Republic of Iraq
Ministry of Higher Education and
Scientific Research
University of Babylon
College of Engineering
Civil Department

CONTROL OF SHRINKAGE CRACKING IN END AND BASE RESTRAINED REINFORCED CONCRETE WALLS

A Thesis

Submitted to the College of Engineering /
University of Babylon in partial
Fulfillment of the Requirements
for the Degree of Master
in Construction Materials
Engineering

By

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April -2009 Gamadi Aual -1430

جمهورية العراق
وزارة التعليم العالي و البحث العلمي
جامعة بابل
كلية الهندسة
القسم المدني

السيطرة على تشققات الانكماش في الجدران الخرسانية المسلحة والمقيدة من القاعدة والجوانب

رسالة
مقدمة إلى كلية الهندسة في جامعة بابل
كجزء من متطلبات نيل درجة الماجستير
في هندسة المواد الإنشائية

من قبل

عباس كاظم مشجل

إشراف

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نيسان- 2009 جمادي أول- 1430

بسم الله الرحمن الرحيم

وأن ليس للإنسان إلا ما سعى

صدق الله العلي العظيم

سورة النجم - الآية (39)

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Certification

We certify that this thesis titled "**Control of Shrinkage Cracking in Restrained Base and End Reinforced Concrete Walls**" was prepared by "**Abbas Kadhim Mushchil**" 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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Certification

We certify that we have read this thesis, titled " **Control of Shrinkage Cracking in Restrained Base and End Reinforced Concrete Walls** ", and as an examining committee examined the student "**Abbas Kadhim Mushchil**" 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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Signature **Name** : Asst. Prof. Mr. Samir A. Al-Mashhadi (Supervisor) **Date** : / / 2009	**Signature** **Name** : Asst. Prof. Dr. Ghalib M. Habeeb (Supervisor) **Date** : / /2009
Approval of the Civil Engineering Department. Head of the Civil Engineering Department. **Signature** **Name** : Prof. Dr. Ammar Y. Ali **Date** : / / 2009	
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Appendix (A)

Strains Measurement

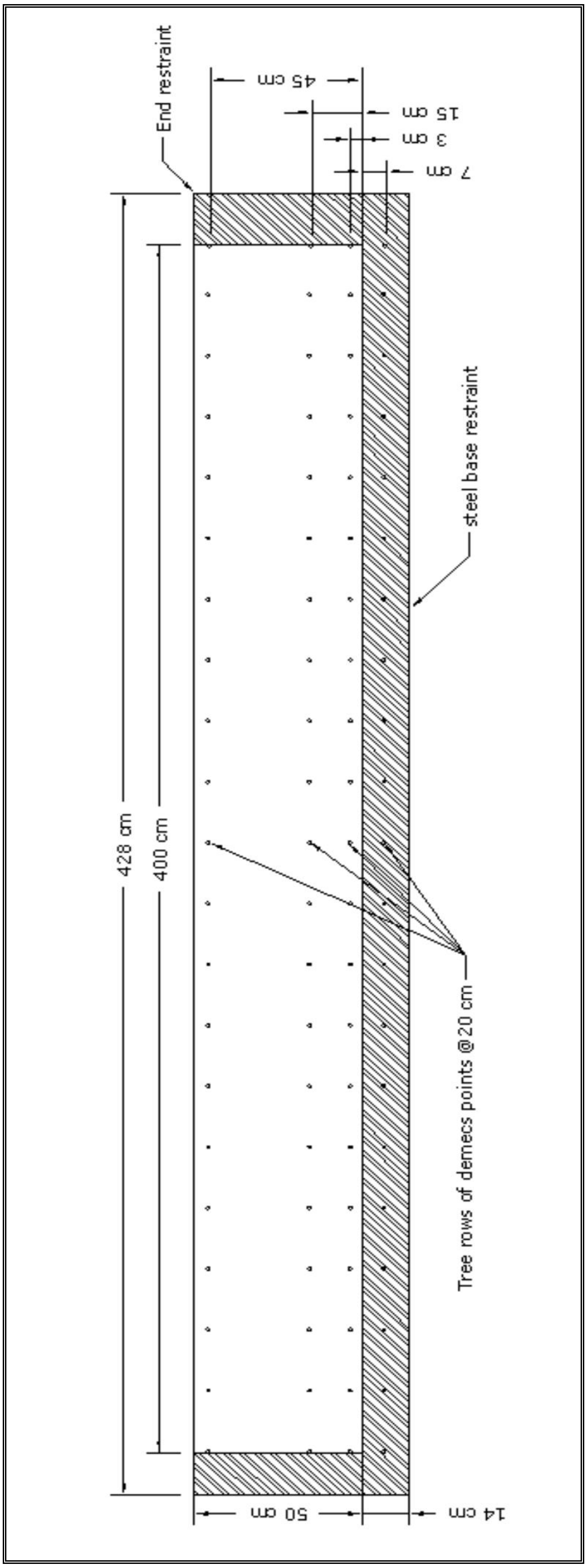


Figure (A-1) Fixing of demecs on wall (8w) and (8S)

