

Ministry of Higher  
Education and  
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University of Babylon  
College of Engineering  
Civil Department

# DRYING SHRINKAGE OF HIGH STRENGTH CONCRETE WALLS WITH BASE RESTRAINTS

*A Thesis*

Submitted to The College of Engineering  
of The University of Babylon in Partial  
Fulfillment of The Requirements  
for The Degree of Master  
of Science in Civil  
Engineering

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# الانكماش للجدران ذات القواعد المقيّدة المصنوعة من خرسانة عالية المقاومة

## رسالة

مقدمة إلى كلية الهندسة في جامعة بابل

كجزء من متطلبات تخرج الماجستير في علوم

الهندسة المدنية

من قبل

محمد حسام محمد الطائي

إشراف

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بِسْمِ اللَّهِ الرَّحْمَنِ الرَّحِيمِ

وَالسَّحَابِ مِثْقَالًا وَأَنَّا لَمُوسِعُونَ  وَالْأَرْضِ

فَرَشْنَاهَا فَنِعْمَ الْمَاهِدُونَ 

صَدَقَ اللَّهُ الْعَلِيِّ الْعَظِيمِ

سورة الذاريات

(آية ٤٧-٤٨)

# DEDICATION

*To those who are always in  
my heart :*

*My supervisors*

*My parents*

*My brothers*

*My wife , son and daughter*

*My friends*

*Mohammed J. Mohammed*

## الخلاصة

في هذا البحث تم استخدام نماذج بحث مصغرة يعتقد بأنها توفر ظروفًا مشابهة للظروف الطبيعية الموقعية لدراسة سلوك الانكماش للجدران المصنوعة من خرسانة عالية المقاومة وعلى وجبتين: شتوية بعدد ثمانية جدران لنسب مختلفة من الطول/الارتفاع (٦,٤,٢,١) بوجود وعدم وجود حديد التسليح (٠,٢٪ أفقي و ١٪ عمودي) والوجبة الصيفية تحوي على جدارين مسلحين (٠,٢٪ أفقي و ١٪ عمودي) وبنسبتين من الطول/الارتفاع (٦,٤). تم تقييد تلك الجدران بواسطة أساس حديدي للتخلص من تأثير الانكماش الذي يحدث في الأساس الخرساني.

الخواص الميكانيكية للخرسانة التي تضمنها البحث ، مقاومة الانضغاط ، مقاومة الشد ( الانشطار ) ، مقاومة الشد (الانحناء) ، معامل المرونة ، تراوحت القيم بالنسبة للخرسانة عالية المقاومة ، مقاومة الانضغاط ( ٩٠-٥٠ ) ميكاباسكال ، مقاومة الشد بالانشطار ( ٥,٥ - ٧ ) ميكاباسكال ، مقاومة الشد بالكسر ( ٦ - ٨,٦ ) ميكاباسكال ، ومعامل المرونة الاستاتيكي ( ٣٢-٤٤ ) كيكاباسكال للأعمار من (٧-٩٠ يوم).

لقد تبين من خلال المراقبة إن جميع هذه الجدران المسلحة وغير المسلحة ولهذه النسب من الطول/الارتفاع (٦,٤,٢,١) لا تبدي تشققًا خلال فترة تعرض للجفاف قدرها (٩٠) يوم. وكانت قيم الانكماش غالبًا ما تزداد كلما اتجهنا نحو نهايات الجدار الجانبية والعليا.

كانت نسب الزيادة في القيم العظمى للانكماش في الصيف بحوالي (٧٣٪ ، ٦١٪) من القيمة العظمى للانكماش في الشتاء عند عمر ٩٠ يوم لنسب الطول إلى الارتفاع (٤ ، ٦) على التوالي. كما أدى استخدام حديد التسليح الأفقي بنسبة ٠,٢٪ إلى نقصان في قيم الانكماش العظمى بزيادة نسبة الطول/الارتفاع حيث كانت نسب النقصان حوالي (١٢٪ ، ١٤٪ ، ٢٠٪ ، ٢٨٪) عن قيم الانكماش العظمى للجدران الغير مسلحة ولنسب الطول/الارتفاع (٦,٤,٢,١) على التوالي.

وقد بينت النتائج المختبرية للعتبات الخرسانية المقيدة من النهايتين إن الانكماش الحر في وقت حدوث الشق أعلى في فصل الصيف مما هو عليه في الشتاء بحوالي (٢٧٪)، وانفعال الشد الأقصى في الصيف أعلى مما هو عليه في الشتاء بحوالي (٣٠٪) وانفعال الشد المرن في الصيف أعلى مما هو عليه في الشتاء بحوالي (٤٤٪). بالإضافة إلى ذلك عند مقارنة نتائج الانكماش هذه مع نتائج الانكماش للخرسانة العادية في البحوث السابقة نجد إن الخرسانة العالية المقاومة أقل انكماشًا من الخرسانة العادية.

وأخيرا تم اقتراح نموذج رياضي لتخمين القيمة العظمى للانكماش خلال فترة جفاف قدرها ٩٠ يوم والذي يحدث في الجدران ذات الخرسانة العالية المقاومة, ويعتمد النموذج الرياضي المعد بواسطة برنامج إحصائي يدعى (DATA FIT 9.0.59, 2008) على نسبة الطول/الارتفاع, نسبة حديد التسليح الأفقي, نسبة حديد التسليح العمودي, معدل درجة الحرارة ومعدل الرطوبة النسبية, وكانت قيمة معامل الارتباط (٠,٩٧٢٤) وقيمة الخطأ القياسي (٦٢,٢٤٧).

# *A*BSTRACT

In this research, reduced scale wall models (which are believed to resemble as much as possible field conditions) were used to study the shrinkage behavior of high strength concrete(HSC) base restrained walls. Two sets of base restrained HSC walls were casted; Winter set which consists eight walls in different length/height ratios (1,2,4,6) with and without reinforcement(0.2% horizontal and 1% vertical) and Summer set which consists two reinforced walls (0.2% horizontal and 1% vertical) in two length/height ratios (4,6). These walls were base restrained by steel base frame to hinder the shrinkage movement in the concrete base.

The mechanical properties of high strength concrete investigated were; compressive strength between ( 50 – 90 ) MPa, splitting tensile strength between ( 5.5 – 7 ) MPa, flexural tensile strength ( 6 – 8.6 ) MPa, static modulus of elasticity ( 32 – 44 ) GPa at ages from (7 to 90 days).

Based on the observations of this work, it was found that the cracks did not develop in the reduced scale reinforced and unreinforced high strength concrete walls with L/H (1,2,4 and 6) during 90 days period of drying conditions. Also, the shrinkage values increase toward the edges.

Based on the results of this work, the increasing in maximum shrinkage values in Summer were (73%, 61%) of the maximum shrinkage values in Winter for length/height ratios (4,6) respectively.

The use of horizontal reinforcement (0.2%) led to reduce in the maximum shrinkage values with increasing of length/height ratio where this reduction ratios were (12 %,14%,20%,28%) of the maximum shrinkage values in the unreinforced walls for length/height ratios (1,2,4,6) respectively.

The experimental observation in end restrained beams showed that the Summer free shrinkage(at date of cracking) was higher than of Winter free

shrinkage about (27%), the Summer tensile strain capacity(at date of cracking) was higher than of Winter about (30%), the Summer elastic tensile strain capacity(at date of cracking) was higher than the Winter elastic tensile strain capacity about (44%). As well as, when the shrinkage results of this study were compared with shrinkage results of normal concrete in previous study, the shrinkage of high strength concrete was less than of normal concrete.

Finally mathematical model was proposed to predict the maximum shrinkage value at 90 days drying period which occurs in high strength concrete walls. This model was derived by statistical program called (DATA FIT 9.0.59, 2008) and depends on length/height ratio, horizontal reinforcement ratio, vertical reinforcement ratio, temperature degree and relative humidity. The correlation coefficient was (  $R = 0.9724$  ),the variance explained was ( $R^2 = 0.9456$ ) and Standard Error was( $S.E=62.247$ ).

# A CKNOWLEDGEMENT

Praise be to ALLAH, most gracious, most merciful, who gave me the ability and desire to complete this research work.

I would like to express my sincere appreciation to my supervisors, Asst. Prof. Dr. Ghalib M. Habeeb and Asst. Prof. Dr. Mahdi S. Essa for suggesting the problem, their valuable assistance and continuous guidance.

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Finally, thanks to all my friends, especially Abbas Khadum and Awaad Ali for their forceful encouragement during the progress of this work.



**Mohammed J. Mohammed**  
**2008**

# CERTIFICATION

We certify that we have read this thesis, titled "***Drying Shrinkage for High Strength Concrete Walls With Different Restraints***", and as an examining committee examined the student "***Mohammed Jassam Mohammed Al-Tae*** " 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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# CERTIFICATION

We certify that this thesis titled " *Drying Shrinkage for High Strength Concrete Walls With Different Restraints* " was prepared by " *Mohammed Jassam Mohammed Al-Tae* " 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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# C HAPTER ONE 1

## *INTRODUCTION*

### ***1-1 General:***

With the gradual improvement in concrete technology and concrete practice over the years there has been an increasing use of concrete in higher strengths, in which one or more specific characteristics have been enhanced through the selection and proportioning of its constituents.

The term " high strength concrete" (HSC) is generally used for concrete having a 28 days compressive strength in excess of 40 MPa. The use of high strength concrete in construction industry has steadily increased over the past years, since it reduces the dead weight of structural elements. This in turn leads to the design of smaller sections, allowing longer spans and more usable area of buildings [***Brooks and Wainwright, 1983***].

In general HSC has been advantageously used for columns and shear walls of high rise buildings, elevated structures and any construction where durability is a function of compressive strength. Such concrete is being specified where reduced weight is important or where architectural considerations require more slender load carrying elements [***Yogendran et.al,1987***].

Volume change cracking of base restrained concrete walls is a widespread problem. In practice, such cases are frequently encountered in structures consisting of rigidly interconnected parts which have been concreted after their foundations have hardened, such as tunnels, basements, retaining walls, etc. Because of this procedure, the foundation and the wall are out of phase with regard to shrinkage and early age thermal movement. Differences in deformations thus tend to develop between the foundation and the wall, but they are prevented from actually developing by the rigid interconnection of these structural parts. This restraint is liable to give rise to cracking of the concrete walls [*Kheder,1997*].

Finally, 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 .

### ***1-2 Research Significance :***

Many researchers studied base restrained shrinkage of walls on the normal concrete. However, no work was done on shrinkage cracking of high strength concrete walls with base restraint. The main aims of this study are the effect of base restraint on the shrinkage of high strength concrete walls, studying the shrinkage strain values and its distribution and location of maximum shrinkage value.

**1-3 Objectives of This Work :**

This work planned to produce high strength concrete in 80 MPa of cube compressive strength. The main goals of this work are as follows :

- 1- Studying the mechanical properties of high strength concrete produced in  $1.55\text{m}^3$  as a total volume which was divided between ten walls (Winter and Summer sets) and the specimens( 24 concrete cubes of 100 mm, 16 concrete cylinders of (  $100 \times 200$  ) mm for splitting tensile strength, 16 concrete cylinders of (  $150 \times 300$  ) mm for modulus of elasticity and 16 concrete prisms of (  $100 \times 100 \times 400$  ) mm for flexural tensile strength).
- 2- Studying the base restrained drying shrinkage of ten high strength concrete walls(eight walls in the Winter season and two walls in the Summer season) with and without reinforcement (0.2% as horizontal reinforcement and 1% as vertical reinforcement) in different length/height ratios (1,2,4 and 6).
- 3- Studying the behavior of free drying shrinkage of high strength concrete ( 2 concrete beams of size (  $3 \times 0.125 \times 0.12$  ) m ) in two seasons(Winter and Summer).

**1-4 Research Layout :**

The research work presented in this thesis is given through five chapters:

Chapter One provides a general introduction.

Chapter Two, “Review of Literature”, introduces a definition of the high strength concrete ( HSC ), mechanical properties of HSC, shrinkage

definition, types of shrinkage and factors influencing it, creep, tensile strain capacity, cracking age and the shrinkage mechanism in end restrained members and base restrained walls.

Chapter Three, “Experimental work”, gives a description of the materials used to achieve HSC, mixing, casting, curing and testing the specimens for compressive strength, tensile strength, flexural strength and modulus of elasticity. As well as it gives a description of the experimental wall models which were casted to observe cracking and procedure of measurements carried out in this study.

Chapter Four the analysis and discussion of the experimental work were presented. Mechanical properties ( compressive strength, splitting tensile strength, flexural strength, modulus of elasticity ), shrinkage strain values and their distribution in the walls and (free shrinkage, tensile strain capacity, elastic tensile strain capacity, creep and cracking time) of end restrained specimens. A mathematical model for prediction the maximum shrinkage strain in the walls were developed and presented in this chapter .

Finally, Chapter Five is devoted to the general conclusions and suggestions for further research.

# C HAPTER TWO 2

## LITERATURE REVIEW

### ***2-1 Introduction:***

For many years, concrete with compressive strength in excess of 40 MPa was available at only few locations. However, in recent years, the applications of high-strength concrete have increased, and high-strength concrete has now been used in many parts of the world. The growth has been possible as a result of recent developments in material technology and a demand for higher-strength concrete. High strength concrete (HSC) has been advantageously used for columns, shear walls of high rise buildings, elevated structures, bridges, tunnels, precast and prestressed products, and in construction where durability is a function of compressive strength. Such concrete is being specified where reduced weight is important or where architectural considerations require more slender vertical load carrying elements[Yogendran *et.al*,1987].

High strength concrete (HSC) has a simple definition would be " concrete with a compressive strength above the present existing limits in national cods about 40 MPa and up to 130 MPa " [ *ACI Committee 363, 1997*].

The strength classification was explained according to many factors ,thus it is a helpful tool for describing high strength concretes as shown in Table (2-1). [*Concrete Technology,1994*].

**Table (2-1)** Strength Classification of Concrete[*Concrete Technology,1994*].

|                                            | <b>Conventional<br/>Concrete</b> | <b>High-Strength<br/>Concrete</b> | <b>Very-High<br/>Strength<br/>Concrete</b> | <b>Ultra-High-<br/>Strength<br/>Concrete</b> |
|--------------------------------------------|----------------------------------|-----------------------------------|--------------------------------------------|----------------------------------------------|
| <b>Strength (MPa)</b>                      | <50                              | 50-100                            | 100-150                                    | >150                                         |
| <b>Water-cement ratio</b>                  | >0.45                            | 0.30-0.45                         | 0.25-0.30                                  | <0.25                                        |
| <b>Chemical admixture</b>                  | Not necessary                    | WRA* or<br>HRWR* *                | HRWR                                       | HRWR                                         |
| <b>Mineral admixture</b>                   | Not necessary                    | Fly ash                           | Silica fume                                | Silica fume                                  |
| <b>Permeability<br/>coefficient (cm/s)</b> | $>10^{-10}$                      | $10^{-11}$                        | $10^{-12}$                                 | $<10^{-13}$                                  |
| <b>Freeze-thaw<br/>protection</b>          | Needs air<br>entrainment         | Needs air<br>entrainment          | Needs air<br>entrainment                   | No freezable<br>water                        |

\* WRA:Water reducing admixture.

\* \* HRWR:High range water reducer.

High compressive strengths are achieved by using a low water- to- cementitious materials ratio, requiring the use of water- reducing admixtures to provide adequate workability. High strength concrete offers significant economic advantages over conventional normal strength concrete (NSC) because more slender members can be designed, resulting in reduced material and transportation costs. As structural components become more slender, deflection becomes a more crucial issue, making long-term creep and shrinkage deformations especially important in HSC structures[*Long and Nicholas,2000*].

## 2-2 Mechanical Properties of HSC:

### 2-2-1 Compressive Strength measurement:

Testing 150 mm cubes or, 150 by 300 mm cylinders may present a problem with respect to the capacity of the testing machines as 80 percent of the capacity should not be exceeded. The use of smaller specimens is therefore, preferable: specifically, 100 mm cubes or 100 by 200 mm cylinders, are satisfactory, given that, in high-strength concrete [*Neville, 1995*].

### 2-2-2 Modulus of Elasticity:

The modulus of elasticity of concrete is one of the most important mechanical properties of concrete. It is a basic parameter necessary for estimating prestress losses, and immediate and time-dependent deformations due to early loading. The modulus of elasticity of concrete is closely related to the properties of the cement paste, the stiffness of selected aggregates, and also the method of determining the modulus [*Iravani, 1996*].

Most national standards express the modulus of elasticity as a function of the compressive strength determined from compression tests. These expressions are listed below .

$$\text{ACI 363R-97} \quad E_c = 3.32 f_c'^{0.5} + 6.9 \quad [\text{American}]$$

$$\text{EN(1990)} \quad E_c = 10(f_c' + 8)^{1/3} \quad [\text{European}]$$

$$\text{ACI 318-95} \quad E_c = 43 \rho_c^{1.5} f_c'^{0.5} \times 10^{-6} \quad [\text{American}]$$

Where:

$f_c'$ : is the compressive strength of concrete cylinders.

$\rho_c$ : is the density of concrete in (kg/m<sup>3</sup>)

**Mehta and Aitcin [1990]** showed that the nature of the coarse aggregate influences the elastic properties of HSC. This can be attributed to

the highly dense hydrated cement phase and the exceptional hydrated cement paste- aggregate bond that makes HSC behave like an ideal composite material.

The elastic deformation of high strength concrete is of particular interest. Because the modulus of elasticity of the very strong hardened cement paste and of the aggregate differ less from one another than in normal strength concrete, the behaviour of HSC is more monolithic and the strength of the aggregate-matrix interface is higher. There is, therefore, less bond microcracking and the linear part of the stress- strain curve extends up to a stress which may be as high as 85 per cent of the failure stress, or even higher (Fig(2-1)). Subsequent failure takes place through the coarse aggregate particles as well as through the matrix. Thus, the coarse aggregate particles do not act as crack arresters so that failure is rapid [**ACI Committee 363,1997** ].

Moreover the shape of the ascending part of the stress-strain curve in Fig .(2-1) is more linear and steeper for HSC. The strain at the maximum stress is slightly higher and the slope of the descending parts becomes steeper.

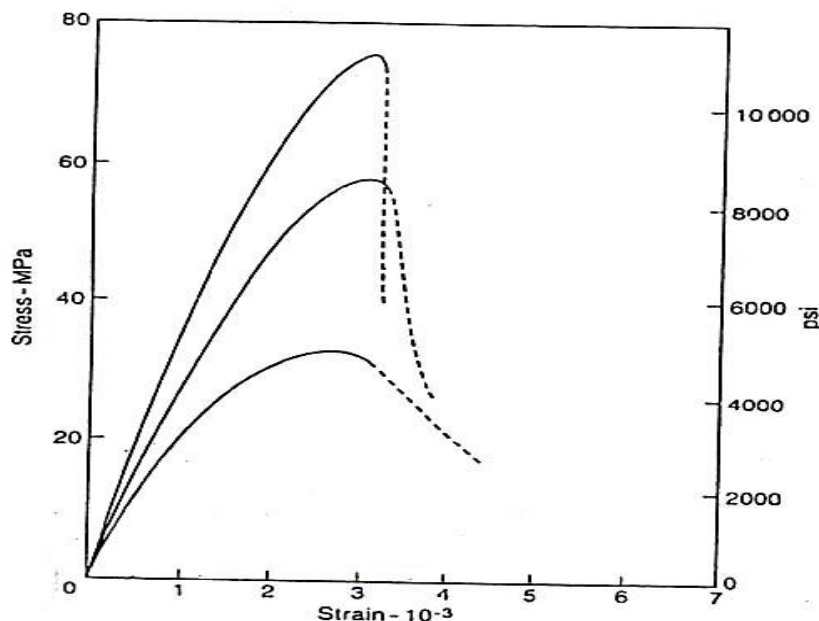


**Fig. (2-1)** Typical stress-strain curves for concrete of different strength [**ACI Committee 363,1997** ].

From the preceding paragraph it seems that for HSC the  $E_c$  which represents the secant modulus and  $E_t$  which represents the initial tangent modulus are more comparable in HSC than in NSC.

### **2-2-3 Modulus of Rupture:**

**Burg and Ust [1992]** reported that moist-cured specimens have higher flexural tensile strength or modulus of rupture than air-dried specimens. **ACI Committee 363[1997]** gives the modulus of rupture of high strength concrete fall in the range of  $0.62\sqrt{f_c'}$  to  $1.0\sqrt{f_c'}$  and equal to  $0.94\sqrt{f_c'}$  for  $21 < f_c' < 83$  MPa , where both the modulus of rupture and compressive strength are measured in MPa.

### **2-3 Strength Gain of High Strength:**

High-strength concrete shows higher rate of strength gain at early ages as compared with normal strength concrete, but at later ages the difference is not significant [**ACI Committee 363 ,1997**].The ratio of 7-day to 28-day strength were found between 0.8 - 0.9 for HSC and 0.7 - 0.75 for normal strength concrete. It seems likely that higher rate of strength development of high-strength concrete at early ages is caused by:

1. An increase in the internal curing temperature in the concrete due to the higher heat of hydration.
2. Shorter distance between hydrated particles in high-strength concrete due to low water / cement ratio (low porosity).

## **2-4 Shrinkage of Concrete:**

Concrete shrinks when it is exposed to a drying environment. The amount of shrinkage depends on many factors, including the properties of the material, temperature, relative humidity of the environment, the age when concrete is exposed to the drying environment, and the size of the structure. If concrete is restrained from shrinkage tensile stresses will develop and concrete may crack [*Wiegrink et.al.,1996*].

Shrinkage is the decrease of concrete volume with time. 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 cracking is a major cause of concern for concrete structures. In addition to weakening the structure, these shrinkage cracks have the potential to allow infiltration of moisture and chloride ions that accelerate the corrosion of steel reinforcement and reduce the durability of concrete [*Gilbert, 2001*].

## **2-5 Types of Concrete Shrinkage:**

There are different types of concrete shrinkage, which can be summarized as below.

### **2-5-1 Plastic Shrinkage:**

Plastic shrinkage is associated with moisture loss from freshly poured concrete into the surrounding environment, within the first few hours after it has been placed. Plastic shrinkage occurs primarily due to the heat liberated during hydration, absorption of mixing water and bleeding. The volumetric contraction while cement paste is plastic may reach 1% of absolute volume of dry cement [*ACI Committee 209,1996*].

### **2-5-2 Autogenous Shrinkage:**

Autogenous shrinkage is the early shrinkage of concrete caused by the loss of water from capillary pores due to the hydration of cementitious materials, without the loss of water into the surrounding environment. This phenomenon is known as self-desiccation of concrete. Still the autogenous shrinkage will continue as the cement hydration reaction lowers the internal relative humidity, which is known as self-desiccation [**Holt, 2001**].

**Bentz et.al.[2002]** explained autogenous shrinkage of high performance concrete as follows: In high-performance concrete a dense microstructure can form within a few days, preventing the introduction of external curing water. The reaction products that are formed during the hydration of cement occupy less space than the corresponding reactants. Cement paste hydrating under sealed condition self-desiccate and creates empty pores within the hydration paste structure. If external water is not available to fill these empty pores, then large tensile stresses are generated in the pores, resulting in considerable amount of shrinkage.

Self-desiccation occurs in all concrete irrespectively of the W/C ratio. On the other hand, in case of high performance concrete, pore network is essentially composed of fine capillaries due to low W/C and high amounts of cementitious hydration products. When self-desiccation starts to take place, very high tensile stresses are generated in these fine pores, resulting in higher autogenous shrinkage [**Aitcin, 2003**].

### **2-5-3 Drying Shrinkage:**

Both autogenous shrinkage and drying shrinkage occur due to decrease in humidity in the hardened cementitious paste. Drying shrinkage is different from autogenous shrinkage with regard to the mechanism of decrease in humidity. Drying shrinkage is caused by the diffusion of water from concrete into the outer surrounding environment .Drying shrinkage

refers to the reduction in concrete volume resulting from the loss of capillary water by evaporation. This shrinkage causes an increase in tensile stress of restrained concrete, which leads the concrete to cracking, internal warping, and external deflection, even if the concrete is not subjected to any kind of external loading [Neville, 1995].

According to [Power and Brownnyard, 1946] there are three types of water occurring in cement paste which are:

- Non-evaporable water [fixed water].
- Gel water.                      }
- Capillary water.               } Free water

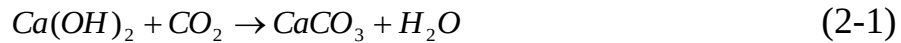
The non-evaporable water is very difficult to evaporate under the normal conditions.

Gel water is difficult to evaporate, its evaporation may take 20 years, and this is the principal cause of drying shrinkage.

The change in the volume of drying concrete is not equal to the volume of water removed. The loss of free water, which takes place first, causes little or no shrinkage. As drying continues, adsorbed water is removed and the change in the volume of unrestrained hydrated cement paste at that stage is equal approximately to the loss of a water layer one molecule thick from the surface of all gel particles since the "thickness" of water molecule is about 1% of the gel particle size, a linear change in dimension of cement paste on complete drying to be of the order of  $10000 \times 10^{-6}$ .

#### **2-5-4 Carbonation Shrinkage:**

The presence of carbon dioxide ( $CO_2$ ) and moisture in air can produce carbonic acid which reacts with hydrated lime in cement and produce ( $CaCO_3$ ).



This type of shrinkage is usually limited to the surface of concrete, and the actual rate of carbonation depends on the moisture and the  $\text{CO}_2$  content and the relative humidity of the ambient medium.[ **Wolsiefer,1984**].

## **2-6 Factors Affecting Shrinkage of Concrete:**

There are many factors influencing shrinkage of the concrete but the main factors are summarized as follows:

### **2-6-1 Influence of Aggregate:**

The aggregate appears to have a great influence on shrinkage of concrete. It occupies about (65-75) % of total concrete volume and restrain the shrinkage of cement paste.

**Han[2002]** examined the effect of aggregate in high strength concrete on creep and shrinkage. the shrinkage testing was conducted on (100x100x400) mm specimens cured at  $20 \pm 3^\circ\text{C}$  and 65% relative humidity. The different concrete mixtures included three types of aggregate; crushed gravel, granite, and limestone. The high strength concrete with the limestone aggregate had less shrinkage than the mixtures containing crushed gravel or granite. The rate of shrinkage of the limestone mixture was less than that of the mixtures containing the other aggregate types. It was determined that the limestone mixture had the highest early elastic modulus which explains the reduced shrinkage behavior.

Various equations have been proposed to relate the shrinkage of concrete to the shrinkage of its cement paste and the aggregate content.

**Pickett[1956]** derived on equation of the form:

$$S_c = S_p(1-a)^n \quad (2-2)$$

$$n = \frac{3(1 - \mu_c)}{1 + \mu_c + 2(1 - \mu_g)E / E_g} \quad (2-3)$$

Where:

( $S_c$ ): The shrinkage of concrete.

( $S_p$ ): The shrinkage of neat cement paste.

(a): The aggregate content in the concrete (by volume).

(n): Factor.

( $E_c$ ,  $E_g$ ): Modulus of elasticity for concrete and aggregate respectively.

( $\mu_c$ ,  $\mu_g$ ): Poisson's ratio for the concrete and aggregate respectively.

This relation also shows in Fig.(2-2) which shows the relation between the aggregate content and the ratio of the shrinkage of concrete to the shrinkage of neat cement paste.

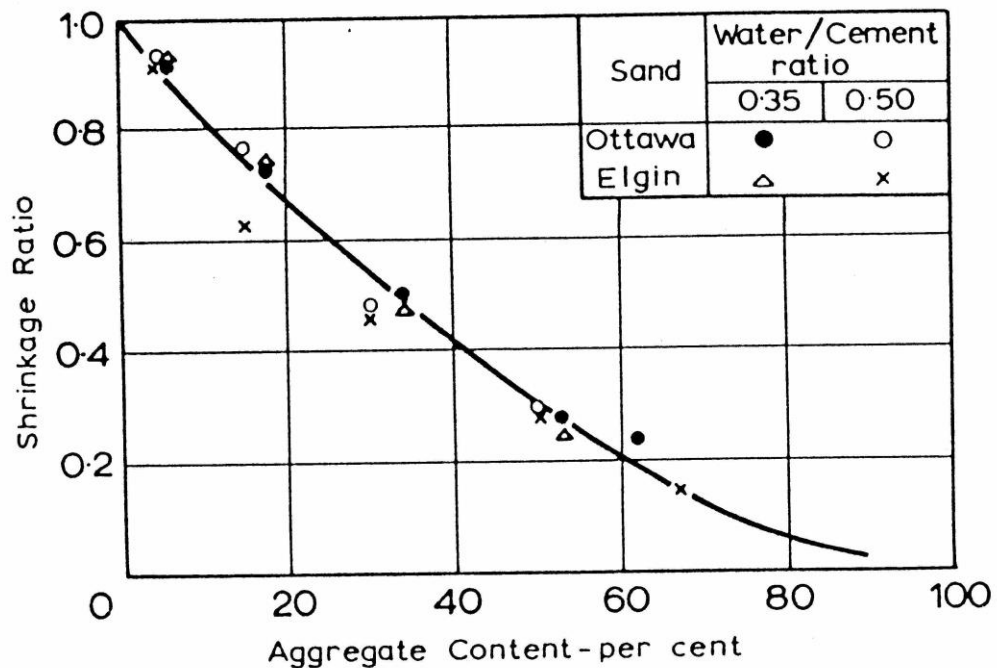


**Fig.(2-2):** Influence of the aggregate content in concrete (by volume) on the ratio of the shrinkage of concrete to the shrinkage of neat cement paste, *Pickett [1956]*.

### **2-6-2 Cement Type and Fineness:**

**Troxell et al.[1968]** concluded that shrinkage of low heat cement (type IV) is greater than that of (type I). This is believed to be due to high ( $C_2S$ ) content in (type IV) cement, which exhibits high shrinkage.

**Mehta and Monteiro [1993]** stated that the variation in fineness and composition of Portland cement affect the rate of hydration, but not the volume and characteristics of hydration products. Therefore, normal changes in fineness and composition of cement have negligible effect on drying shrinkage of concrete .

**Tazawa and Miyazawa [1997]** found that cement composition has a greater influence on autogenous shrinkage than drying shrinkage. Compared to normal Portland cement, larger autogenous shrinkage was observed for high early strength cement at an early age, and blast furnace slag cement at later ages. Less autogenous shrinkage was observed for moderate heat cement paste, and low heat Portland cement with a high  $C_2S$  content. Autogenous shrinkage depends on the hydration of  $C_3A$  and  $C_4AF$  and it increases with an increase in these compounds.

**ACI Committee 224 [2001]** suggested that lower shrinkage of cement paste is associated with lower ( $C_3A/SO_3$ ) ratio, lower ( $Na_2O$ ) and ( $K_2O$ ) content and higher ( $C_4AF$ ) content.

**Bentz et al. [2001]** studied the influence of cement particle-size distribution on early age autogenous strain and stress in concrete. The experimental results indicate that a small autogenous expansion as opposed to shrinkage may be produced through the use of coarser cements. Therefore early age cracking could possibly be avoided. Although coarser particles of cement are relatively beneficial in minimizing early age cracking, they may be detrimental to long-term strength .

Autogenous shrinkage of concrete is also influenced by the mineral composition of cement. Increased fineness and increased content of  $C_3A$  and  $C_4AF$  contributes to the increase in early shrinkage of concrete [Aitcin 2003].

### **2-6-3 Influence of Curing:**

**Powers[1959]** reported that prolonged moist curing delays the initiate of shrinkage but the curing effect on the shrinkage magnitude is small, though rather complex. As far as neat cement paste is considered, the greater the quantity of hydrated cement the smaller is the volume of unhydrated cement particles, which restrain the shrinkage; thus prolonged curing could expect to lead to greater shrinkage. But the hydrated cement paste contains less water and becomes stronger with age and is able attain a larger fraction of its shrinkage tendency without cracking. However, in concrete, if cracking takes place e.g. around aggregate particles, the overall shrinkage, measured on concrete specimens, apparently decreases.

**Hanson [1965]** reported that concrete at (28) days curing instead of (7) days before drying was gave about, (20%) reduction in free shrinkage strain. The longer the curing time , concrete was the less shrinkage occurrence.

### **2-6-4 Water Content:**

**Carlson et.al. [1979]** reported that a decrease of water content of concrete by 15 percent causes a decrease in shrinkage of about 30 percent, as similar trend was given by the **ACI Committee 224 [2001]** for 10 percent reduction of water content, the one year drying shrinkage decreased by 15 percent.

In general, the water content of any mix would indicate the expected shrinkage, following the general pattern of direct proportionality Fig. (2-3) [Shacklock et.al.[1957] as reported by Neville1995].

