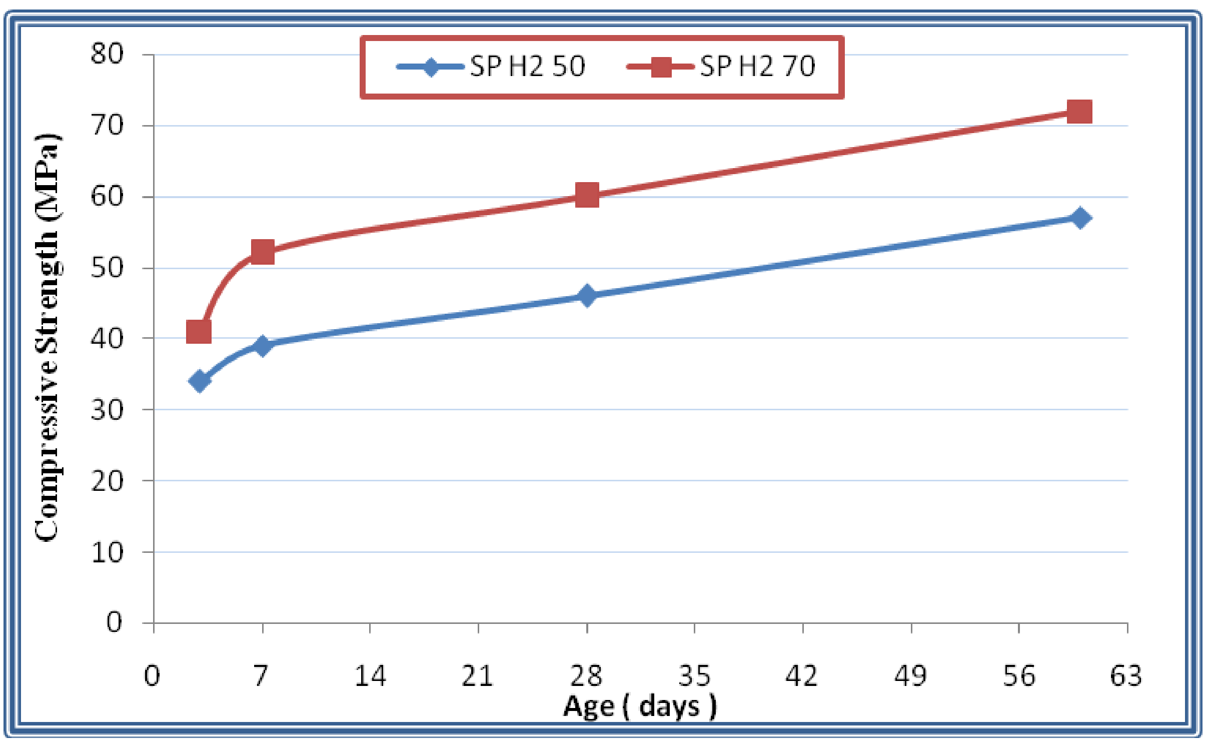


Figure (4 - 6) : Development of Compressive Strength of Series SPH2 (50 , 70) MPa

4.4 Splitting Tensile Strength

The tensile strength governs the cracking behavior and affects other properties such as stiffness and durability of concrete. The tensile strength is determined either by direct tensile test or by indirect tensile tests such as flexural or split cylinder tests ***(Carrasquiho et - al , 1987)*** .

(Dewar, 1964) reported that the splitting tensile strength may be as high as 10 percent of the compressive strength but at higher strengths it may reduce to 5 percent.

In this work splitting tensile strength of concrete measured as described in chapter three section (3 - 5 - 2). The results of tests at age (7, 28, 60) days are shown in Table (4 - 2) and plotted in Figures (4 - 7) and (4 - 8) .

The tensile strength of concrete for series SP ranged between (3.2 – 5.1) MPa, for series SPH_1 (3 – 4.6) MPa, for series SPH_2 (2.9 – 4.3) MPa and (2.1 – 3.2) MPa for normal concrete .

It is clear from Figures (4 - 7) and (4 - 8) and Table (4 - 2), that the rate of development of tensile strength between 28 and 60 days are a round (15 - 17) % , (16 - 19) % and (14) % for series (50 , 70) MPa and normal concrete respectively. SP70 exhibits higher splitting tensile strength than SP50 by about (7 - 21) % at (7 - 60) days respectively .This results can be attributed to high stiffness and higher compressive strength.

The addition of (5% RHA) content to concrete leads to a reduction in splitting tensile strength by (6 - 8) %, (9 - 11) % for series (50, 70) MPa respectively for ages (7 - 28) days.

Increasing RHA content up to (7.5 %), leads to a deficiency in splitting tensile strength around (9 – 14) %, (14 - 18) % for series (50 , 70) MPa for (7 - 28) day ages. The reactivity of RHA is not enough to complete or combine with liberated lime during the process of hydrations thus causing a deficiency in tensile strength .

From these results, it can be observed that the influence of addition RHA to concrete on the splitting tensile strength was found similar to behavior of compressive strength, also the tensile strength related to compressive strength of concrete mixes.

Table (4-2):- *Results of the splitting tensile strength(MPa) of concrte mixes with and without RHA content*

Specimen designation	***Splitting tensile strength(MPa) at age of(days)***		
	7	***28***	***60***
Sp50	3.2	3.6	4.2
SP H1 50	3	3.3	3.8
SP H2 50	2.9	3.1	3.6
SP 70	3.8	4.3	5.1
SP H1 70	3.4	3.9	4.6
SP H2 70	3.1	3.7	4.3
Normal	2.1	2.8	3.2

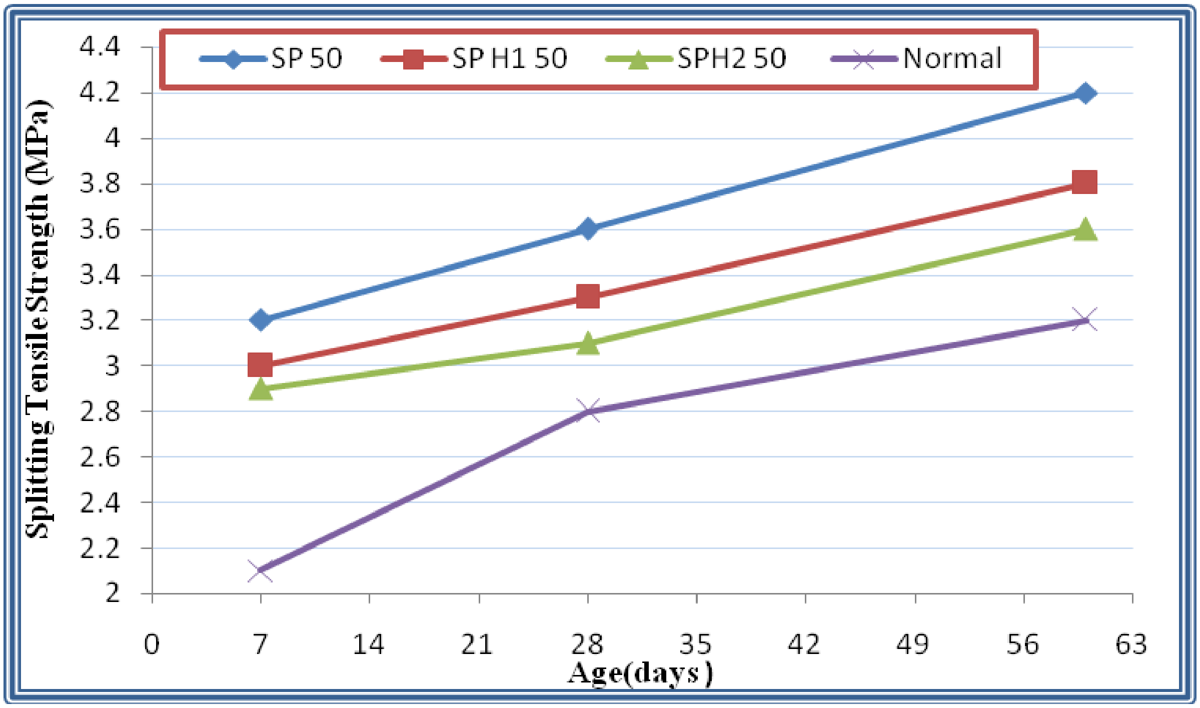


Figure (4 - 7) : Development of splitting tensile strength of series 50MPa and normal concrete

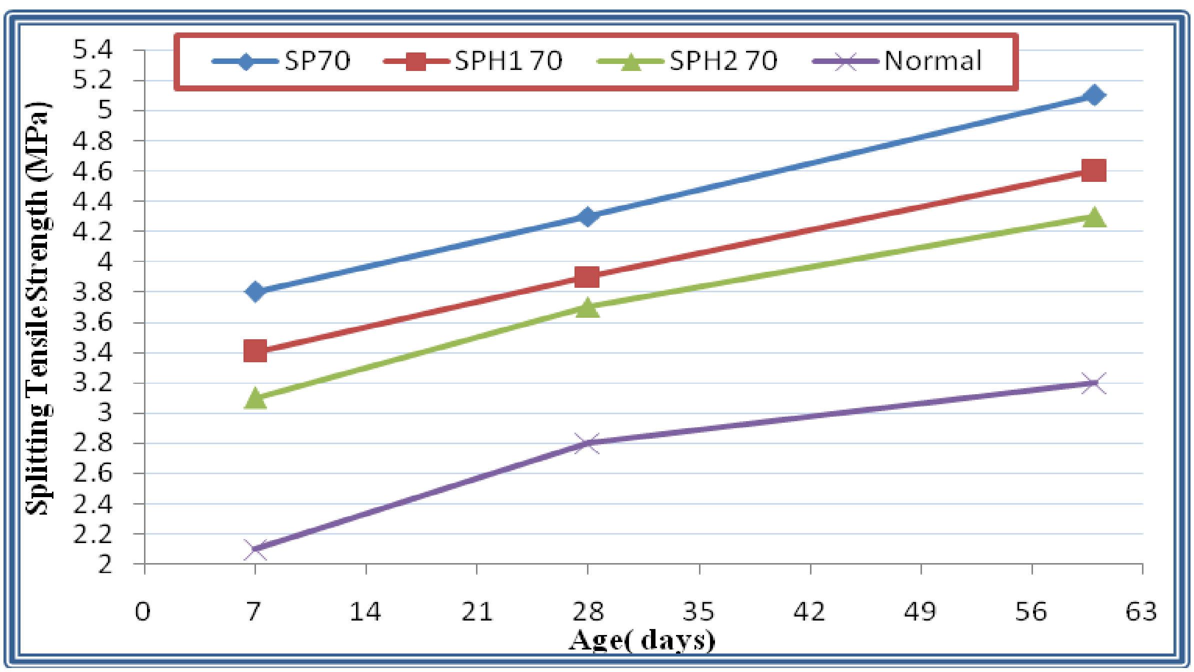


Figure (4 – 8) : Development of splitting tensile strength of series 70 MPa and normal concrete