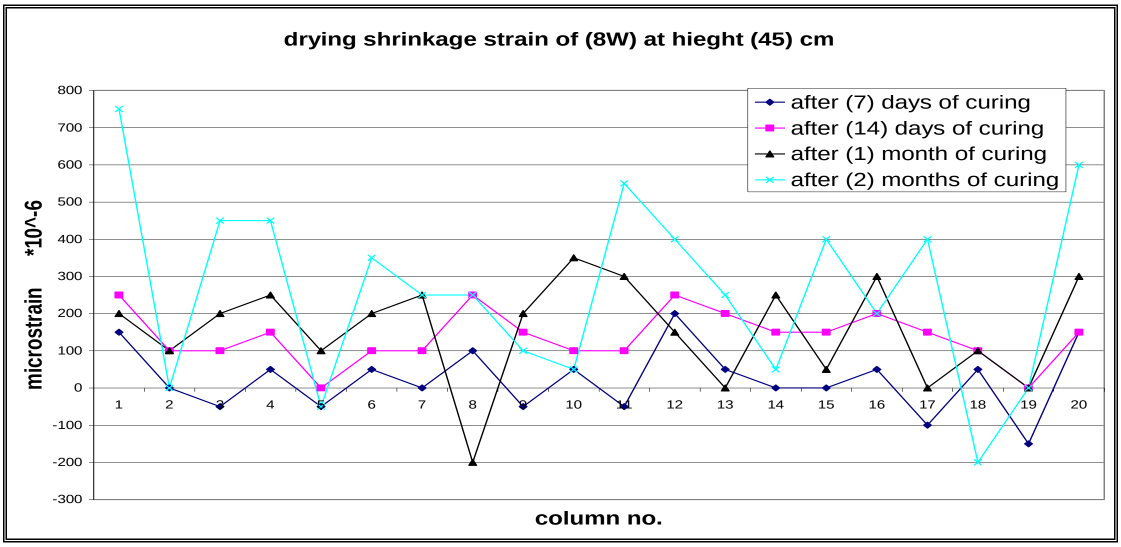


Figure (A-2) drying shrinkage strain of wall (8W) at height (45) cm

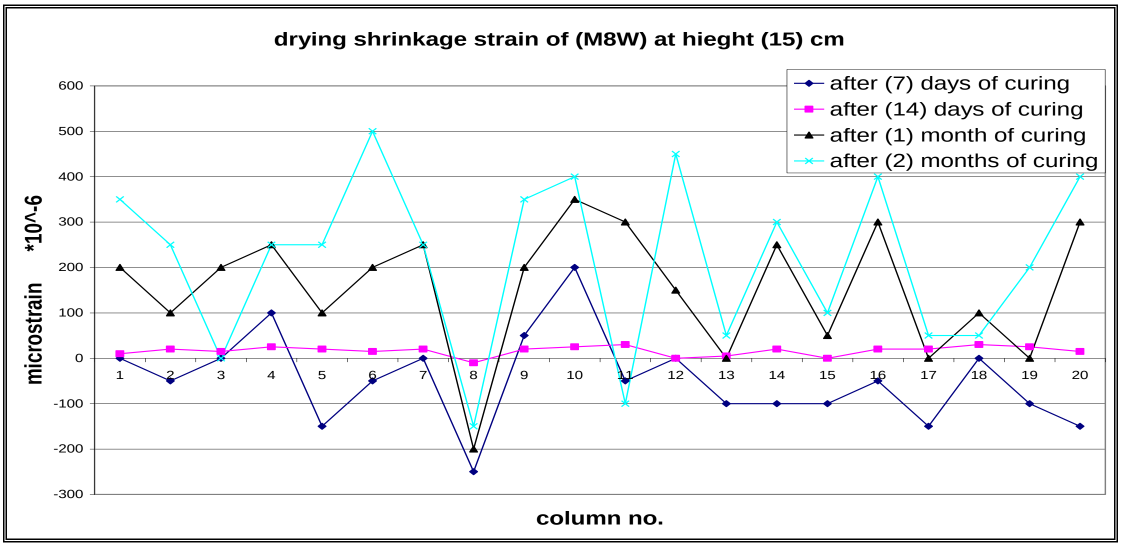


Figure (A-3) drying shrinkage strain of wall (8W) at height (15) cm

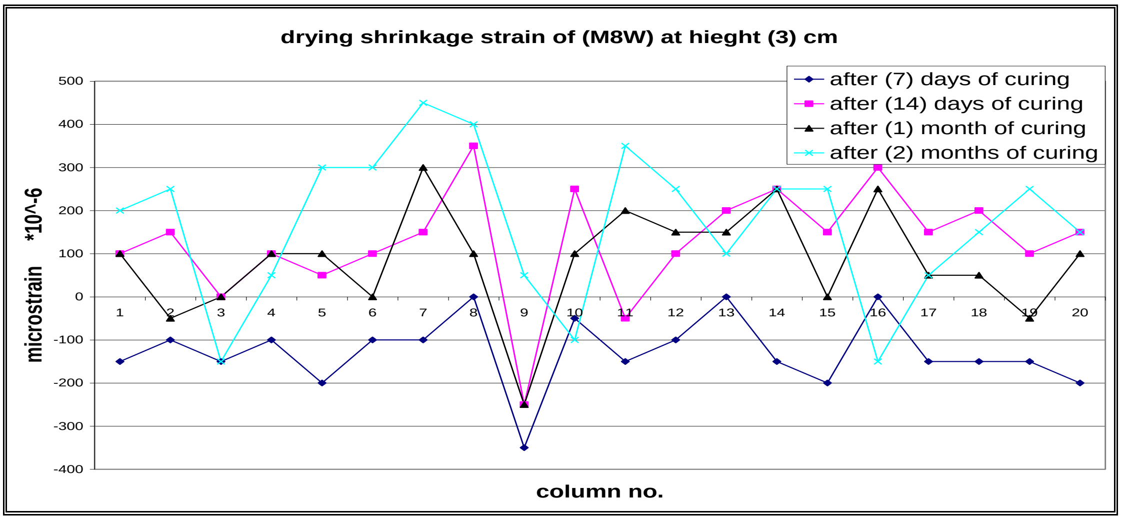


Figure (A-4) drying shrinkage strain of wall (8W) at height (3) cm

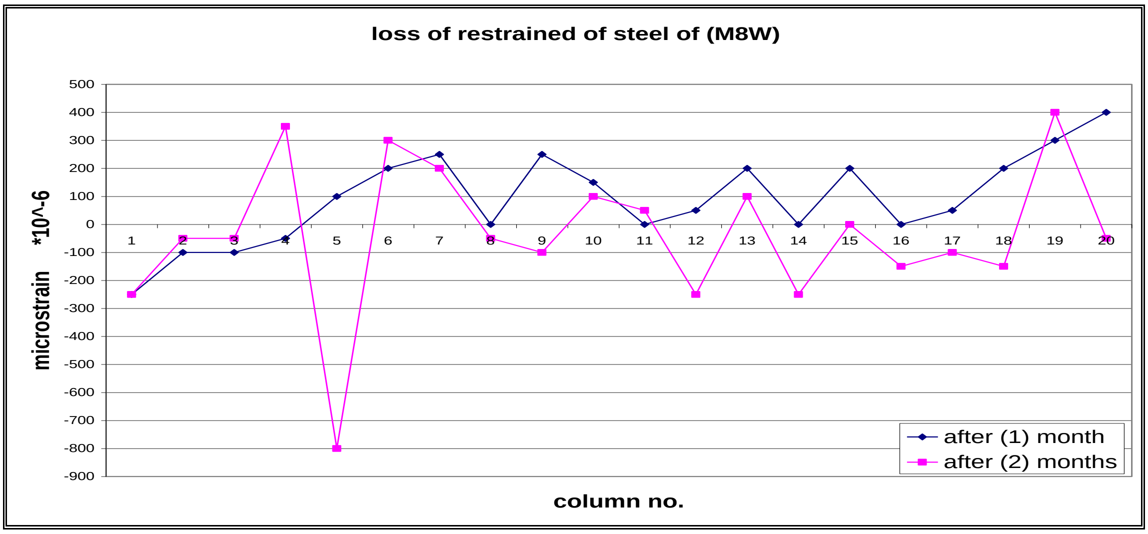


Figure (A-5) Loss of base restraint of wall (8W)