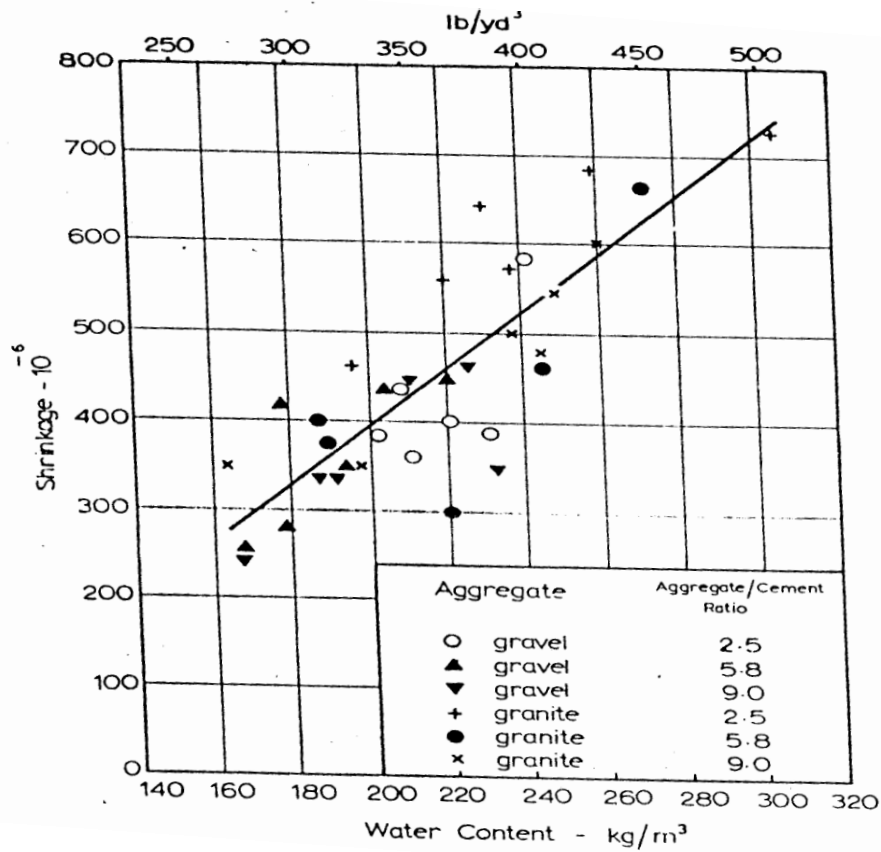


**Fig.(2-3)** The relation between the water content of fresh concrete and drying shrinkage [*Shacklock[1957] as reported by Neville,1995*] .

From the above figure, it can be observed that the shrinkage will increase with increasing water content and also in case of increasing the aggregate/cement ratio, the shrinkage will increase with different amounts depending on the type of aggregate. As well as, the w/c ratio of the mix is one of the most important factors in shrinkage of high strength concrete because its increase tends to increase shrinkage and, at the same time, to reduce the strength of the concrete.

#### **2-6-5 Influence of Ambient Conditions:**

The relative humidity of the medium surrounding the concrete greatly affects the magnitude of shrinkage, as shown for instance in Fig. (2-4). The same figure illustrates also the greater absolute magnitude of shrinkage compared

with swelling in water: swelling is about six times smaller than shrinkage in air of relative humidity of 70 per cent or eight times smaller than shrinkage in air at 50 per cent.

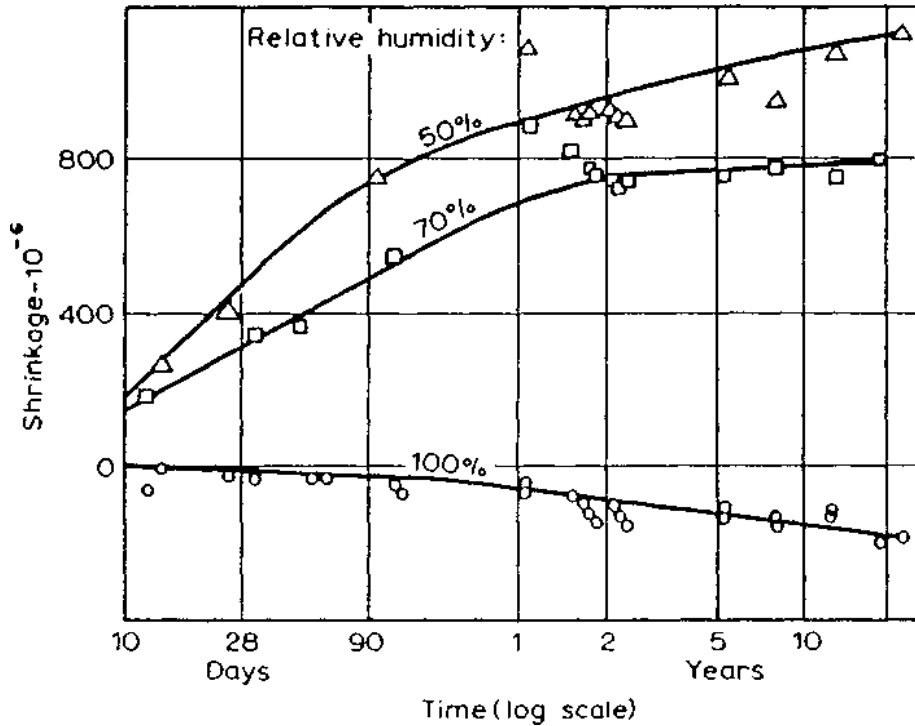


**Fig. (2-4)** Relation between shrinkage and time for concrete stored at different relative humidities[Troxell et. al,1958 as reported by Neville, 1995].

A high temperature and a low relative humidity of the ambient environment accelerate the diffusion of the adsorbed water and capillary water into the atmosphere, and consequently, increases the drying shrinkage of concrete. An increase in the atmospheric humidity slows down the rate of moisture flow from the interior to the outer surface of concrete. At 100% relative humidity, it is assumed that the drying shrinkage of concrete is zero [Mehta and Monteiro, 1993].

**Almudaiheem and Hansen [1987]** exposed 450 x 450 x 20 mm specimens to different temperature, relative humidity, and wind velocity conditions. The increased relative humidity caused a decrease in water

evaporation when there was no wind. Relative humidity became insignificant to evaporation of water under windy conditions. The increase in wind increased the rate of water evaporation, crack length and area, and early cracking as compared to non-windy conditions. The increase in temperature increased the rate of water evaporation and the crack length and area. It was concluded that water evaporation should be minimized to reduce the intensity of plastic shrinkage cracking.

#### **2-6-6 Influence of Specimen Size and Shape:**

The size and shape of concrete specimen definitely influence the rate of loss or gain of moisture under a given storage condition, and this can affect the rate of volume change as well as the total expansion or contraction.

**Mindess and Young[1981]** showed the example of a T-beam compared to a solid beam with the different volume to surface ratios. If the two beams have equal widths and heights, the T-beam will dry more rapidly and will exhibit slightly more ultimate shrinkage since it has the lower volume to surface ratio. The faster drying is attributed to the shorter diffusion path for water to travel to reach an equivalent relative humidity through the cross-section.

**Almudaiheem and Hansen [1987]** observed the shrinkage of various specimen size over a one year period. The shrinkage decreased with increasing specimen size. The ultimate shrinkage of paste, mortar, and concrete were found to be independent of specimen size and shape according to the shrinkage/ weight loss curves. They concluded that the ultimate drying may be estimated from shrinkage versus drying time curves for small laboratory specimens of (25x25x279) mm with the same mixture proportions as the larger structural members.

From the above studies, it can be expected that the walls have a more shrinkage because of its lower volume to surface ratio.

### **2-6-7 Influence of Admixtures:**

The use of admixtures has become popular in industry, especially for high strength and low permeable concretes.

#### **2-6-7-1 High-range water-reducing admixture:**

Water-reducing admixtures probably cause a small increase in shrinkage. Their main effect is indirect in that the use of an admixture may result in a change in the water content or in the cement content of the mix, or in both, and it is the combined action of those changes that influences shrinkage [Neville,1995].

Superplasticizers have been found to increase shrinkage by some 10 to 20 per cent. However, the changes in the observed shrinkage are too small to be accepted as reliable and generally valid [Brooks and Neville,1992].

From the preceding statements, it can be expected that shrinkage of very-high-strength concrete, which contains a superplasticizer, is simply the outcome of the relevant and opposing factors: a very low water/cement ratio and concomitant high self-desiccation, which lead to low shrinkage, and a high cement content, which leads to high shrinkage. Thus, the usual approach to estimating shrinkage applies also to very-high-strength concrete. However, the more rigid structure of such concrete restrains the magnitude of the effective shrinkage [Neville,1995].

The strength of hardened concrete containing HRWR admixtures is normally higher than that predicted by the lower w/cm alone. As with conventional admixtures, this is believed to be due to the dispersing effect of HRWR admixtures on the cement and other cementitious or pozzolanic materials. Because the w/c of mixtures containing HRWR admixtures are typically low, shrinkage may also be reduced and the overall durability of the concrete may be increased [ACI Committee E-701,2003].

**2-6-7-2 Silica Fume:**

Silica fume is an industrial by-product with a particle size about 100 times finer than Portland cement. This material, which is highly pozzolanic also creates a greater water demand or the use of a high range water reducer [Mehta, 1986].

*Tazawa and Miyazawa [1991]* found that autogenous shrinkage of concrete during the first 24 hours after placement increases with decreasing water to binder ratio. When the water to binder ratio was reduced to 0.17 with superplasticizers and silica fume, the greater part of the shrinkage in the dried condition was attributed to autogenous shrinkage.

Silica fume considerably increased the free plastic shrinkage of the concrete compared to the reference Portland cement concrete with the same water to binder ratio, where the mixtures with a water to binder ratio of 0.40 cracked with silica fume added, but no cracking was observed in the non-silica fume mixtures[ *Bloom and Bentur,1995*].

*Haque [1996]* measured the drying shrinkage on 85 x 85 x 285 mm prism specimens. The addition of both 5 and 10 % silica fume (by weight) in concrete mixtures resulted in substantial reduction of drying shrinkage. Further addition of 20 % fly ash or slag increased the shrinkage of these concretes.

**2-6-7-3 Fly Ash:**

In most cases, good pozzolans, such as fly ash, are available and can be used to replace a portion of the cement. This can result in a considerable cost saving, and more importantly for mass construction, can reduce the heat generation and improve the resistance against cracking [ACI Committee 224,2001].

The particle size distribution, morphology, and surface characteristics of fly ash used as a mineral admixture has a considerable influence on the water requirement, workability, and rate of strength development of concrete [Mehta, 1986].

#### **2-6-7-4 Shrinkage reducing admixture (SRA):**

Free and restrained shrinkage of high performance concrete incorporating SRA have been studied by *Folliard and Berke [1997]*. Results indicated a reduction in drying shrinkage in the order of 35 to 50 % for an addition rate of SRA of 1.5 % of the cement weight.

SRA was most effective in reducing the drying shrinkage of concrete during early periods (about four days) of exposure to dry air. In effect, SRAs eliminates much of the initial high drying shrinkage of concrete and reduces the cost of using water reducing admixtures and/or air-entraining admixtures [Tarun et. al, 2006].

It was found that the addition of 1.5 % SRA of the cement weight gave a reduction of the free shrinkage in the order of 25 %. Unexpectedly, the reduction did not seem to have a significant effect on the age at cracking [Carlsward, 2006].

#### **2-7 Shrinkage Induced Cracking:**

Concrete is always under some degree of restraint, either externally (by the foundation or by another part of the structure) or internally (by reinforcing steel embedded in concrete and the rigidity of the concrete aggregate) in structures during its contraction due to exposure to the environment of temperature and humidity variation conditions. So, the tendency of the concrete to contract upon drying and the restraint which resists action will develop tensile stresses within the concrete. 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 when the shrinkage develops slowly. If the tensile stress produced by the restrained shrinkage and reduced by the relaxation exceeds the tensile strength of concrete, which is a function of time, the cracking will occur as shown in **Fig. (2-5)** [Neville,1995].

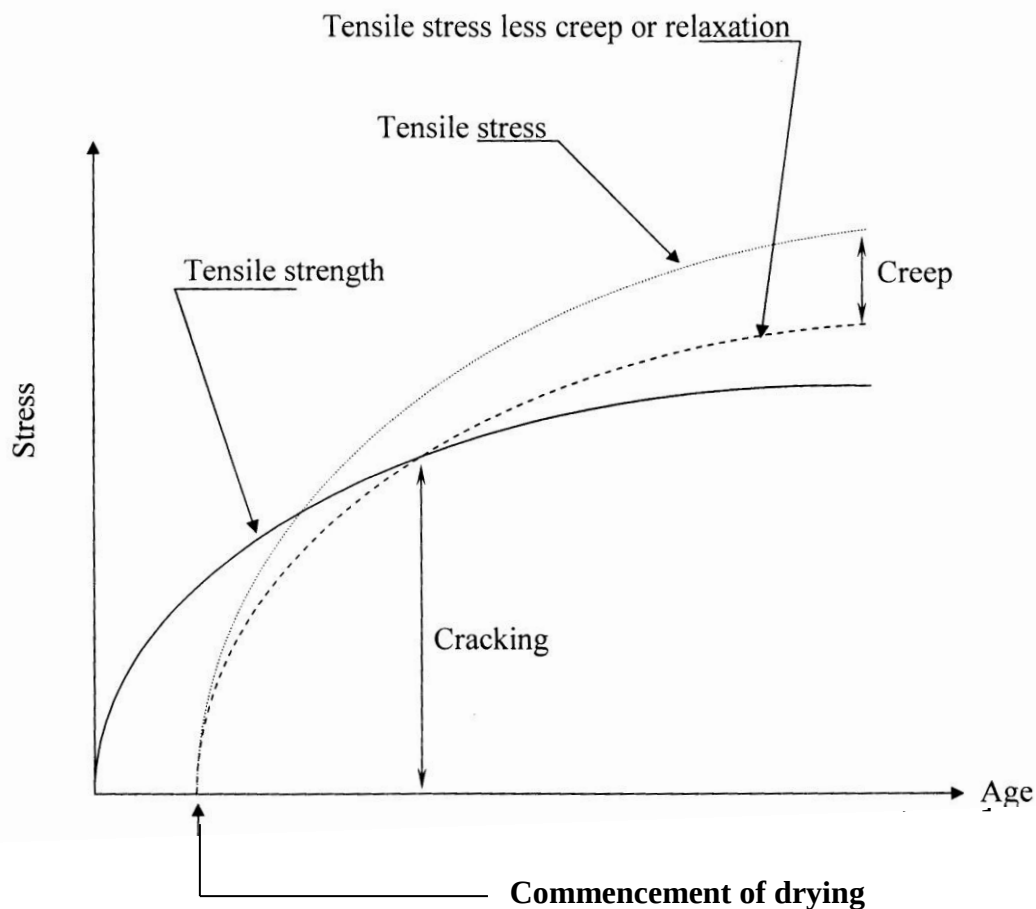


**Fig.(2-5)** Sketch of crack development (stress wise) [Neville,1995].

The magnitude of tensile stress development during drying of the concrete is influenced by a 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 or relaxation of concrete.

### **2-7-1 Shrinkage Cracking Mechanism In End-Restrained Members:**

**Saliger[1936] as reported by Beeby[1979]** was the first to describe completely the bond-slip mechanism of shrinkage and thermal cracking, Saliger based his theory for the prediction of crack spacings and widths on the following arguments:

1. Upon cracking, a slip between the concrete and the reinforcing steel- will take place. The tensile stress that was carried by the concrete at the crack position before cracking will be totally transferred to the reinforcement passing the cracked section. The concrete stress adjacent to the crack will decrease to a value in the neighborhood of zero. The tensile force carried by the steel will be retransferred gradually to the concrete, within the bond-slip distance, i.e., the concrete stress will increase gradually from zero at the crack to a maximum at the end of the bond-slip distance, beyond which it will remain uniform. As a result, the concrete stress within the bond-slip distance will generally be less than the tensile strength of the concrete, and therefore no cracking will occur within the bond-slip distance. Hence, the minimum crack spacing ( $S_{min}$ ) will equal the bond-slip distance. When two cracks form at a distance equal to  $2 S_{min}$ , no further cracks can form between them because of the low stress within that distance. The maximum crack spacing in a fully developed crack pattern will therefore be equal to  $2 S_{min}$ . In a complete cracking pattern, the crack spacing  $S$  will be within:

$$S_{min} < S < 2 S_{min} \quad (2-3)$$

Saliger defined crack spacing as:

$$S_{min} = kd / \rho \quad (2-4)$$

Where:  $k$  is a constant.

$d$ : bar diameter which its unit is (mm).

$\rho$ : steel reinforcement ratio.

2. The average crack width is related to the final average crack spacing and the total volume-change strain, and can be written as follows:

$$W_{ave} = S_{ave}(e_v - e_{ult} / 2) \quad (2-5)$$

Where:

$W_{ave}$  : average crack width(mm).

$S_{ave}$  : final average crack spacing(mm).

$e_v$  : total volume-change strain(microstrain).

$e_{ult}$ : elastic tensile strain capacity of concrete (microstrain).

**Evans and Hughes[1968]** adopted the following equations to predict the minimum crack spacing and maximum crack width in end strained members:

$$S_{min} = 1/4 \times d/\rho \times f_{tu}/f_b \quad (2-6)$$

where  $f_{tu}/f_b = 0.57, 0.53,;$  and  $0.35$  for deformed, indented, and plain round bars, respectively.

$S_{min}$  :minimum crack spacing (mm).

$d$ : bar diameter which its unit is (mm).

$\rho$ : steel reinforcement ratio.

$f_{tu}$ : tensile strength of concrete (MPa).

$f_b$ : average bond strength between steel and concrete (MPa).

$$W = S [e_{sh} + e_{th} (S \times e_{ult}) / (4 S_{min})] \quad (2-7)$$

Where:

$W$  : final crack width(mm).

$S$  : final crack spacing(mm).

$e_{sh}$  : free shrinkage of concrete(microstrain).

$\epsilon_{th}$  : thermal strain in concrete(microstrain).

$\epsilon_{ult}$  :elastic tensile strain capacity of concrete(microstrain).

$S_{min}$  : final minimum crack spacing(mm).

**B.S. Code 5337 [1976]** obtained the following equations to predict the minimum crack spacing:

$$S_{min} = 1/4 \times d/\rho \times f_{tu}/f_b \quad (2-8)$$

Where:

$S_{min}$  :minimum crack spacing (mm).

d: bar diameter (mm).

$\rho$ : steel reinforcement ratio.

$f_{tu}$ : tensile strength of concrete (MPa).

$f_b$ : average bond strength between steel and concrete (MPa).

And obtained the width of crack due to drying shrinkage and heat of hydration in restrained slabs and walls:

$$W_{max} = S_{max} [\epsilon_{cs} + \epsilon_{te} - (100 \times 10^{-6})] \quad (2-9)$$

Where:

$W_{max}$  : maximum crack width(mm).

$S_{max}$  : maximum crack spacing(mm).

$\epsilon_{cs}$  : shrinkage strain(microstrain).

$\epsilon_{te}$  : total thermal contraction after peak temperature due to heat of hydration(microstrain).

**Al-Rawi [1986]** used concrete beams reinforced with deformed bars and welded wire mesh. He found that using the deformed bar reinforcement can control both the minimum crack spacing and the maximum crack width. He found also a formula for the minimum crack spacing, (based on

experimentally measured bond slip distance), using a new model which was believed to resemble field conditions:

$$S_{\min} = 0.85 K \times d/\rho \quad (2-10)$$

$K$  : constant depending on the type of reinforcement which may be taken as (0.8) for indented deformed bars and (0.67) for ribbed deformed bars.

The width of crack at the onset of cracking ( $W_i$ ) is:

$$W_i = 2/3 S_{\max} \times \epsilon_{ult} \quad (2-11)$$

The final maximum crack width  $W_{\max}$  is:

$$W_{\max} = S_{\max} [\epsilon_{sh} - (\text{creep}_1 + \text{loss of restraint}) - (\epsilon_{ult} + \text{creep}_2) / 2] \quad (2-12)$$

$$S_{\max} = 2 \times S_{\min}$$

$S_{\max}$  : final maximum crack spacing(mm).

$S_{\min}$  : final minimum crack spacing(mm).

$\epsilon_{sh}$ : final shrinkage strain (microstrain)

$\epsilon_{ult}$ : elastic tensile strain capacity of concrete (microstrain).

$\text{creep}_1$ : creep prior to cracking (microstrain).

$\text{creep}_2$ : creep after cracking (microstrain).

**Al Ali[1985]** and **Al Timmimi[1987]** affirmed equation(2-10) for lean and medium-rich mixes, but observed a 25 percent increase in  $S_{\min}$  for rich concrete mixes.

**Kadhum[2003]** studied the behavior of drying shrinkage cracking of reinforced concrete slabs for different restraint cases (crack width, crack length and crack spacing).

For two and three end restrained slabs, the maximum crack width can be calculated by the following equation:

$$W_{\max} = S_{\max} [-10.06 \times \epsilon_{shn} + 26.92 \times C_f - 50.22 \times \epsilon_{ult} / 2] \quad (2-13)$$

Where:

$W_{\max}$  : maximum crack width(mm).

$S_{\max}$  : maximum crack spacing(mm).

$e_{shn}$  : net shrinkage stain, ( $e_{shn}=e_{sh}-L.O.R$ ) (microstrain).

$C_f$  : final creep strain, ( $C_f = K \times e_{shn}$ ) (microstrain).

$e_{ult}$  :elastic tensile strain capacity (microstrain).

For four end restrained slabs, the maximum crack width can be calculated by the following equation:

$$W_{\max}=S_{\max}[0.83(R_b-0.7R_a)e_{shn}+3.9 \times C_f-23.46 \times e_{ult}/2] \quad (2-14)$$

Where:

$R_a$  : degree of restraint after cracking

$R_b$  : degree of restraint before cracking (at wall center).

For two end restrained slabs:

$$S_{\min} = 0.85K \frac{d}{(\rho + \rho_R)} \quad (2-15)$$

For three end restrained slabs:

$$S_{\min} = 0.85K \frac{d}{(\rho + K_{R1})} \quad (2-16)$$

For four end restrained slabs:

$$S_{\min} = 0.85K \left( \frac{d}{\rho + 0.8K_{R2}} \right) \quad (2-17)$$

K: constant depending on the type of reinforcement which may be taken as (0.8) for indented deformed bars and (0.67) for ribbed deformed bars.

d: bar diameter (mm).

$\rho$ : steel reinforcement ratio.

$\rho_R, K_{R1}, K_{R2}$  : factors was estimated by STATISTICA program.

Table (2-2) shows a survey of crack spacing and width predicting formulas that were reported to be directly applicable to volume change cracking of end restrained concrete members.

**Table(2-2)** The crack spacing and width predicting formulas of end restrained concrete members

| Source                                                                                                                                                                                     | Crack spacing                                                                                                                                         | Crack width                                                                                                                                                                                                                                                                     |
|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <b>Saliger as reported by Beeby [1979]</b>                                                                                                                                                 | $S_{min} = kd / \rho$<br>$S_{min} < S < 2 S_{min} = S_{max}$                                                                                          | $W_{ave} = S_{ave}(e_v - e_{ult} / 2)$                                                                                                                                                                                                                                          |
| <b>Evans and Hugus (1968)</b>                                                                                                                                                              | $S_{min} = \frac{1}{4} \times d / \rho \times f_{tu} / f_b$<br>$S_{min} < S < 2 S_{min} = S_{max}$                                                    | $W = S [e_{sh} + e_{th} (S \times e_{ult}) / (4 S_{min})]$                                                                                                                                                                                                                      |
| <b>BS 5337:1976</b>                                                                                                                                                                        | $S_{min} = \frac{1}{4} \times d / \rho \times f_{tu} / f_b$<br>$S_{min} < S < 2 S_{min} = S_{max}$                                                    | $W_{max} = S_{max} [\epsilon_{cs} + \epsilon_{te} - (100 \times 10^{-6})]$                                                                                                                                                                                                      |
| <b>Al Rawi (1986)</b>                                                                                                                                                                      | $S_{min} = 0.85 d / \rho$<br>$S_{min} < S < 2 S_{min} = S_{max}$                                                                                      | $W_{max} = S_{max} [e_{sh} - (creep_1 + \text{loss of restraint}) - (e_{ult} + creep_2) / 2]$                                                                                                                                                                                   |
| <b>Kadhum[2003]</b> <ul style="list-style-type: none"> <li>• For two end restrained slabs:</li> <li>• For three end restrained slabs:</li> <li>• For four end restrained slabs:</li> </ul> | $S_{min} = 0.85K \frac{d}{(\rho + \rho_R)}$ $S_{min} = 0.85K \frac{d}{(\rho + K_{R1})}$ $S_{min} = 0.85K \left( \frac{d}{(\rho + 0.8K_{R2})} \right)$ | $W_{max} = S_{max} [-10.06 \times e_{shn} + 26.92 \times C_f - 50.22 \times e_{ult} / 2]$ $W_{max} = S_{max} [-10.06 \times e_{shn} + 26.92 \times C_f - 50.22 \times e_{ult} / 2]$ $W_{max} = S_{max} [0.83(R_b - 0.7R_a)e_{shn} + 3.9 \times C_f - 23.46 \times e_{ult} / 2]$ |

### **2-7-2 Shrinkage Cracking Mechanism In Continuous Base-Restrained Members:**

According to *ACI Committee 207[1973]*, several researchers have analyzed the stresses and strains generated in the uncracked stage in walls rigidly connected to a concrete base, due to the shortening of the wall relative to the restraining base. In all these works, the researchers based their analyses on linear-elastic behavior of the wall and the base, neglecting any loss in restraint occurring due to the shrinkage of the base or the slippage between the wall and the base. From these works, it is clear that the degree of restraint in walls with continuous restraining base is not uniform. Accordingly, a nonuniform state of tensile stress and strain will form as the wall shrinks, and the cracking will therefore take place only in positions of high restraint, where the tensile strain exceeds the elastic tensile strain capacity of the concrete.

In a wall casted on a continuous base, the restraint is not uniform throughout the wall, but varies from point to point within the wall. This variation depends on several factors as shown in *Fig(2-6)[Kheder,1997]*, these factors including the wall's L/H, the position of the point in consideration within the wall itself, wall and base dimensions, and the roughness of the contact surface between the wall and the base, including the vertical reinforcement (dowels) joining the two.

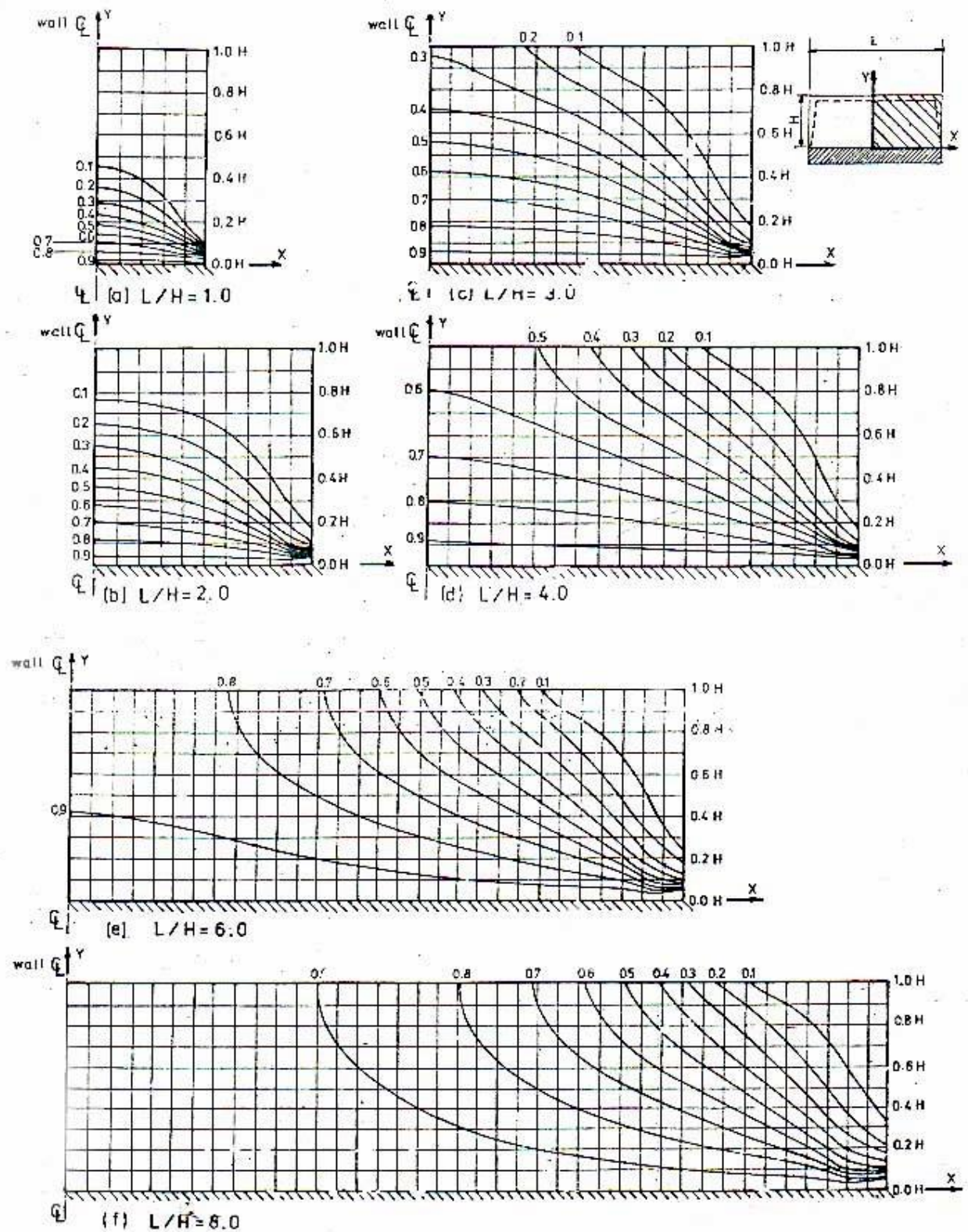


**Fig (2-6)** Distribution of restraint factor ( $R$ ) in walls with different  $L/H$  ratios [Kheder,1997].

### **2-7-3 Theory OF Change In Restraint Due To Cracking In Walls With Continuous Base Restraint:**

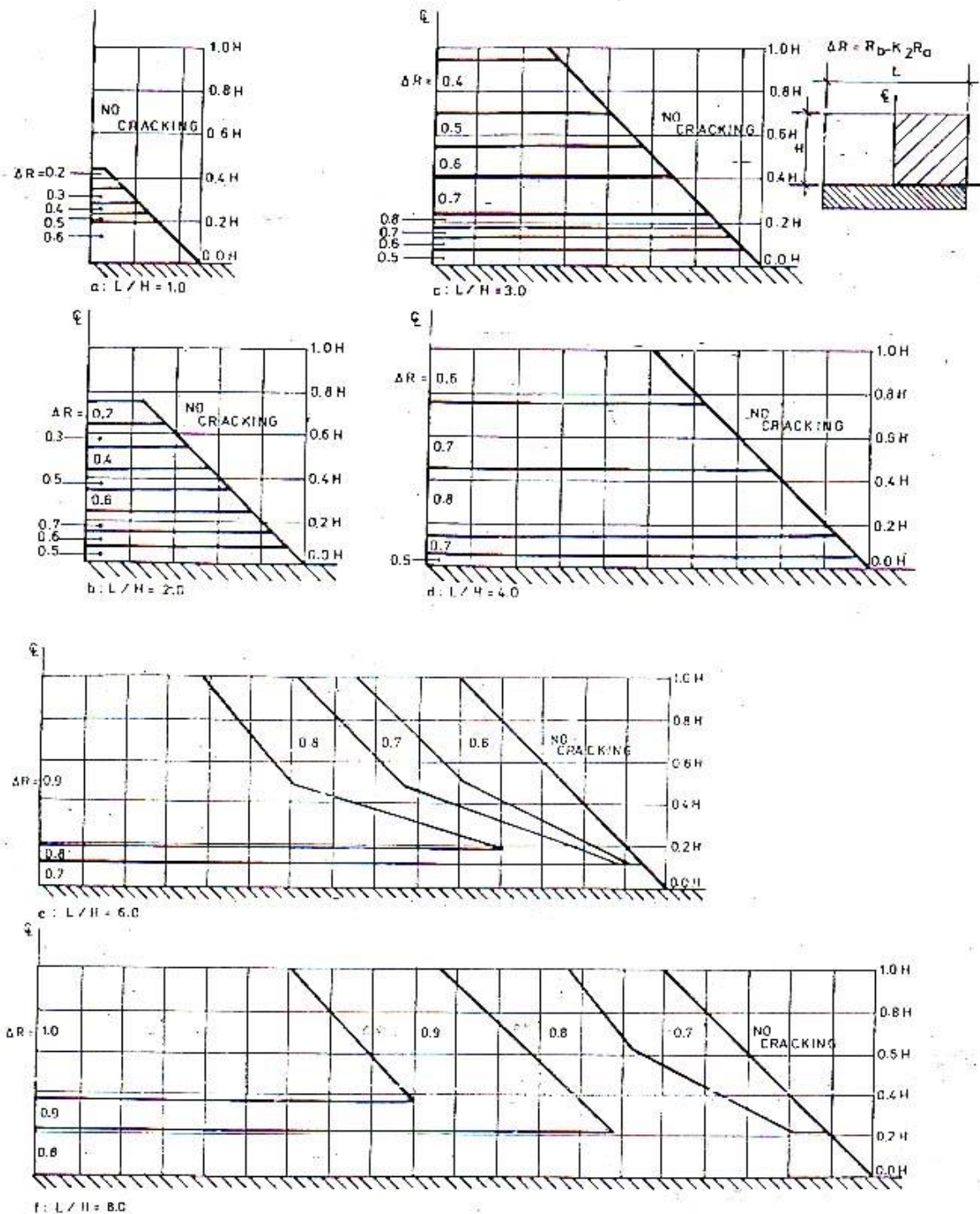
The quest to understand fully shrinkage cracking in walls with continuous base restraint has been misled by extending the theories developed for the cracking-behavior of end-restrained members to explain the cracking of walls with continuous base restraint. Two main differences exist between the cracking of the two types of external restraint.

a- The first difference is that the cracking in end-restrained members can take place at any position within the member, due to the uniform degree of restraint along the member prior to cracking. On the other hand, in walls with base restraint, cracking will only take place in positions of high restraint.

b- The second difference is that in end-restrained members, the degree of restraint in the vicinity of the crack decreases to a value in the neighborhood of zero [**Harrison,1981**]and the resulting member portions will be prevented from free movement only by the presence of steel reinforcement; i.e., unreinforced members will move almost freely after the first (and only) crack has formed. Conversely, in base-restrained members, the degree of restraint at the crack will not decrease to zero after cracking. A shear force between the wall portions and the base will be mobilized [**ACI 207,1973**]. This shear force attempts to prevent the wall portions from moving apart. The value of the residual restraint in the wall portions will depend on the resulting L/H. Residual tensile strain will remain in the concrete adjacent to the crack even if the wall is unreinforced; therefore, crack opening must depend on the difference between the restrained movement before and after cracking. In other

words, the crack width depends on the change in restraint in the wall associated with crack formation [Al-Rawi and Kheder,1990].