The relation between the compressive strength expressed as $\sqrt{f_c}$ in MPa, and the splitting tensile strength f_{sp} in MPa, is plotted in Figure (4 - 9). The equation of the best fit line for the results obtained in this study is :

$$f_{sp} = 0.676\sqrt{f_c} - 1.596 \qquad 46\ MPa < f_c < 76\ (MPa) \qquad \ldots.(4-1)$$

The following equation is recommended by ACI 363 – R 25 , for the prediction of the tensile splitting strength f_{sp} of normal weight concrete based on a study by ***(Carrasquiho et –al, 1987)*** .

$$f_{sp} = 0.59\sqrt{f_c}\ MPa \qquad 21\ MPa < f_c < 83(\ MPa\ ) \qquad \ldots.(4-2)$$

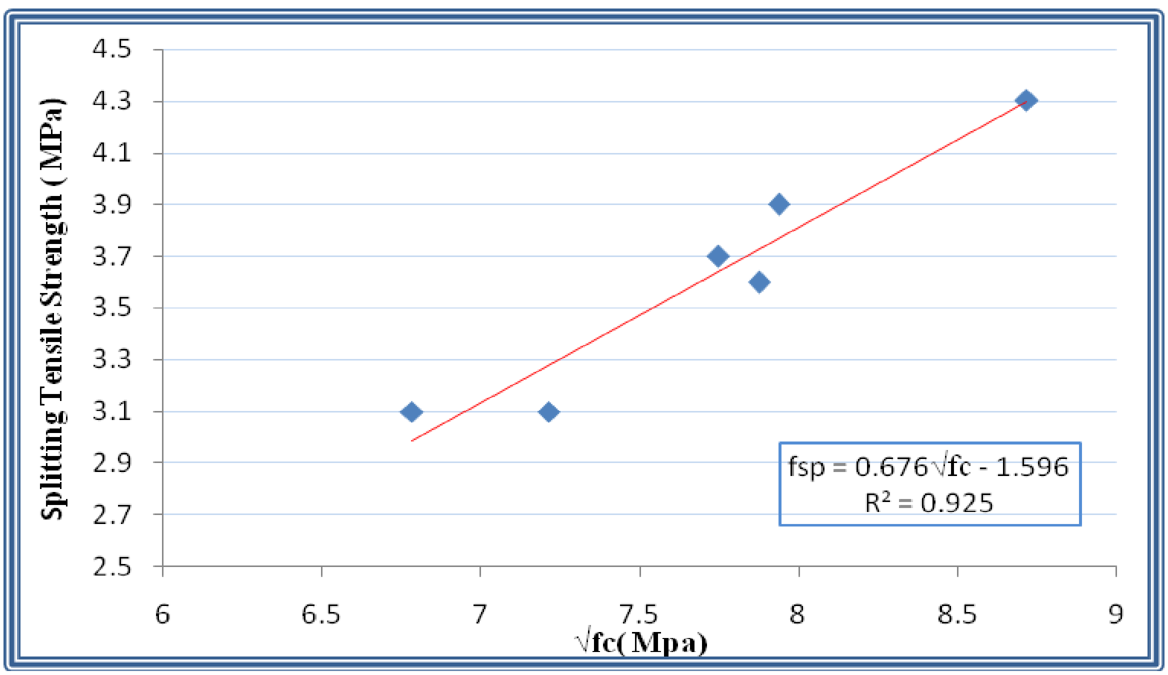


Figure (4 - 9) : Relation between splitting tensile strength and square root of compressive strength

4.5 Modulus of elasticity

The modulus of elasticity is generally related to the compressive strength of concrete. This relation depends on the aggregate type, the mix proportions, curing conditions, rate of loading and method of measurement. The elastic modulus is determined either by static elastic modulus or dynamic elastic modulus ***(Zia et – al , 1993) .***

In this work static elastic modulus measured as described in chapter three section (3 - 5 - 3) was adopted , The results of tests at age (7, 28, 60) days are given in Table (4 - 3) and plotted in Figures (4 - 10) and (4 – 11)

Results illustrate that the modulus of elasticity of concrete for HSC ranged between (31.7 – 42) GPa for series SP, (30.5 – 39) GPa for series SPH_1, (29.6 – 37.5) GPa and (27 – 30.5) GPa for normal concrete .

From the same Figures, SP70 MPa exhibited higher elastic modulus than series 50 MPa, these results can be attributed to high compressive strength for series 70 MPa than series 50 MPa .

The addition of RHA to concrete in two percentage caused reduction in elastic modulus by about (4 - 10) % , (4 - 9) % for series (50, 70) MPa respectively.

From these results it can be concluded that increasing RHA exhibits a similar effect to that of compressive strength and splitting strength but with different percentages .

Modulus of elasticity of normal concrete is lower than these of series (50, 70) MPa by about (17 – 18) %, (37 - 38) % for series (50, 70) MPa respectively .

From the results of this work it was found that while the compressive strength at ages from 28 to 60 days increased from (46 MPa to 102 MPa), splitting tensile strength (3.1 MPa – 5.1 MPa), modulus of elasticity was enhanced only from (30.7 GPa to 42 GPa) .

The elastic modulus of concrete increases with compressive strength . The modulus is greatly affected by the properties of the coarse aggregate, the large amount of coarse aggregate with a high elastic modulus, the high modulus of elasticity of concrete. The modulus also increases with concrete age .

Table (4 -3) :- *Results of the modulus of elasticity (GPa) of concrete with and without RHA content*

Specimen designation	***Modulus of elasticity (GPa) at age of (days)***		
	7	***28***	***60***
SP50	31.7	34	36
SPH1 50	30.5	32	34
SPH2 50	29.6	30.7	33
SP70	37	39	42
SPH1 70	34.5	37.5	39
SPH2 70	33.8	36.2	37.5
Normal	27	28.5	30.5

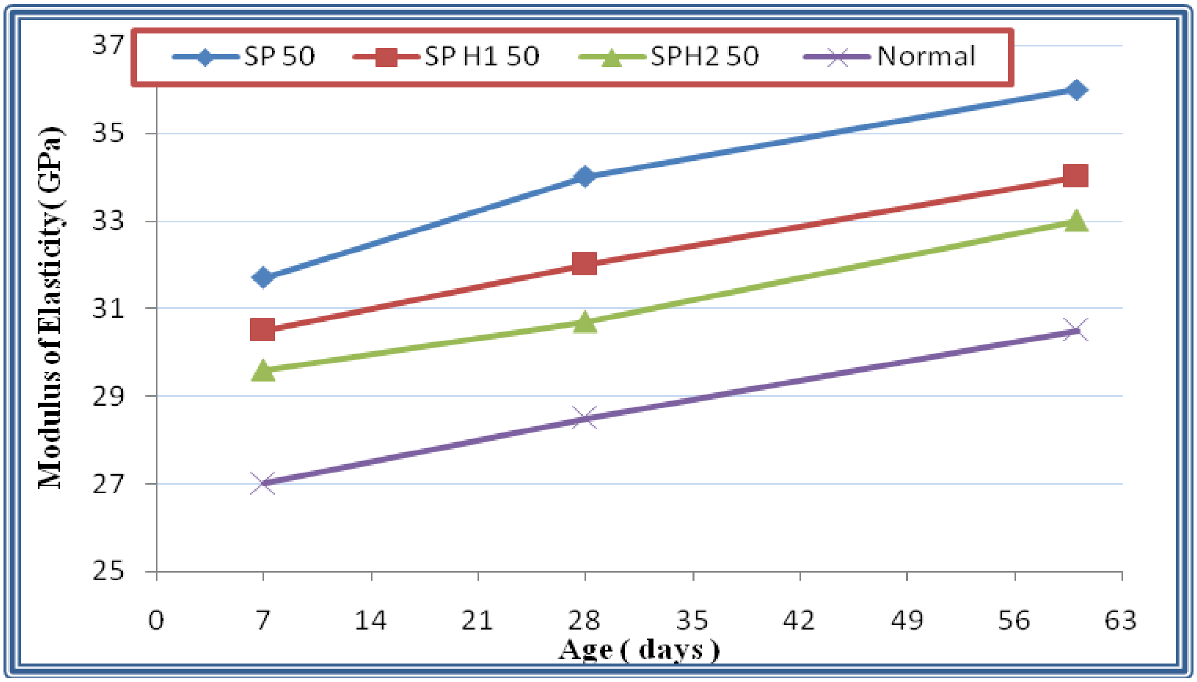


Figure (4 - 10) : Development of Moduluse of elasticity of series 50 MPa with normal concrete

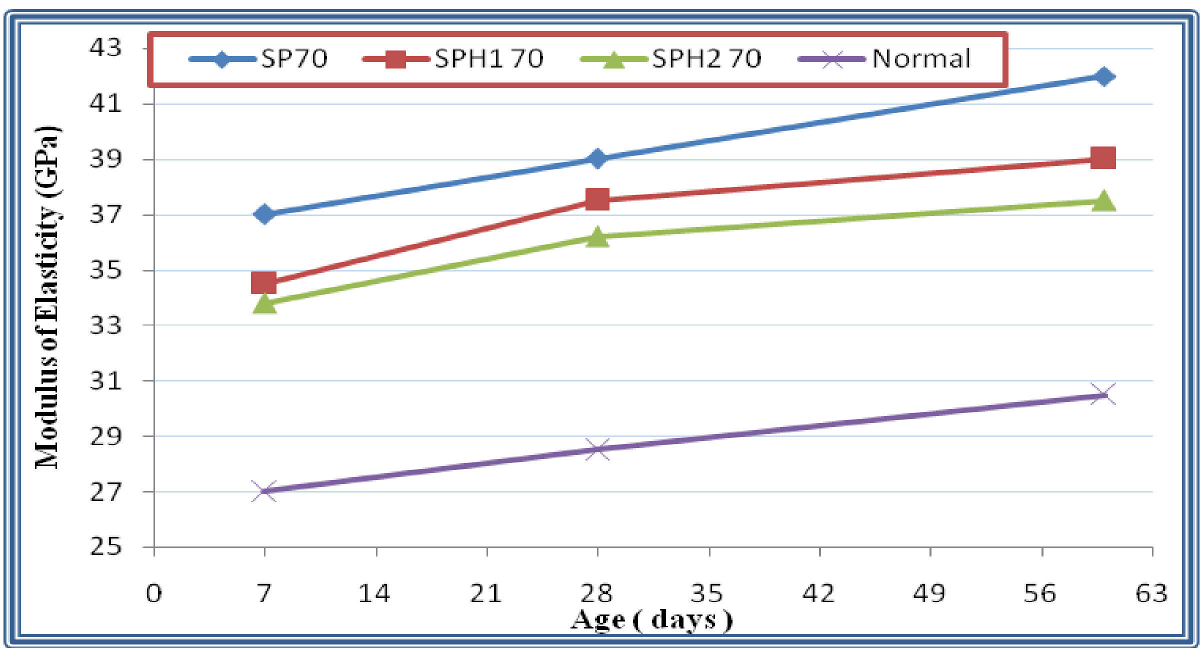


Figure (4 -11) : Development of Modulus of elasticity for series 70 MPa with normal concrete

The relation between the values of E_c of this work and the compressive strength is presented in Figure (4 - 12) .