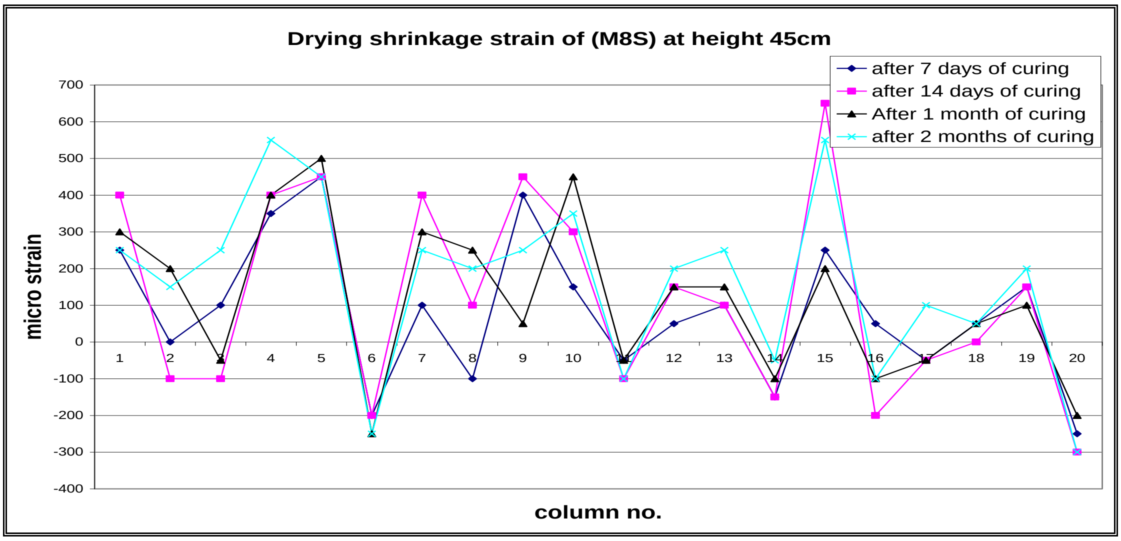


Figure (A-6) drying shrinkage strain of wall (8S) at height (45) cm

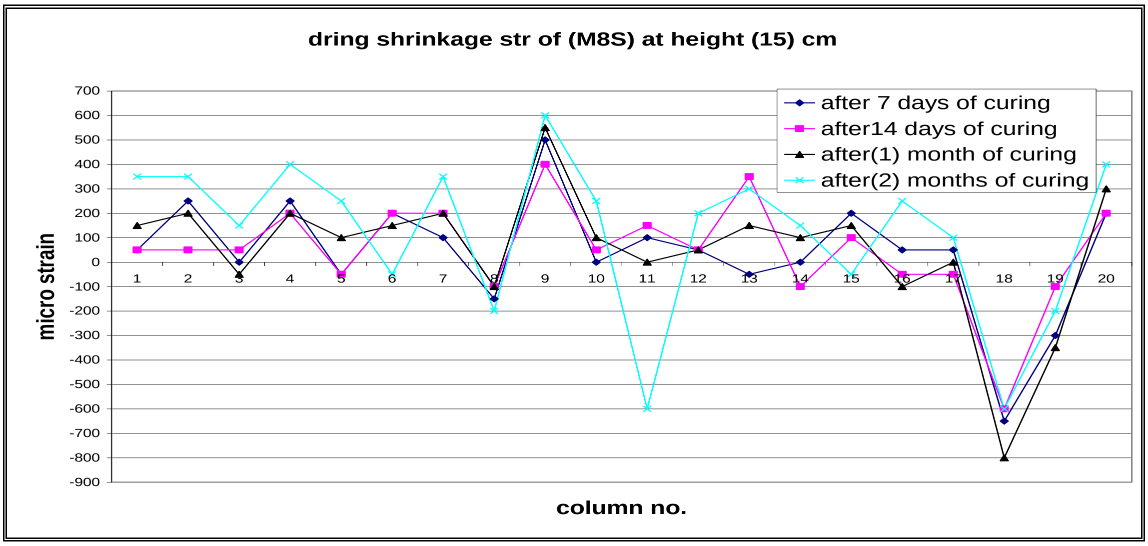


Figure (A-7) drying shrinkage strain of wall (8S) at height (15) cm

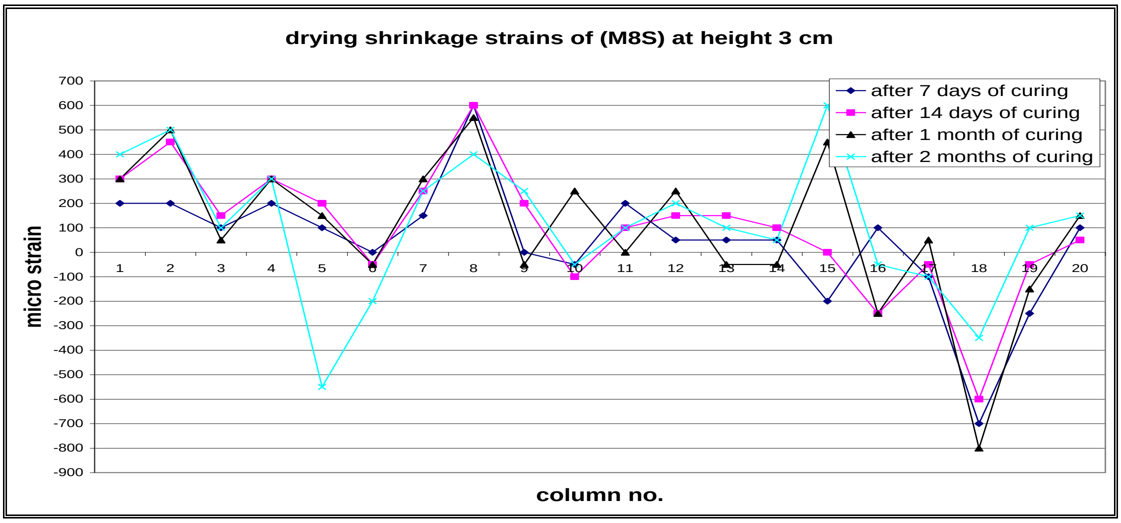


Figure (A-8) drying shrinkage strain of wall (8S) at height (3) cm

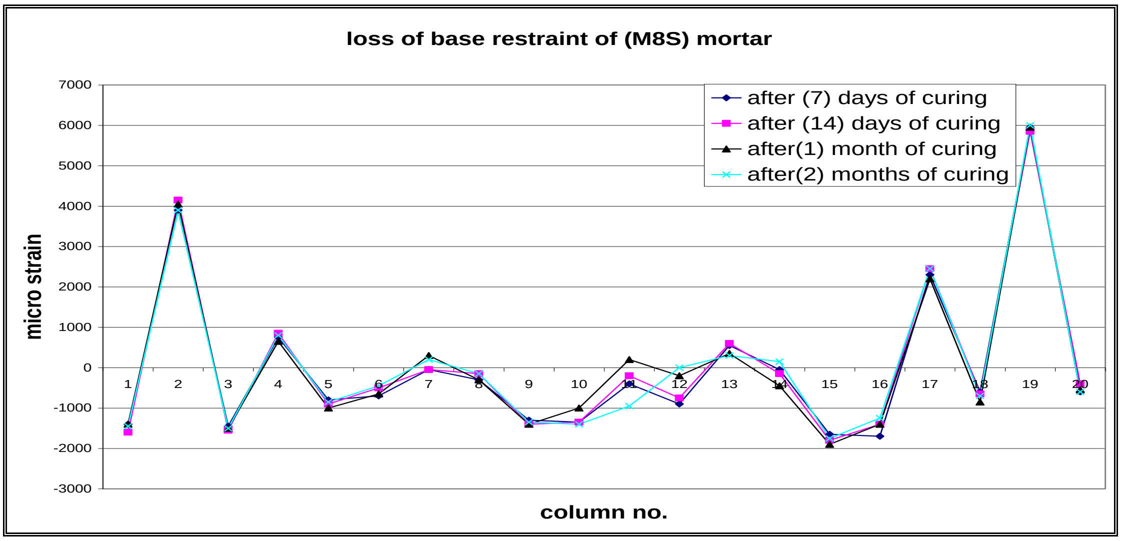


Figure (A-9) Loss of base restraint of wall (8S)