This change of restraint had been explain by Kheder [1997] as shown in Fig(2-7).



**Fig(2-7)** Idealized diagrams of change in restraint factors ( $\Delta R$ ) in walls with different  $L/H$  ratios [Kheder ,1997].

### **2-7-4 Crack Spacing And Width Predication Formulas of Walls With Continuous Base Restraint:**

**Evans and Hugus[1968]** used the mechanism which built on the assumption that slip exists between the steel and concrete adjacent to the crack.

$$S_{min} = 1/4 \times d/\rho \times f_{tu}/f_b \quad (2-18)$$

$$W = S[e_{sh} + e_{th} (S \times e_{ult}) / (4 S_{min})] \quad (2-19)$$

**ACI Committee 207[1973]** also considered the effect of variation in restraint in the wall before cracking only, and gave the following equation:

$$W_{max} = [1.085(d_c A)]^{1/3} f_s \times 10^5 \quad (2-20)$$

$$S_{ave} = W_{max} / [1.5(R_b C_t T_e - f_{tu}/E_c)] \quad (2-21)$$

Where:

$W_{max}$  : final maximum crack width(mm).

$S_{ave}$  :final average crack spacing(mm).

$d_c$  :thickness of concrete cover, measured from concrete surface at

which cracks are being considered, to center of nearest reinforcing bar (mm).

$A$ : effective tension area of concrete surrounding reinforcing bars, with the same centroid as reinforcement, divided by number of bars (mm<sup>2</sup>).

$f_s$  : steel stress at crack position (MPa).

$C_t$  :coefficient of thermal expansion of concrete.

$T_e$  : effective temperature change in members including an equivalent temperature change to compensate for drying shrinkage (°C).

$f_{tu}$  : tensile strength of concrete (MPa).

$E_c$  : modulus of elasticity of concrete (GPa).

The British standard **BS 5337:1976** used the formulas developed by **Hughes[1972]**. BS 5337 totally neglects the effect of the restraining base and considers the wall as an end-restrained member with a degree of restraint of  $R_b = 1.0$ . BS 5337 considers the creep to be 50 percent of the thermal strain only, and almost neglects the shrinkage by setting its value to 100 microstrains, which is a low value for hot-dry weather regions. BS 5337 gives the following equation to predict maximum crack width:

$$W_{\max} = S_{\max} [e_{sh} + e_{th}/2 - e_{ult}/2] \quad (2-22)$$

$$S_{\min} = 1/4 \times d/\rho \times f_{tu}/f_b \quad (2-23)$$

According to **Stoffers[1978]**, the crack spacing in reinforced concrete walls is determined only by the effect of the floor rather than by present in the wall, although a certain amount of stress will exist in the reinforcement. Stoffers assumes that the spacing between primary cracks ranges from 1.0 to 1.5 times the wall height, and assumes that this relationship is also valid for secondary crack spacing. Thus, the average crack spacing at any height in the wall Y is given by:

$$S_{ave} = (1.0 \text{ to } 1.5) Y \quad (2-24)$$

For convenience, Stoffers adopts the following equation :

$$S_{ave} = Y \quad (2-25)$$

Stoffers derived the following equation to calculate the steel stress  $f_{sy}$  and the crack width  $W_y$ , at any level in the wall:

$$f_{sy} = -2 f_b n \rho Y / d + \sqrt{(2 f_b n \rho Y / d)^2 + 4 f_b E_s Y e_v / d} \quad (2-26)$$

$$W_y = d f_{sy}^2 \cdot (1 - \rho) / \{4 f_b E_s [1 + (n - 1) \rho]\} \quad (2-22b)$$

where:

$f_b$  : average bond strength between reinforcing steel and concrete (MPa).

$n$  : modular ratio ( $E_s / E_c$ ).

$\rho$  : steel ratio.

Y:crack height(mm).

d :bar daimeter(mm).

$E_s$  :modulus of elacticity of steel.

**Base and Murray[1982]** have proposed another method from their work on end-restrained walls. They gave the following equations for the average crack spacing and width:

$$S_{ave} = 0.08 d / \rho \quad (2-27)$$

$$W_{max} = 2S(f_s/E_s + e_{sh}/3) \quad (2-28)$$

The British standard **BS 8007:1987** used the formulas developed by **Harrison et. al.[1981]** which used the Eq.(2-20a) by introducing the effect of the degree of restraint before cracking at the wall centerline and considered creep relief to be 50 percent of both thermal and shrinkage strains. Thus Eq. (2-20a) will become:

$$W_{max} = S_{max}[R_b(e_{sh} + e_{th})/2 - e_{ult}/2] \quad (2-29)$$

$R_b$  is maximum at the wall base, and decreases toward the wall top. Therefore, according to Harrison's formula, the crack width will be maximum at the base and minimum at the wall top.

**Al Rawi and Kheder (1990)** observed that the crack spacing in base restrained walls is determined by the combined effects of the restraining base and the horizontal steel reinforcement provided in the wall. In unreinforced concrete walls, the spacing of cracks is in the range of 1.0-2.0 times the wall or crack height for primary cracks and secondary cracks respectively. The provision of steel reinforcement in the wall will result in decreasing the crack spacing. The equation of crack spacing in end restrained members was modified to take into consideration the effect

of the restraining concrete base. The modified crack spacing equations for primary or secondary cracks are respectively:

$$\text{(primary cracks)} \quad S_{\min} = KdH/(\rho H + Kd) \quad (2-30)$$

$$\text{(secondary cracks)} \quad S_{\min} = KdY/(\rho Y + Kd) \quad (2-31)$$

And the crack width of two types is:

$$W_{\max} = S_{\max} [C_1(R_b - 0.8R_a)(e_{sh} + e_{th}) - e_{ult}/2] \quad (2-32)$$

$C_1$ : coefficient to include the effect of the creep of concrete taken equal to 0.6.

Table (2-3) shows a survey of crack spacing and width predicting formulas that were reported to be directly applicable to volume change cracking of base restrained concrete walls.

**Table(2-3)** The crack spacing and width predicting formulas of base restrained concrete walls.

| Source                           | Crack spacing                                                                                              | Crack width                                                            |
|----------------------------------|------------------------------------------------------------------------------------------------------------|------------------------------------------------------------------------|
| Evans and Hughus (1968)          | $S_{\min} = 1/4 \cdot d/\rho \times f_{tu}/f_b$<br>$S_{\min} < S < 2 S_{\min} = S_{\max}$                  | $W = S[e_{sh} + e_{th}(S \times e_{ult}) / (4 S_{\min})]$              |
| ACI 207.2R (1973)                | $S_{ave} = W_{\max} / [1.5(R_b C_t T_e - f_{cr}/E_c)]$                                                     | $W_{\max} = [1.085(d_c A)]^{1/3} f_s \times 10^5$                      |
| Hughes (1972)<br>BS 5337:1976    | $S_{\min} = 1/4 \cdot d/\rho \times f_{tu}/f_b$<br>$S_{\min} < S < 2 S_{\min} = S_{\max}$                  | $W_{\max} = S_{\max} [e_{sh} + e_{th}/2 - e_{ult}/2]$                  |
| Stoffers (1978)                  | $S_{\min} = H$<br>$S_{\min} < S < 2 S_{\min} = S_{\max}$                                                   | $W_y = f_{sy} d (1 - \rho) / \{4 f_b E_s [1 + (n-1) \rho]\}$           |
| Base and Murray (1982)           | $S_{ave} = 0.08 d/\rho$                                                                                    | $W_{\max} = 2S(f_s/E_s + e_{sh}/3)$                                    |
| Harrison (1981)<br>BS 8007: 1987 | $S_{\min} = 1/4 \cdot d/\rho \times f_{tu}/f_b$<br>$S_{\max} = 2 S_{\min}$                                 | $W_{\max} = S_{\max} [R_b(e_{sh} + e_{th})/2 - e_{ult}/2]$             |
| Al Rawi and Kheder (1990)        | $S_{\min} = KdH/(\rho H + Kd)$<br>$S_{\min} = KdY/(\rho Y + Kd)$<br>$S_{\min} < S < 2 S_{\min} = S_{\max}$ | $W_{\max} = S_{\max} [C_1(R_b - 0.8R_a)(e_{sh} + e_{th}) - e_{ult}/2]$ |

## ***2-8 Factors Influencing Cracking Resistance:***

### ***2-8-1 Creep (Relaxation) of Concrete:***

Creep is a slow increase of strain (due to stress) under a sustained load. According to **Neville [1995]**, if the restraints are such that a stressed concrete specimen is subjected to constant strain, creep will be expressed as a progressive decrease in stress with time known as (relaxation).

**Al-Rawi [1985]** measured creep under shrinkage cracking conditions, he assumed creep to be the difference between tensile strain capacity and elastic tensile strain capacity.

The basic effect of creep on cracking tendency of concrete is to delay or prevent the cracking of concrete through delaying the shrinkage strain propagation and preventing the stage at which tensile strain capacity is exceeded.

Many factors which affect creep such as; relative humidity, w/c ratio, mix proportion, age of concrete, ambient temperature, influence of shrinkage is in a similar manner. Hence, their influence on cracking tendency is very little.

### ***2-8-2 Tensile Strain Capacity:***

The tensile strain capacity is the maximum strain that concrete can sustain in tension before cracking occurs. In this case, it is defined, as the total shrinkage required to induce cracking. According to **Al-Rawi [1985]** the amount of strain which is instantly relived due to elastic recovery of restraint concrete upon cracking is termed elastic tensile strain capacity.

The tensile strain capacity is normally assessed by testing the concrete in flexure or indirect tension.

**Houk et al. [1970]** reported that the tensile strain capacity is higher in flexural test 230 microstrain than indirect tension 130 microstrain, possibly because of the "weakest link" theory. They also reported that the rate of straining significantly affected the tensile strain capacity.

**Lee and Lamb [1976]** reported that tensile strain capacity as measured in a direct test at a progressively increasing stress is of the order of 100 microstrain while the order of 200 microstrain will be reached if the rate of strain is controlled.

**B.S Code 5337 [1976]** doubles the typical strain capacity for mature concrete (from 100 to 200 microstrain) to allow for age and creep.

There is an agreement that the tensile strain capacity of concrete depends on the mix proportion, in particular on the shape, type and size of the aggregate. It was found by **Lee and Lamb [1976]** that the crushed type aggregate has the beneficial effect on the strain capacity of concrete by increasing from  $95 \times 10^{-6}$  for rounded to  $139 \times 10^{-6}$  for the crushed type.

**Al-Rawi [1985]** reported an increase in the elastic tensile strain capacity of mortar beam by 16 % when curing time was increased from 3 to 7 days.

**AL-Abdaly [2007]** found that the tensile strain capacity for series 70MPa higher than of series 50 MPa about ( 42 – 51 ) % .

### **2-8-3 Elastic Tensile Strain Capacity:**

The elastic tensile strain capacity is the amount of strain that is instantly relived due to the elastic recovery of restrained concrete upon cracking. It is defined by **Al-Rawi [1985]** as the observed free contraction of concrete at the onset of cracking. Elastic tensile strain capacity does not include creep strain, which was lost during tension built up, before the occurrence of cracking. It is considered as a basic concrete property that may be used for calculation of shrinkage crack width [**Al-Rawi, 1986**].

From the researchs it was found that the elastic tensile strain capacity for 7- days was 98, and 99 microstrain and for 28-days 135, and 145 by[*Al-Mashhedi, 1989 ,Kadhum , 2003*] respectively.

*AL-Abdaly*[2007] found that the elastic tensile strain capacity at date of cracking decrease with increasing in RHA content about ( 22 - 47 ) %, elastic tensile strain calculated by indirect method ( splitting tensile strength divided by the modulus of elasticity ) higher than these of direct method ( elastic tensile strain capacity of concrete was obtained directly by measuring the immediate movement after cracking of concrete on end restrained beams ) by about ( 3 - 65 ) % .

### **2-9 Cracking Age:**

Cracking time is the time required for first crack to occur. The first crack is dependent on the some factors that influence the cracking tendency of concrete (degree of restraint , shrinkage , creep , tensile strain capacity ..etc) (*AL- Rawi, 1985*) .(*AL –Nassar,2002*) studied the effect of admixtures (plasticizer, superplasticizer ,BVD and waterproofing) on the shrinkage of concrete ,steel I shaped molds 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 end (two end, three end and four restrained end),the slab with dimensions (2250×2250×100)mm, with (W/C = 0.48) ratio , the reinforced slab concrete exposed to drying for two months , the results

showed the 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 casted and exposed to laboratory conditions ,four levels of  $\text{SO}_3$  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 crack time then it decreases with the higher value of sulphate than the optimum .

Also(*Al-khafaji 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 casted and exposed to laboratory conditions, local admixtures of finely divided mineral admixtures were used which are limestone dust, silica flour and bentonite, each types was additions in three levels by weight of cement , for limestone dust (1.5, 3,4) % , (2.5 ,5,8) % for silica flour admixtures and (2.5,4.5,6)% for bentonite admixtures ,the results showed the cracking time is decreased with the addition of different contents admixtures (limestone dust, silica flour, bentonite)to concrete mixes.

(*Kubba,2007*)investigated the effect of different end restraint with addition of superplastizer and fillers( pigment limestone powder and metakolin) on free and restrained drying shrinkage of self compact(SCC)and normal concrete for plain and reinforced concrete , also studied the effect of pigment powder on reinforced SCC plates(free plates ,two end restrained plate,three end restrained plate, four end restrained plate),end restraint steel molds having channel section were used and four plate, were casted and exposed to laboratory condition. The investigation showed that two , three and four end restrained plates cracked early in the first day after casting ,crack width is difference along the plate length , crack which appeared at the free

edge have maximum crack width at the edge level and decreases to zero towards the center of the plate .

It is the time required for the shrinkage induced strains to build up and exceed the tensile strain capacity of concrete. The age of the first crack and the cracking sequence is dependent on the some factors that influence the cracking tendency of concrete (i.e., degree of restraint, shrinkage, creep, tensile strain capacity etc.). Thus, cracking age, as stated by **Al Rawi[1985]** could be used as in index to assess the possibility of cracking of various mixes. He also reported that the cracking time depends on both shrinkage and tensile strain capacity. It increases with a decrease in water/cement ratio, an increase in a normal curing time and the use of smaller size and amount of the crushed coarse aggregate.

The cracking time decreases with increased restraint, since cracking time (16, 21) days for four and two end restrained reinforced concrete slabs respectively [ **Kadhum , 2003**].

**AL-Abdaly[2007]** found that the 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.

# C HAPTER THREE 3

## *EXPERIMENTAL WORK*

### ***3-1 Introduction:***

This chapter describes the experimental work, which consists of two parts. The first part deals with the materials used in the production of the specimens, mix proportions and the methods of testing.

The second part has been carried out to study the shrinkage behavior of reinforced and unreinforced high strength concrete walls restrained from movement at their bases for different L/H ratios and for two seasons(Winter and Summer).

### ***3-2 Materials:***

#### **3-2-1 Cement:**

Alqaseem ordinary Portland cement (OPC) manufactured in Saudia Arabia was used in this work. The chemical composition and physical properties of the cement are shown in Table (3-1) and (3-2) respectively. The results conform to the Iraqi specification No .5/1984. The tests were carried out in the testing cement laboratory of Babylon University.

**Table ( 3 – 1 ):** Chemical analysis of the ordinary Portland cement used.

| <u><b>Oxide</b></u>                | <u><b>%</b></u> | <u><b>I.Q.S. 5: 1984 Limits</b></u> |
|------------------------------------|-----------------|-------------------------------------|
| CaO                                | 62.23           | —                                   |
| SiO <sub>2</sub>                   | 20.78           | —                                   |
| Al <sub>2</sub> O <sub>3</sub>     | 4.9             | —                                   |
| Fe <sub>2</sub> O <sub>3</sub>     | 3.84            | —                                   |
| MgO                                | 1.57            | < 5.0                               |
| SO <sub>3</sub>                    | 2.50            | < 2.8                               |
| Loss On Ignition (L.O.I)           | 1.7             | < 4.0                               |
| Lime Saturation Factor (L.S.F)     | 0.89            | 0.66 - 1.02                         |
| Insoluble residue (I.R)            | 1.23            | < 1.5 %                             |
| Free lime (F.L)                    | 1.0             |                                     |
| <b>Total</b>                       | <b>99.40</b>    |                                     |
| <u><b>Compound Composition</b></u> | <u><b>%</b></u> | <u><b>I.Q.S. 5: 1984 Limits</b></u> |
| C <sub>3</sub> S                   | 45.8            | —                                   |
| C <sub>2</sub> S                   | 25.02           | —                                   |
| C <sub>3</sub> A                   | 6.49            | —                                   |
| C <sub>4</sub> AF                  | 11.68           | —                                   |

**Table ( 3-2 ) :** Physical Properties of ordinary Portland cement used.

| <u><b>Physical Properties</b></u>       | <u><b>Test Results</b></u> | <u><b>I.Q.S.5: 1984 Limits</b></u> |
|-----------------------------------------|----------------------------|------------------------------------|
| Fineness , Blaine , cm <sup>2</sup> /gm | 3250                       | >2300                              |
| Setting Time :                          |                            |                                    |
| Initial hrs ; min                       | 2;05                       | ≥45 min                            |
| Final hrs ; min                         | 3;45                       | ≤10hrs                             |
| Compressive Strength MPa                |                            |                                    |
| 3-days                                  | 25.5                       | ≥15                                |
| 7-days                                  | 34                         | ≥23                                |

**3-2-2 Fine Aggregate:**

The fine aggregate used was Al-Ekhaider natural sand. The grading, physical and chemical properties of fine aggregate is shown in Table (3-3) and (3-4) respectively. The results conform to Iraqi specification No . 45 /1984. The tests were carried out in the construction materials laboratory of Babylon University.

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

| <b><u>Sieve size ( mm)</u></b> | <b><u>Passing %</u></b> | <b><u>I.Q.S.45:1984 Limits Zone ( 2)</u></b> |
|--------------------------------|-------------------------|----------------------------------------------|
| 10                             | 100                     | 100                                          |
| 5                              | 96                      | 90-100                                       |
| 2.36                           | 79                      | 75-100                                       |
| 1.18                           | 65                      | 55-90                                        |
| 0.6                            | 45                      | 35-59                                        |
| 0.3                            | 18                      | 8-30                                         |
| 0.15                           | 7                       | 0-10                                         |

Fineness modulus of fine aggregate ( FM ) = 2.90

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

| <b><u>Physical Properties</u></b>     | <b><u>Test Results</u></b> | <b><u>L.Q.S.45 1984 Limits</u></b> |
|---------------------------------------|----------------------------|------------------------------------|
| Specific gravity                      | 2.65                       | —                                  |
| Absorption %                          | 1.6                        | —                                  |
| Sulfate content ( SO <sub>3</sub> ) % | 0.25                       | ≤ 0.5                              |
| Clay %                                | 2.3                        | ≤3.0                               |

**3-2-3 Coarse Aggregate :**

Crushed gravel obtained from AL-Nebai source was used. The maximum coarse aggregate size was chosen to be 14 mm . Table (3-5) shows the grading of coarse aggregate which conforms to the I.Q.S NO.45 / 1984 specification .Table (3-6) illustrates the specific gravity ,sulfate content and absorption of coarse aggregate. The tests were carried out in the construction materials laboratory of Babylon University.

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

| <b><u>Sieve size (mm)</u></b> | <b><u>Passing %</u></b> | <b><u>I.Q.S.45:1984 Limits Zone ( 3)</u></b> |
|-------------------------------|-------------------------|----------------------------------------------|
| 14                            | 100                     | 90-100                                       |
| 10                            | 65                      | 50-85                                        |
| 5                             | 10                      | 0-10                                         |
| 2.36                          | 0                       | 0                                            |

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

| <b><u>Physical Properties</u></b>    | <b><u>Test Results</u></b> | <b><u>I.Q. S NO 45 : 1984 Limits</u></b> |
|--------------------------------------|----------------------------|------------------------------------------|
| Specific gravity ( S.G)              | 2.6                        | —                                        |
| Absorption %                         | 1.1                        | —                                        |
| Sulfate Content (SO <sub>3</sub> ) % | 0.06                       | ≤0.1                                     |
| Clay %                               | 0.5                        | ≤1.0                                     |

**3-2-4 Superplasticizer:**

High range water reducing agent( HRWR) called Glenium 51( Sulfonated melamine – formaldehyde ) was used in this work. This superplasticizer is conformed to ASTM C-494 classified as type F, it has acceleration effect on the HSC .The dosage for Glenium 51 was 0.8 liters per 100 kg of cement . The typical properties are shown in Table ( 3 – 7).

The superplasticizer was added to the mixing water before the addition to the mixes.

**Table ( 3 – 7 ) :** Typical Properties of Super plasticizer (\*).

|                         |                                        |
|-------------------------|----------------------------------------|
| <b>Form</b>             | Viscous liquid                         |
| <b>Colour</b>           | Light brown                            |
| <b>Relative density</b> | 1.1 @ 20°C                             |
| <b>pH</b>               | 6.6                                    |
| <b>Viscosity</b>        | 128 cps @20°C                          |
| <b>Transport</b>        | Not classified as dangerous            |
| <b>Labelling</b>        | No hazard label required               |
| <b>Dosage</b>           | 0.5 to 0.8 liters per 100 kg of cement |

( \* ) From the catalog of manufacture.

**3-2-5 Water:**

Tap water was used for mixing and curing of concrete through out the investigation .

**3-2-6 Reinforcement:**

Deformed steel bars of 10 mm diameter were used as reinforcement and dowels. The average yield strength of three samples is 425 MPa. This test was carried out in the materials laboratory of Babylon University.

### 3-3 Mix Design and Proportion:

One target design strength of 80 MPa was used in this work according to [ACI Committee 211, 1998]. The proportions of the concrete mix are summarized in Table (3-8).

**Table (3-8): Mix Proportions**

| <u>Weight proportion</u>      | <u>W/C</u>   | <u>Mix proportion kg/m<sup>3</sup></u> | <u>Superplasti-</u> | <u>Slump</u> |
|-------------------------------|--------------|----------------------------------------|---------------------|--------------|
| <u>Cement : Sand : Gravel</u> | <u>ratio</u> | <u>Cement : Sand : Gravel : Water</u>  | <u>cizer(liter/</u> | <u>mm</u>    |
|                               |              |                                        | <u>100kg cement</u> |              |
| 1.0 : 1.2 : 1.8               | 0.3          | 550 : 660 : 990 : 165                  | 0.8                 | 200          |

### 3-4 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 specimens for static modulus of elasticity test .
- 4-(100 ×100×400) mm prisms to obtain concrete specimens for flexural tensile strength test.
- 5- End restraint molds having a channel section with a necking at the mid-span with the dimensions shown in Figure ( 3 - 1 ). This mould was used to determine free shrinkage and elastic tensile strain capacity.

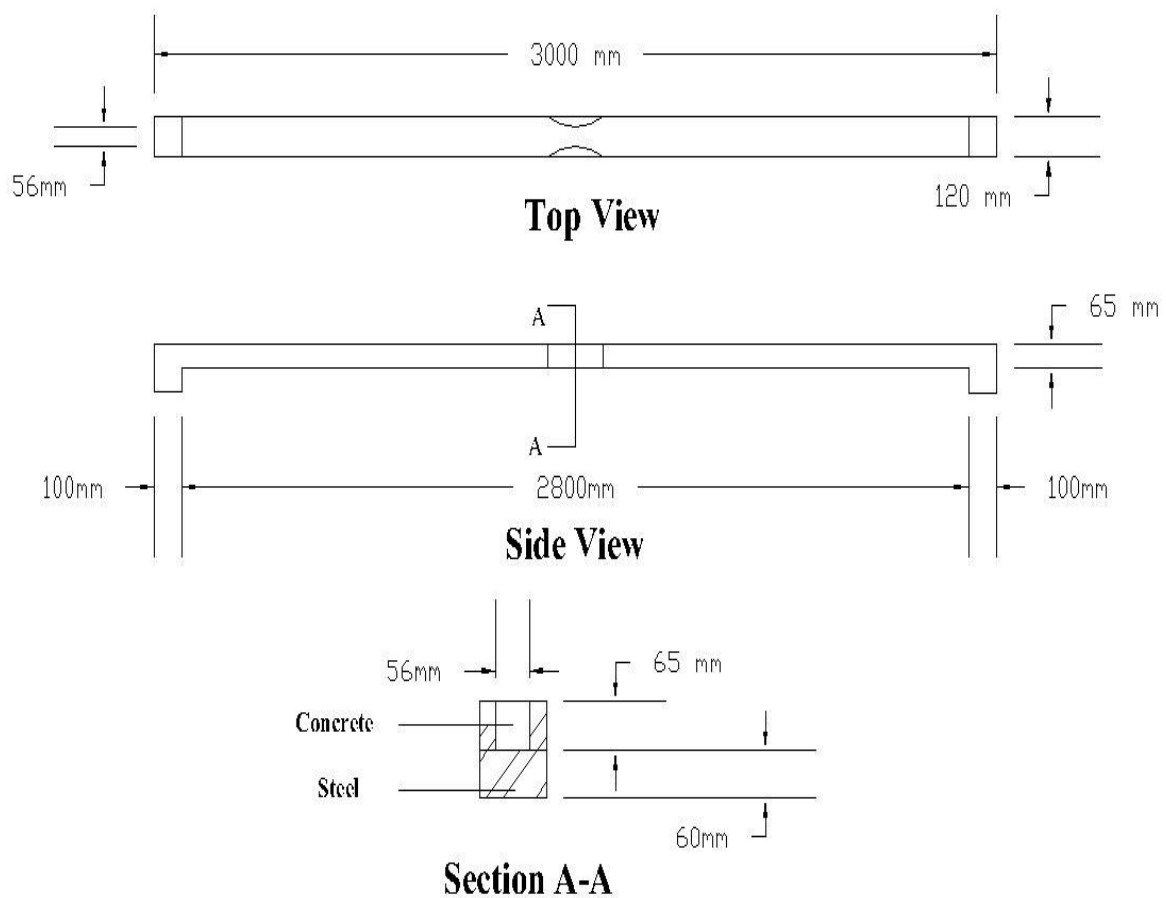


**Figure ( 3 - 1 ) :** Schematic Diagram of the End Restrained Beam.

### **3-5 Testing Fresh and Hardened Concrete**

#### **3-5-1 Slump Test:**

The workability of the fresh concrete mixes was measured by the slump test before casting these mixes in their molds. This test was conducted according to ASTM C143-04 .

#### **3-5-2 Compressive Strength Test:**

For the hardened concrete, the compressive strength test was carried out according to [BS. 1881: part 116:1983 , using a digital testing machine of

2000 kN maximum capacity. Three cubes (100 mm) were tested for each mix at each age for the determination of compressive strength. The load was applied approximately at a constant rate (3kN/sec). The test was accomplished in concrete laboratory of AL-Kufa University.

### **3-5-3 Splitting Tensile strength:**

The splitting tensile strength was determined according to ASTM C496-04 specification. Each splitting tensile strength value was the average of two specimens. The test was done in the concrete laboratory of AL-Kufa University.

### **3-5-4 Flexural Strength Test:**

Two point load flexure test was performed on two (100x100x400 mm) prisms with a span of 330 mm using the machine meeting the requirement of ASTM C293-04 . The test was done in concrete laboratory of AL-Kufa University.

### **3-5-5 Static Modulus of Elasticity Test:**

The static modulus of elasticity was determined according to ASTM C469-04 specification. The cylinders were casted into three layers, then remolded 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. 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 were recorded to determine the modulus of elasticity, the results are plotted on a graph, in which the slope of the chord is the modulus of

elasticity. The test was accomplished in concrete laboratory of AL-Kufa University.

The recorded results were the average readings of two cylinders. The modulus of elasticity was calculated from the following relation:

$$E_c = (S_2 - S_1) / \epsilon_2 - 0.00005.$$

Where:

$E_c$  : the static elastic modulus of concrete, MPa .

$S_2$  : stress corresponding to 40% of ultimate load, MPa.

$S_1$  : stress corresponding to a longitudinal strain (0.00005), MPa.

$\epsilon_2$  : longitudinal strain produced by stress  $S_2$ .

### 3-6 Steel Rigid Restraining Base:

The base restraint was provided by two welded steel channel sections (127×64mm) have properties as shown in Table(3.9).

**Table(3.9)** The properties of channel section used.

| Serial size<br>(mm) | Mass per<br>metre<br>(kg/m) | Depth<br>(mm) | Width<br>(mm) | Thickness<br>(mm) |        | Area of<br>Section<br>(mm <sup>2</sup> ) |
|---------------------|-----------------------------|---------------|---------------|-------------------|--------|------------------------------------------|
|                     |                             |               |               | Web               | Flange |                                          |
| 127×64              | 14.9                        | 127           | 63.5          | 6.4               | 9.2    | 1898                                     |

The roughness of base was taken from welding reinforcement bars along the top surface of base. The dowels were welded on the top surface of base at 200mm distance. Fig.(3-2) shows the base, dowels and roughness of the restraining base.

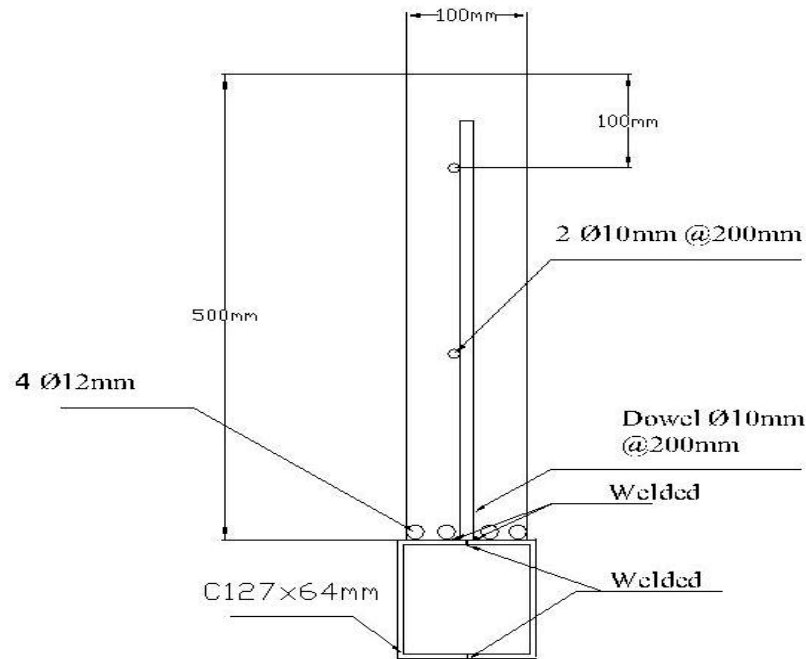
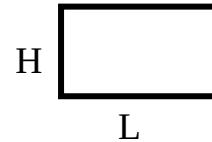


**Fig.( 3 - 2)** The schematic diagram of the steel restraining base with the welded dowels and roughness of the base.

### 3-7 Wall Notation:

Two sets of concrete walls were casted .The first set was casted in Winter season which contains four unreinforced concrete walls(WP1,0 ,WP2,0 ,WP4,0 and WP6,0) and four reinforced concrete walls(WP1,2 ,WP2,2 ,WP4,2 and WP6,2) reinforced with deformed steel bars(10mm diameter) distributed horizontally with a steel ratio ( $\rho=0.2\%$ ) and vertical steel ratio ( $\rho=1\%$ ). Whilst the second set was casted in Summer season which contains two reinforced concrete walls(SP4,2 and SP6,2) reinforced with deformed steel bars(10mm diameter) distributed horizontally with a steel ratio ( $\rho=0.2\%$ ) and vertical steel ratio( $\rho=1\%$ ). The characteristics of the eight reduced scale reinforced and unreinforced high strength concrete walls are shown in Table ( 3.10).