The equation of the best fit line for the results of this work is :

$E_c = 4.25\sqrt{f_c}$ (GPa) 46 MPa $< f_c <$ 76 (MPa) ....(4 – 3)

While the correlation between the modulus of elasticity E_c and the compressive strength f_c for normal weight concretes was reported by ***(ACI 363- R-23)*** is :

$E_c = 3320\sqrt{f_C} + 6900$ (MPa) 21 MPa $< f_c <$ 83 (MPa) ....(4 – 4)

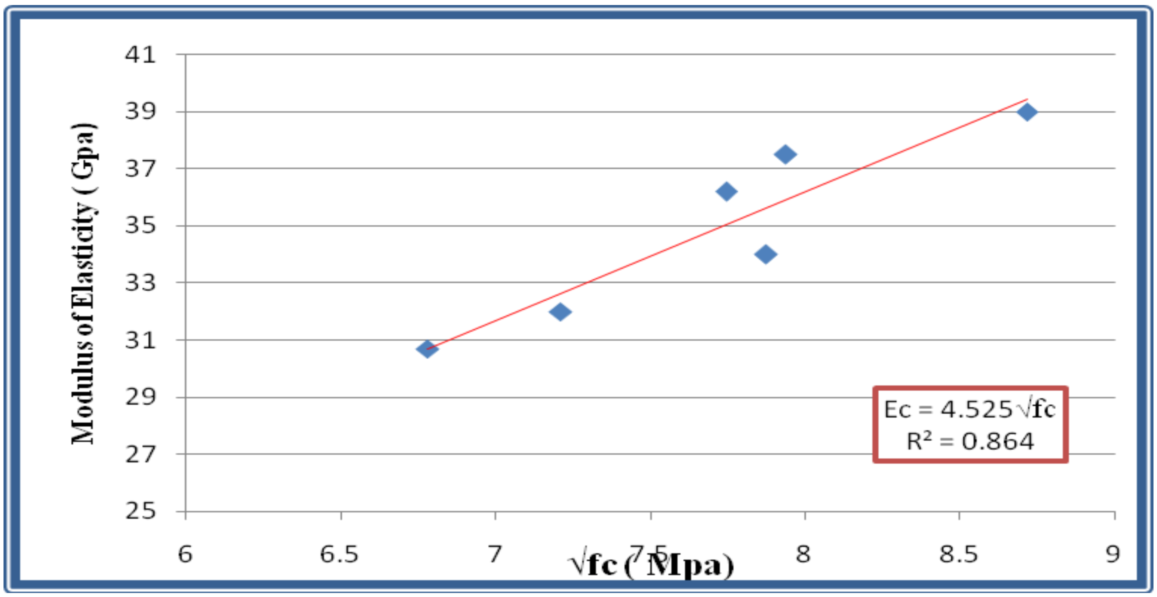


Figure (4- 12) : Relation between modulus of elasticity and square root of compressive strength .

4.6 Free Shrinkage Test

A steel mold described in chapter three (concrete beam with a gap at its middle to ensure free movement) was used for measuring the free shrinkage for all mixes as shown in Table (4 - 4) and plotted in Figures (4 - 13) and

(4 – 14), the expermintal results for HSC and NSC mixes showed that the drying shrinkage increases with the age progress for all mixes but with different rates. Results illustrate that the drying shrinkage of concrete for high strength concrete ranged between (20 - 480) micro strain, (43 - 450) micro strain for series (50, 70) MPa, respectively and (10 - 522) micro strain for normal concrete at ages (3 - 70) days .

It is clear form Figure (4 - 13) and (4 - 14), that the shrinkage of normal concrete at the early age (3 day) lower than the free shrinkage of high strength concrete by about (50 - 71) %, (78 - 85) % for series (50 , 70) MPa respectively, these results can be attributed to accelrating effect of SP on concrete, which changes the size and distrbution of capillary pores due to the effect of acceleration on the hydration of the cement ***(Suchorski and Frang, 2000)***

The drying shrinkage of series (SP50 , SP70) higher than concrete with RHA by about (8 - 43) %, (7 - 34) %, for series (50 , 70) MPa respectively at age (3 - 70) days. The increasing of addtion of RHA up to (7.5) % causing a deficiency in drying shrinkage by about, (12 - 20) %, (9 - 20) %, for series (SPH50, SPH70), respectively at age (3 - 70) days. This results can attrbuted to the lower water to cement ratio altes the hydration rate because it effectively delays the dissolution of the RHA particles lead to reduction in drying shrinkage .

The drying shrinkage for normal concrete at later ages is higher than high strength concrete by about (8 - 25) %, (14 – 28) % for series (50, 70) MPa resprctively, these results are attributed to higher W/c ratio, lower compressive strength compared with high strength concrete.

Table (4 - 4):- Drying shrinkage data

Mix Notation	Date of Crack(day)	Free Shrinkage ($\times 10^{-6}$)at cracking date (1)	Loss of Restrained ($\times 10^{-6}$)at cracking date (2)	Tensile Strain ($\times 10^{-6}$)at Cracking date =(1) - (2) (3)	Elastic tensil Strain ($\times 10^{-6}$)at cracking date (4)	Creep ($\times 10^{-6}$)at cracking date = (3) - (4) (5)	Crack Width (mm)at cracking date
SP 50	11	103	20	83	50	33	0.14
SPH1 50	13	120	22	98	43	55	0.12
SPH2 50	16	142	25	117	36	81	0.1
SP70	30	224	53	171	72	99	0. 2
SPH1 70	37	245	56	189	65	124	0.18
SPH2 70	44	258	58	200	54	146	0.15
SP 85	Not cracked						
Normal	15	97	18	79	29	50	0.08

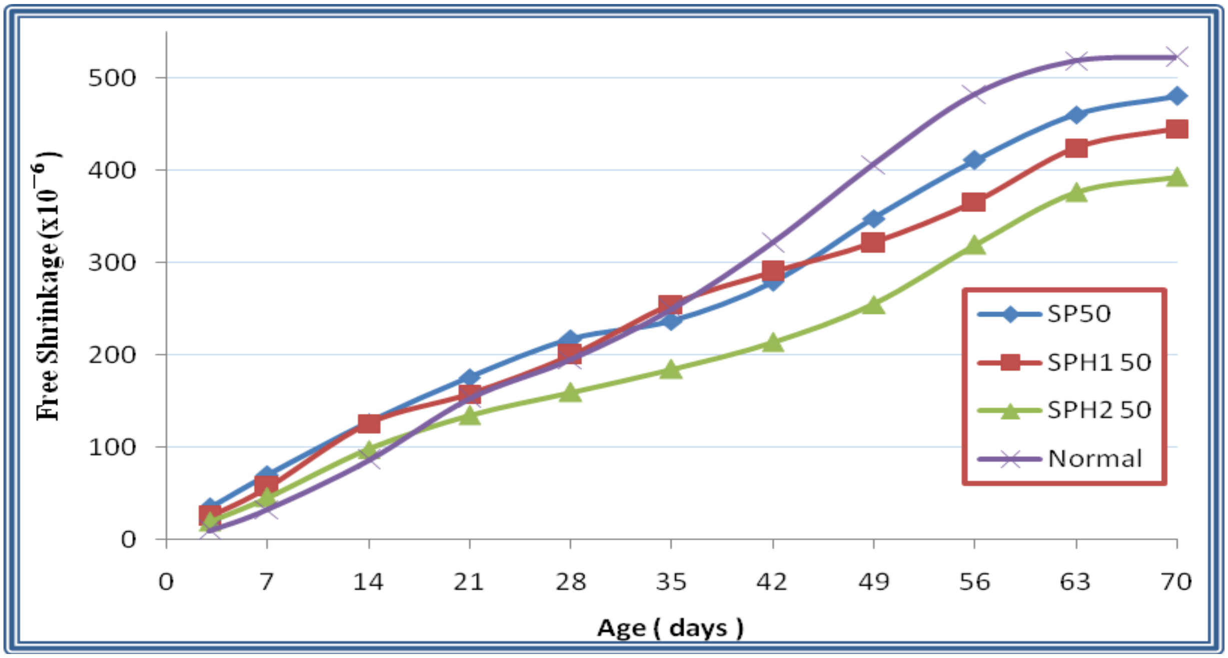


Figure (4 – 13): Development of Free Shrinkage of Series 50 and Normal Concrete

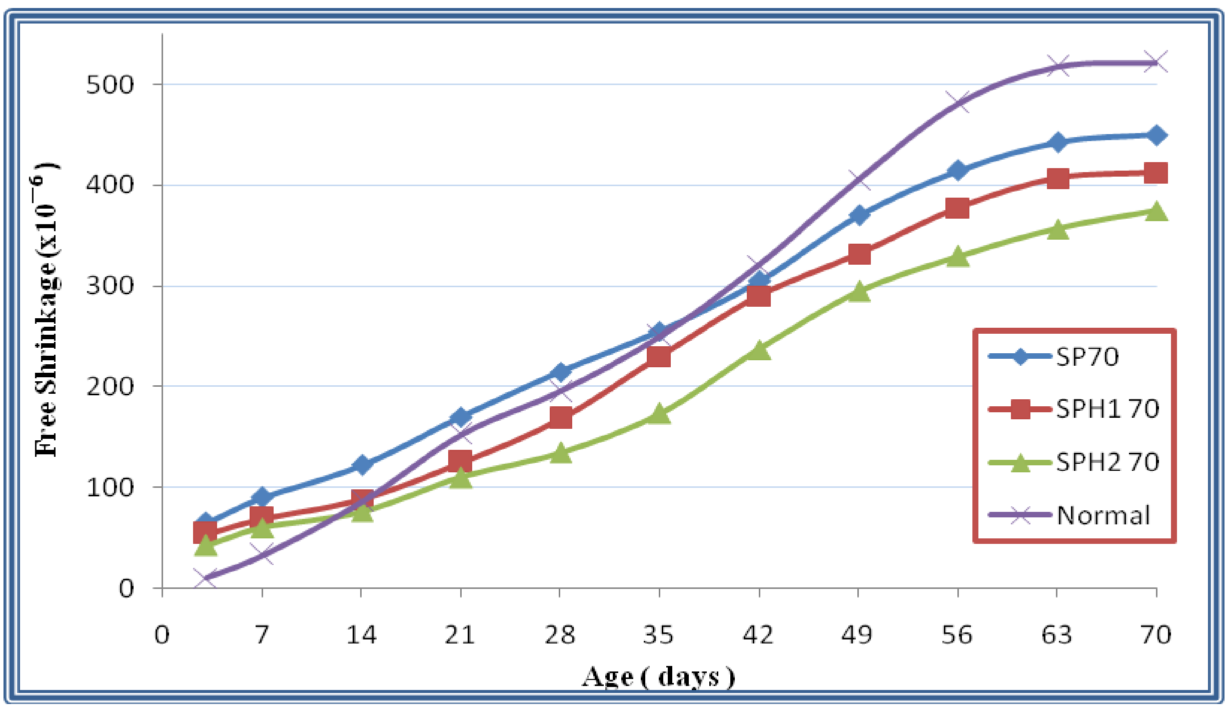


Figure (4 – 14) : Development of Free Shrinkage of Series 70 and Normal Concrete

Mix characteristics that have an effect on shrinkage values such as, water / binder ratio, and the target strength of mixes, togther with shrinkage values at age of 70 days, are listed in Table (4 - 5). From this table it can be concluded that shrinkage is unaffected by an increase in the cementitious materials content, for high strengh concrete mixes investigated having cement content above (540 kg/m^3), but even decrease, because the water / binder ratio is reduced , and the concrete is, therefore, better able to resist shrinkage more over at this low water / binder ratio, the quantity of unhydrated cement particles which restrain the shrinkage is great ***(Mehta, 1975)*** .