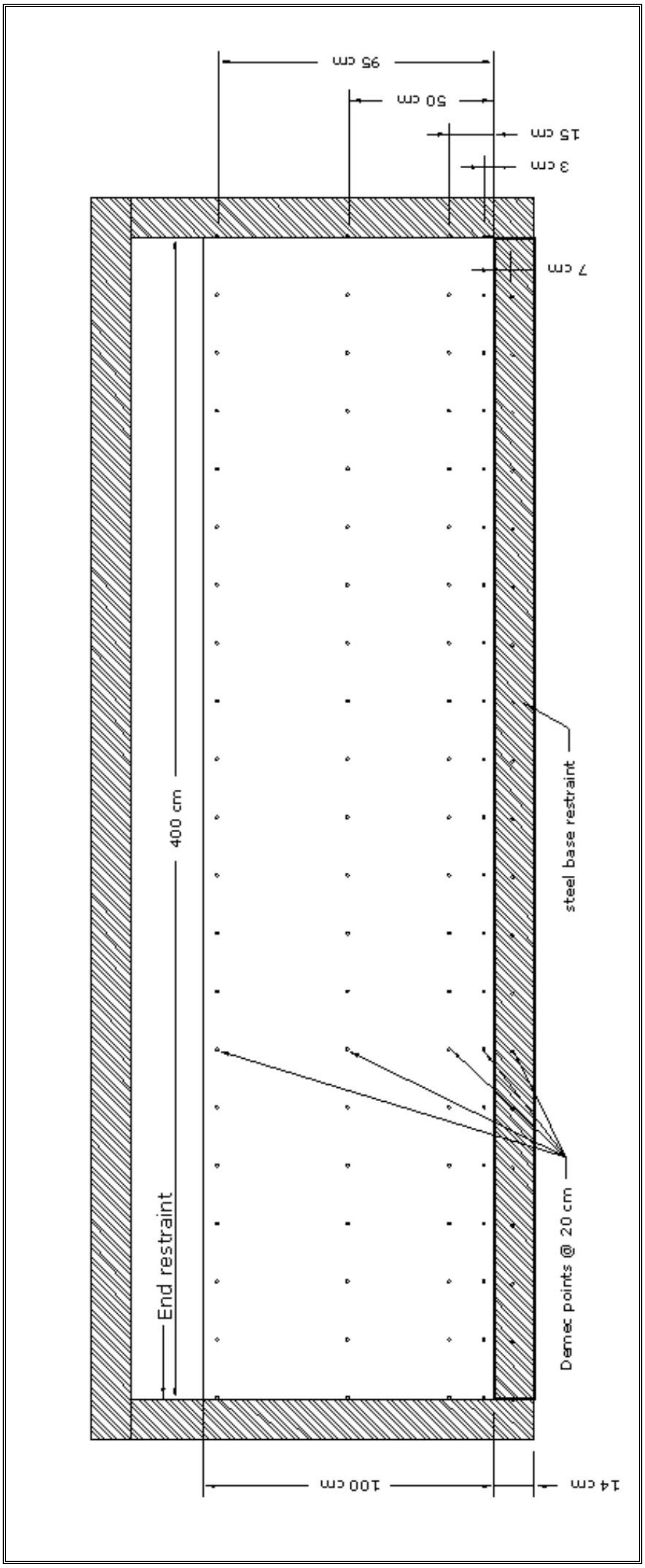


Figure (A-10) Fixing of demecs on wall (4w) and (4S)