**Table(3.10)** Walls characteristics.

| Wall Notation | Length mm | Height mm | L/H Ratio | Horizontal Reinf.( $\rho\%$ ) |
|---------------|-----------|-----------|-----------|-------------------------------|
| WP1,2*        | 1000      | 1000      | 1         | 0.2                           |
| WP1,0         | 1000      | 1000      | 1         | 0                             |
| WP2,2         | 2000      | 1000      | 2         | 0.2                           |
| WP2,0         | 2000      | 1000      | 2         | 0                             |
| WP4,2         | 2000      | 500       | 4         | 0.2                           |
| WP4,0         | 2000      | 500       | 4         | 0                             |
| WP6,2         | 3000      | 500       | 6         | 0.2                           |
| WP6,0         | 3000      | 500       | 6         | 0                             |
| SP4,2         | 2000      | 500       | 4         | 0.2                           |
| SP6,2         | 3000      | 500       | 6         | 0.2                           |

WP1,2\* W:Winter set  
P:Panel.  
1: L/H ratio.  
,2: Reinforcement ratio(0.2%).

### ***3-8 Casting Procedure of the Walls:***

#### **3-8-1 Mixing and Casting of Walls:**

The concrete was mixed using an electrical drum type mixer with a maximum capacity of 0.1m<sup>3</sup>. The interior surface of the mixer was cleaned and moistened before placing the materials.

Materials were put in the pan of the mixer, firstly coarse and fine aggregates were mixed together with amount of mixing water(1/4 mixing water without superplasticizer) for 1 minute. Meanwhile the superplasticizer was mixed with the remaining water (3/4 of mixing water) . Half of cement

and half of mixing water (with superplasticizer) were added as mixing proceeded for 4 minutes to make a homogeneous mix. Finally, the remaining (cement and water) were added to the mix as mixing proceeds for other 2 minutes. The total mixing time was about 7 min.

Then the concrete mix was casted into the plywood formwork of the wall as can be seen in Plate (3-1). The concrete mixes were casted in three layers, compaction was achieved using a portable mechanical vibrator.



***Plate (3-1) Wall formwork.***

### **3-8-2 Curing and Exposure to Drying:**

To prevent plastic shrinkage cracking due to rapid evaporation from the upper surface of the wall. Wetted hessian sheets and polythene sheets were used to cover the walls after lifting the formwork (about 24 hrs from the casting as can be seen in Plate (3-2). The hessian sheets were wetted every day during 7 days of curing period.

The same curing procedure was adopted to cubes, cylinders and beams.



**Plate (3-2)** Curing Method

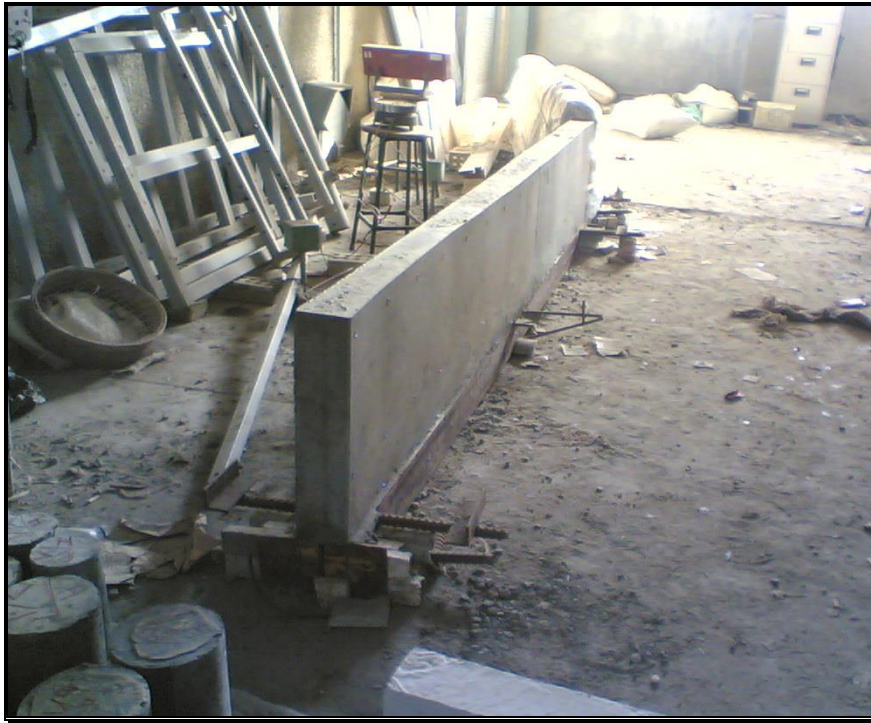
Temperature and relative humidity variation in the laboratory were measured and recorded during the period of work. Table (3-11) gives these ambient conditions.

**Table(3-11):-** Temperature and relative humidity during measurement period (2008). \*

| <b>Month</b>     | <b>Temperature ( av. )<br/>°C</b> | <b>Relative humidity<br/>( av . ) %</b> |
|------------------|-----------------------------------|-----------------------------------------|
| <b>Febreuary</b> | 27                                | 44                                      |
| <b>March</b>     | 30                                | 48                                      |
| <b>April</b>     | 35.5                              | 36                                      |
| <b>May</b>       | 36.5                              | 31                                      |
| <b>June</b>      | 39.5                              | 26                                      |
| <b>July</b>      | 48                                | 28                                      |
| <b>August</b>    | 45.5                              | 31                                      |
| <b>September</b> | 41.8                              | 35                                      |

(\*) From Center of Weather Forecast in Babylon.

The drying process of the Winter walls (set no.1) proceeds during the period from February to the end of April inside the construction materials laboratory of Babylon University. As the drying period of the Summer walls (set no.2) it proceeds from June to the end of August outside this laboratory. See plate (3-3) and plate(3-4).



**Plate(3-3):** Base restrained wall in  $L/H=6$  (Winter set).



**Plate(3-4):** The base restrained walls(Winter set).



**Plate(3-5):**The base restrained walls(Summer set).

### **3-9 Strain and Crack Width Measurements:**

Surface strain measurements were carried out using demec points inserted in 3 rows ( 25, 150 and 450 mm distance from the base) for the walls which have 500mm height and 4 rows(25,150,500and 900 mm distance from the base) for the walls which have 1000mm height. The spacing between demec points in the same row was 200 mm apart. The demec points were positioned in the walls and bases after the curing.

An extensometer, with an accuracy of (0.002 mm/division) was used to measure strain in the panels of the wall (the panel is the distance between two consecutive demec points in the same row, panel length = 200 mm). The measurement devices are shown in Plate (3-6).



Extensometer

Portable dial microscope

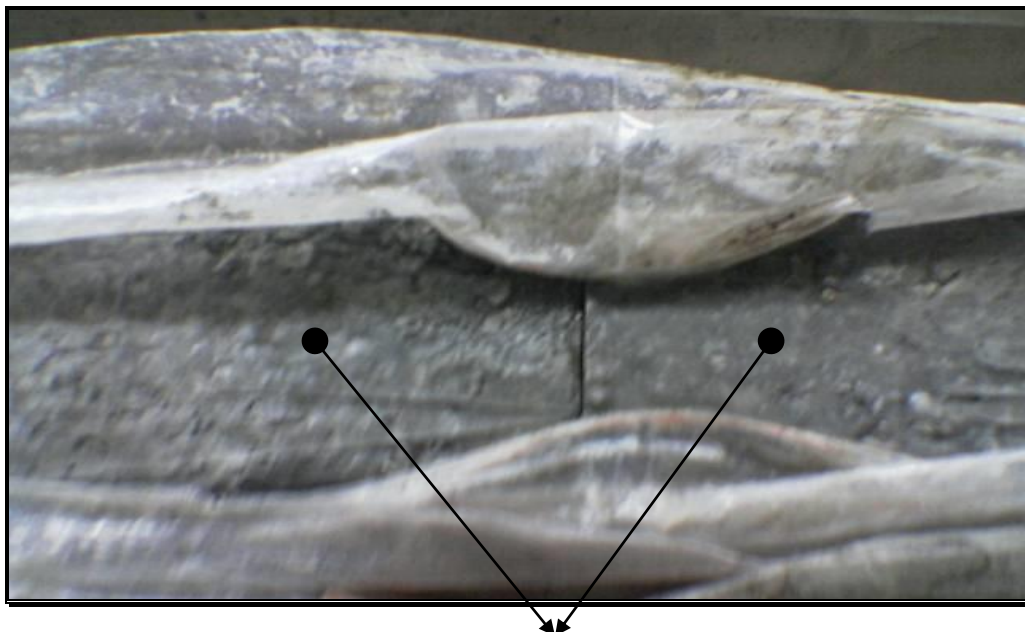
**Plate (3-6)** Measurement devices.

The measurements were registered early in the morning after curing period(7 days) by using the extensometer every 3 days as shown in Plate(3-7) through the first week. Then, measurements were taken at an average of 15 days for a total period of about 90 days.

**Plate (3-7):** The strain measurement.

**3-10 Free Volume Change Test:**

The free shrinkage of concrete was determined by fixing demec points at both side of the gap for beams with artificial crack ( opening ) in the web of channel-shaped steel mold. Daily measurements were taken for the widening of the artificial crack in the middle of the beam, till little or no movement could be recorded , see Plate (3-8).



Two demec points

**Plate(3-8)** Free shrinkage test

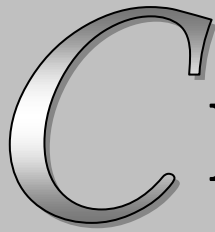
**3-11 Elastic Tensile Strain Capacity Test:**

The elastic tensile strain capacity is the amount of strain that is instantly relieved due to the elastic recovery of restrained concrete upon cracking. It is defined as the observed free contraction of concrete at the onset of cracking .

Elastic tensile strain capacity of concrete was measured using two methods:

The first is a direct method using the same  channel-shaped steel mold. Plain beams were casted and allowed to shrink. Soon after the first crack occurred, the amount of strain which was relieved as a result of elastic recovery of restrained concrete, was measured between plug fixed points together with crack width measurement by microscope. This crack opening was assumed to represent elastic tensile strain capacity of concrete.

The second is an indirect method by dividing the flexural strength of concrete by the modulus of elasticity of concrete. According to *[ACI Committee 224,2001]*, this method produces significantly inaccurate values of tensile strain capacity due to the non-linearity of stress distribution across the beam, which causes the modulus of rupture to be 20-40 percent higher than the true tensile strength of concrete.



# CHAPTER FOUR 4

## *RESULTS AND DISCUSSION*

### **4-1 Introduction:**

In this chapter, the results obtained from the experimental work are analyzed and studied. These results include the following characteristics of the concrete (compressive strength, tensile strength, modulus of elasticity, volume changes ) in two seasons; Winter(set no.1) and Summer (set no.2).

### **4-2 Compressive Strength:**

The compressive strength test results of the concrete specimens (cubes 100 mm) were tested at age ( 7, 28,60 and 90 ) days, the results of compressive strength for two sets (1) and (2) are given in Table ( 4 - 1).

**Table(4-1)** The compressive strength results.

| Set No. | Compressive Strength (MPa) |         |         |         |
|---------|----------------------------|---------|---------|---------|
|         | 7 days                     | 28 days | 60 days | 90 days |
| 1       | 50                         | 74      | 80      | 87      |
| 2       | 63                         | 82      | 86      | 90      |

It can be seen that set (2) gave compressive strength higher than set (1) in all ages and the strength development rate is faster in set (2) whereas the latter age compressive strength for the two sets are approximately close. This can be attributed to the action of temperature which increases the early strengths during placing and setting [Neville,1995].

### 4-3 Splitting Tensile Strength:

The splitting tensile strength test results of the concrete specimens (cylinder 100×200 mm) were tested at age ( 7, 28,60 and 90 ) days, the results of two sets (1) and (2)are given in Table ( 4 - 2).

**Table(4-2)** The splitting tensile strength results.

| Set No. | Splitting Tensile Strength (MPa) |         |         |         |
|---------|----------------------------------|---------|---------|---------|
|         | 7 days                           | 28 days | 60 days | 90 days |
| 1       | 5.5                              | 6.2     | 6.5     | 6.7     |
| 2       | 5.9                              | 6.6     | 6.75    | 7.0     |

### 4-4 Flexural Strength:

The flexural tensile strength test results of the concrete specimens (prism 100×100×400 mm) were tested at age ( 7, 28,60 and 90 ) days, the results of two sets (1) and (2) are given in Table ( 4 - 3).

**Table(4-3)** The flexural strength results.

| Set No. | Flexural Strength (MPa) |         |         |         |
|---------|-------------------------|---------|---------|---------|
|         | 7 days                  | 28 days | 60 days | 90 days |
| 1       | 6                       | 6.7     | 7.5     | 7.8     |
| 2       | 7.2                     | 7.9     | 8.2     | 8.6     |

#### 4-5 Static Modulus of Elasticity:

The static modulus of elasticity of the concrete specimens (cylinder 150×300 mm) was tested at age ( 7, 28,60 and 90 ) days, the results of two sets (1) and (2) are given in Table ( 4 - 4).

**Table(4-4)** The static modulus of elasticity results.

| Set No. | Static Modulus of Elasticity (GPa) |         |         |         |
|---------|------------------------------------|---------|---------|---------|
|         | 7 days                             | 28 days | 60 days | 90 days |
| 1       | 32                                 | 35      | 39      | 40      |
| 2       | 38                                 | 41      | 42      | 44      |

#### 4-6 Restrained Shrinkage of The Walls:

The measurement of longitudinal movement of the walls was carried out to investigate the effect of base restraint of the walls on the shrinkage behavior. These measurements were conducted for a drying period of 90 days for the first set( in Winter) and 90 days for the second set( in Summer). Figures (4.1 to 4.10) and Table (4-5) show the restrained shrinkage strain development

with age at 3 rows of ( 25, 150 and 450 mm distance from the base) for the walls which have 500mm height and 4 rows of (25,150,500and 950 mm distance from the base) for the walls which have 1000mm height.

**Table(4-5)** The maximum restrained shrinkage strains and loss of restraint of the walls.

| Wall Notation | L/H Ratio | Max. Shrinkage Strain |          | Loss of Restraint L.O.R (Microstrain) |
|---------------|-----------|-----------------------|----------|---------------------------------------|
|               |           | Value (Microstrain)   | Location |                                       |
| WP1,2         | 1         | 350                   | Top row  | 70                                    |
| WP1,0         | 1         | 400                   | =        | 60                                    |
| WP2,2         | 2         | 300                   | =        | 77                                    |
| WP2,0         | 2         | 350                   | =        | 72                                    |
| WP4,2         | 4         | 200                   | =        | 75                                    |
| WP4,0         | 4         | 250                   | =        | 68                                    |
| WP6,2         | 6         | 250                   | =        | 83                                    |
| WP6,0         | 6         | 350                   | =        | 78                                    |
| SP4,2         | 4         | 750                   | =        | 85                                    |
| SP6,2         | 6         | 650                   | =        | 91                                    |

From Table (4-5), it can be noticed the followings:

- 1- The maximum shrinkage strains were found to be decreasing with increasing in L/H ratios in Summer and Winter sets. This may be attributed to that the action of restraint is increasing with the increasing of L/H ratio, this agrees with *Kheder [1997]* .
- 2- The maximum shrinkage strain was found in all the casted walls at the upper row(95cm from base for 1m wall height and 45cm from base for 0.5m wall height). This is due to the base restraint decreasing toward the upper row.
- 3- There is increasing in maximum shrinkage values in Summer about (66%, 46%) of the maximum shrinkage values in Winter for wall with length/height

ratio (4,6) respectively. This can be attributed to that the shrinkage in hot weather is more intensive than in cold weather.

4- The values of loss of restraint (L.O.R) in the present work (60-90 microstrain) was found to be higher than its values in the [*Al-Mashhdi, 1989*] and [*Kheder,1986*] which may indicate stiffer restraint.

5- The loss of restraint (L.O.R) in the Summer set was found to be higher than in Winter set. This is due to high shrinkage rates which occur in Summer set.

From Figures (4-1 to 4-10), it can be noticed the followings:

1- The shrinkage strains in reinforced walls were found to be less than in unreinforced walls. This is due to the action of internal restraint which comes from the reinforcement.

2- The shrinkage strains in the exterior columns were found to be more than the shrinkage strains in the interior columns of most walls. This may be attributed to the large surface area which is subjected to the drying conditions. That is clearly in *Fig.(4.9B)*.

3- There is a continuous increasing in the shrinkage strains with age. This gives an indication to no crack occurrence.

4- There are certain values in the row 2.5cm from the base in all walls although its position is very close to the base. This is may be due to some slippage which occurs between base and wall and due to loss of restraint which comes from contraction of the base.

5-The shrinkage strain curves are more divergent in unreinforced walls than in the reinforced walls. This is because of the absence of reinforcement which distribute the strains along the wall uniformly.

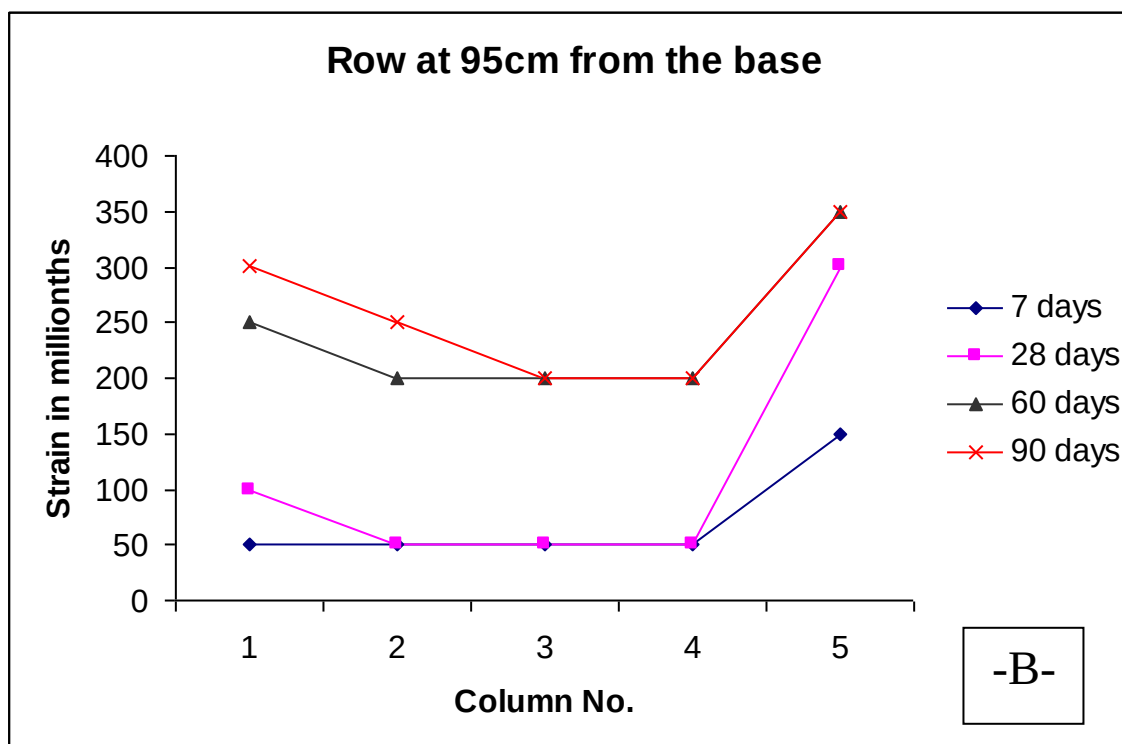
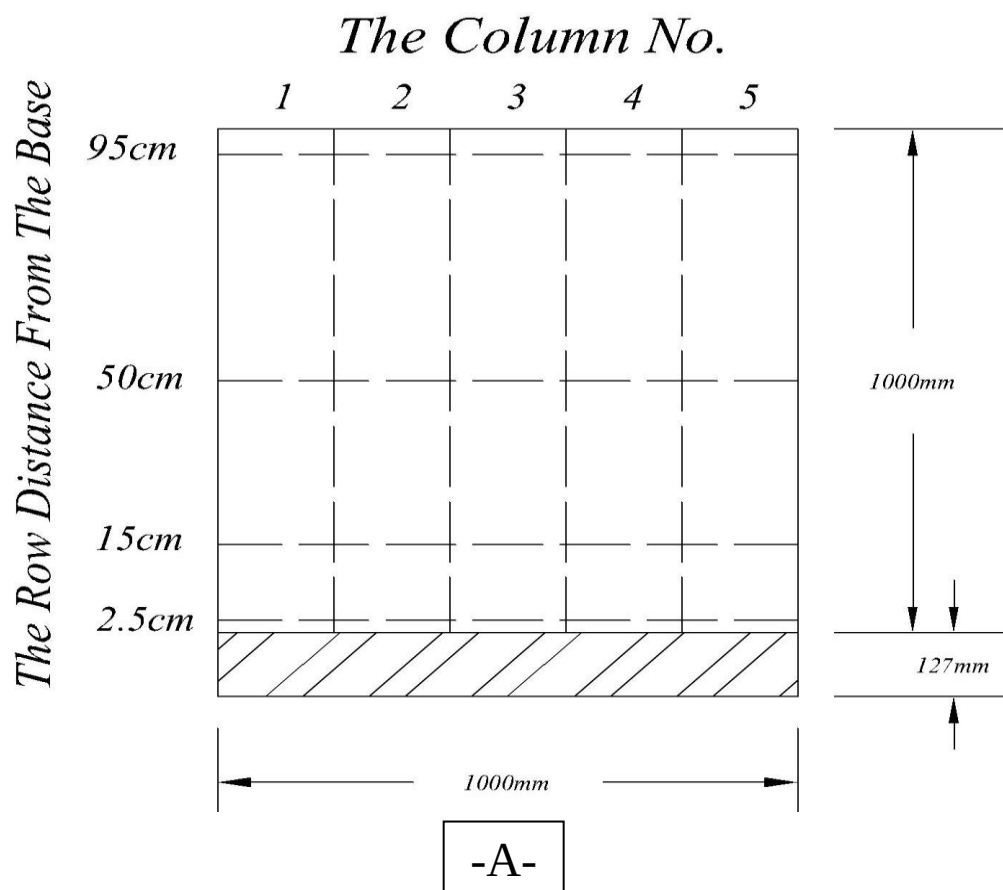
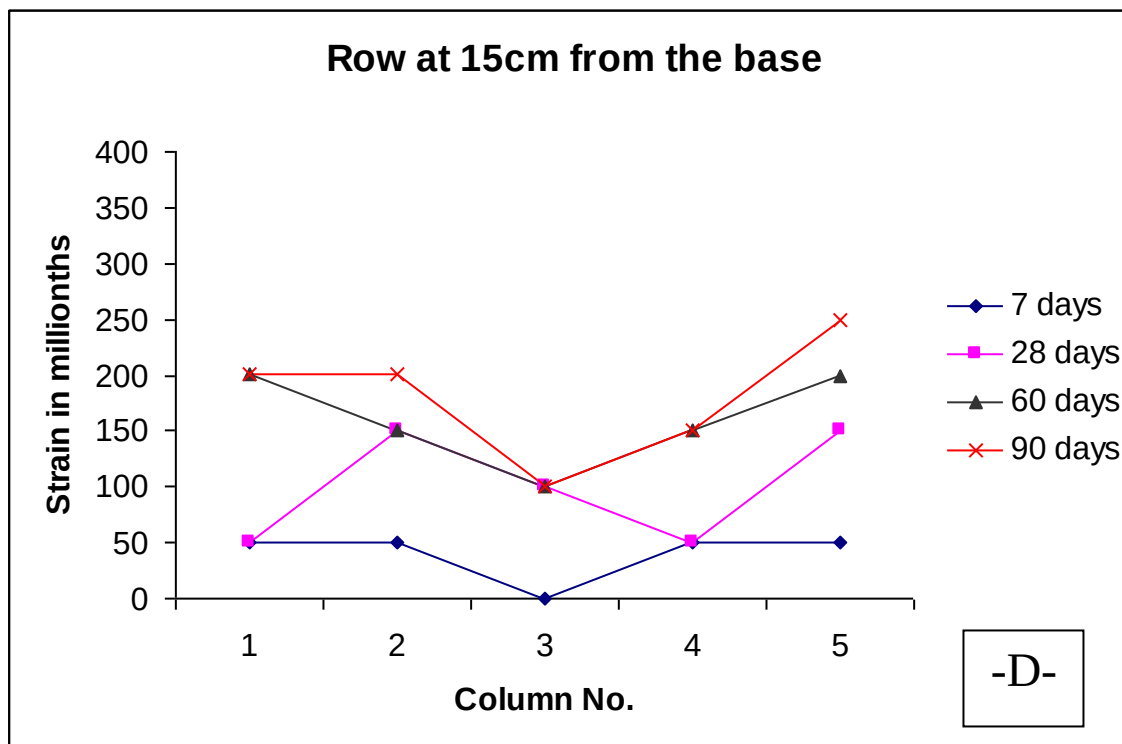
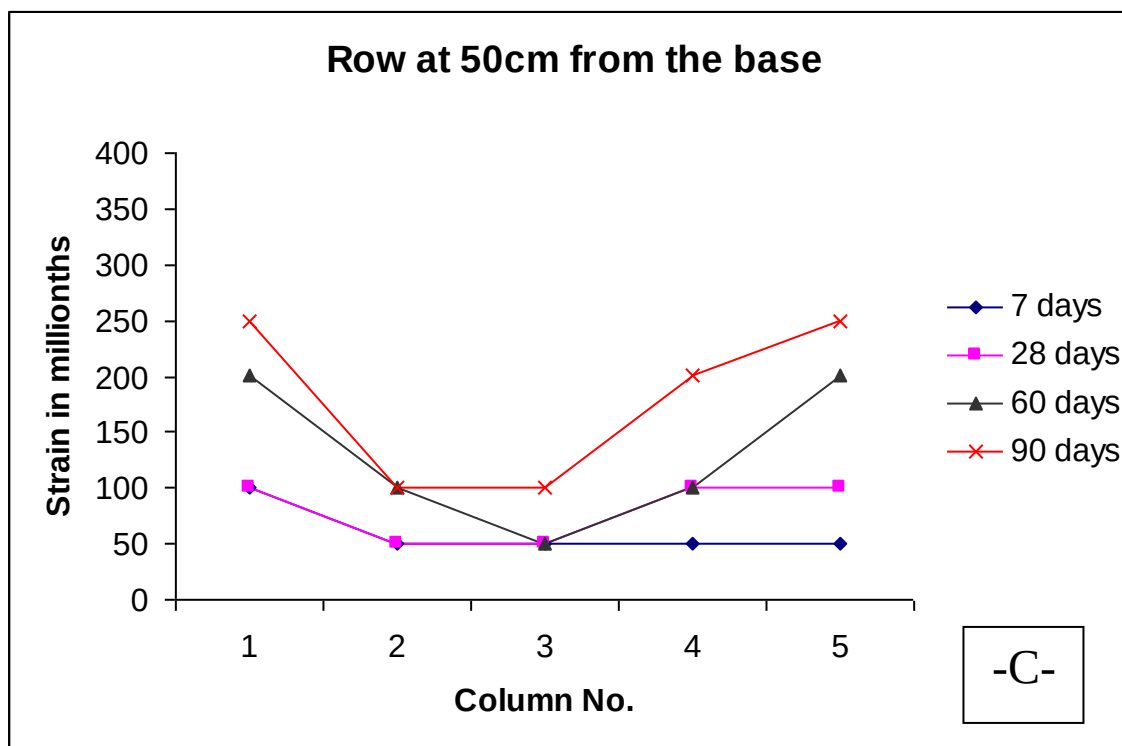