Table (4 - 5):- *Mixes properties and free shrinkage at age 70 days*

Mix Notation	***W/b Water / binder***	***Agg. Content by volume (%)***	***Free Shrinkage Microstrain***	***Cementitious Materials (kg/m^3)***	***Target strength (MPa)***
SP 50	0.37	63	480	540	50
SP H$_1$ 50	0.35	62	445	567	50
SP H$_2$ 50	0.34	61	393	580.5	50
SP 70	0.3	64	450	580	70
SP H$_1$ 70	0.28	62	413	609	70
SP H$_2$ 70	0.27	61	375	623.5	70
Normal	0.57	65	522	385	25

4.7 Restrained Shrinkage Test

4.7.1 Elastic Tensile Strain Capacity

Elastic tensile strain capacity of concrte was obtained directly by measuring the immediate movement after cracking of concrete on end restrained beams , the test results are summarized in Table (4 – 4) .

It can concluded from the results that the elastic tensile strain at cracking date is ((50, 43, 36) , (72, 65, 54) and 29 $\times 10^{-6}$) for series (50, 70) MPa and normal concrete respectively. The reducion in elastic tensile strain for series 50 MPa and normal concrete can be attributed to the decrease in cracking time. From the same table it can observed the addtion of RHA to concrete causes a deficiency in elastic tensile strain capacity of concrete at date of cracking . The percentages of these decreasing were about (20 - 28) % for series 50 MPa and (10 - 25) % for series 70 MPa the reason for such behvior could be due to the fact that the quantity of RHA present in the concrete alters the hydration rate effectively reduce shrinage rate.

The elastic tensile strain capacity was also calculated indirectly by determination of splitting tensile strength (ASTM C 496), (as shown in Table (4 – 2) divided by the modulus of elasticity (ASTM C 469), (as shown in Table (4 – 3). The results are illustrated in Table (4 – 7) and plotted in Figures (4 - 15) and (4 - 16). From this Table, it can be noticed that the calculated values of the elastic tensile strain capacity are higher than those measured by the direct method around (50, 60, 63) %, (35, 40, 50) %, and 64 % for series (50, 70) MPa and normal concrete at date of cracking

From the results of this work it was found that the rate of development of tensile strength at ages from 28 and 60 days are around (15 - 17) %, (16 - 19) % and (15) %, elastic modulus enhanced (6 - 8) % , (4 - 8) % and (7) % for series (50, 70) MPa and normal concrete respectively .

This result is not agreed with Al-Rawi who studied behavoier of normal concrete and found elastic tensile strain capacity from direct method greater than that from indirect tensile strain (Al - Rawi, 1985).

Table (4 - 6):- Results of the calculated elastic tensile strain capacity of the concrete mixes with and without RHA (indirect method)

Specimen designation	***Elastic tensile strain capacity ×10^{-6} at age of (days)***		
	7	***28***	***60***
SP 50	100	105.6	116.7
SP H1 50	98	103.1	112
SP H2 50	97.6	101	109.1
SP 70	102.7	110.3	121.4
SP H1 70	98.6	104	118
SP H2 70	97.6	102.2	115
Normal	77.8	87.8	98.4

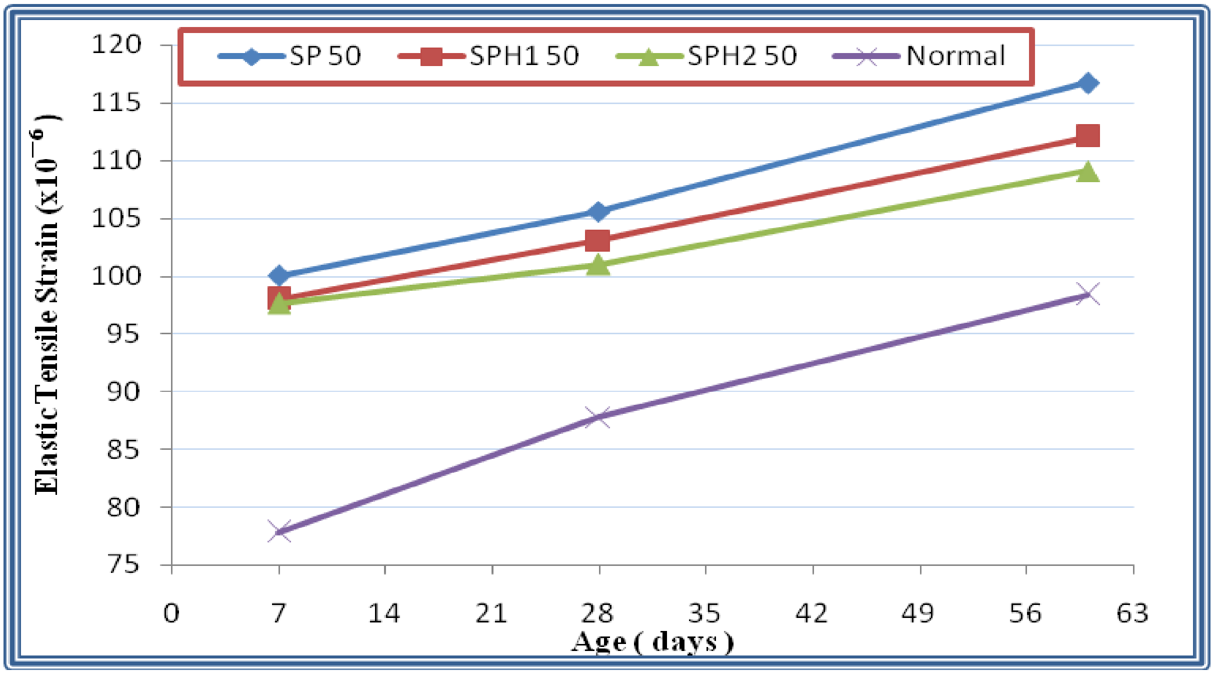


Figure (4 - 15) :Development of elastic tensile strain for series 50 and normal concrete

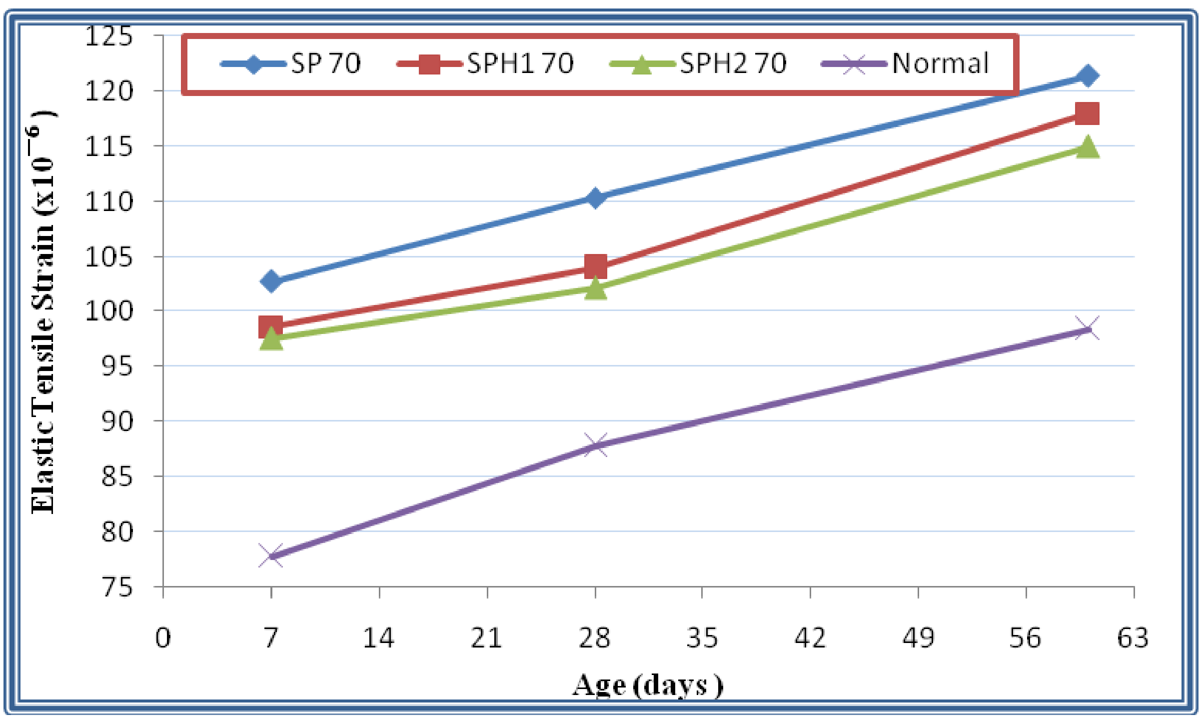


Figure (4 - 16): Development of elastic tensile strain for series 70 MPa and normal concrete

4 .7. 2 Tensile Strain Capacity

The determination of tensile strain capacity was based on the end restrained shrinkage beam, Figures (4 - 17) and (4 – 18) show the values of tensile strain capacity obtained in the present work, from these results it can observed that the tensile strain capacity increase with the increasing in age and time of crack, the results obtained in this work are in agreement with what is found by others ***(AL-Rawi, 1987 and Houk , 1970) .***

Indication to the same Figures and Table (4 - 4) it can be observed that the tensile strain capacity (83, 98, 117) micro strain, (171, 189, 200) micro strian and (79) micro strain , for series (50, 70) MPa , and normal concrete respectively at date of cracking .

The tensile strain for series 70 MPa higher than series 50 MPa and normal concrete by about (51,48, 42)%, (54)% respecively . This resultcan be attributed to the longer time for the occurrence of cracking (30, 37, 44) days for series 70 MPa, compared with 15 day for normal concrete and (11, 13, 16) days for series 50 MPa .

The higher cement content (580 kg/m^3) in series 70 MPa lead to higher tensile strain compared with series 50 MPa and normal concrete, this result is compatible with the study carried out by ***(Carlson, 1979)***, who found , that the tensile strain increased significantly when the cement content increased from (130 to 550) kg / m^3.