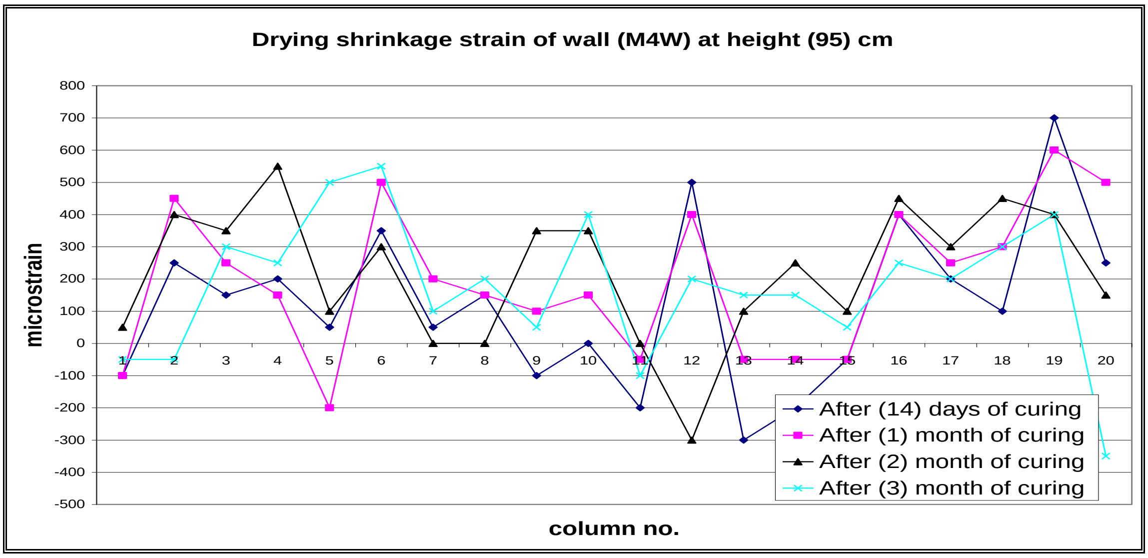


Figure (A-11) drying shrinkage strain of wall (4W) at height (95) cm

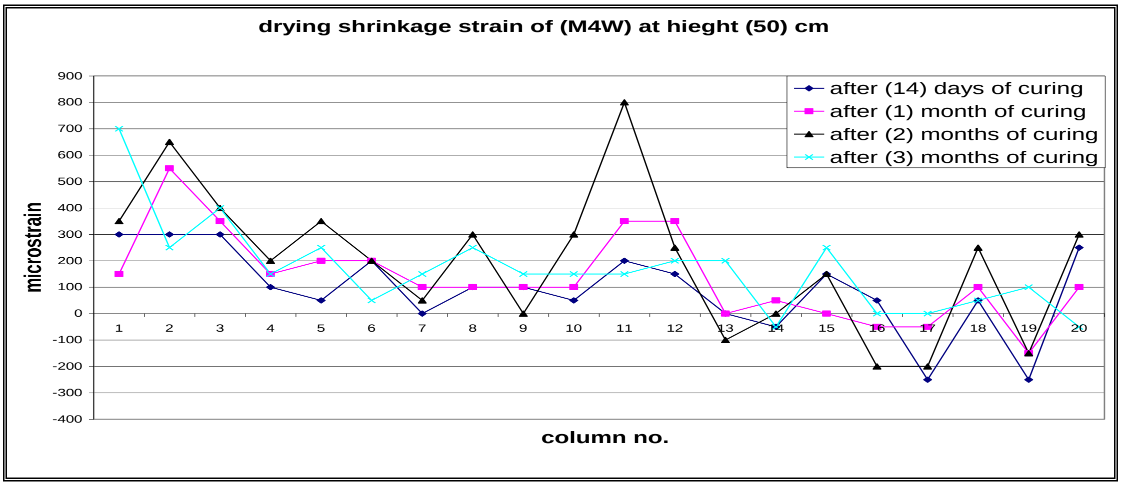


Figure (A-12) drying shrinkage strain of wall (4W) at height 50) cm

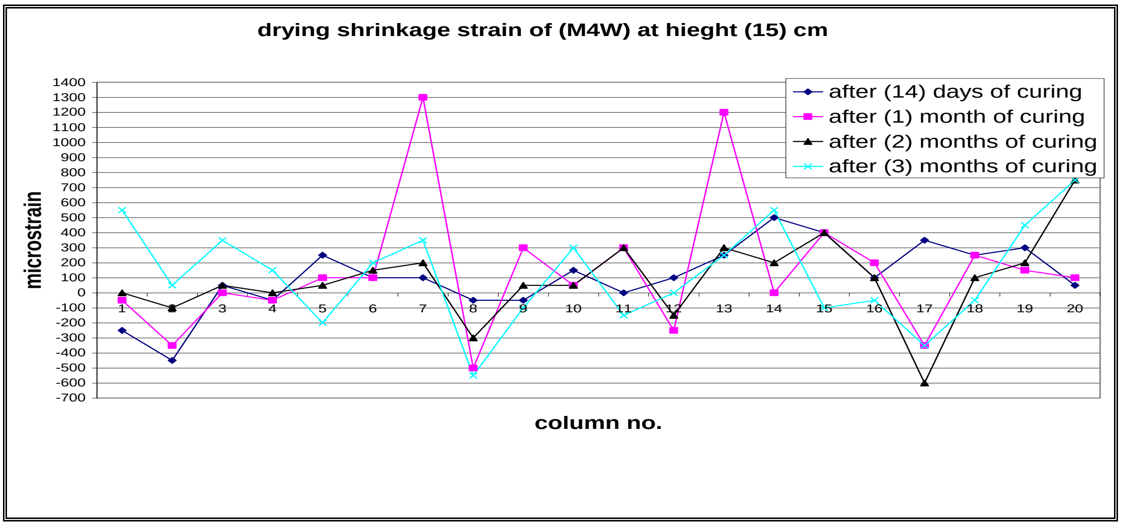