**Fig.(4.1)** Strain development history for reinforced HSC base restrained wall(WP1,2).

*Fig.(4.1 ) Continued.*

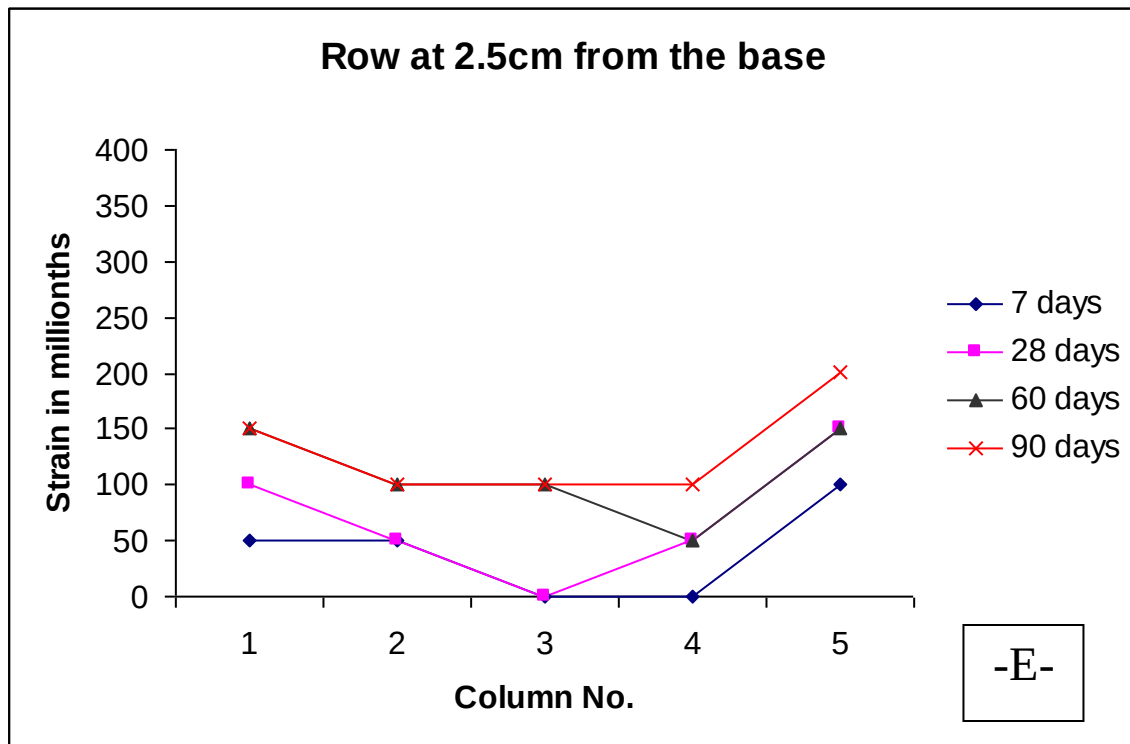


**Fig.(4.1 )** Continued.

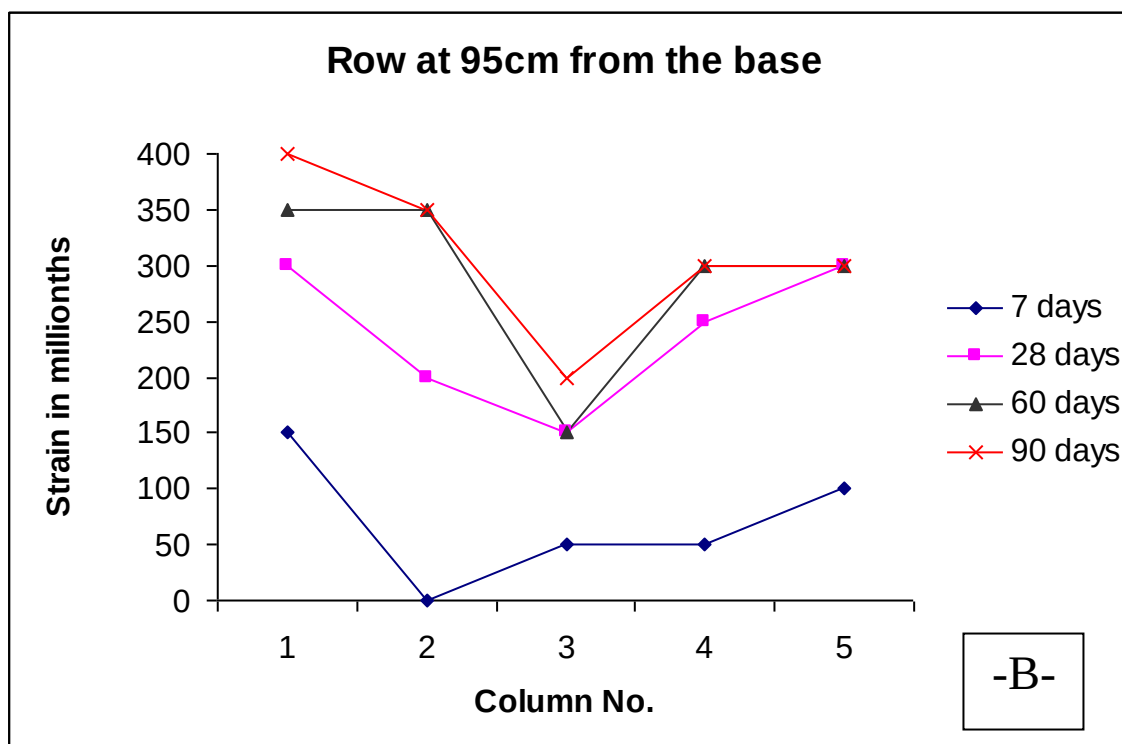
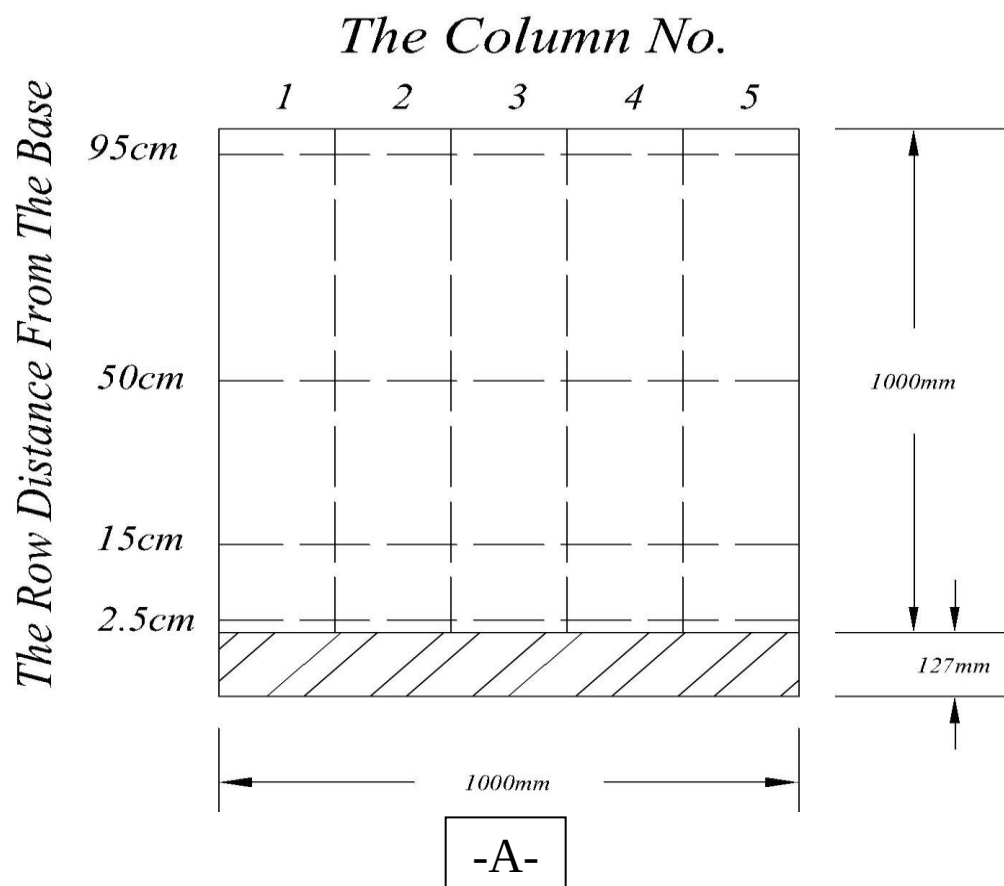
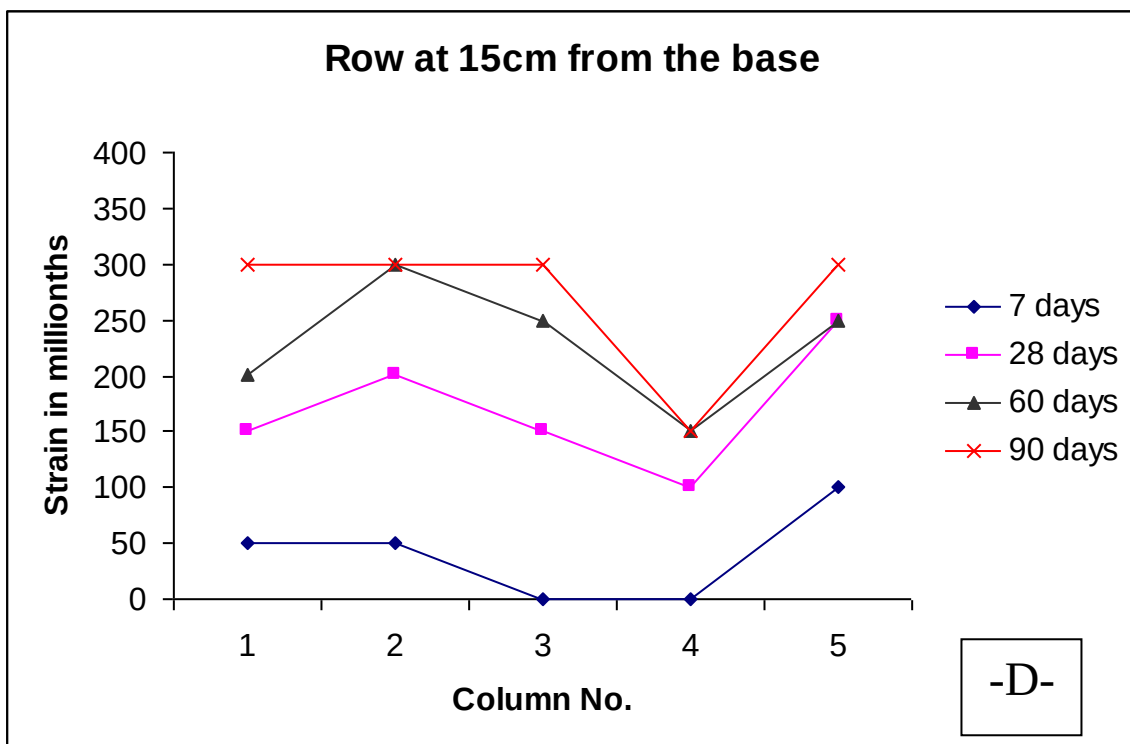
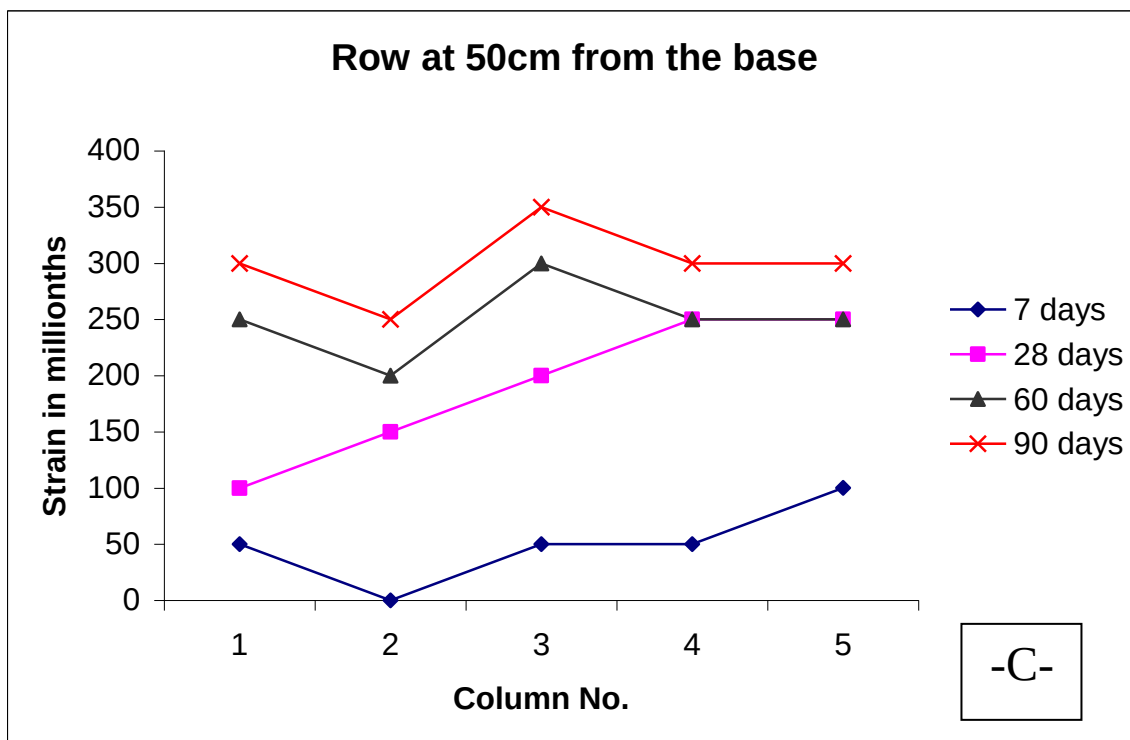


**Fig.(4.2)** Strain development history for unreinforced HSC base restrained wall(WP1,0).

*Fig.(4.2)* Continued.

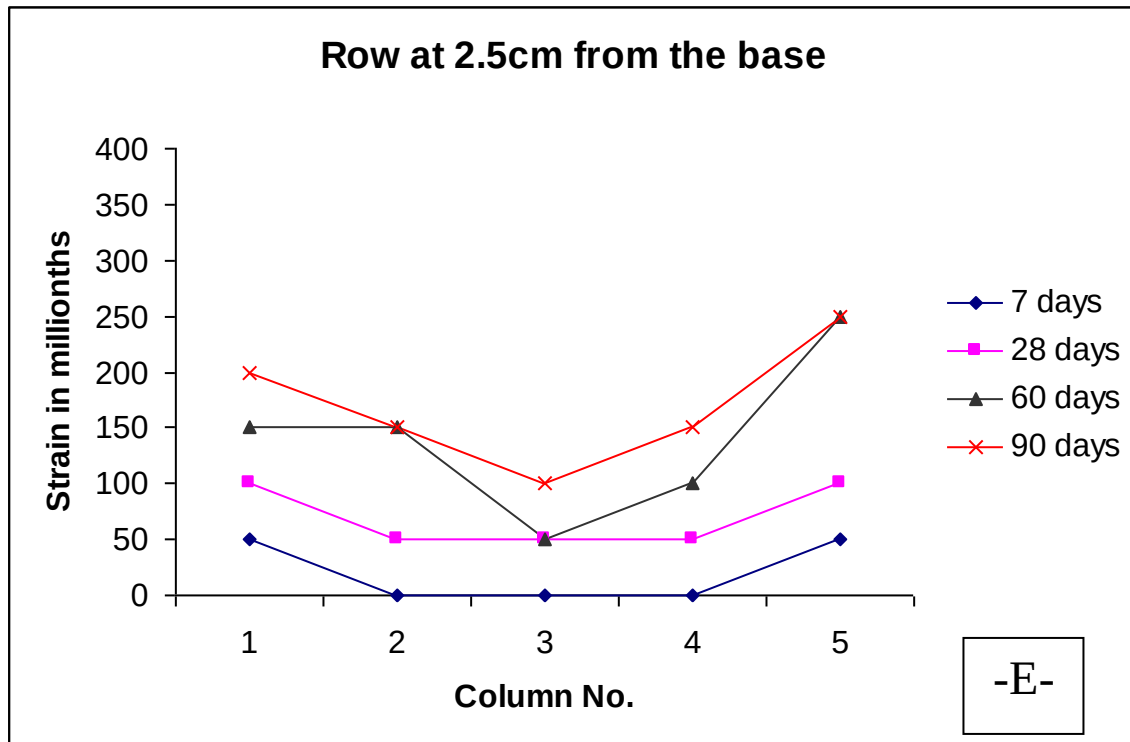


**Fig.(4.2)** Continued.

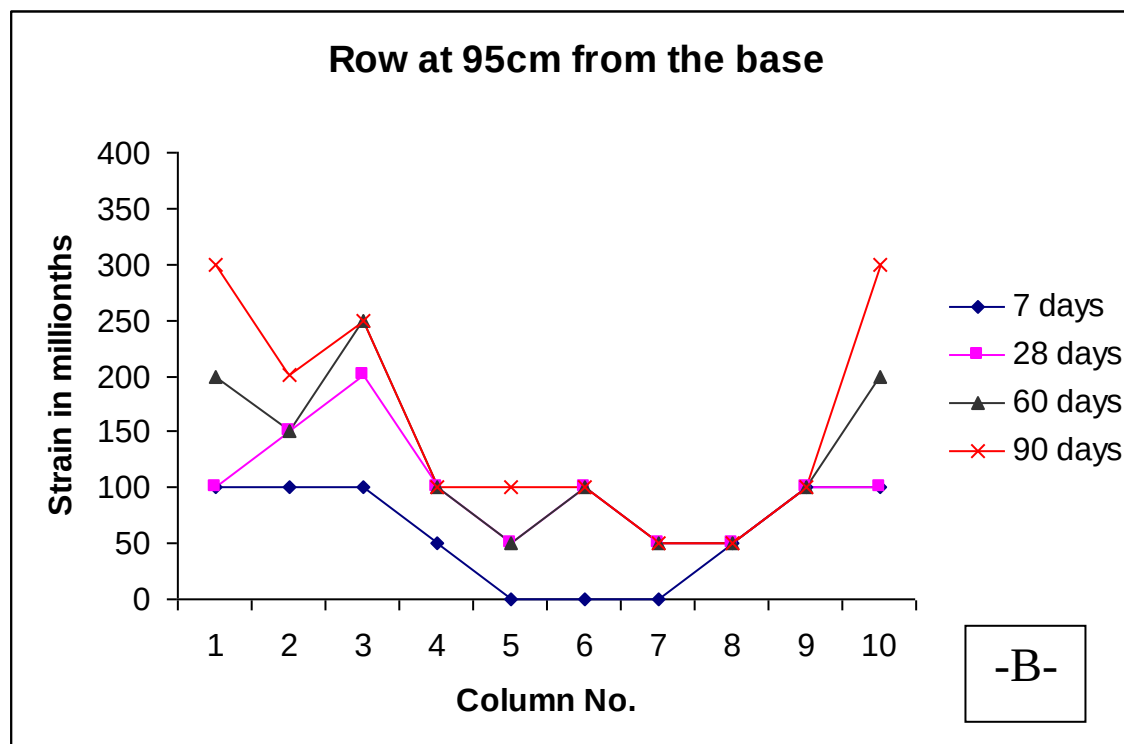
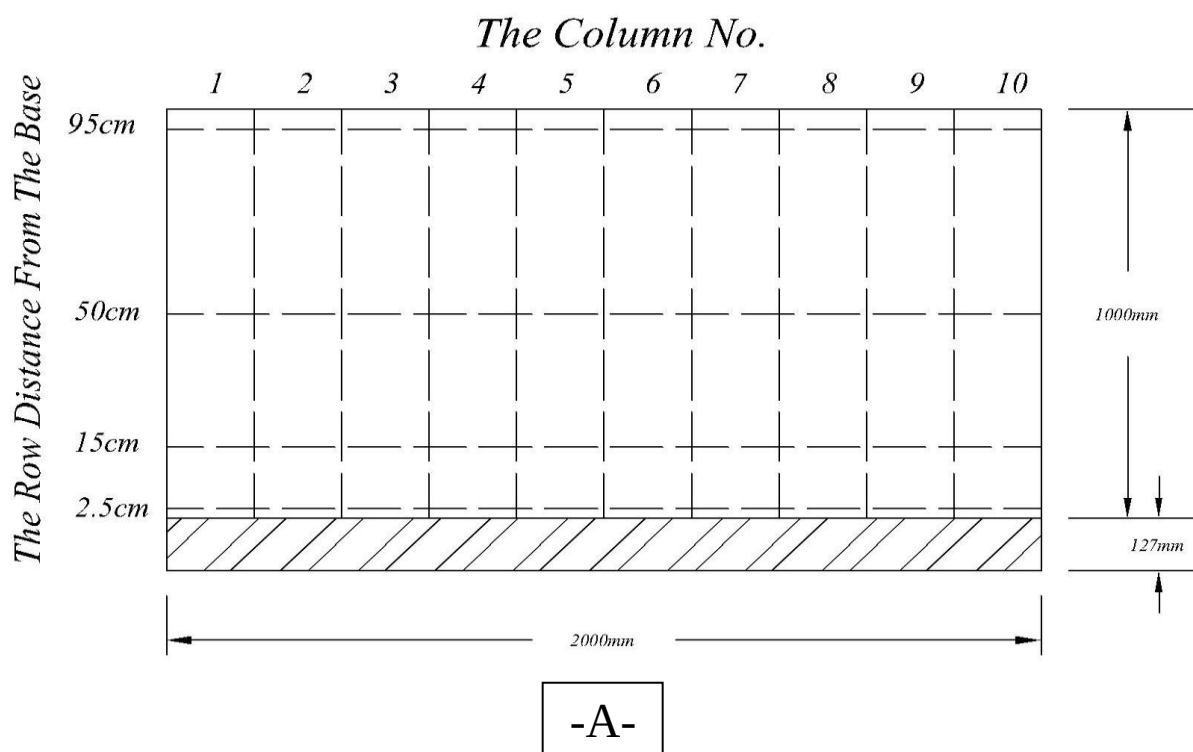


**Fig.(4.3)** Strain development history for reinforced HSC base restrained wall(WP2,2).

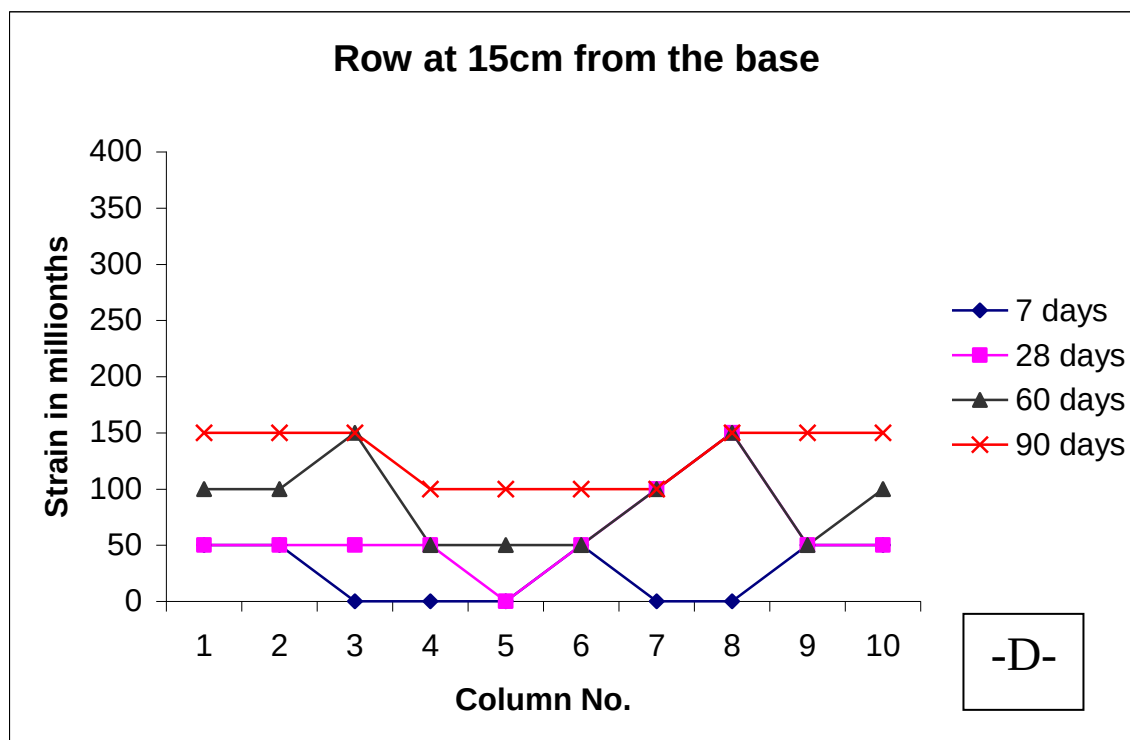
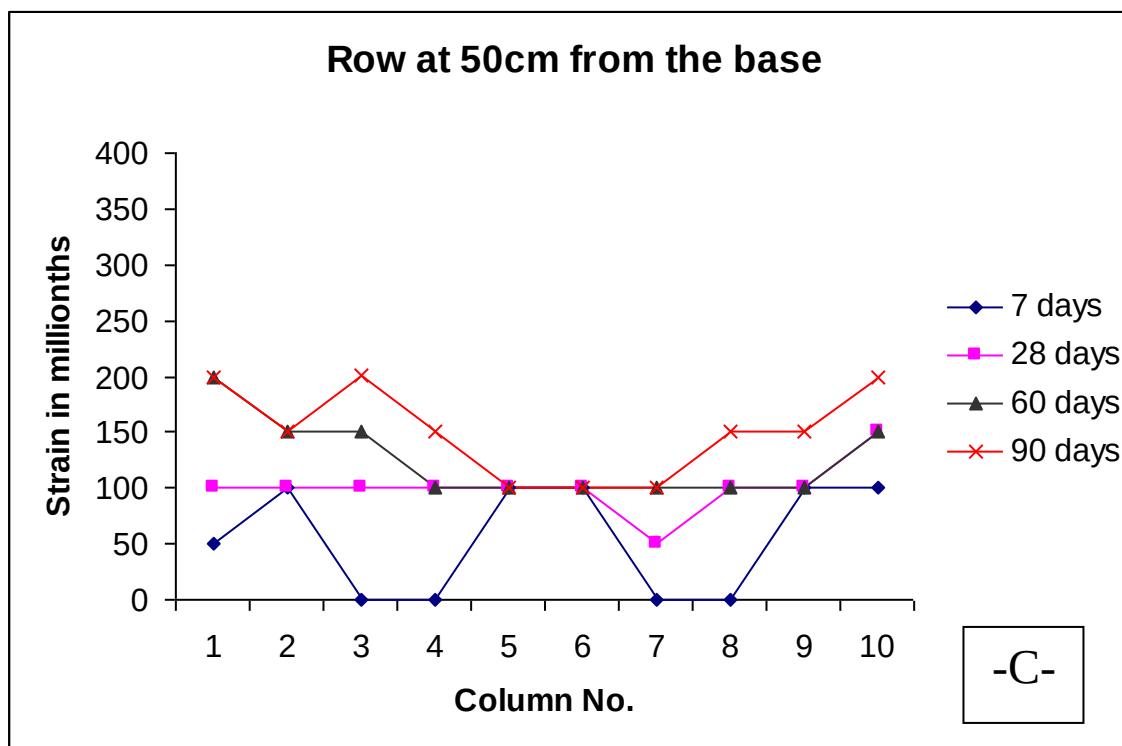


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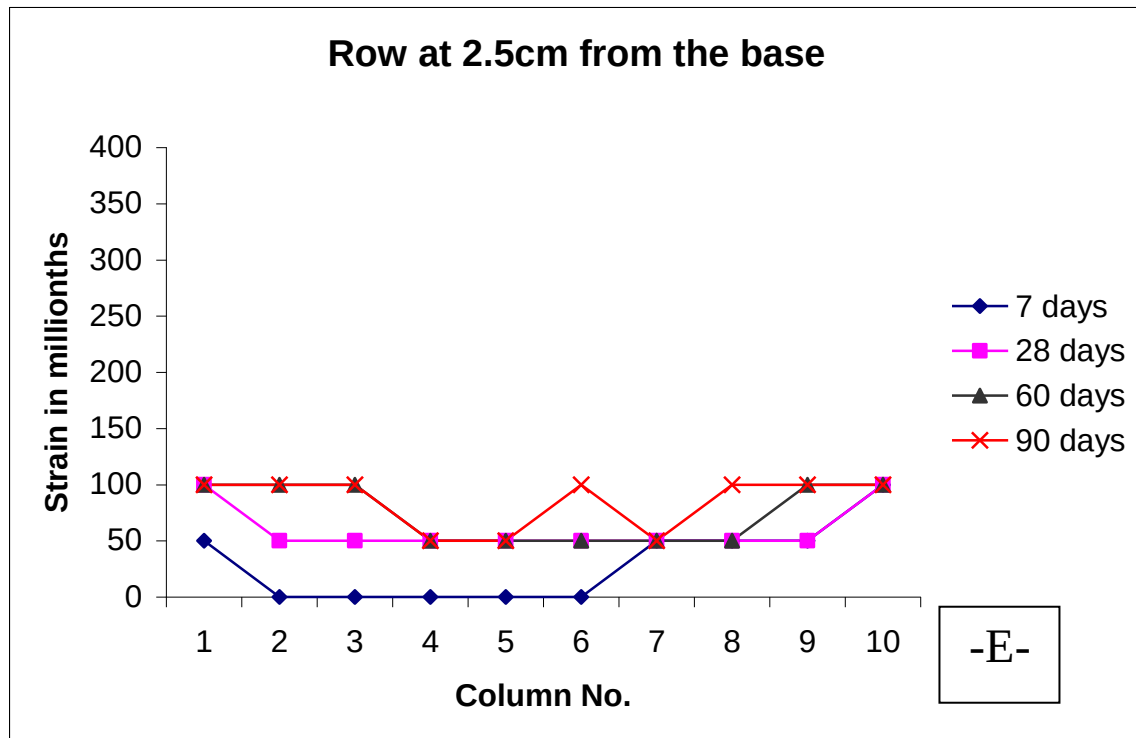


*Fig.(4.3)* Continued.

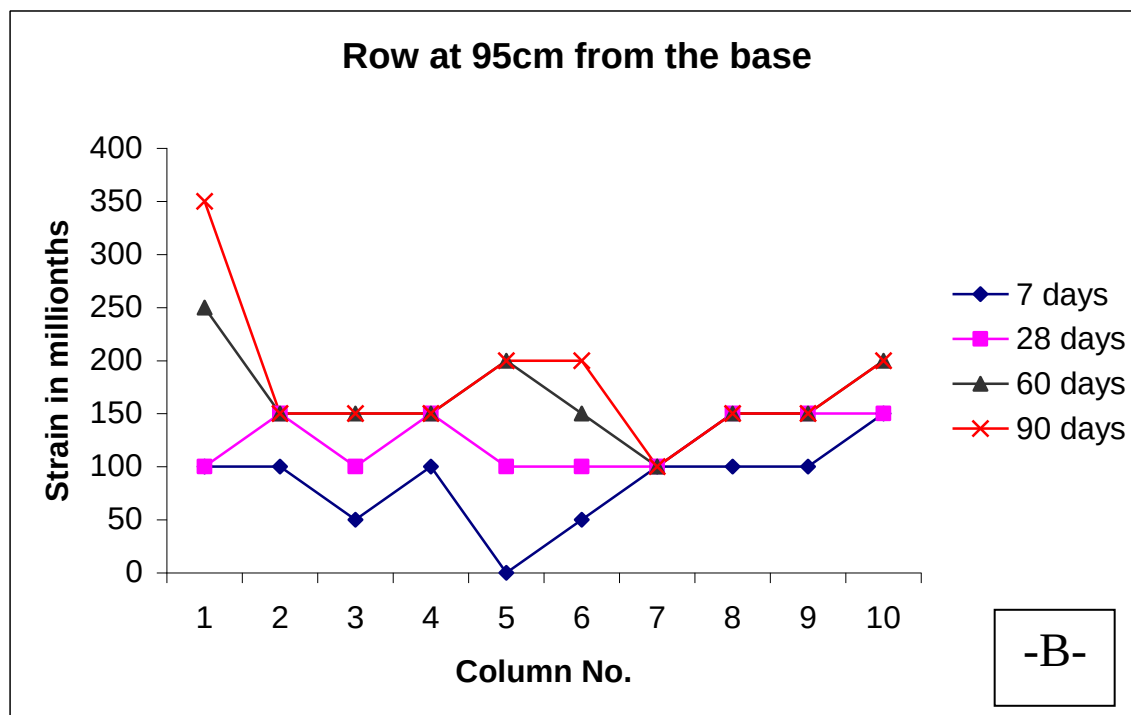
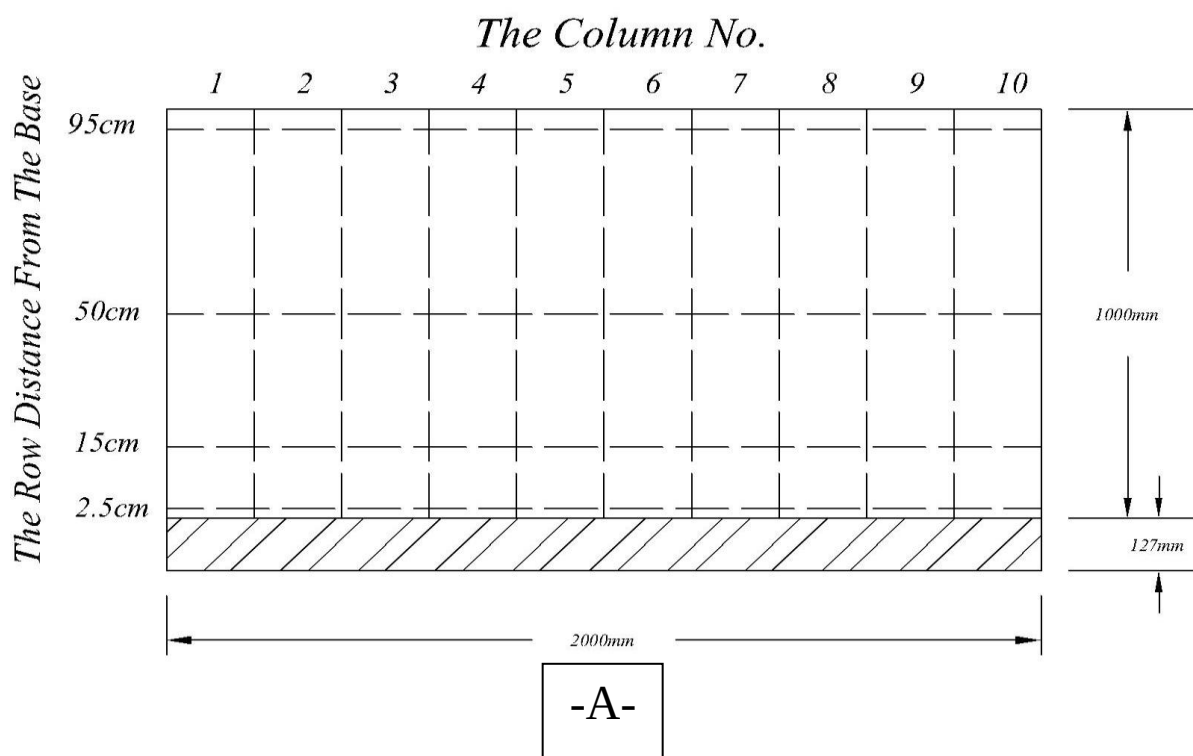