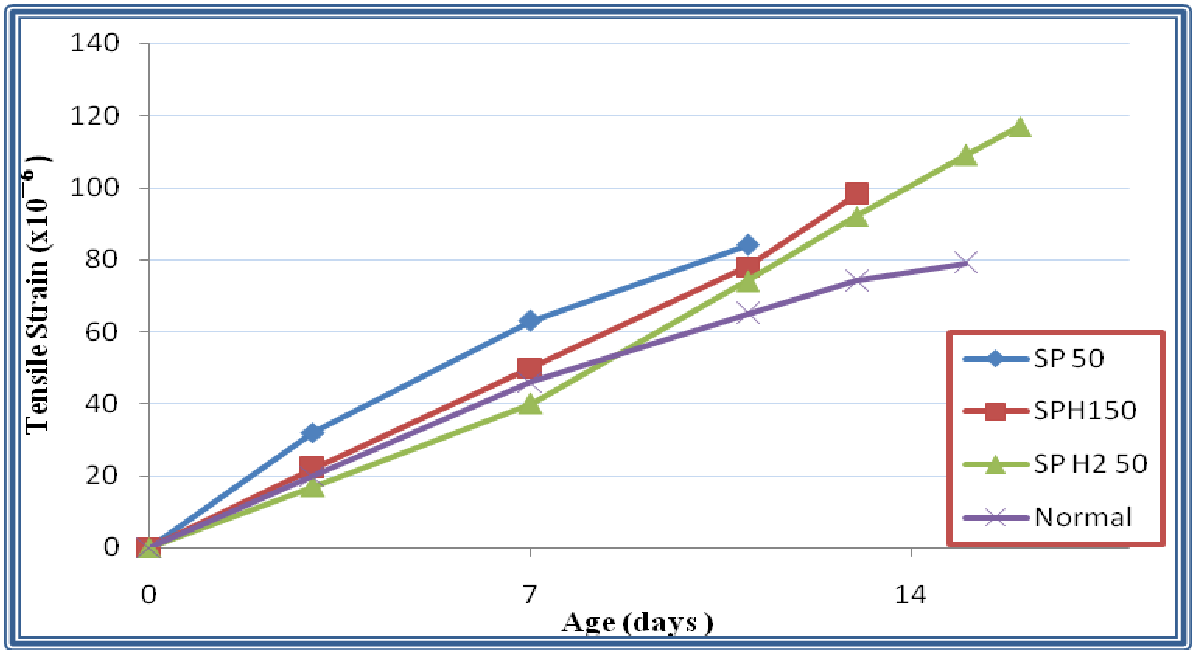


Figure (4 – 17): Development of tensile strain capacity of series 50 MPa & normal concret

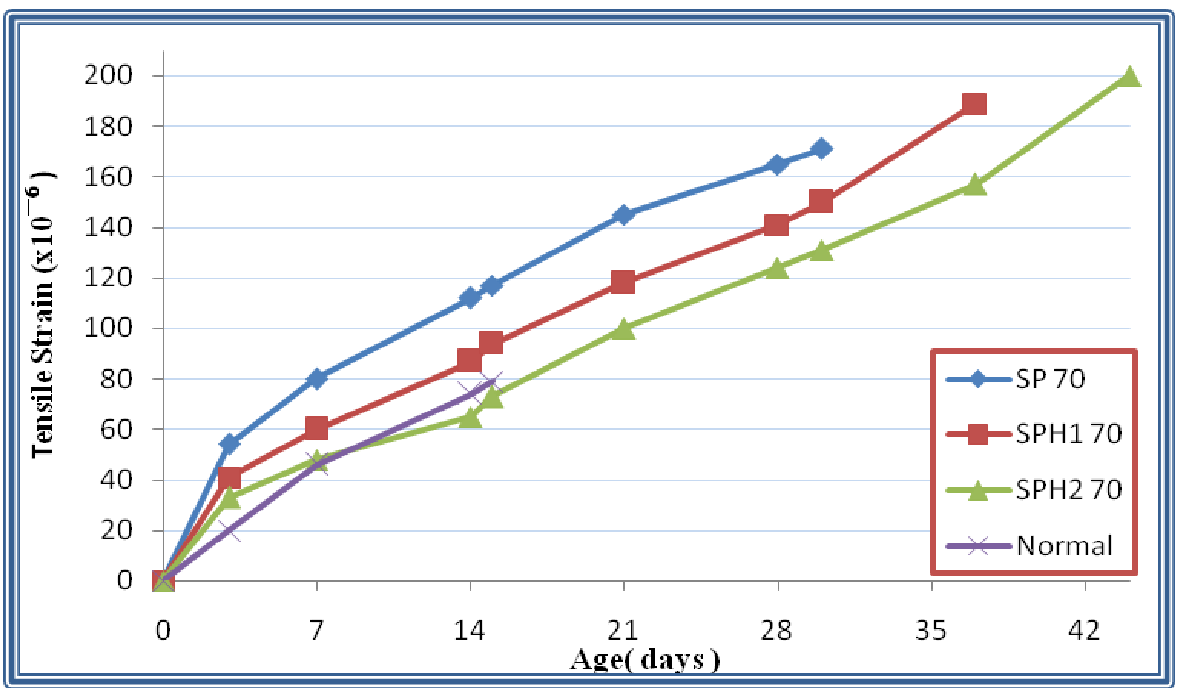


Figure (4 – 18): Development of tensile strain capacity of series 70 MPa & normal concrete

It is clear from Figure (4 – 19) that the addition of RHA increased the tensile strain capacity (with increased the percentage of RHA) by about (18 - 40) % , (11 - 17) % for two series (50, 70) MPa respectively. This may be due to the later shrinkage and later shrinkage cracking .

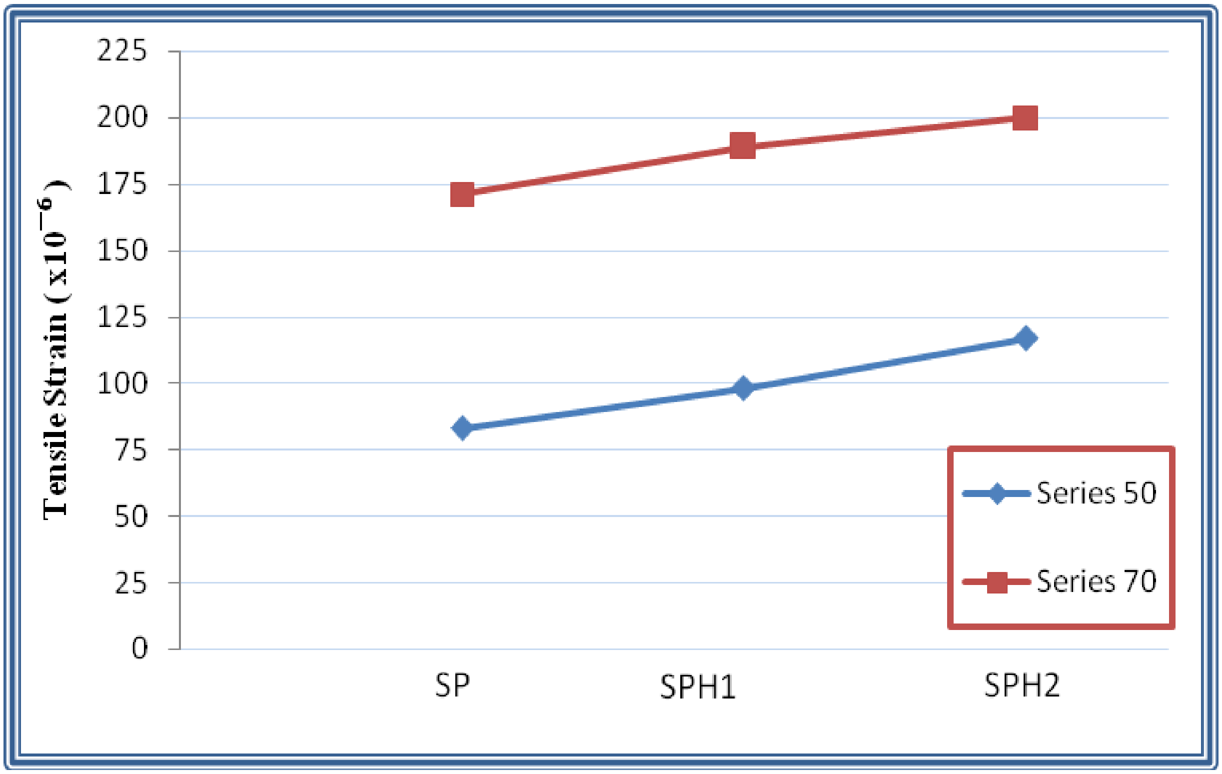


Figure (4- 19) : Effect of Different Percenatges of RHA on the Tensile Strain Capacity (at date of cracking)

4.7.3 Creep

Creep strain of concrete subjected to the restrained shrinkage was calculated as the difference between the tensile strain capacity and the elastic tensile strain capacity. Table (4 – 4) and Figure (4 – 20), showed the relation between the creep strain and the percentage of RHA content in concrete mixes and development of creep strain .

It is clear from results that the creep strain at date of cracking for series 50 MPa (33, 58, 81) micro strain, (99, 124, 146) micro strain for series 70 MPa and (50) micro strain for normal concrete. It is obavious that the creep strain for series 70 MPa is higher than series 50 MPa by about (80 - 200) % (at the cracking time).

The lower creep strain (at cracking) for series 50 MPa, may be attributed to more evaporation which causes early cracks (11, 13, 16) days therfore, the given period for creep is lower than creep period for series 70 MPa .

The creep also increases when addtion RHA content up to 7.5 % by about (40 - 76) % for series 50 MPa, (17 - 25) % for series 70 MPa. This increament can be attributed to the elongation of cracking time or the restrained shrinkage stress with increasing RHA content. Also the increasing in creep strain at cracking time was at different rayes because of the difference in tensile strain capacity values.

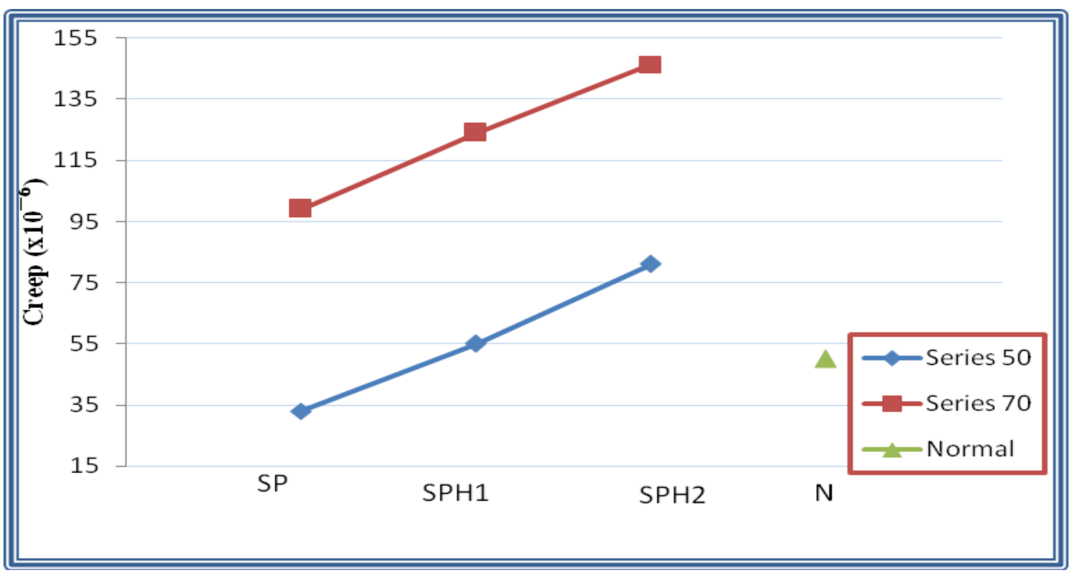


Figure (4 – 20) : Creep strain at date of cracking

4.7.4 Cracking Time

Cracking time is the time required for first crack to occur . From Table (4 – 4) and Figure (4 - 21), it can be observed that the cracking time increases with increasing in compressive strength and the percentage of RHA content ,the cracking time for series 50 MPa (11, 13, 16) and (30, 37 , 44) for series 70MPa , this may related to the fact that the addtions of RHA in two percntages (5 , 7.5) % would cause a reduction in W/b ratio by about (5, 8) %, (7, 11) %, increament in cracking time by (18, 46) %, (23, 47) % for series (50, 70) MPa respectvily, and a decreased in the drying shrinkage, thus the bossibillity of craking at later ages will increase, the time required for cracking will be longer. This opinion is compatible with series 85 MPa , till 56 days had no cracks . This may be due to very higher compressive strength (102) MPa at 60 days, high cement content (740 kg/m^3), high coarse aggregate content (1100 kg/m^3) and very low W/b ratio (0.22), which lead to lower shrinkage and no cracks