Figure (A-13) drying shrinkage strain of wall (4W) at height (15) cm

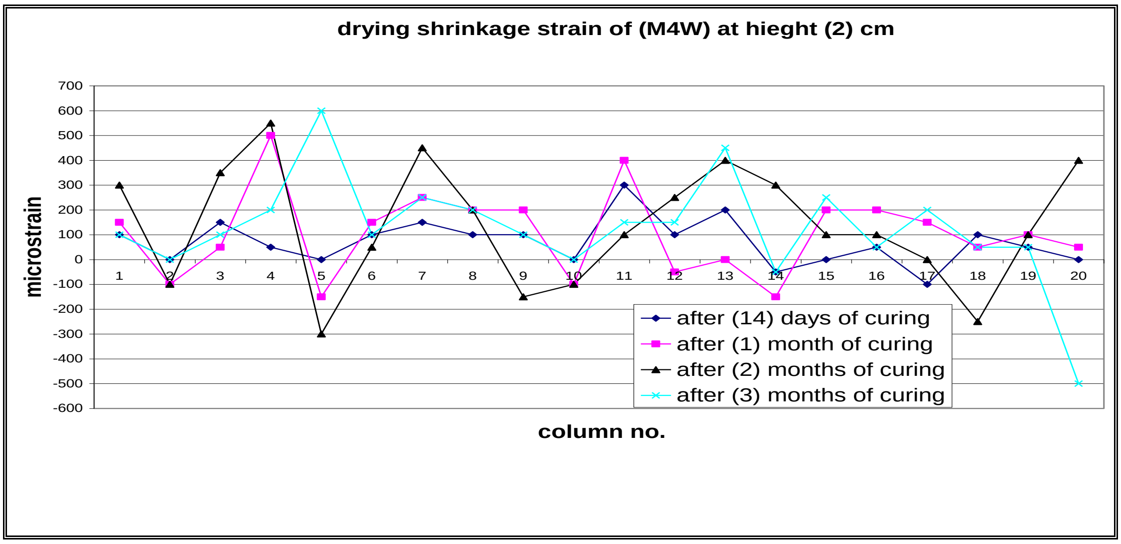


Figure (A-14) drying shrinkage strain of wall (4W) at height (3) cm

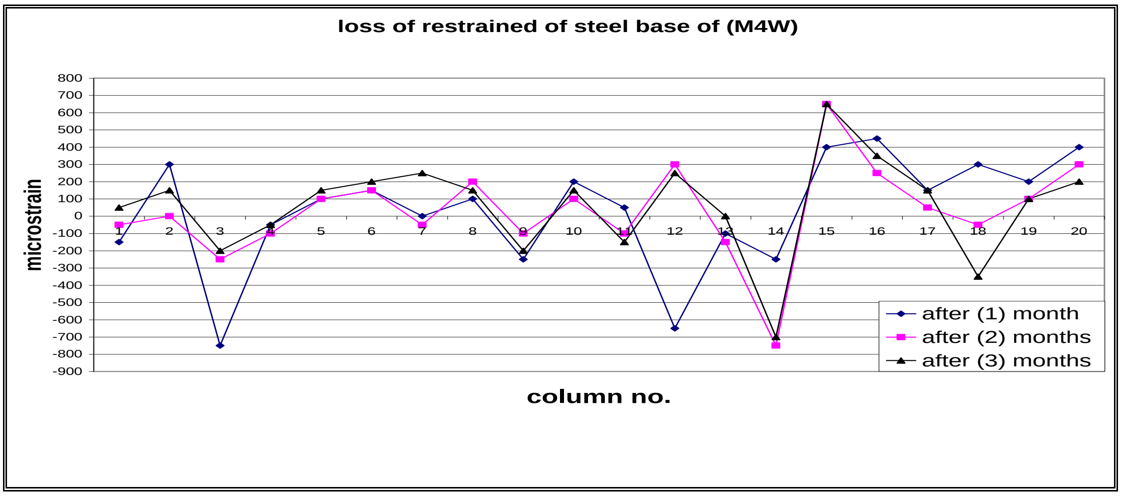


Figure (A-15) Loss of base restraint of wall (4W)