**Fig.(4.4)** Strain development history for unreinforced HSC base restrained wall(WP2,0).

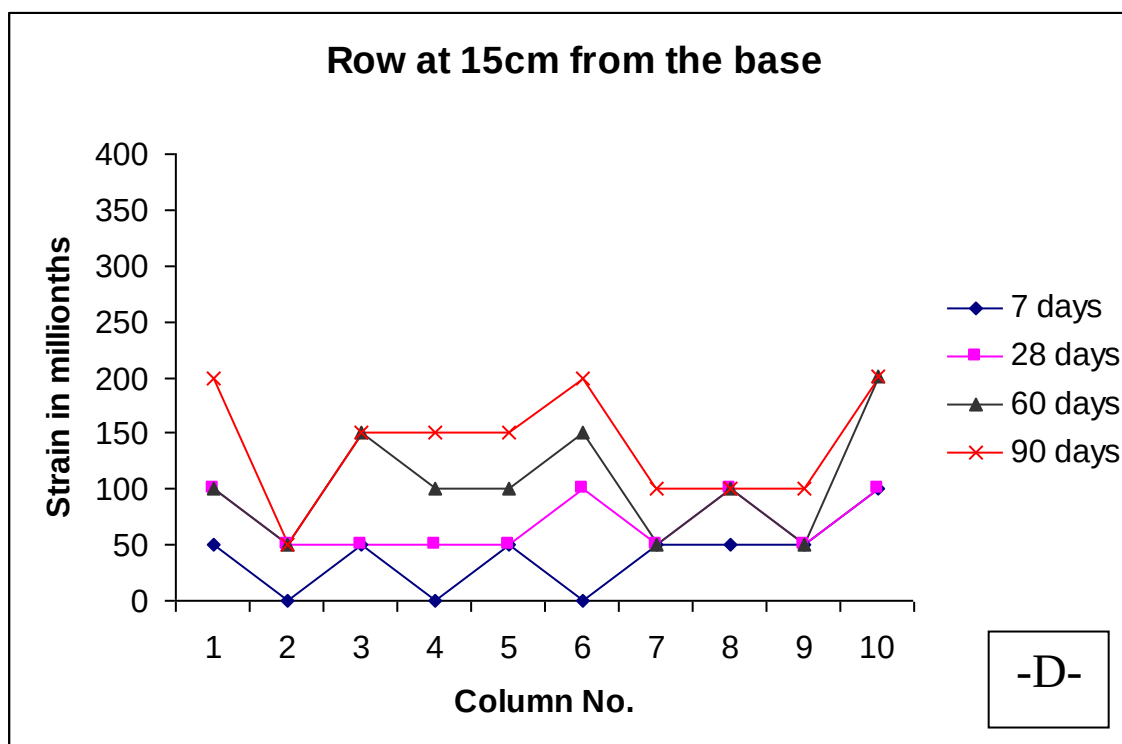
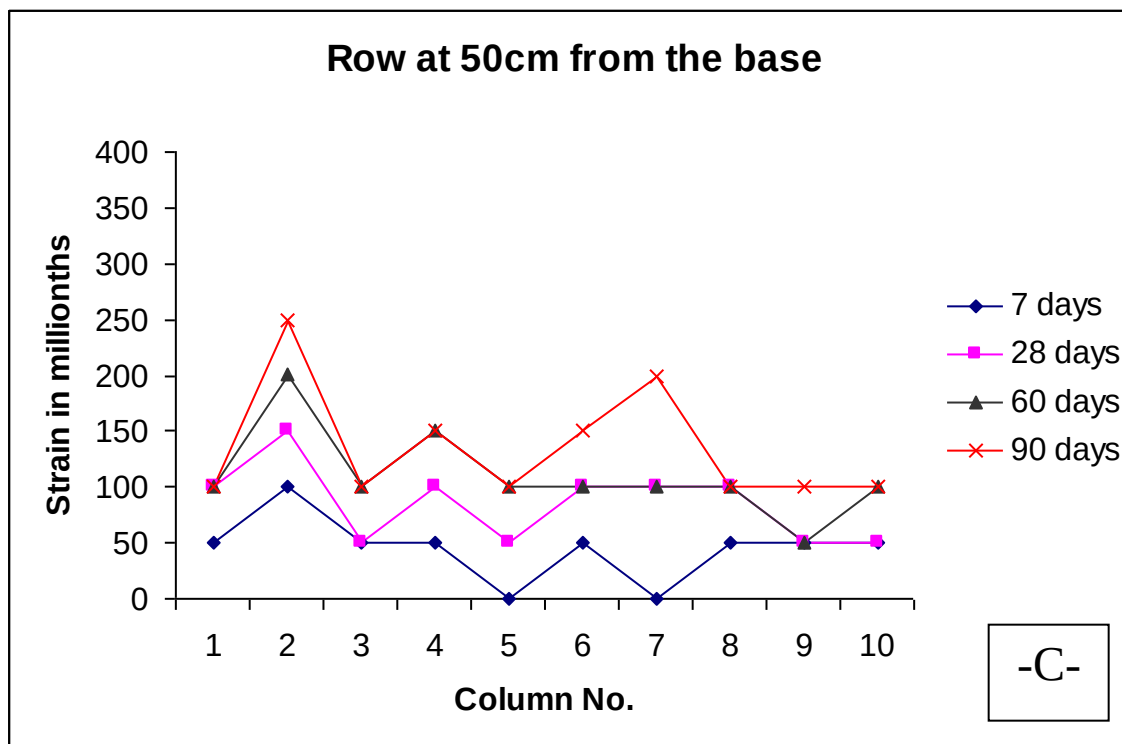


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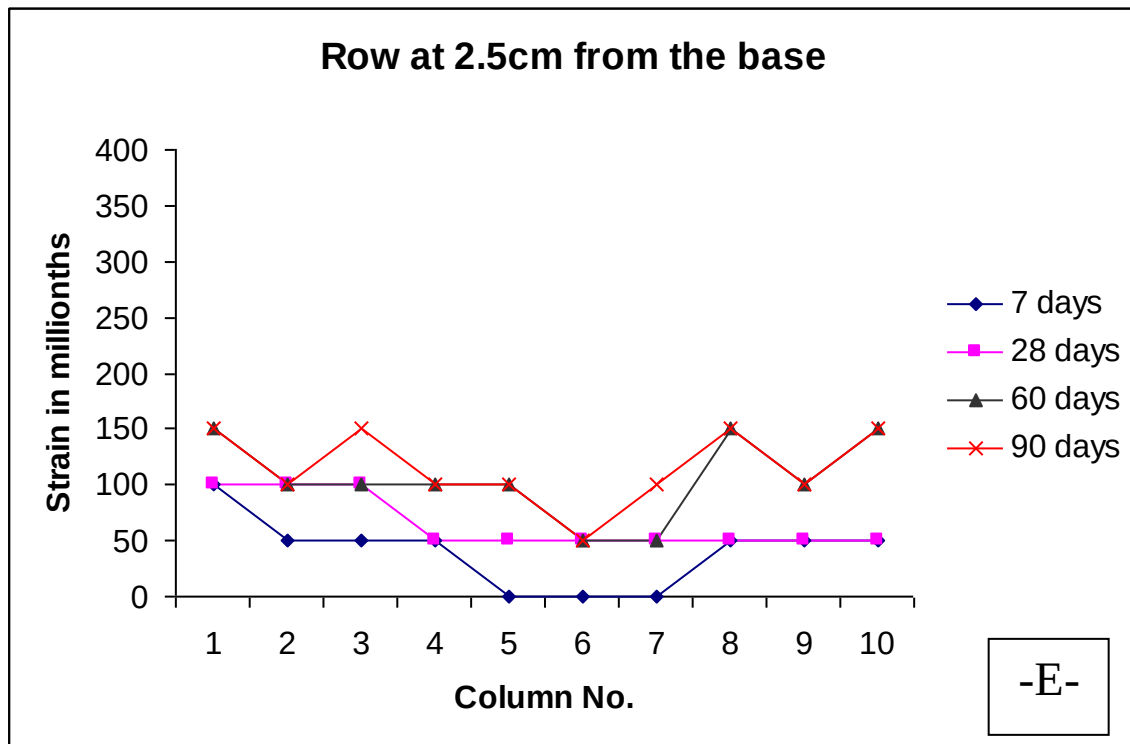


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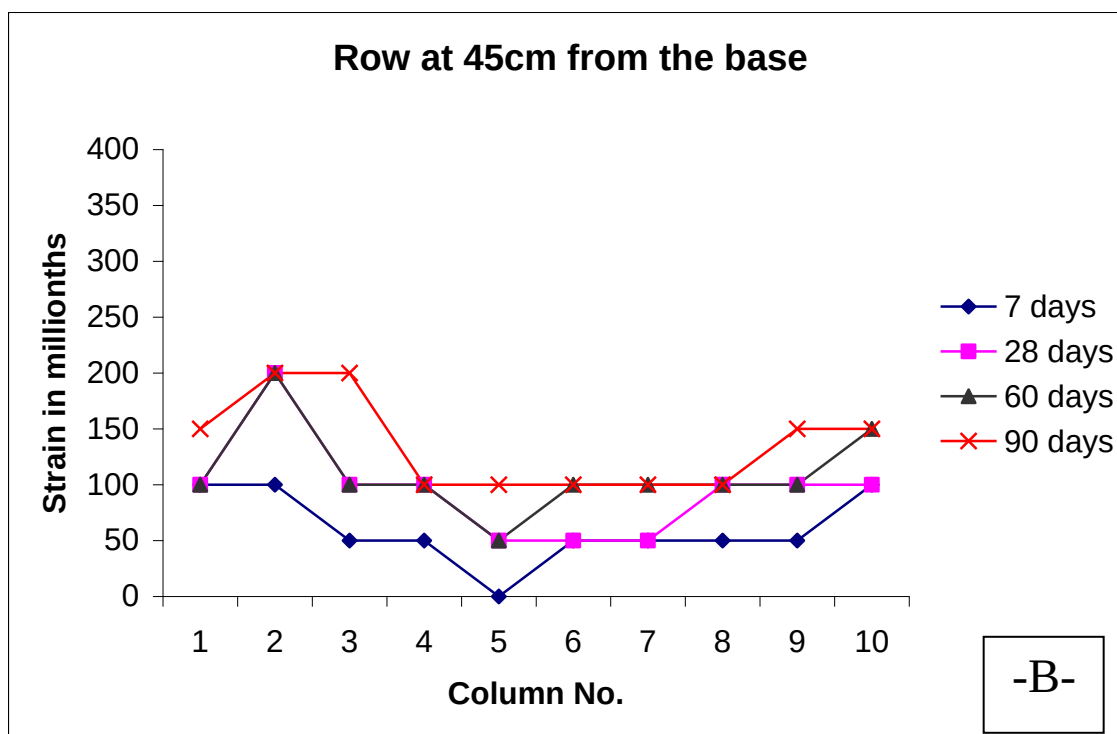
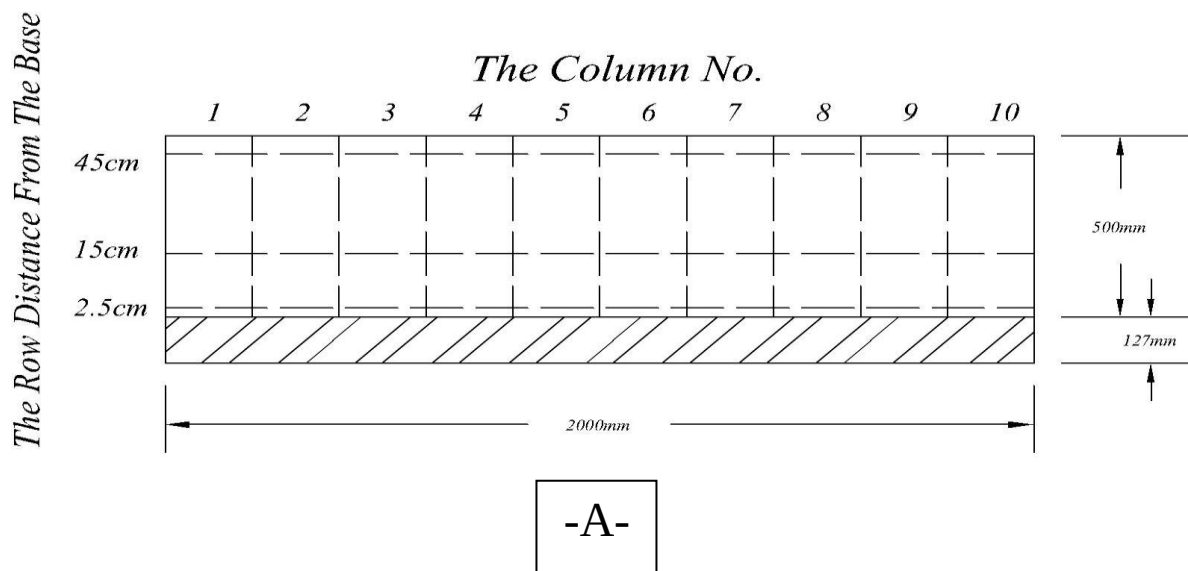


**Fig.(4.5)** Strain development history for reinforced HSC base restrained wall(WP4,2).

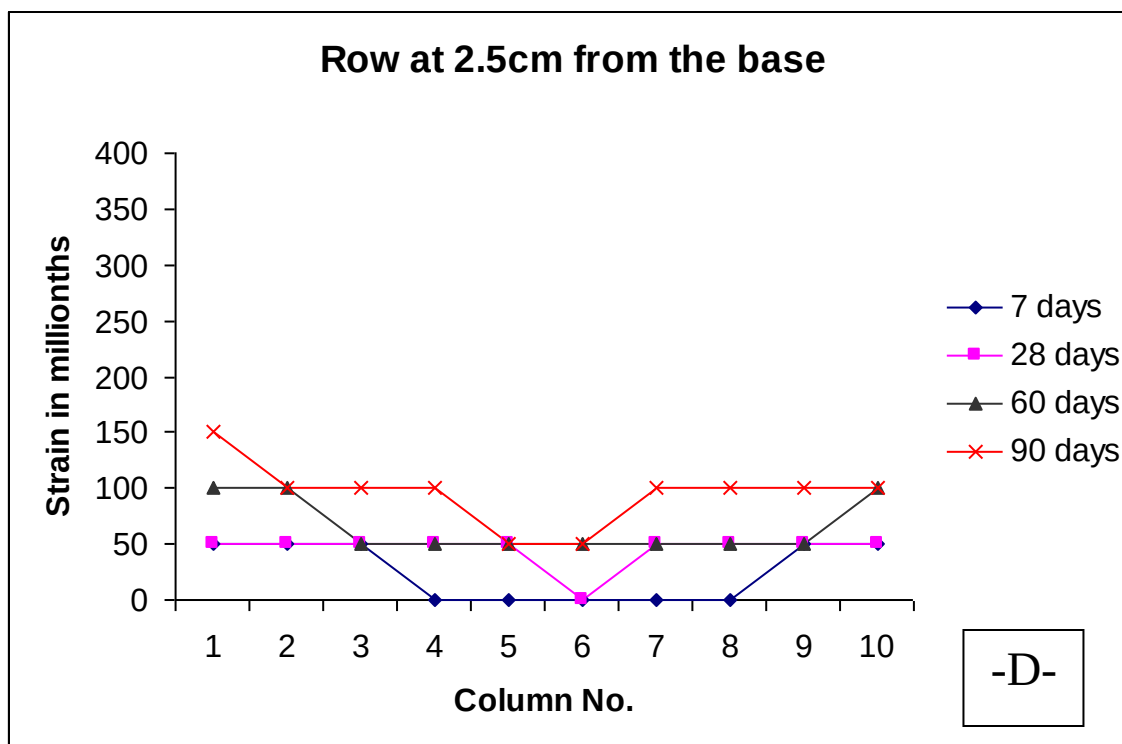
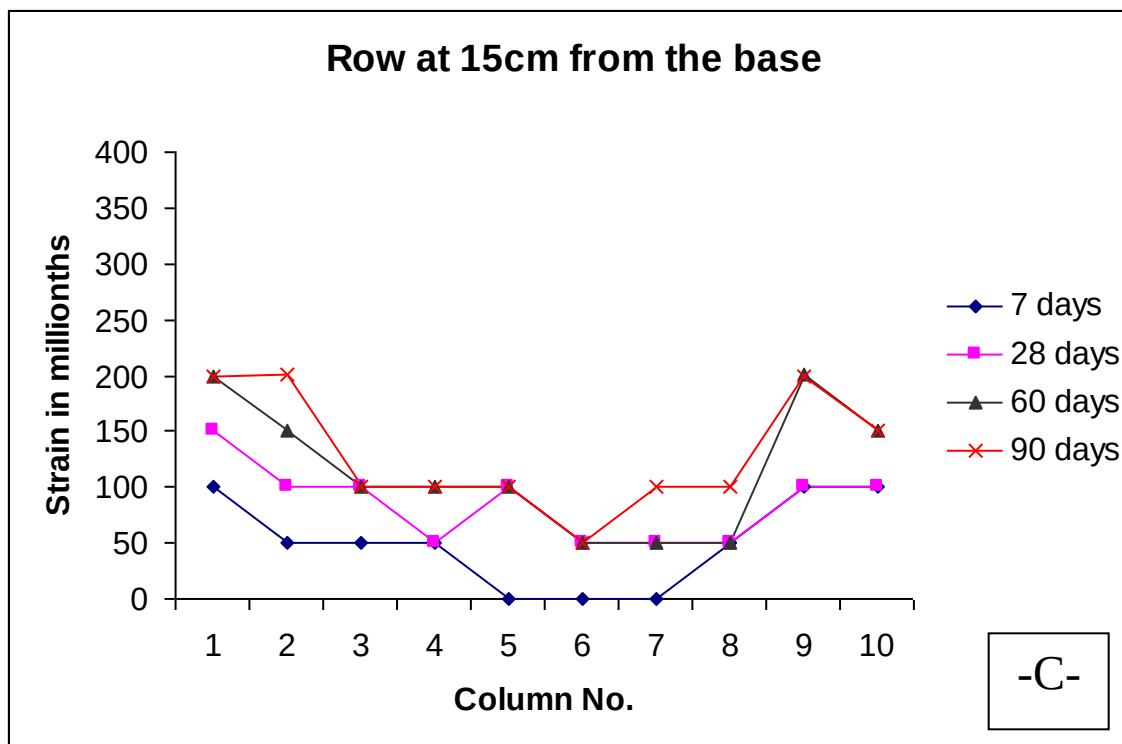


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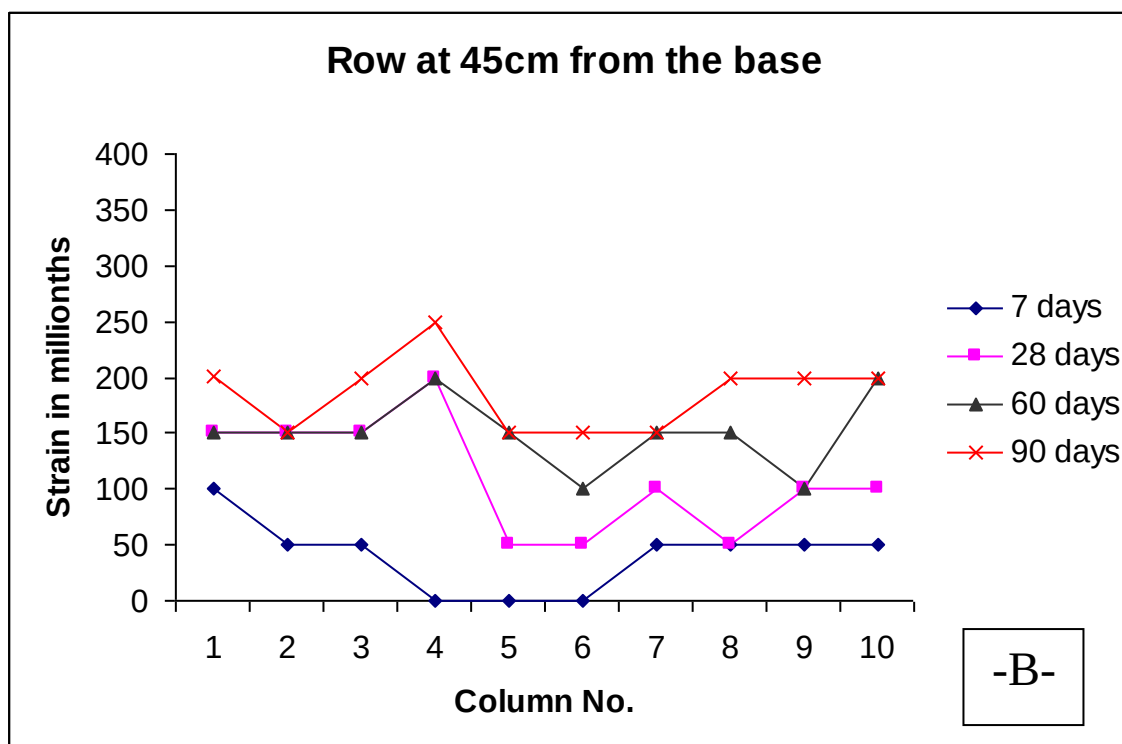
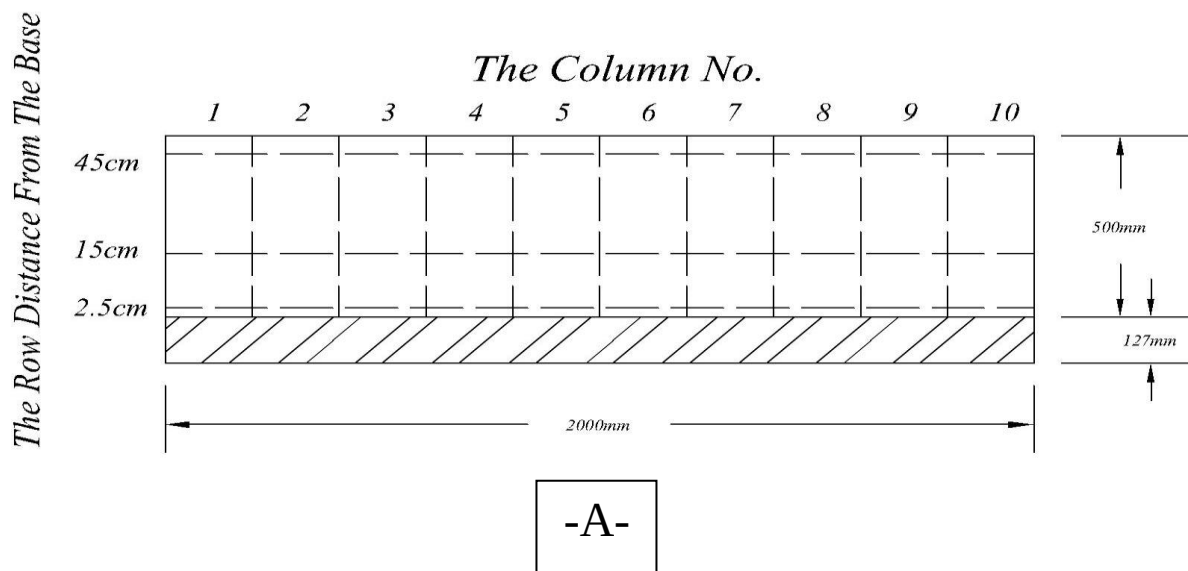


**Fig.(4.6)** Strain development history for unreinforced HSC base restrained wall(WP4,0).

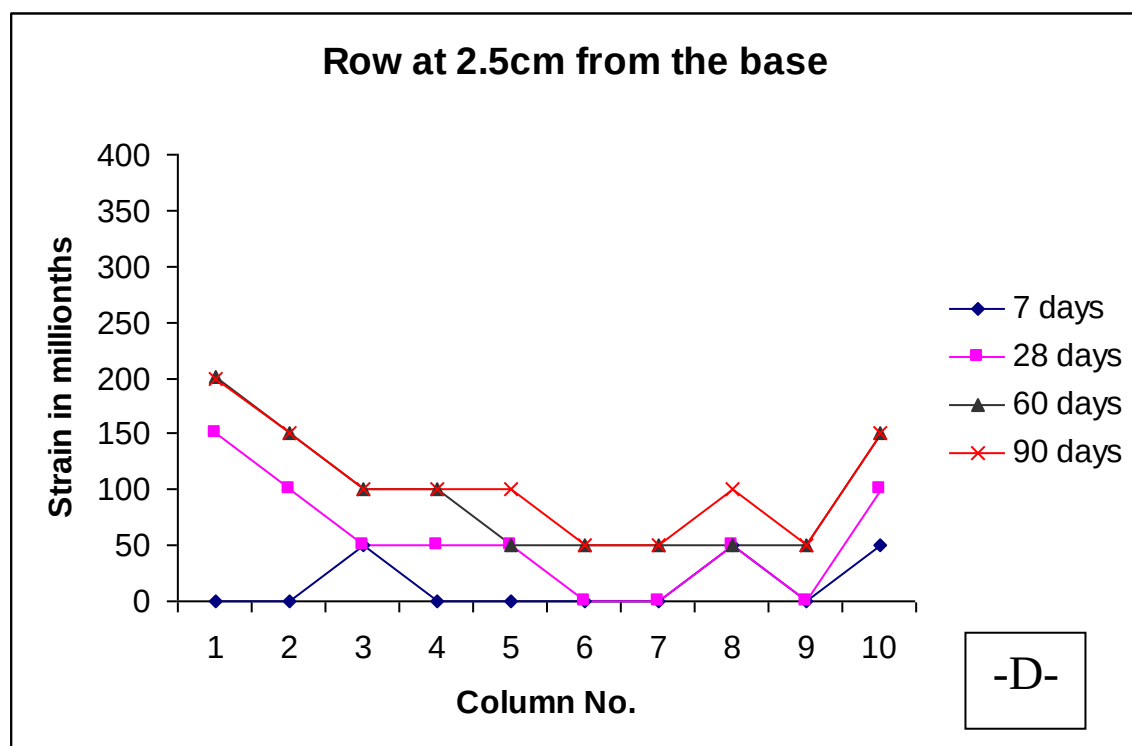
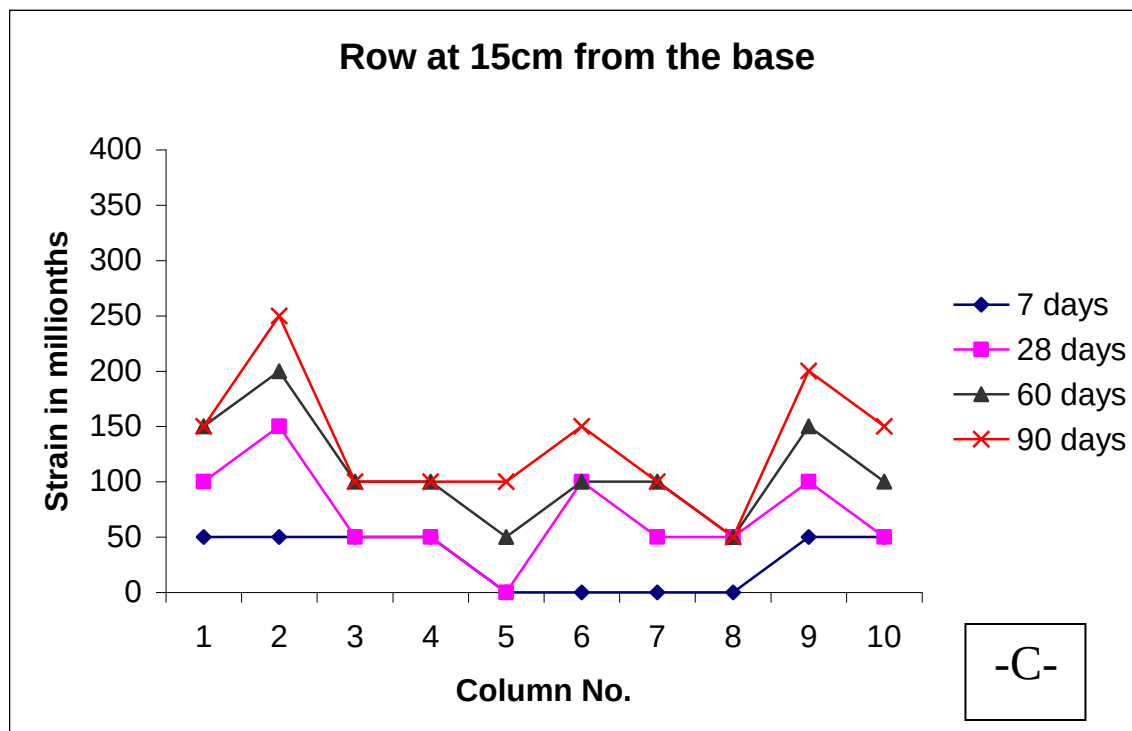


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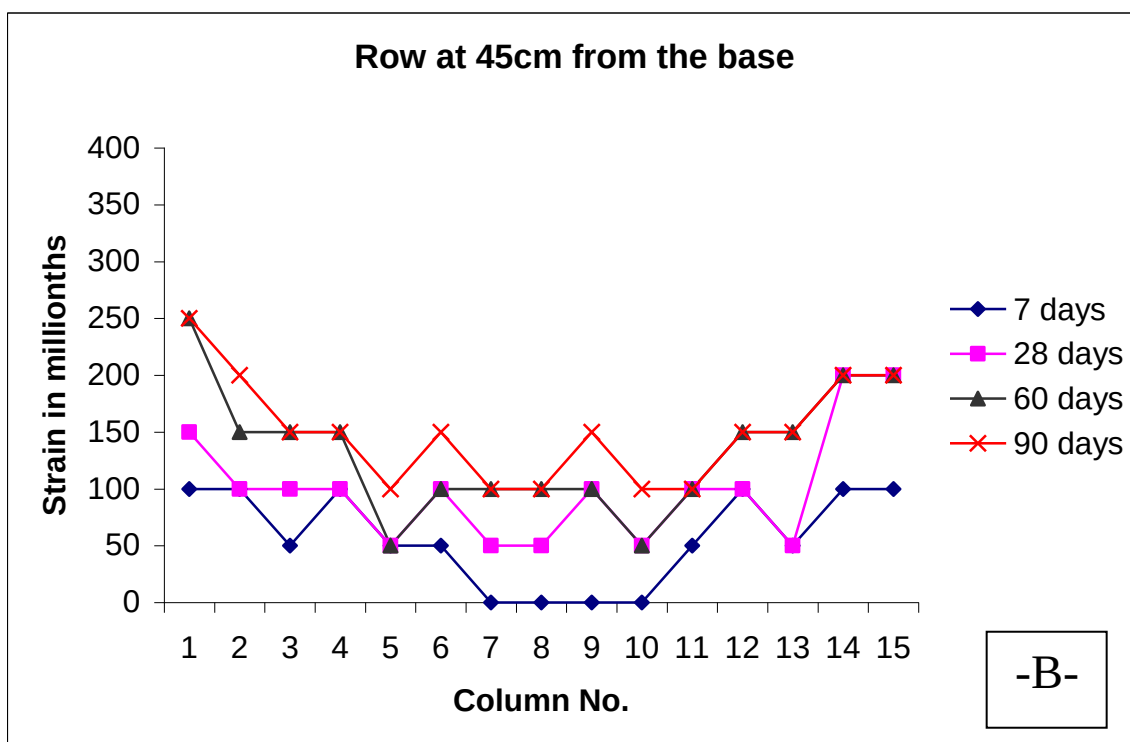
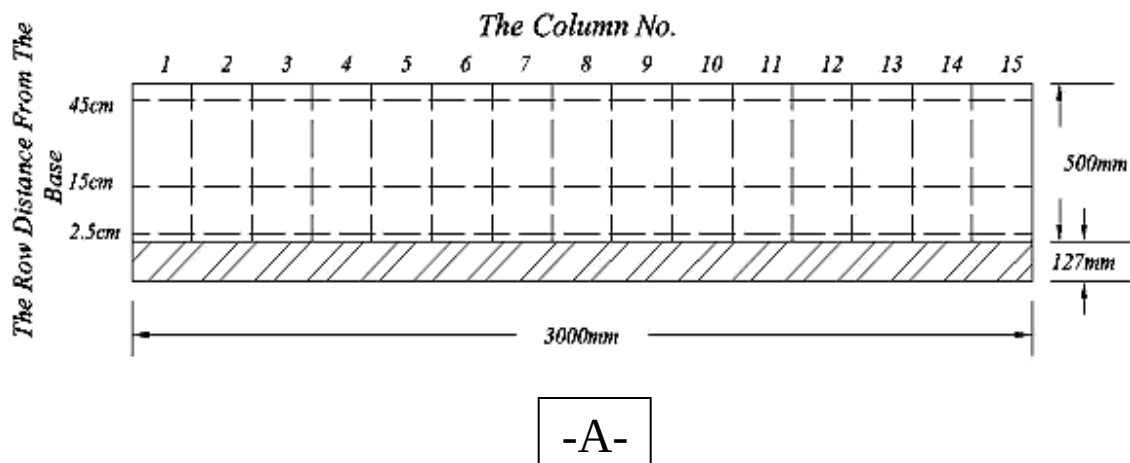