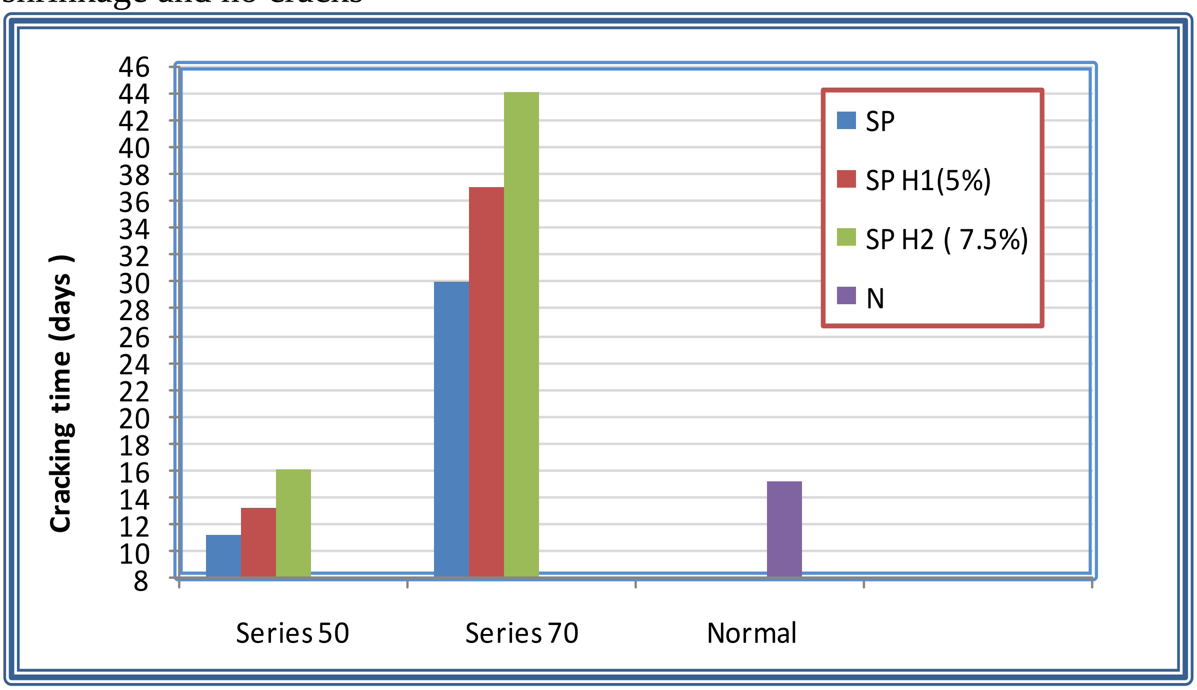


Figure (4 - 21): Effect of different RHA content on the cracking time of concrete

4.7.5 Crack Location

The results showed that the crack occurred within the narrower at the middle of the web. This mean that the tensile strain at the middle is higher than that at the sides of beam, this behavior can be attributed to narrower in this area of specimen lead to increasing in the stresses . Cracks occurred at a distance from the center of the web ranged between (5 - 21) mm.

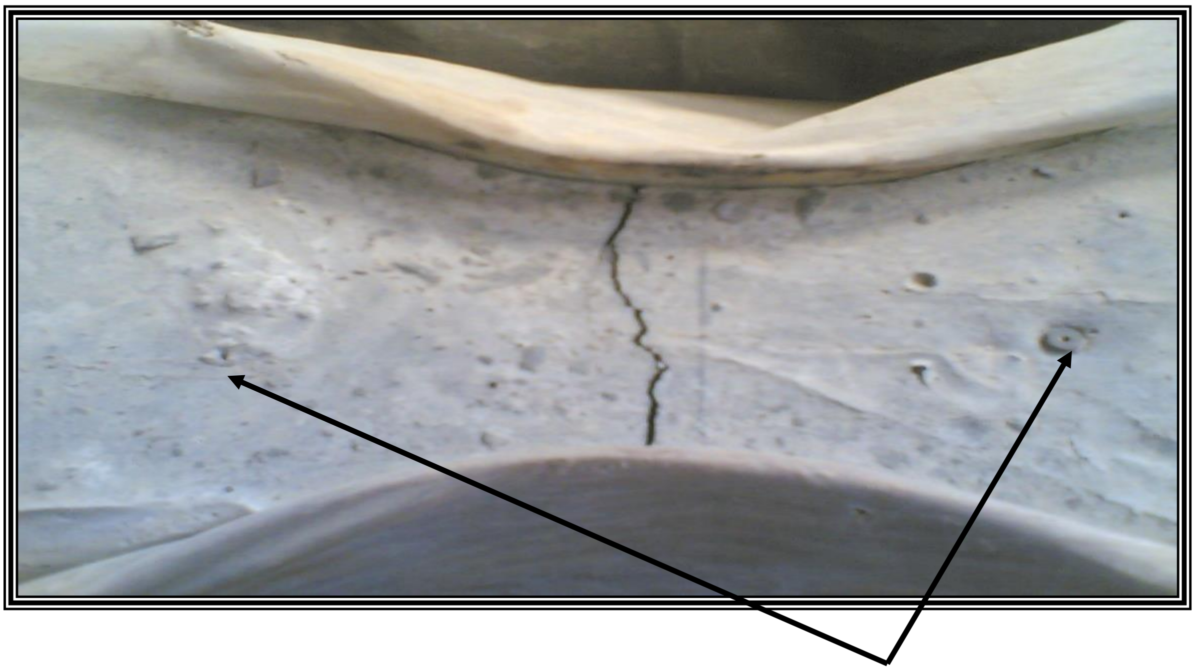


Plate (4 – 1): Crack location Demecs point

4.7.6 Crack Development

The results of crack width for NSC and HSC are plotted in Figure (4 - 22), (4 - 23). From this Figure, it can be seen that the crack width development of series (50 , 70) MPa with superplasticizer higher than concrete with superplasticizer and RHA, this may be due to the high acceleration the cement hydration, high drying shrinkage, high compressive strength. This finding was confirmed by other researches' ***(Kanastad et-al , 2000) .***

It is obvious , that the rate of crack widening is high at early ages and decreases at later ages, and consequently less crack widening would be experienced in this case, and also the rate of shrinkage reduced at the later ages, from the same Figures, it can be seen that the development of crack width of normal concrete is less than HSC for series (50, 70) MPa, this may be due to the presece of high quantity of cementitious materials .

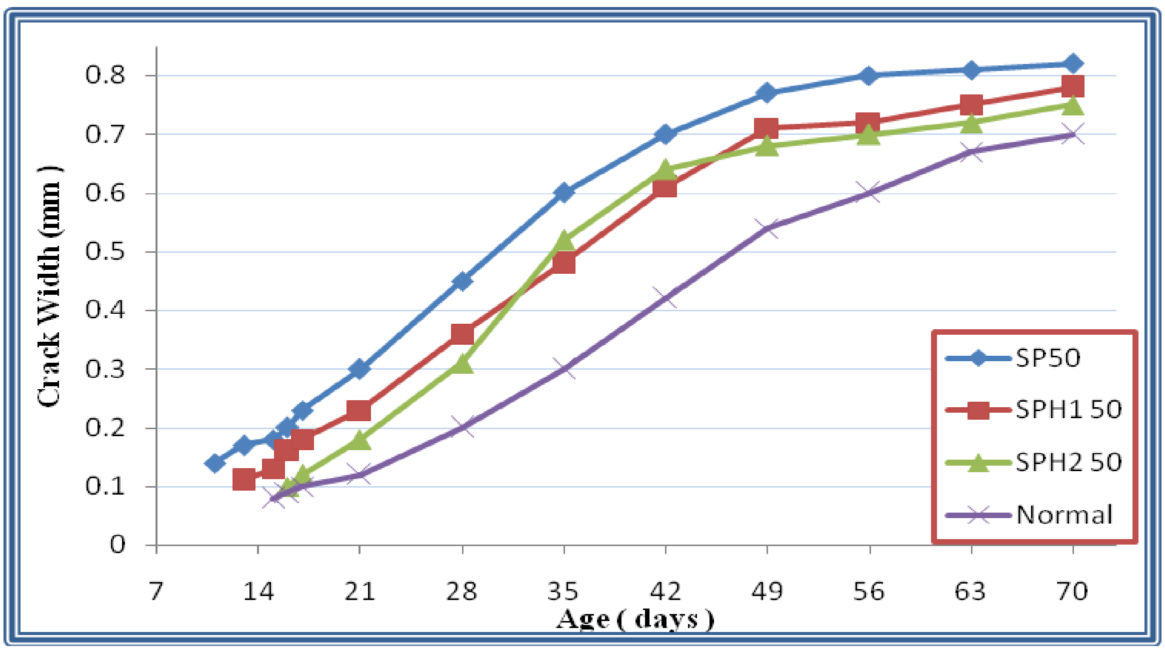


Figure (4 – 22) : Effect of drying period on crack development for series 50MPa and normal concrete .