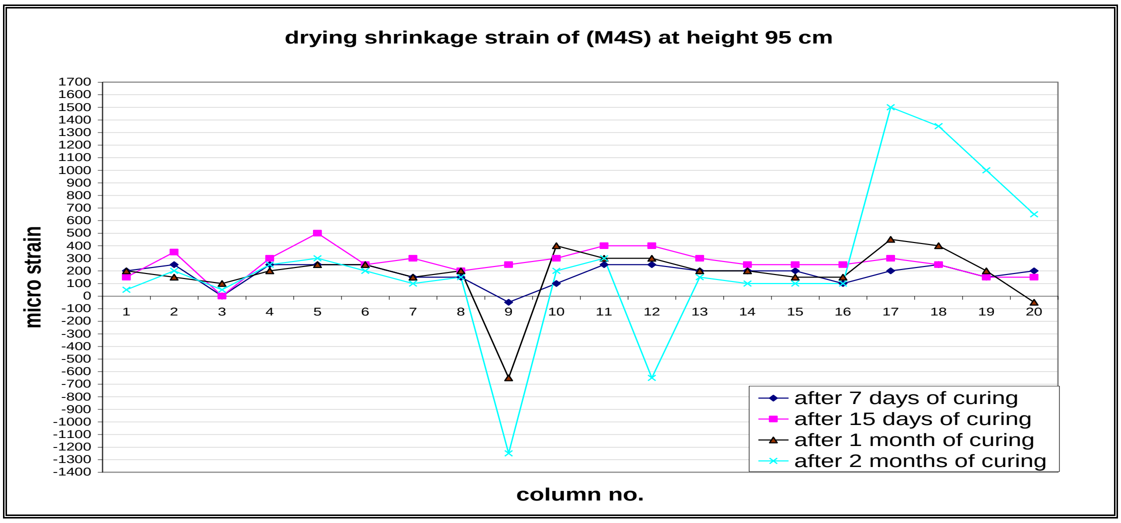


Figure (A-16drying shrinkage strain of wall (4S) at height (95) cm

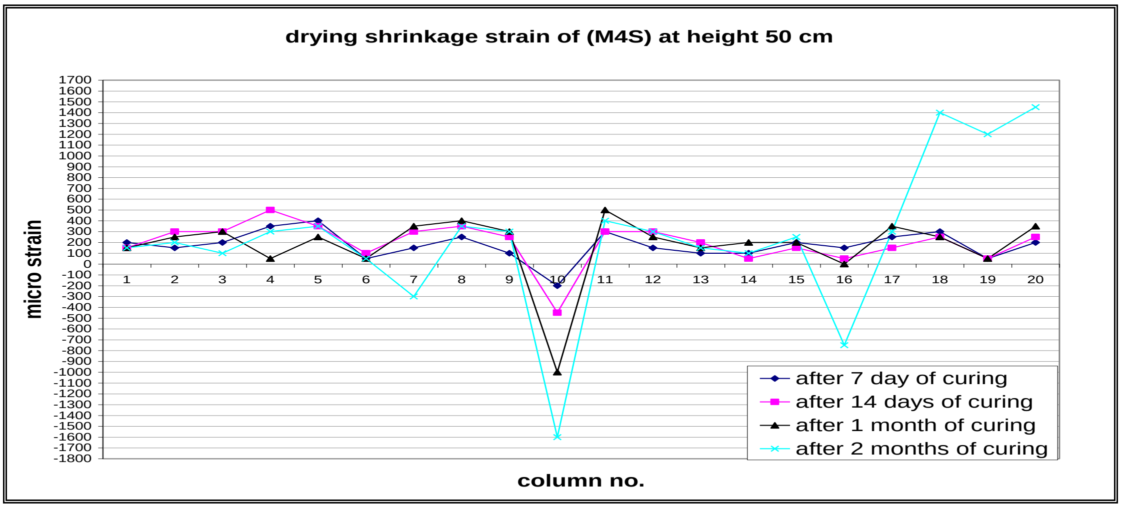


Figure (A-17) drying shrinkage strain of wall (4S) at height (50) cm

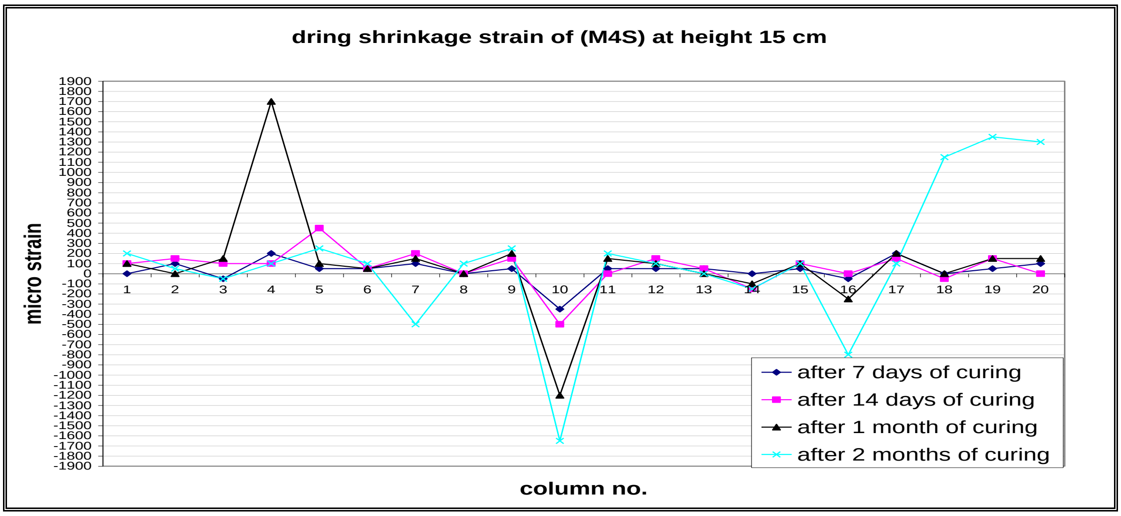