**Fig.(4.7)** Strain development history for reinforced HSC base restrained wall(WP6,2).

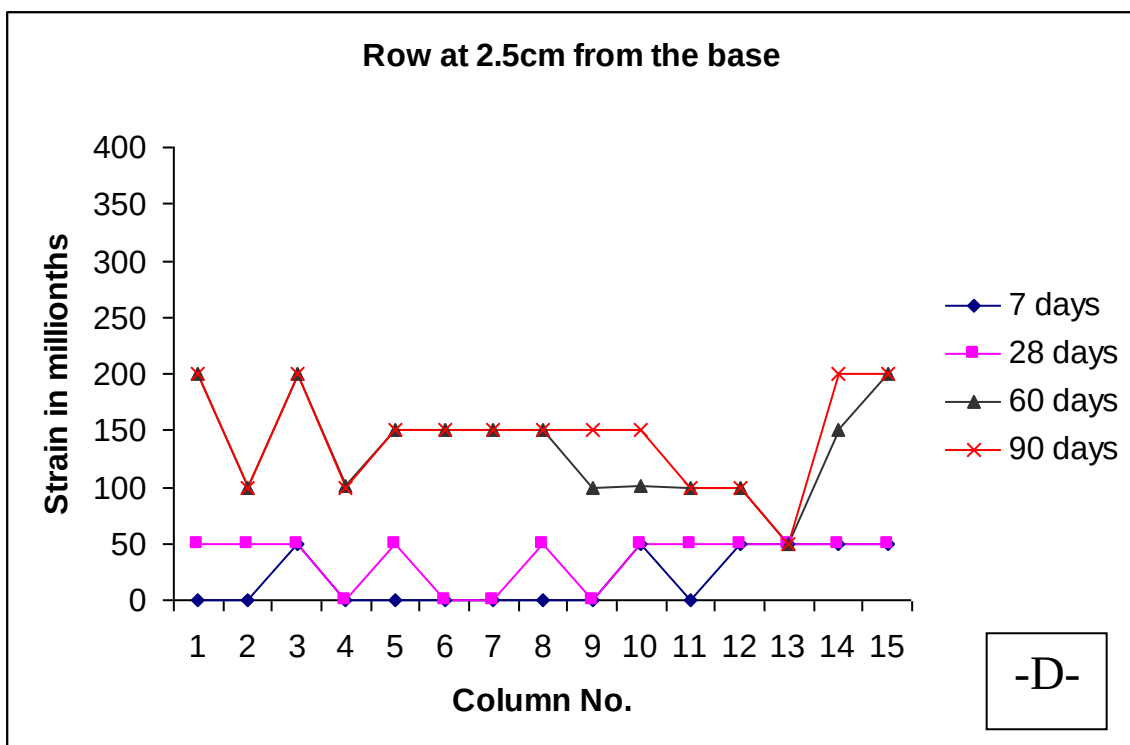
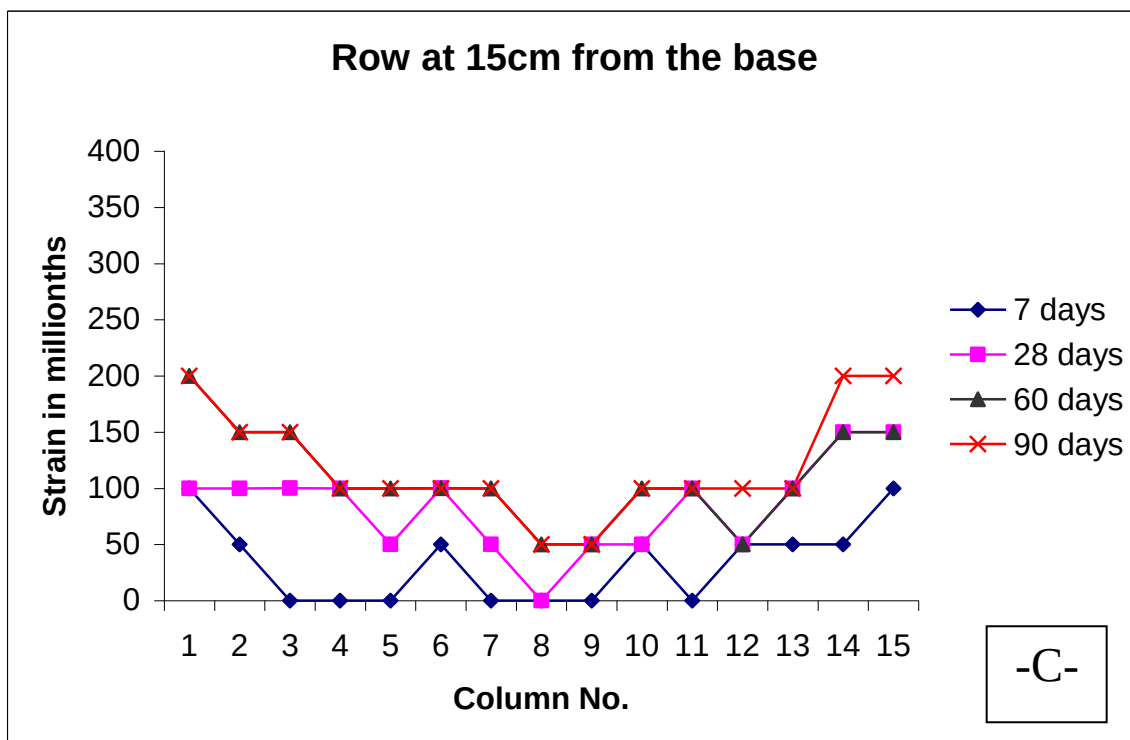


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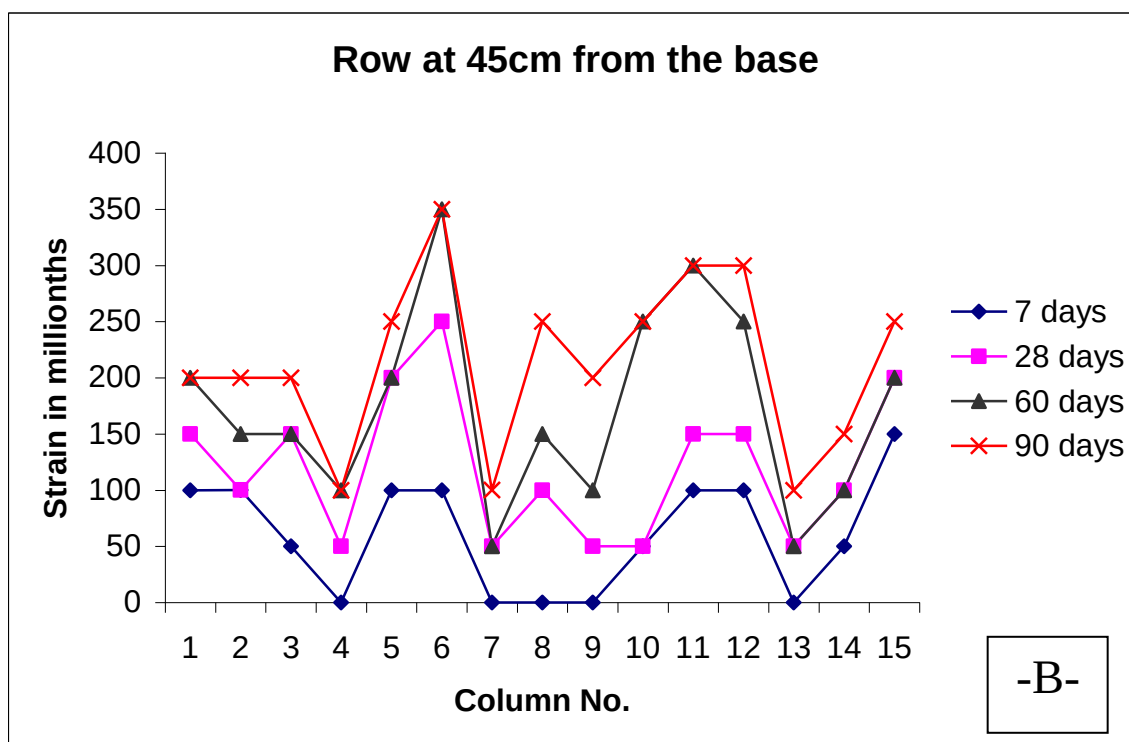
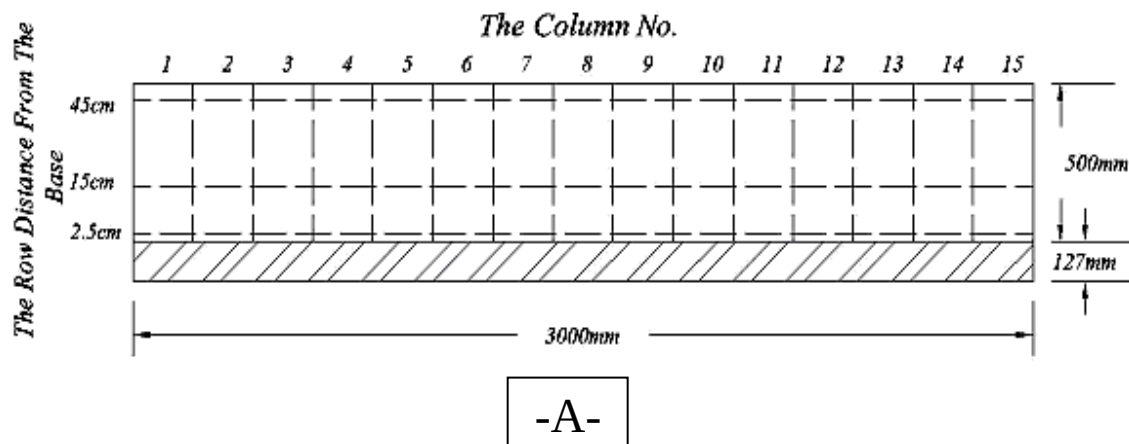


**Fig.(4.8)** Strain development history for unreinforced HSC base restrained wall(WP6,0).

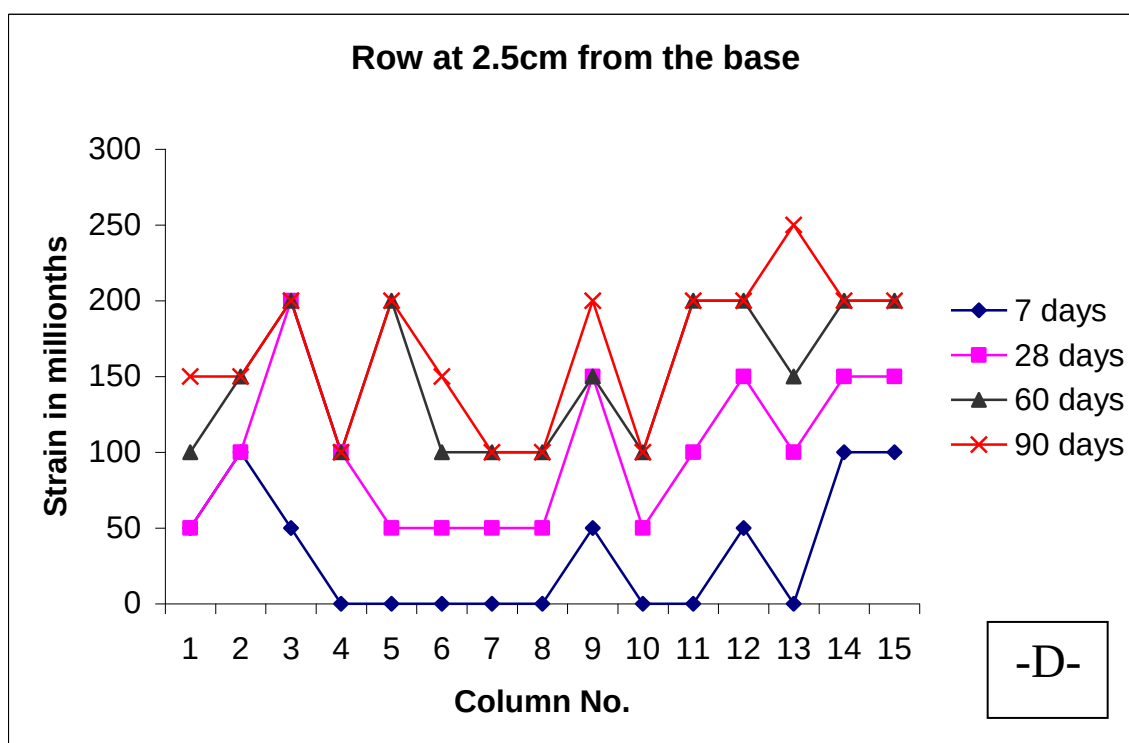
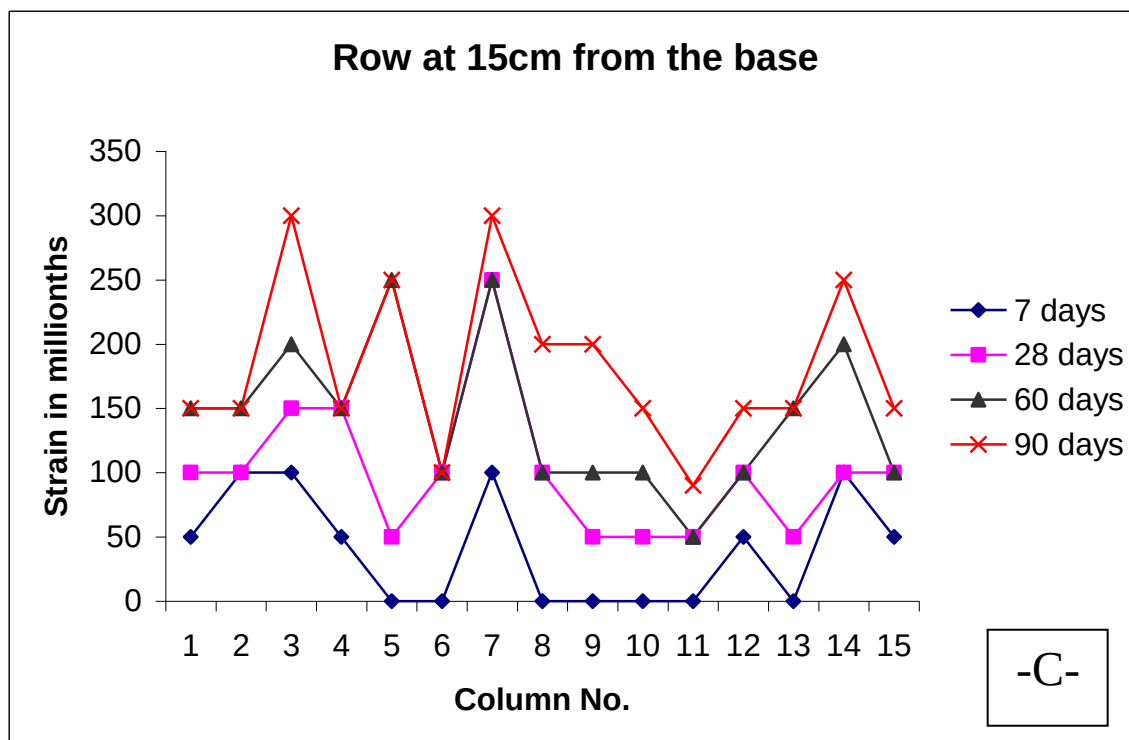


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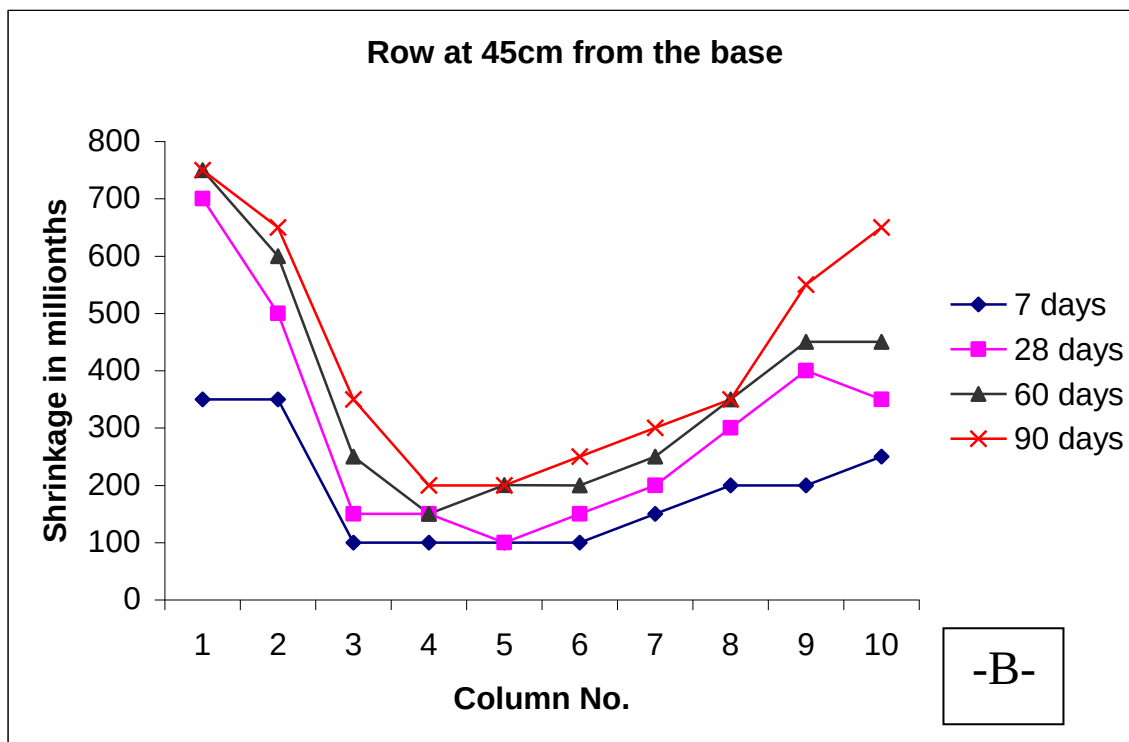
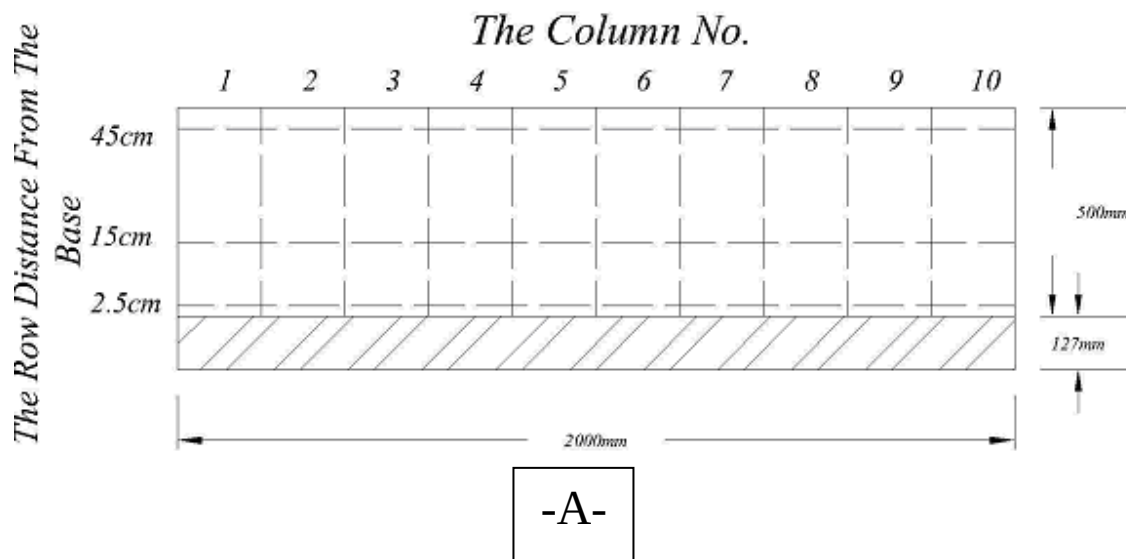


**Fig.(4.9)** Strain development history for reinforced HSC base restrained wall(SP4,2).

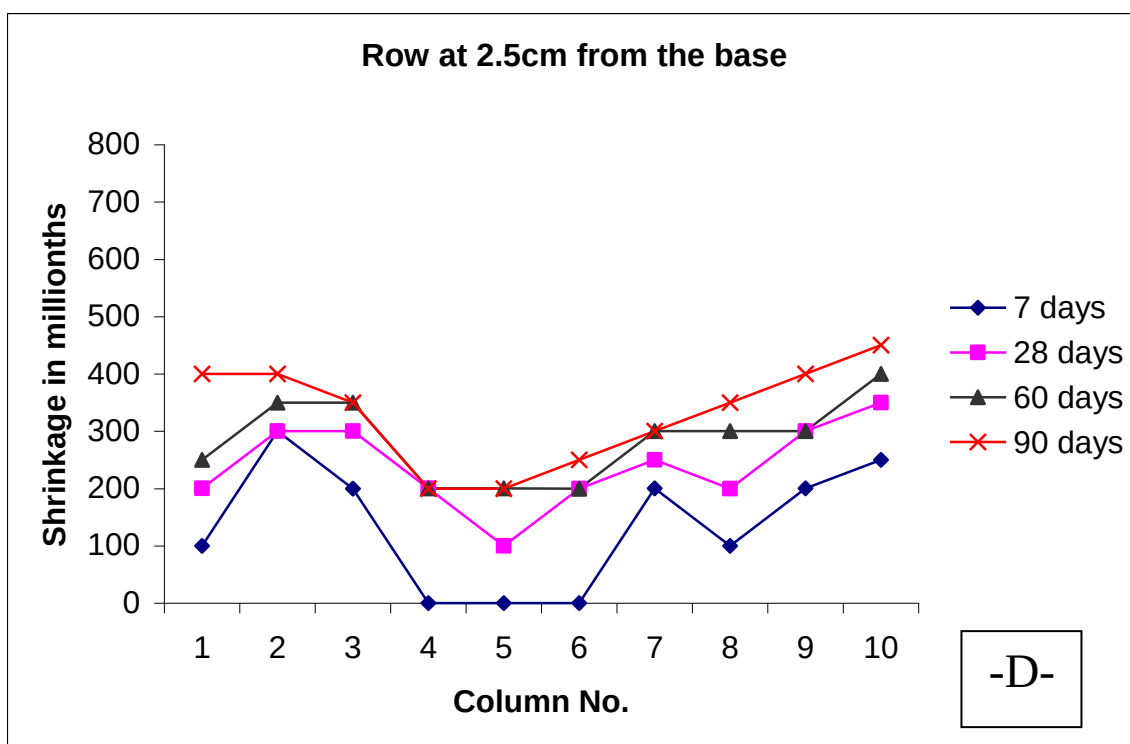
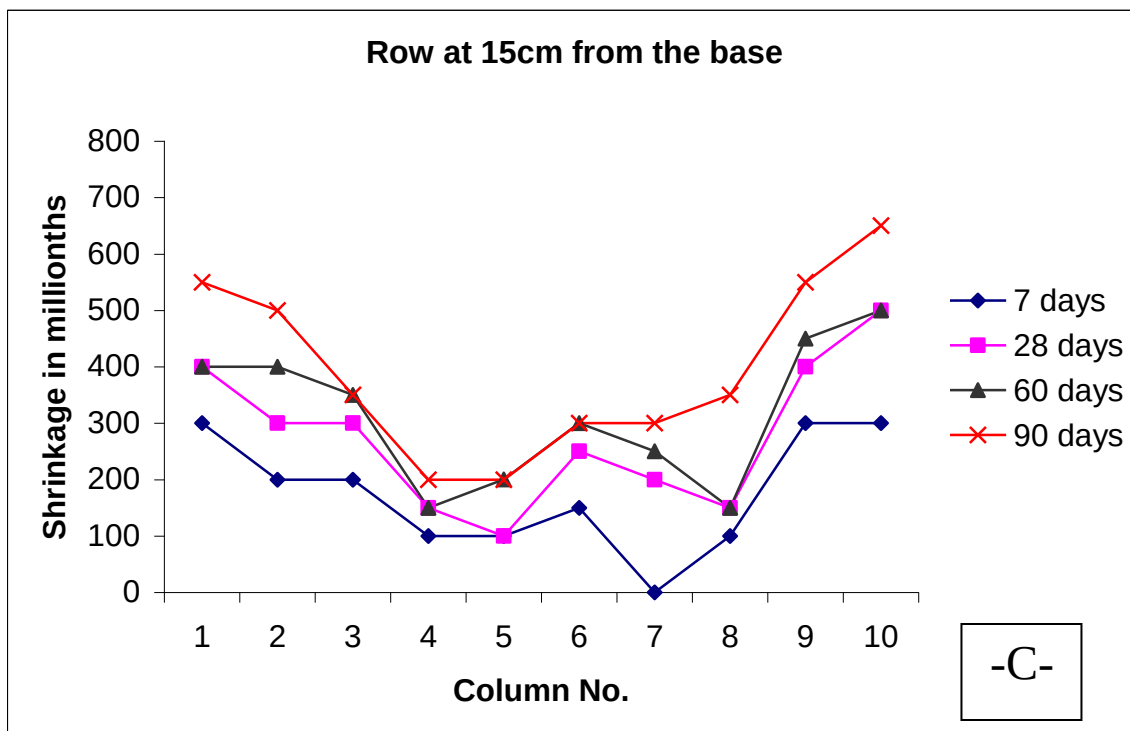


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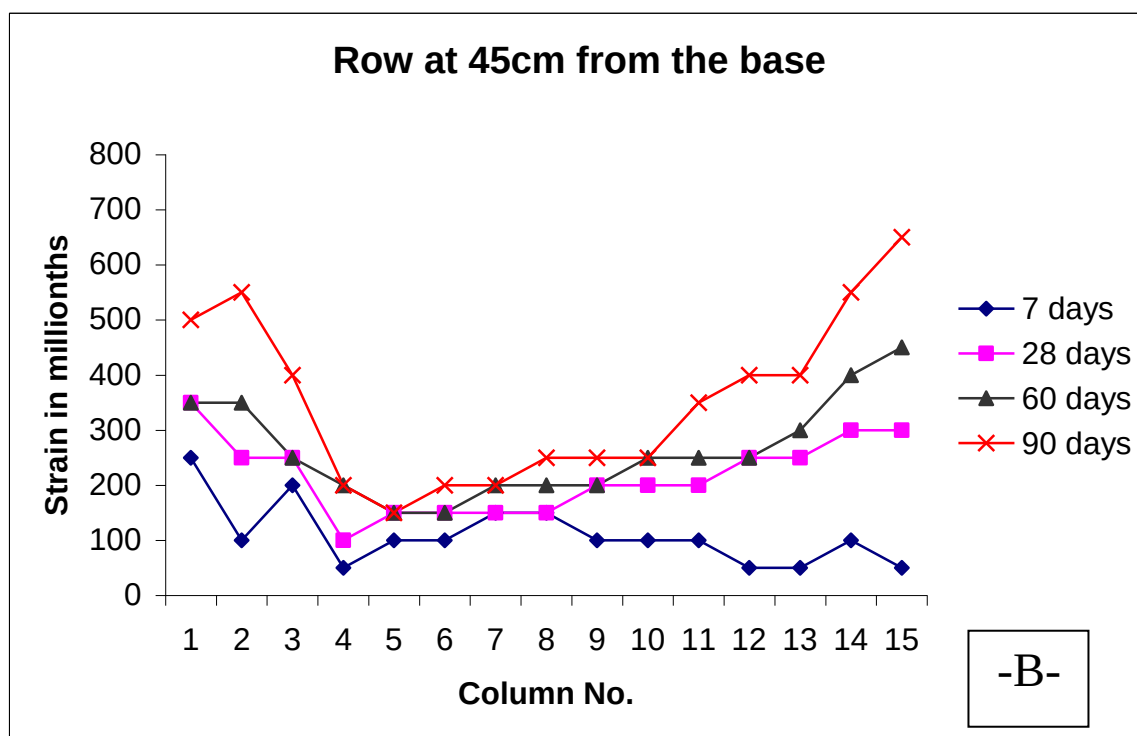
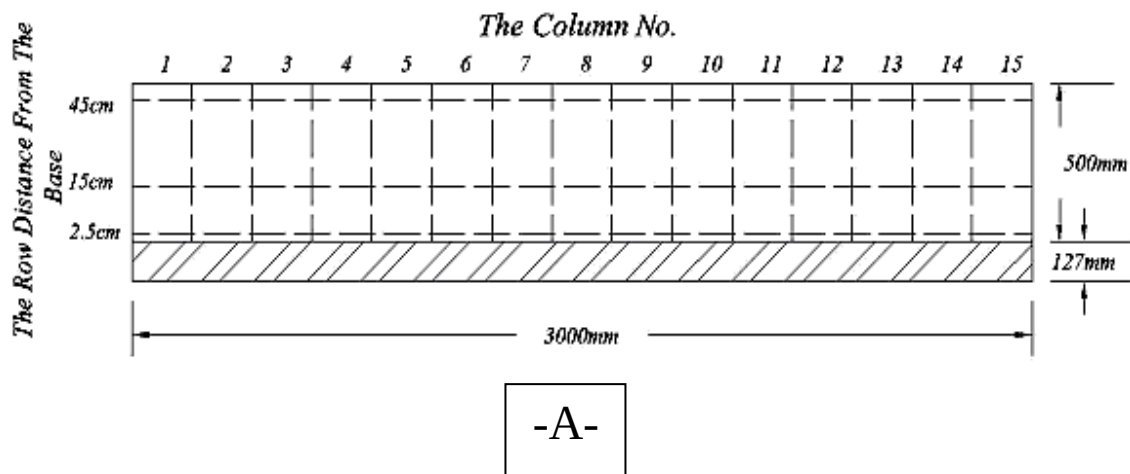


**Fig.(4.10)** Strain development history for reinforced HSC base restrained wall(SP6,2).

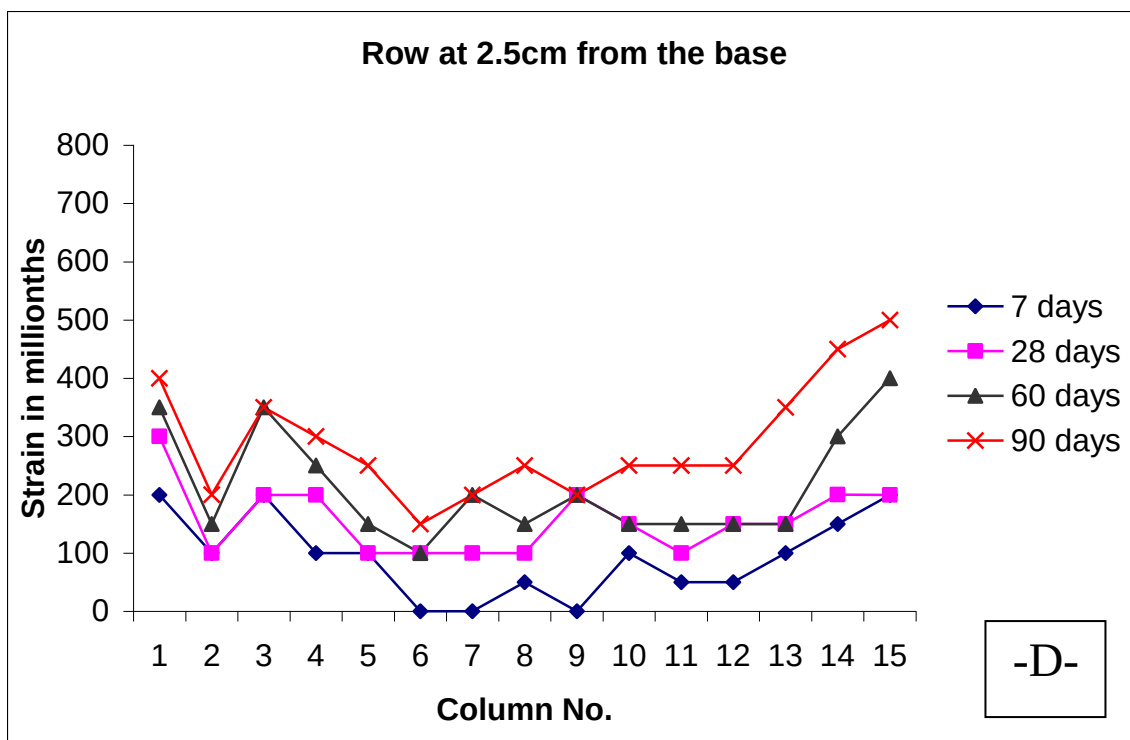
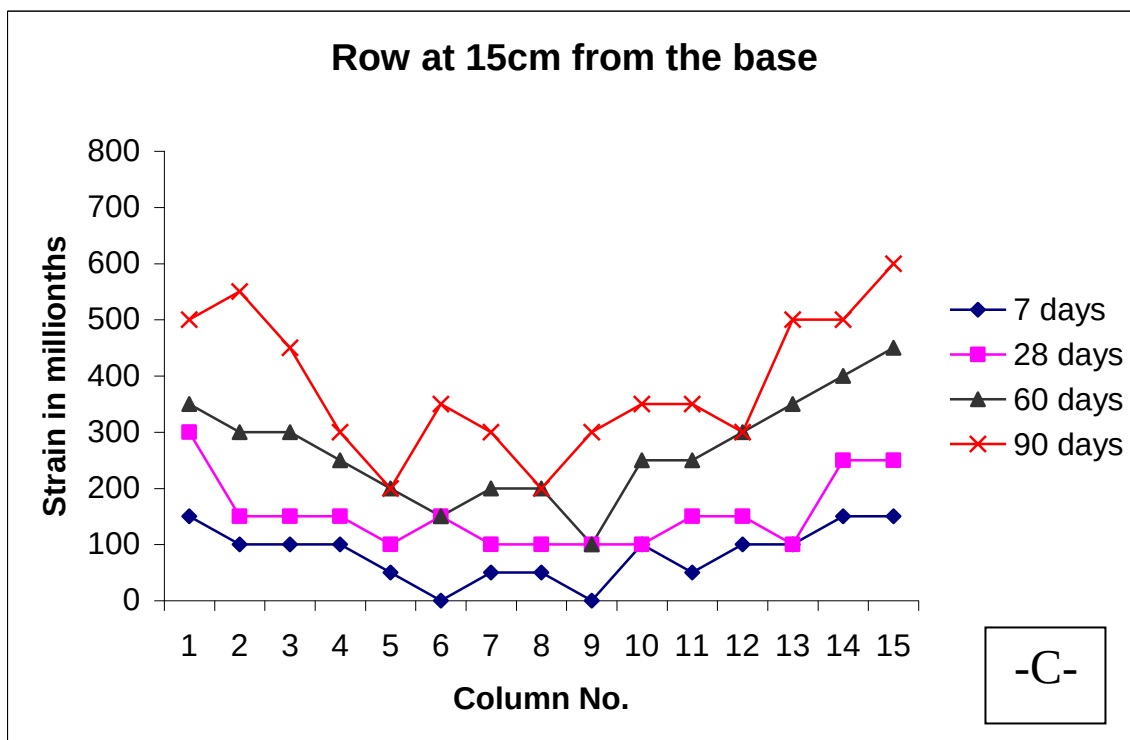


Fig.(4.10) Continued.