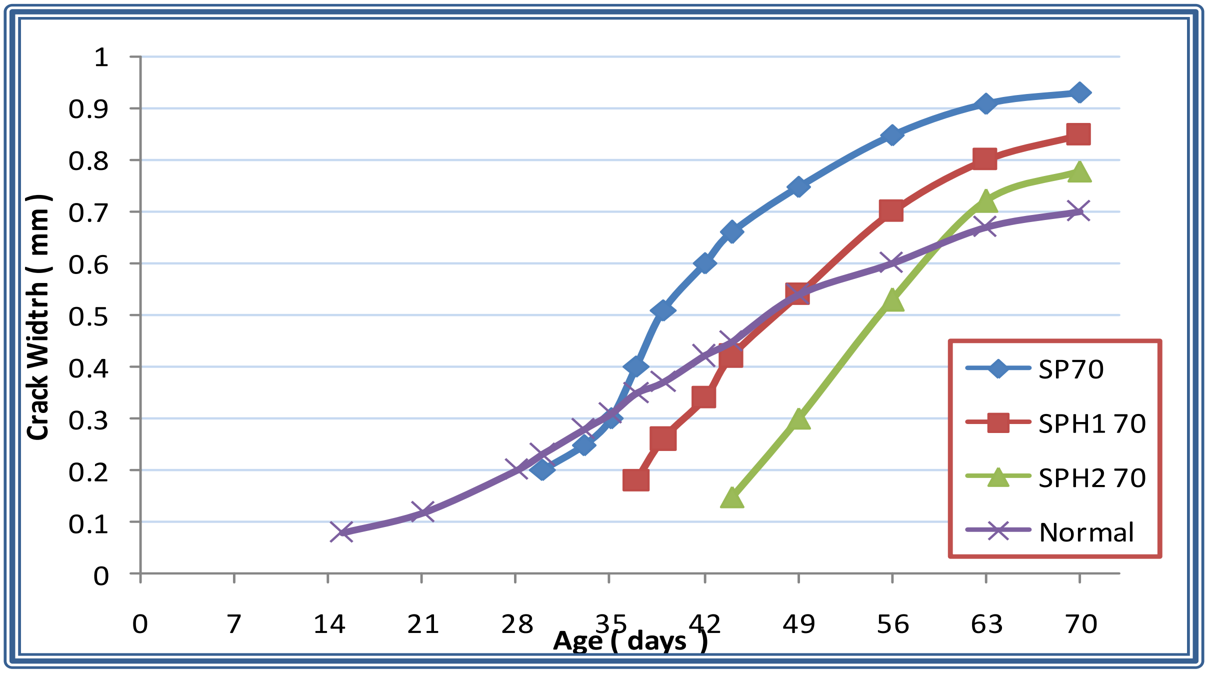


Figure (4 – 23) : Effect of drying period on crack development for series 70 MPa and normal concrete

4.8 The Statistica Regression Models

4.8.1 Models Development for Prediction of Free Drying Shrinkage

Multipile nonlinear regression analysis was applied to build the present model. The regression anlysis was made with the aid of computer software (STATISTICA 6., 2001) .

The various elements of multiple nonlinear regression equation can be illustrated from the general form of the follwing eq :-

$$Y = a_0 + a_1 x_1^{b_1} + a_2 x_2^{b_2} + a_3 x_3^{b_3} \ldots\ldots\ldots a_n x_n^{b_n} \text{ - - - - - - - - - - -} (4-1)$$

Where : -

y: is the predicted value of the dependent variable .

x_1, x_2, x_3 ,.............................., x_n , independent variiables.

a_0 : is the intercept coefficient , or the value of the depentdent variable when the independent varibles are zero .

a_1 , a_2 , a_3,...a_n
b_1 , b_2 ,b_3,…b_n
are the partial regression coefficient of independent varibles

n : is the number of independent variables included in regression equation .

In this study, in order to yield the best form for predicting drying shrinkage of concrete, many forms which make the final form of the proposed models for the drying shrinkage prediction are :

1-In the first model , two main factors are useds which are :

t : time

W/b : water content (kg/m^3) / cementitious materials (kg/m^3).

$$\varepsilon_{sh} = (a_0 + a_1 t^{a_2} + a_3 W/b^{a_4}) \times 10^{-6} \text{ — — —} (4-2)$$

2- The coarse aggreagate content is added in the second model :

C_a : coarse aggregate (kg/m^3) .

$$\varepsilon_{sh} = (a_0 + a_1 t^{a_2} + a_3 W/b^{a_4} + a_5 C_a{}^{a_6}) \times 10^{-6} \quad (4-3)$$

3- The Compressive strength and relative humidity are introduced in the third model

f_c : the mean compressive strength at day (MPa) for cube .

RH : relative humidity (%) .

$$\varepsilon_{sh} = (a_0 + a_1 t^{a_2} + a_3 W/b^{a_4} + a_5 C_a{}^{a_6} + a_7 f_c{}^{a_8} + a_9 \, RH^{a_{10}} \times 10^{-6} \quad (4-4)$$

Many attempts have been made to produce the best form of such statistical model for predicting the free drying shrinkage. These attempts include suggesting other equation forms .When suggesting these eqs. it was found that there is no significant improvement in correlation coefficient (R) of the proposed model, it is well – known fact in model building that the “ bigger and more complicated model “ does not necessarily mean better model . In the actual practice of building models for specfic purposes , the best advice is to keep them simple as far as possible Abdul-latif stated that – as reported by ***(Abdul – Abass , 2007)***.

Table (4 - 8) gives the regression coefficients of the prediction models above for the prediction of the drying shrinkage at any age from seven to seventy days, from the same Table, the correlation coefficient increases and the standard error decreases with increasing the number of factors used in the proposed model .

Table(4- 7) : Regression Coefficients for The Drying Shrinkage Prediction Model

Variable	***Cofficient***	***Model 1***	***Model 2***	***Model 3***
	a_0	3594.813	-17604.8	-5202.78
t	a_1	-3784.47	-1602.63	861.0485
	a_2	-0.038418	-0.104346	0.10
W/b	a_3	-0.000715	88.49376	547.9314
	a_4	-8.49258	410.3843	1.363211
C_a	a_5		9516.331	1609.739
	a_6		0.1	0.1
f_c	a_7			590.7020
	a_8			0.1
RH%	a_9			0.339272
	a_{10}			-4.12184
Correlation Coefficient (R)		0.9697	0.9866	0.9920
Variance Explained (R^2)		0.9457	0.96715	0.9823
Standrder Error (S .E)		33.385	23.64	19.63

The first model results have agood correlation coefficient (0.9697), when introducing coarse aggregate and compressive strength in second model increasing the correlation coefficient to (0.9866) . correlation coefficient increasing to (0.9920) in third model .

The increase of correlation coefficient and decrease in standard error from model (1) to (3) , proves the right choice of the vaiables and their effect on drying shrinkage of concrete. Also it can be seen from Figures (4 –24) and (4 – 25) that the residuals in the first plot are normally distributed, the second plot yields points close to the (45)° line, then the proposed regression function gives accurate prediction of the values that actually observed, this indicates that the models fit the data well.

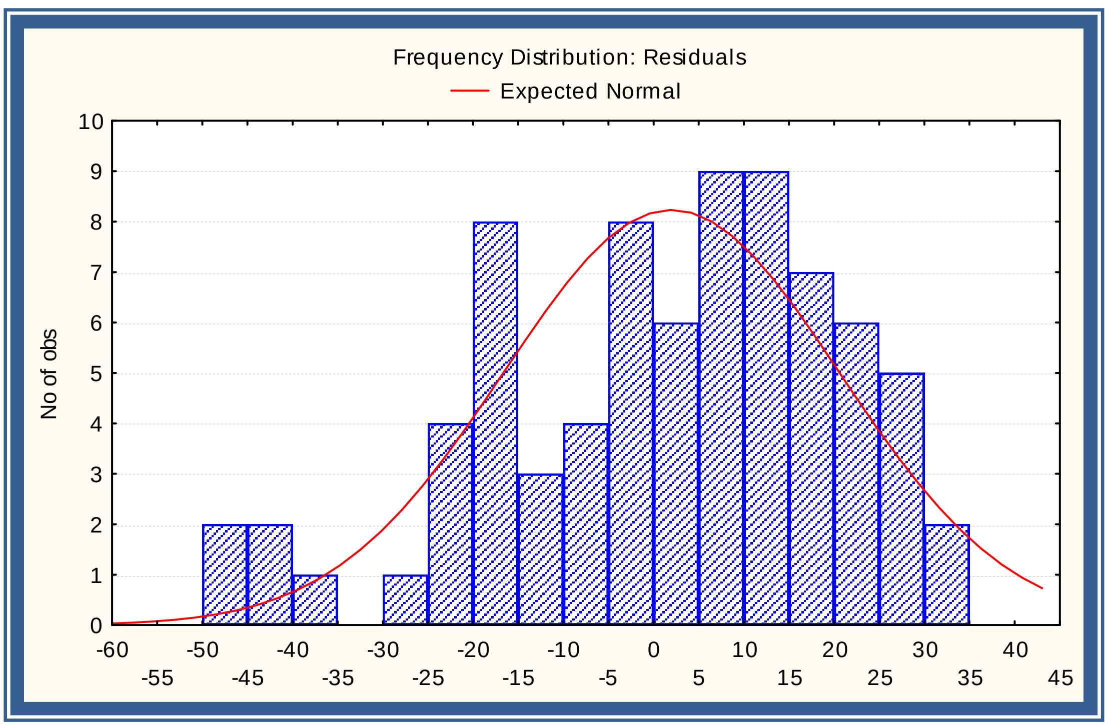


Figure (4 – 24) :Residual Values of Drying Shrinkage for model (3)

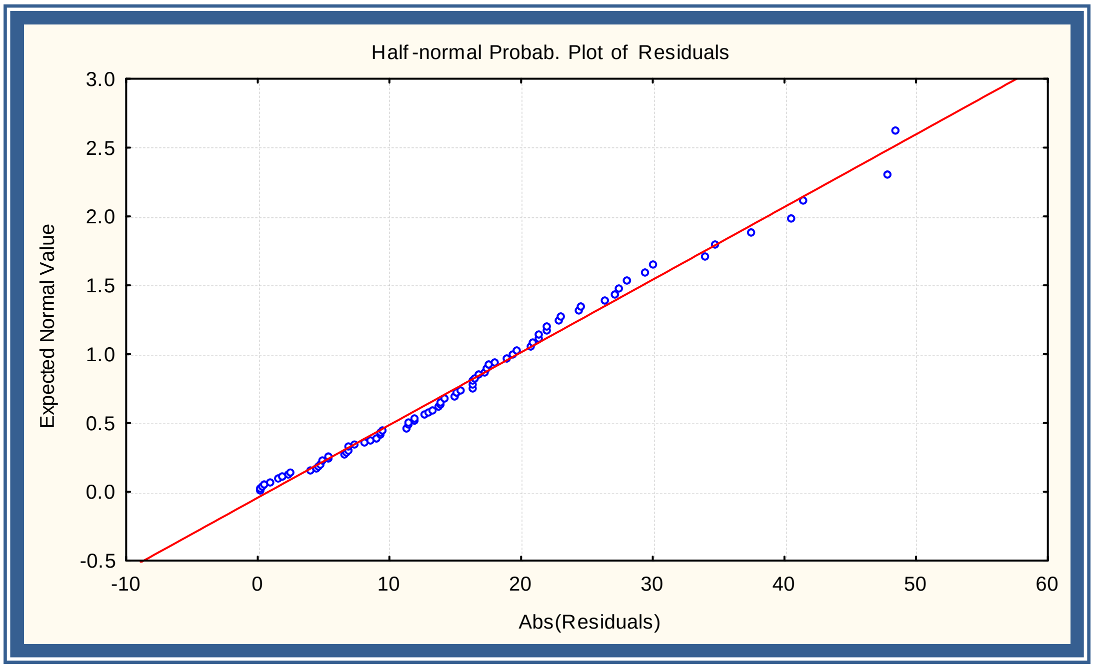


Figure (4-25) :Cumulative Probability Plot of Residuals for model (3)

For example, for specimens with compressive strength equal to 88MPa , coarse aggregate 1015 kg/m^3, fine aggregate 661 kg/m^3,w/b ratio 0.3and relative humidity 42%, the shrinkage strains predicted at (60 days) by the above three models compared with ACI model are given in table (4- 8) .

Table (4 -8) : Compared between value of three models and ACI model

Models	**Model (1)**	**Model (2)**	**Model (3)**	**Experimental**	**ACI Model**
Shrinkage Strain ($\times 10^{-6}$)	341.5	365	353	440	314

4.8.2 Models Development for Prediction of Crack Width:

The equation used to calculated crack width developed by ***(Kadhum 2003)*** was adopted by considering the strain distibution and using non linear analysis .

$$\mathbf{W_C max = Smax[\varepsilon_{sh}(1-k) - \frac{e_{tsc}}{2}] ------- (4-5)}$$

The equation was adopted and modified by ***(Kubb, 2007)*** as follows :

$$\mathbf{W_C max = Smax[a \times \varepsilon_{sh}(1-k) - b \times \frac{e_{tsc}}{2}] ------- (4-6)}$$

In this work the above equation can be adopted and modified as follows:

$$\mathbf{W_C max = [a \times \varepsilon_{sh}^{(a+b)} - c \times (\frac{e_{tsc}}{2})^{(c+d)}] ------- (4-7)}$$

Where:

ϵ_{sh} : free shrinkage strain (micro strain).

e_{tsc} : elastic tensile strain capacity(micro strain) , and a_1, a_2, a_3, a_4 constants (estimated statically).

a = 0.098522, b = 0.341210, c = 0.757562, d = -0.780125, with Coefficient of Correlation (R = 0.87), S.E = 0.233

It is clear from Figures (4 - 26) and (4 - 27) that the residuals in the first plot are normally distributed, and the second plot yields points close to the (45)° line, then the proposed regression function gives accurate prediction of the values that actually observed, this indicates that the models fit the data well.

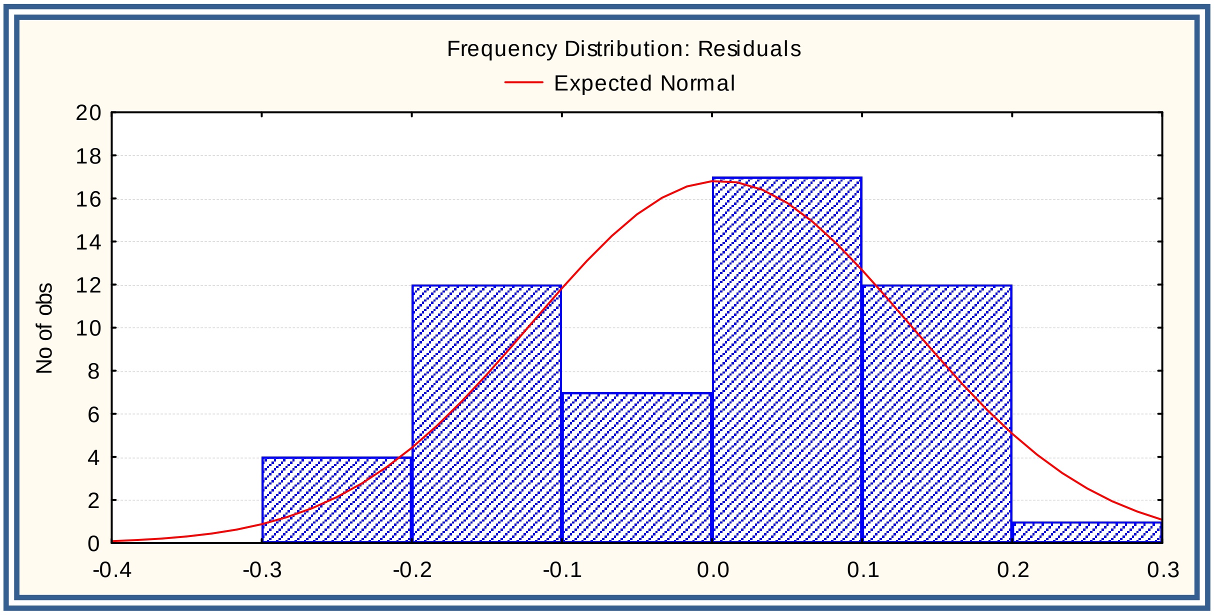


Figure (4- 26): Residual Values of crack width for (crack width model)

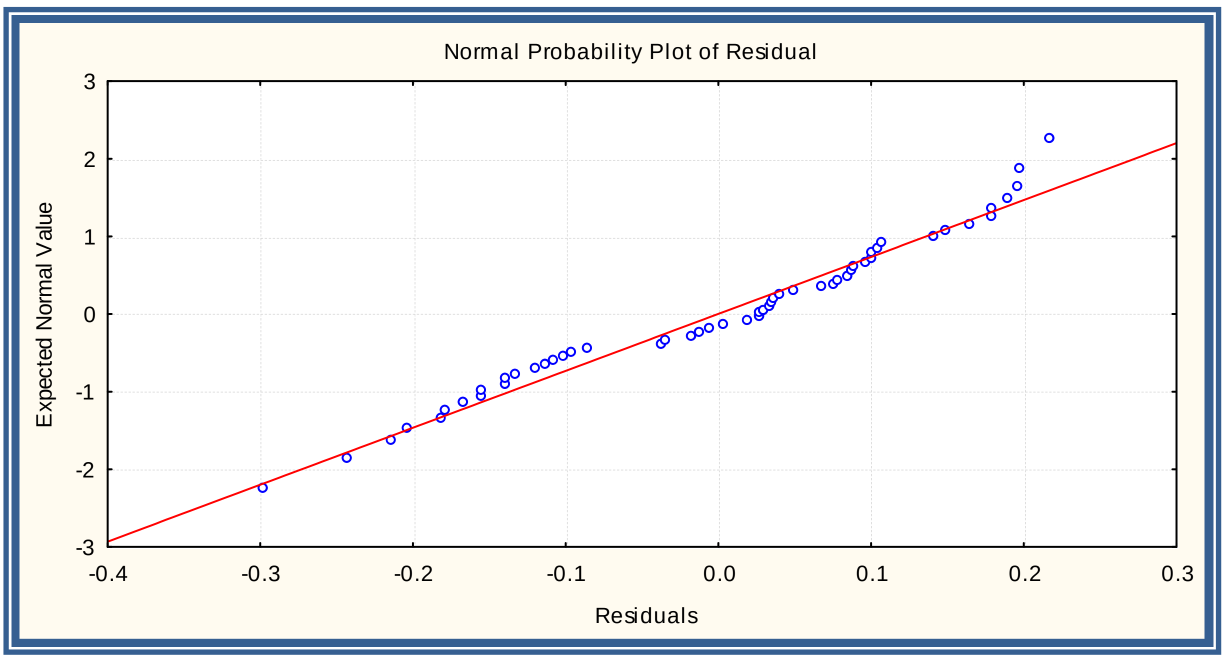


Figure (4 - 27): Cumulative Probability Plot of Residuals (crack width model)

CHAPTER FIVE

5

CONCLUSIONS AND RECOMMENDATION S

5.1 Conclusions

Based on the experimental results presented in the preceding chapters and on the basis of the observations made in the present work , the following conclusions were found :

1- The compressive strength of series SP is higher than series SPH by about (27- 34) % . The addition of RHA to concrete in two percentages causes a reduction in compressive strength at early and later ages with different percentages .

2- The normal strength concrete and series 50 MPa experienced a major increase in compressive strength between 28 and 60 days by about (24 – 36) % .

3- The influence of RHA on splitting tensile strength and static modulus of elasticity was found to be agree with the behavior of compressive strength , new equations relating between compressive strength and splitting tensile strength were developed **:**

$$f_{sp} = 0.676\sqrt{f_c} - 1.596 \qquad 46\ MPa < f_c < 76\ (MPa)$$

another one developed between compressive strength and modulus of elasticity

$E_c = 4.25\sqrt{f_c}$ *(GPa)* $\quad\quad 46\ MPa < f_c < 76$ *(MPa)*

4- The free shrinkage of normal concrete is lower than that of high strength concrete at early ages by about (100- 330) %, while normal strength concrete higher than high strength concrete by about (8 - 28) % at later ages .

5- Free shrinkage of series 70 MPa is higher than series 50 MPa around (53 - 80) % at early ages.

6- The free shrinkage strain at cracking time increases with increasing in RHA content, addition RHA causing a deficiency in drying shrinkage around (9- 20) % .

7- The elastic tensile strain capacity at date of cracking decreases with increasing in RHA content about (22 - 47) %, elastic tensile strain calculated by indirect method (splitting tensile strength divided by the modulus of elasticity) is higher than these of direct method (elastic tensile strain capacity of concrte was obtained directly by measuring the immediate movement after cracking of concrete on end restrained beams) by about (31 - 65) % .

8- Tensile strain capacity for series 70MPa is higher than of series 50 MPa about (42 – 51) % .

9- Creep strain at date of cracking for series 70 MPa is higher than series 50 MPa around (80 – 200) %, the creep increases with increasing in RHA content up to 7.5 % about (18 – 37) % .

10- Cracking time increasing with increase in compressive strength and increasing in RHA content, crack time of series 50 MPa (11, 13, 16), while (30, 37, 44) days for series 70 MPa, with higher compressive strength series 85 MPa not cracked.

11- The crack location of normal and high strength concrete occurred within the narrower at the middle of the web, at distance from the center of the web range between (5 - 21) mm .

12- Crack development of concrete with superplasticizer is higher than concrete with RHA, rate of widening is high at early ages and decreases at later ages, crack width of normal concrete is less than high strength concrete .

13- New models for predicting (ε_{sh}) were developed, using non linear estimation, these models proved to yield excellent predicted values, with correlation coefficient (R) = 0.992, variance explained (R^2) = 0.9823.

$$\varepsilon_{sh} = (a_0 + a_1 t^{(a_1 - a_2)} + a_3 W/b^{(a_3 - a_4)} + a_5 C_a^{(a_5 - a_6)} + a_7 f_c^{(a_7 - a_8)} + a_9 RH^{(a_9 - a_{10})}) \times 10^{-6}$$

a0=-5202.78, a1= 861.0485, a2= 0.1, a3= 547.9314, a4= 1.363211,a5=1609.739, a6=0.1, a7= 590.7020, a8=0.1, a9= 0.339272, a10= -4.12184

14- Formulas for calculating width of crack adopted and modified . maximum crack width (W_{max}) can be calculated from the following equation :

$$W_c max = [a \times \varepsilon_{sh}^{(a+b)} - c \times (\frac{e_{tsc}}{2})^{(c+d)}]$$

a = 0.098522, b = 0.341210, c = 0.757562, d = -0.780125, with Coefficient of Correlation (R = 0.87), S.E = 0.233

15- Addition (1.25 %) of SP gave good water reduction (27 %) .

16- There is no problem from creep and shrinkage for high strength concrete .

5.2 Recommendations for the future Works :

1- Studying the effect of early curing period days on drying shrinkage of high strength

2- Comparing between the effect of two supplementary materials (Rice Husk Ash , Silica Fume) on drying shrinkage .

3- Studying the effect of other maximum size and grading of coarse aggregate on produced high strength concrete and shrinkage cracking .

4- Studying the drying shrinkage cracking of base restrained concrete .

5- To maintain the workability of mixes , increasing the (Superplasticizer) addition dose or increasing water content with increasing Rice Husk Ash content .

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