#### 4-7 Free Shrinkage of HSC End- Restrained Beams:

Using the model described in Chapter Three, the free shrinkage strain of HSC was measured under the same indoor and outdoor exposure conditions for the two sets (Winter and Summer) of the restrained walls. Table(4-6) and Figure (4-11) show the free shrinkage strain development of HSC of the two sets with drying period.

The maximum drying shrinkage in this study at 28 days is 390 microstrain. This result is less than the free shrinkage strain of normal concrete (580 microstrain at 28 days) which was reported by [Alwash, 2005] whereas it is compatible with the study carried out by **Smadi et. al**[1987] who reported that the drying shrinkage of low-strength concrete was more than for high strength concrete. But, it is not compatible with the study carried out by **Nagab et. al**[1980] who found that the shrinkage of high strength having 70 MPa compressive strength was 500 microstrain at 28 days and it is more than for normal strength concrete.

The maximum drying shrinkage obtained in this study at 60 days is 460 microstrain. This result is compatible with the study carried out by **Habeeb** [2000] who found that the shrinkage of high strength having 80 MPa compressive strength ranged between (400-537) microstrain at 56 days.

Results illustrate that the shrinkage of Winter set and Summer set ranged between (82 – 270 microstrain) and (35 – 460 microstrain) respectively at ages (3 – 60) days respectively. These values are close to the results of study carried out by **AL-Abdaly**[2007] who found that the shrinkage of high strength having 70 MPa compressive strength was ( 43 - 450 microstrain) at ages ( 3 - 70 ) days.

It can be seen from the results that the free shrinkage increases with the age progress for the two sets, but with different rates where the free shrinkage of Summer set is higher than the free shrinkage of Winter set (36%, 42% and 41%.

at ages 7, 28 and 60 days respectively). This may be due to the difference in the temperature degrees and relative humidity and consequently different evaporation intensity between Winter and Summer seasons.

It is clear from the results that there is a high initial rate of shrinkage. The reason of this behavior can be expected from the diffusion theory of drying that is the shrinkage is influenced strongly by the moisture loss. The moisture will be evaporated initially from the surface and near the surface with a high rate.

In the Winter set, there was a contraction (25 microstrain) due to the drop of the concrete temperature from its peak temperature (due to heat of hydration) to its temperature when the first demec reading was taken, which was carried out early in the morning.

**Table (4-6):** Shrinkage properties of HSC end restrained beams.

|        | Date of Crack in Beam (day) | Free Shrinkage at cracking date ( $\times 10^{-6}$ ) | Loss of restraint ( $\times 10^{-6}$ ) | Tensile strain capacity ( $\times 10^{-6}$ ) | Elastic tensile strain capacity ( $\times 10^{-6}$ ) | Creep ( $\times 10^{-6}$ ) | Width of crack (mm) |
|--------|-----------------------------|------------------------------------------------------|----------------------------------------|----------------------------------------------|------------------------------------------------------|----------------------------|---------------------|
| Winter | 40                          | 250                                                  | 60                                     | 190                                          | 54                                                   | 136                        | 0.15                |
| Summer | 19                          | 345                                                  | 75                                     | 270                                          | 96                                                   | 174                        | 0.27                |

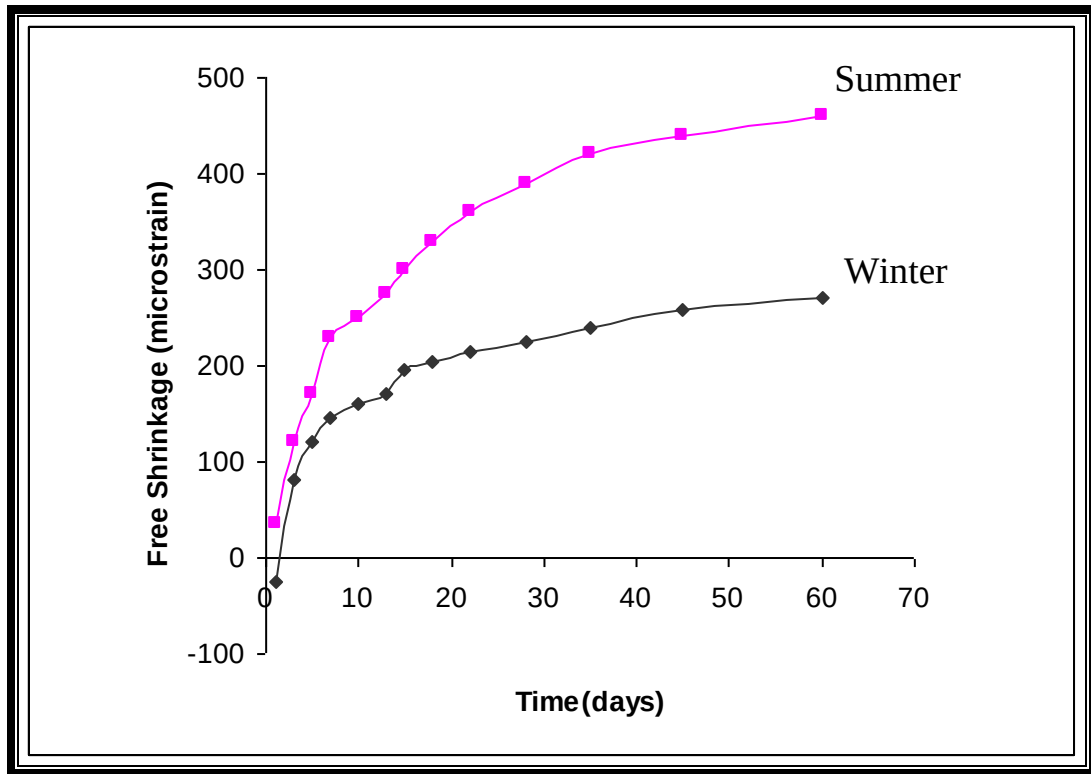


**Fig.(4.11)** Free shrinkage development of the HSC beam models in Winter and Summer .

It is noticed that the maximum restraining shrinkage of walls in Winter set is equal or more than the free shrinkage in Winter beam, also the maximum restraining shrinkage of walls in Summer set is more than the free shrinkage in Summer beam. This is may be attributed to the curling effect of the base which provided by high rigidity of base, or may be due to there is some slippage between the base and wall which cause the reduction in restraint values in all positions at wall, then there is small restraint values at the top row in the wall.

### 4-8 Elastic Tensile Strain Capacity of HSC Beam Models:

Elastic tensile strain capacity of the HSC beam models was obtained directly by measuring the immediate movement after cracking of concrete in the end restrained beams in two seasons (Winter and Summer), the results are summarized in Table (4-6).

The elastic tensile strain capacity was also calculated indirectly by determination of flexural tensile strength divided by the modulus of elasticity. The results are illustrated in Table ( 4-7) and plotted in Figures ( 4 - 12 ).

The results of the present work agreed with *AL-Abdaly*[2007] who studied the behavior of high strength concrete and found the elastic tensile strain capacity from direct method is less than that of indirect tensile strain capacity. On the other hand, these results are not in agreement with Al-Rawi who studied the shrinkage behavior of normal strength concrete and found that the elastic tensile strain capacity from direct method is greater than that from indirect method [*Al-Rawi, 1985*].

**Table (4-7):** Results of the calculated elastic tensile strain capacity of the HSC beam models in Winter and Summer ( indirect method ).

|               | Elastic tensile strain capacity $\times 10^{-6}$ |         |         |         |
|---------------|--------------------------------------------------|---------|---------|---------|
|               | 7 days                                           | 28 days | 60 days | 90 days |
| <b>Winter</b> | 187.5                                            | 191.4   | 192.3   | 195     |
| <b>Summer</b> | 189.5                                            | 192.7   | 195.2   | 195.5   |

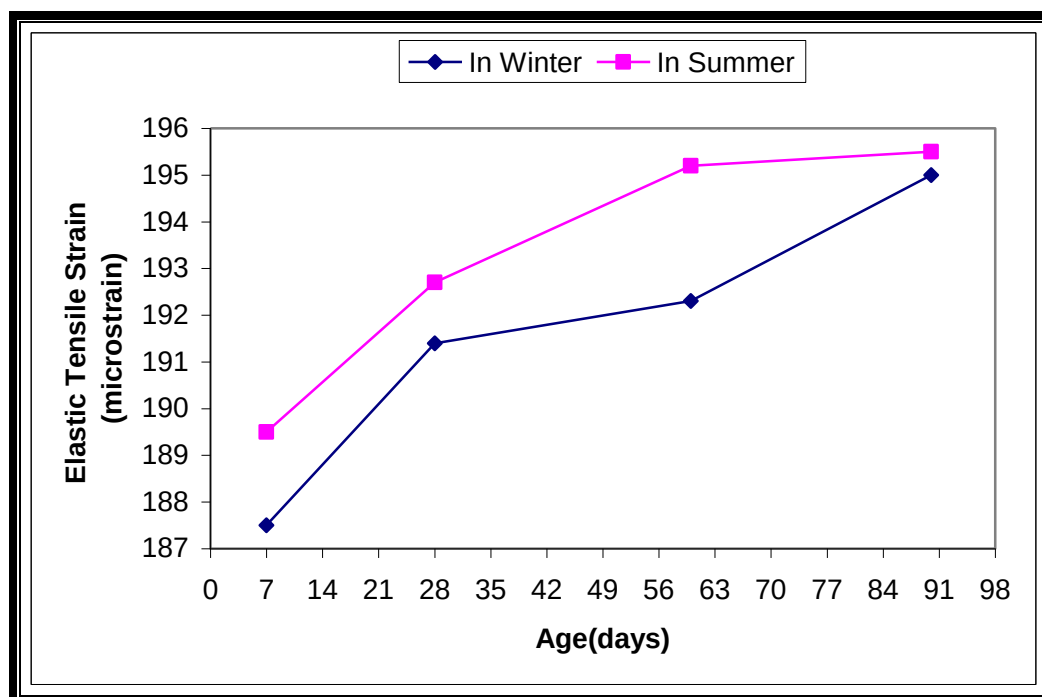


**Figure ( 4 - 12 ) :**Development of elastic tensile strain of the HSC beam models in Winter and Summer ( indirect method ).

#### **4-9 Tensile Strain Capacity of HSC Beam Models:**

The determination of tensile strain capacity was based on the end restrained shrinkage beam model. Table (4-6) shows the values of tensile strain capacity at date of cracking obtained in the present work. It can be observed from the results that the tensile strain capacity in Summer season (270 microstrain) is more than the tensile strain capacity in Winter season (190 microstrain) although the crack date in Summer (19 day) is less than the crack date in Winter (40 day). This result is not compatible with the study carried out by *Hughes and Ghunaim [1982]* and *Hobbs[1974]*.

Although of the longer time for the occurrence of cracking (40 and 19day ) for Winter and Summer season respectively, the tensile strain capacity of HSC is less than the tensile strain capacity of normal concrete by **[Al-Rawi, 1985]** and **Kubba [2007]**.

The higher cement content ( 550 kg/m<sup>3</sup> ) led to lower tensile strain capacity, this result is not in line with the results of the study carried out by **[Carlson, 1979]**, who was found that the tensile strain capacity increases significantly with the increasing in cement content. This may be because the water / cement ratio is reduced in the high strength concrete mix, therefore, it is better able to resist shrinkage more over at this low water / cement ratio, the quantity of unhydrated cement particles which restrain the shrinkage is great **[Mehta, 1975]** .

#### **4-10 Creep of HSC Beam Models:**

Creep strain of HSC subjected to the end restrained shrinkage was calculated as the difference between the tensile strain capacity and the elastic tensile strain capacity. Table (4-6) shows that the creep strain for HSC end restrained beam specimen at cracking in Winter and Summer seasons is (136 and 174 ×10<sup>-6</sup>) respectively. It can be observed that the creep strain of HSC is less than the creep strain of normal strength concrete (275 and 304 microstrain) as reported by **[Al-Rawi, 1985]** and **[Kubba, 2007]** respectively.

It can be noticed that the creep coefficient (creep strain to total strain) is 0.3 which is less than the creep coefficient (0.35) of normal strength concrete as reported by **Kubba [2007]**. This is compatible with the studies carried out

by *Smadi et. al*[1987] and *Nagab et. al*[1981] who reported that the creep coefficient and specific creep of HSC are less than that of the medium and low strength concretes.

It is obvious that the creep strain (at cracking) for Winter concrete is about 78% from Summer concrete. The high shrinkage (in Summer) shows generally a high creep [*Hermite,1960*]. Also, the high creep of Summer concrete may be due to higher temperature and low humidity as reported by [*Troxell et. al ,1958*].

#### **4-11 Cracking Time of HSC Beam Models:**

Cracking time is the time required for first crack to occur . From Table (4-6), it can be observed that the cracking time in HSC in Winter is 40 days, while it decreases in Summer to 19 days. This may be attributed to that concrete casted in Summer hydrate more rapidly and more evaporation take place which cause early cracks. Also, the high temperatures increase the rate of hydration and initiate thermal stresses and the tendency for drying shrinkage cracking will be enhanced [*Anton and Frank, 2002*].

#### **4-12 Cracking of The Walls:**

Theoretically, the first crack must occur at the centre of the wall starting from base, this can be attributed to the value of the degree of restraint being the highest at this position [ACI 207,1973]. Assuming no weak region occurring over the wall height, the crack must originate near the wall bottom and propagate toward the wall top [Stoffer,1978].

In spite of the long observation period (90 days) of all walls, no crack occurrence at this region or any other place was observed except there were some plastic shrinkage cracks in the top surface of the Summer set. This observation is compatible with the study carried out by [Bloom and Bentur,1995].

There is a few study about high strength concrete base restrained walls to make comparison with this study, but it can be compared with normal concrete base restrained concrete walls that have lower compressive and tensile strength. For example, the walls that have  $L/H=1, 2$  and  $4$ , the vertical steel ratio= $1\%$ , horizontal steel ratio= $0.2\%$  were found not cracked, but there is one secondary crack in the wall which has  $L/H=8$  with  $0.05\text{mm}$  crack width [Kheder,1986].

For  $L/H=2.0$ , the crack width was found  $0.1\text{mm}$  for two horizontal steel ratios ( $0.2\%$  and  $0.0\%$ ) and vertical steel ratio= $1\%$  for the two walls [Al-Mashhdi, 1989].

For  $L/H=4$ , the wall was found not cracked with the same vertical and horizontal steel ratios of this study ( $1\%$  and  $0.2\%$  respectively), but there is one secondary crack in the wall which has  $L/H=8$  [Al-Tamimi,1987].

#### 4-13 Mathematical model:

By using the data which was obtained from this study, mathematical model was proposed to predict the maximum shrinkage value at 90 days drying period which occurs in high strength concrete walls. This model made by statistical program called (DATA FIT 9.0.59, 2008) and depends on length/height ratio, horizontal reinforcement ratio, vertical reinforcement ratio, temperature degree and relative of humidity as shown in Table (4-8).

**Table(4-8)** Experimental data which is used in mathematical model.

| Wall No. | Maximum Shrinkage (microstrain) | L/H | $\rho_h$ (%) | $\rho_v$ (%) | T (°C) | RH (%) |
|----------|---------------------------------|-----|--------------|--------------|--------|--------|
| 1        | 350                             | 1   | 0.2          | 1            | 32     | 40     |
| 2        | 400                             | 1   | 0            | 0            | 32     | 40     |
| 3        | 300                             | 2   | 0.2          | 1            | 32     | 40     |
| 4        | 350                             | 2   | 0            | 0            | 32     | 40     |
| 5        | 200                             | 4   | 0.2          | 1            | 32     | 40     |
| 6        | 250                             | 4   | 0            | 0            | 32     | 40     |
| 7        | 250                             | 6   | 0.2          | 1            | 32     | 40     |
| 8        | 350                             | 6   | 0            | 0            | 32     | 40     |
| 9        | 750                             | 4   | 0.2          | 1            | 43     | 30     |
| 10       | 650                             | 6   | 0.2          | 1            | 43     | 30     |

#### 1-The first model :

$$\text{Maximum Shrinkage Strain} = a \cdot L/H + b \cdot \rho_h + c \cdot \rho_v + d \cdot T + e \cdot RH$$

$L/H$  : Length to height ratio.

$\rho_h$  : Horizontal reinforcement ratio(%).

$\rho_v$  : Vertical reinforcement ratio(%).

$T$  : Average temperature degree of drying period( °C).

$RH$  : Average relative humidity of drying period (%).

$a, b, c, d, e, f$  : Regression coefficient of independent variables.

### 2- *The second model :*

$$\text{Maximum Shrinkage Strain} = a \cdot L/H + b \cdot \rho_h + c \cdot \rho_v + d \cdot T + e \cdot RH + f$$

### 3- *The third model:*

$$\text{Maximum Shrinkage Strain} = \exp(a \cdot L/H + b \cdot \rho_h + c \cdot \rho_v + d \cdot T + e \cdot RH + f)$$

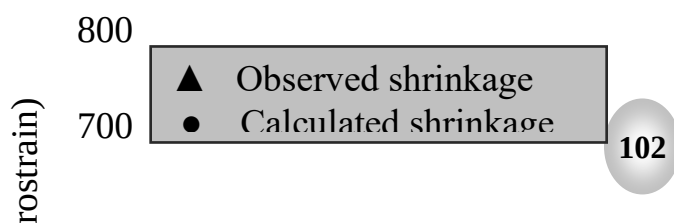
Table ( 4 - 9 ) gives the regression coefficients of the prediction models above for the prediction of the maximum shrinkage value at 90 days drying period which occurs in high strength concrete walls.

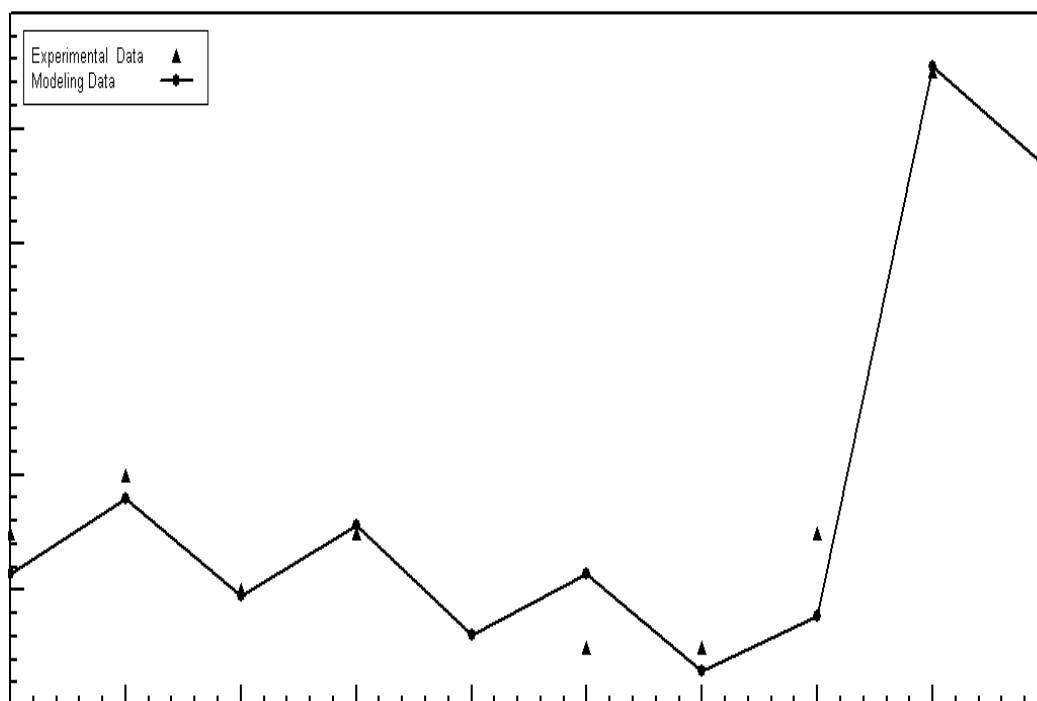
From the same Table, the correlation coefficient increases and the standard error decreases with increasing the number of factors used in the proposed model .

**Table(4- 9) :** Regression Coefficients for maximum shrinkage value model.

| <b>Coefficient</b>                           | <b>Model 1</b> | <b>Model 2</b> | <b>Model 3</b> |
|----------------------------------------------|----------------|----------------|----------------|
| <b>a</b>                                     | -19.42965175   | -19.50766812   | -6.28E-02      |
| <b>b</b>                                     | -3.30E+15      | 1.3353E+14     | -2.01183E+12   |
| <b>c</b>                                     | 6.60536E+14    | -2.6706E+13    | 4.02366E+11    |
| <b>d</b>                                     | 29.42798788    | -9.1423E+14    | -3.8686E+12    |
| <b>e</b>                                     | -13.5257544    | -1.01E+15      | -4.25546E+12   |
| <b>f</b>                                     | -              | 6.95E+16       | 2.94013E+14    |
| <b>Correlation Coefficient(R)</b>            | 0.9689         | 0.9718         | 0.9724         |
| <b>Variance Explained (<math>R^2</math>)</b> | 0.9388         | 0.9444         | 0.9456         |
| <b>Standard Error (S.E)</b>                  | 63.092         | 62.952         | 62.247         |

The increase of correlation coefficient and decrease in standard error from model ( 1 ) to ( 3 ) , proves the right choice of the variables and their effect on drying shrinkage of concrete. Also it can be seen from Figures ( 4 –13 ) and ( 4 –14 ) the difference and relation between observed max. shrinkage and calculated max shrinkage in model (3).





Wall No.

**Figure ( 4 –13 )** The difference between observed max. shrinkage and calculated max. shrinkage in model (3).

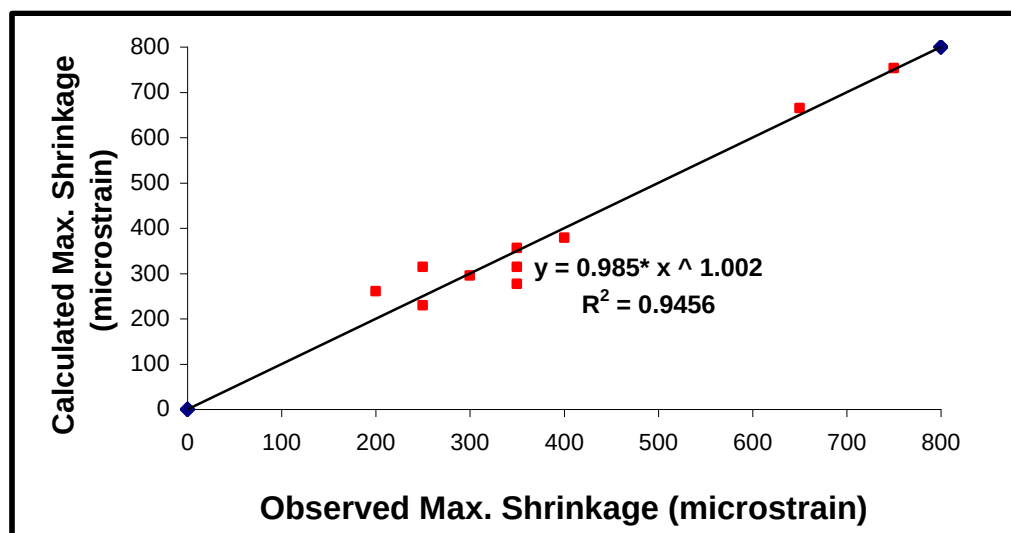


**Figure ( 4 –14 )** The relation between observed max. shrinkage and calculated max. shrinkage in model (3).

# C HAPTER FIVE 5

## *CONCLUSIONS AND RECOMMENDATION*

### *5-1 Conclusions:*

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

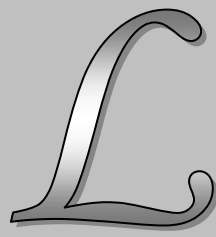
1. The experimental observations clearly indicate that the cracking did not develop in reinforced and unreinforced HSC base restrained walls with L/H ( 1, 2 , 4 and 6) during 90 days period of drying.
2. The maximum shrinkage values increased toward the edges and top of the most walls.
3. The maximum shrinkage strains in Summer walls was found more than for Winter walls about (66%, 46% ) for L/H (4 and 6) respectively.
4. The use of horizontal reinforcement (0.2%) led to reduction in the maximum shrinkage values with increasing of length/height ratio where this reduction ratios were (12 %,14%,20%,28%) of the maximum shrinkage values in the unreinforced concrete walls with length/height ratio (1,2,4,6) respectively.

5. The influence of vertical reinforcement interferes with horizontal reinforcement in controlling of wall shrinkage.
6. It was found from the end-restrained beam specimens results that the drying shrinkage is less for high strength concrete than for normal strength concrete. Also the following conclusions were drawn from the results of the same beam specimens.
7. The Summer free shrinkage at date of cracking is higher than the Winter free shrinkage by about (27%) in end restrained beams.
8. The Summer tensile strain capacity at date of cracking is higher than of Winter about (30)%. On the other hand, the Summer elastic tensile strain capacity at date of cracking is higher than the Winter elastic tensile strain capacity by about (44%).
9. Elastic tensile strain capacity calculated by indirect method ( flexural tensile strength divided by the modulus of elasticity ) is higher than that of direct method ( elastic tensile strain capacity of concrete was obtained directly by measuring the immediate movement after cracking of concrete in end restrained beams ) by about ( 72 and 50)% in Winter and Summer respectively.
10. The creep strain in Winter and Summer (136 and 174 microstrain) respectively and creep coefficient (0.3) are less in high strength concrete than for normal concrete (304 microstrain, 0.35).
11. The Summer creep strain at date of cracking is higher than of Winter by about (22) %.
12. The date of cracking is decreasing with increasing of ambient temperature and decreasing relative humidity, the cracking time was (40 and 19 day) in Winter and Summer respectively.

13. Mathematical model was proposed to predict the maximum shrinkage value at 90 days drying period which occurs in high strength concrete walls. This model was derived by statistical program called (DATA FIT 9.0.59, 2008) and depends on length/height ratio, horizontal reinforcement ratio, vertical reinforcement ratio, temperature degree and relative humidity. The correlation coefficient was ( $R = 0.9724$ ), the variance explained was ( $R^2 = 0.9456$ ) and Standard Error was ( $S.E = 62.247$ ).

### ***5-2 Recommendations for the Further Works:***

1. Studying the effect of steel base restraint on drying shrinkage of high strength concrete walls with L/H above 6.
2. Use normal concrete base to high strength walls with same L/H of this study and make comparison with this study.
3. Studying the shrinkage of high strength concrete walls restrained from base and exposed to fire.
4. Studying creep of high strength concrete columns.



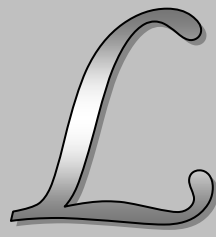
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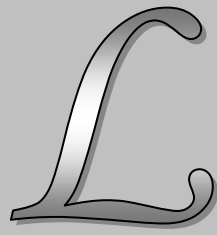
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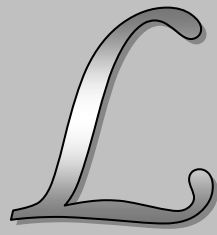
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# NOTATIONS

| <b><u>Symbol</u></b> | <b><u>Description</u></b>                                 |
|----------------------|-----------------------------------------------------------|
| HSC                  | High Strength Concrete                                    |
| SCC                  | Self Compacting Concrete                                  |
| WRA                  | Water Reducing Admixture                                  |
| HRWR                 | High Range Water Reducer                                  |
| SRA                  | Shrinkage Reducing Admixture                              |
| SP                   | Superplasticizer.                                         |
| $C_1$                | Coefficient (effect of the creep of concrete).            |
| $C_a$                | Creep strain after cracking (microstrain).                |
| $C_b$                | Creep strain before cracking (microstrain).               |
| $C_f$                | Final creep strain (microstrain).                         |
| $R_a$                | Degree of restraint after cracking                        |
| $R_b$                | Degree of restraint before cracking (at wall center)      |
| $d$                  | Bar diameter (mm).                                        |
| $E_c$                | Modulus of elasticity of concrete (GPa).                  |
| $E_s$                | Modulus of elasticity of steel (GPa).                     |
| $E_t$                | Initial tangent modulus (GPa).                            |
| $e_{sh}$             | Free shrinkage strain of concrete (microstrain).          |
| $e_{shn}$            | Net shrinkage strain of concrete (microstrain).           |
| $e_{th}$             | Thermal strain in concrete (microstrain).                 |
| $e_{ult}$            | Elastic tensile strain capacity of concrete(microstrain). |

|                 |                                                                                            |
|-----------------|--------------------------------------------------------------------------------------------|
| $\epsilon_v$    | Total volume-change strain of concrete(microstrain).                                       |
| $\epsilon_{cs}$ | Shrinkage strain(microstrain).                                                             |
| $\epsilon_{te}$ | Total thermal contraction after peak<br>temperature due to heat of hydration(microstrain). |
| $f_b$           | Average bond strength between concrete and steel(MPa)                                      |
| $f_c'$          | Compressive strength of concrete cylinders (MPa)                                           |
| $f_{tu}$        | Tensile strength of concrete (MPa).                                                        |
| $H$             | Height of wall (mm).                                                                       |
| $Y$             | Crack height (mm).                                                                         |
| $K$             | Constant                                                                                   |
| $\Delta R$      | Change in restraint factors.                                                               |
| $S$             | Final crack spacing(mm)                                                                    |
| $S_{ave}$       | Final average crack spacing(mm)                                                            |
| $S_{max}$       | Maximum crack spacing(mm)                                                                  |
| $S_{min}$       | Minimum crack spacing(mm)                                                                  |
| $S_c$           | The shrinkage of concrete (microstrain)..                                                  |
| $S_p$           | The shrinkage of neat cement paste (microstrain)..                                         |
| $a$             | The aggregate content in the concrete(by volume).                                          |
| $E_g$           | Modulus of elasticity for aggregate (GPa).                                                 |
| $\mu_c$         | Poisson's ratio for concrete.                                                              |
| $\mu_g$         | Poisson's ratio for aggregate.                                                             |
| $W$             | Final crack width (mm).                                                                    |
| $W_i$           | Width of crack at the onset of cracking(mm)                                                |
| $W_{ave}$       | Average crack width (mm).                                                                  |
| $W_{max}$       | Maximum crack width (mm).                                                                  |
| $W/C$           | Water-cement ratio                                                                         |

|                    |                                                      |
|--------------------|------------------------------------------------------|
| $\rho_c$           | Density of concrete(kg/m <sup>3</sup> )              |
| $\rho$             | Steel ratio                                          |
| $d_c$              | Thickness of concrete cover(mm)                      |
| $A$                | Effective tension area of concrete(mm <sup>2</sup> ) |
| $f_s$              | Steel stress at crack position(MPa)                  |
| $C_t$              | Coefficient of thermal expansion of concrete.        |
| $T_e$              | Effective temperature change in members.             |
| $n$                | Modular ratio ( $E_s/E_c$ ).                         |
| $\rho_R, KR1, KR2$ | Factors was estimated by STATISTICA program          |
| $L/H$              | Length to height ratio.                              |
| $\rho_h$           | Horizontal reinforcement ratio(%).                   |
| $\rho_v$           | Vertical reinforcement ratio(%).                     |
| $T$                | Average temperature degree of drying period(         |
| $^{\circ}C$ ).     |                                                      |
| $RH$               | Average relative humidity of drying period (%).      |
| $a, b, c, d, e, f$ | Regression coefficient of independent variables.     |

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