Figure (A-18) drying shrinkage strain of wall (4S) at height (15) cm

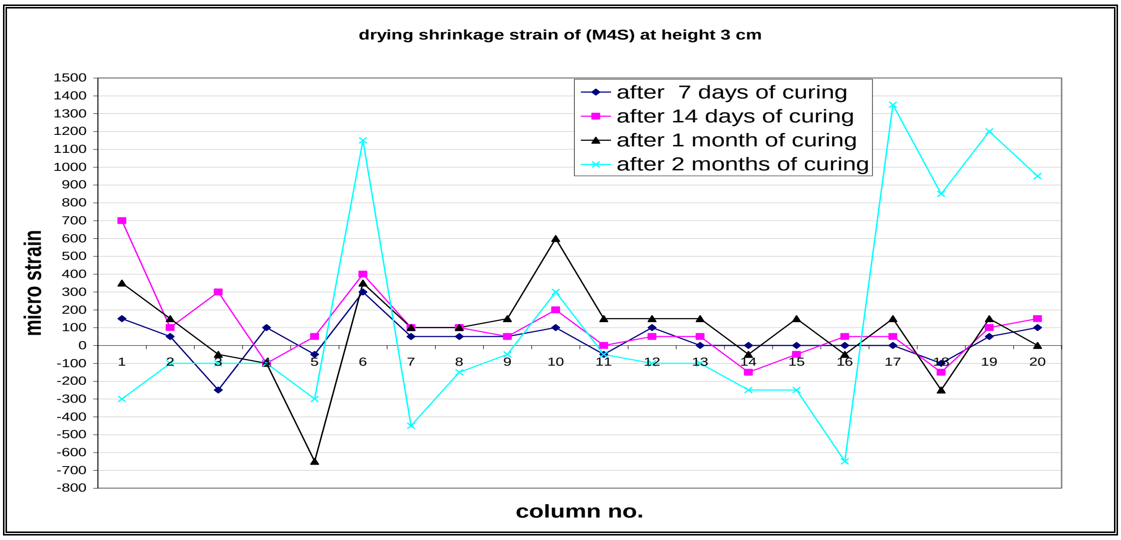


Figure (A-19) drying shrinkage strain of wall (4S) at height (3) cm

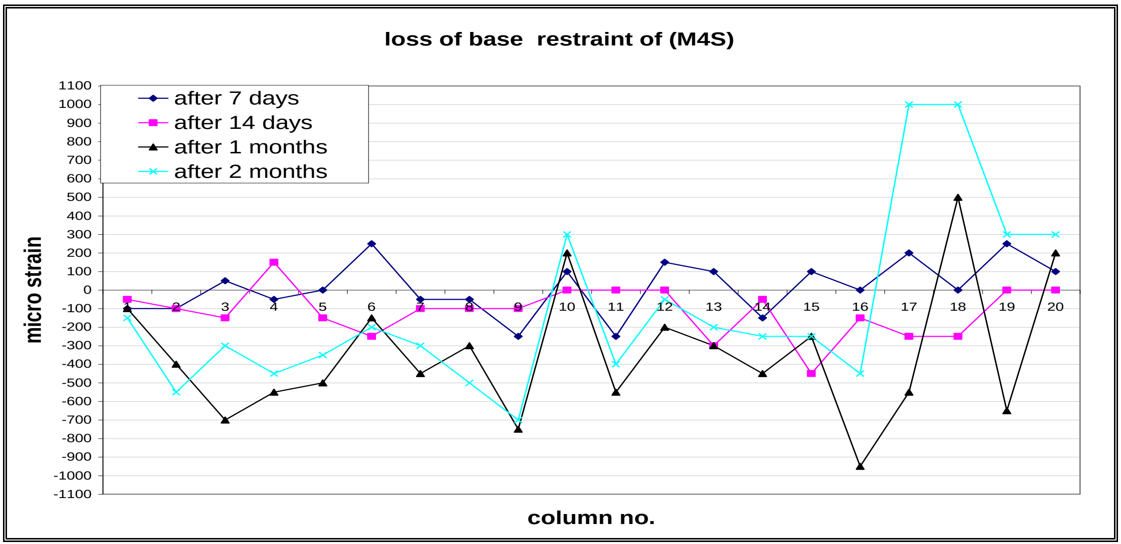


Figure (A-20) Loss of base restraint of wall (4S)

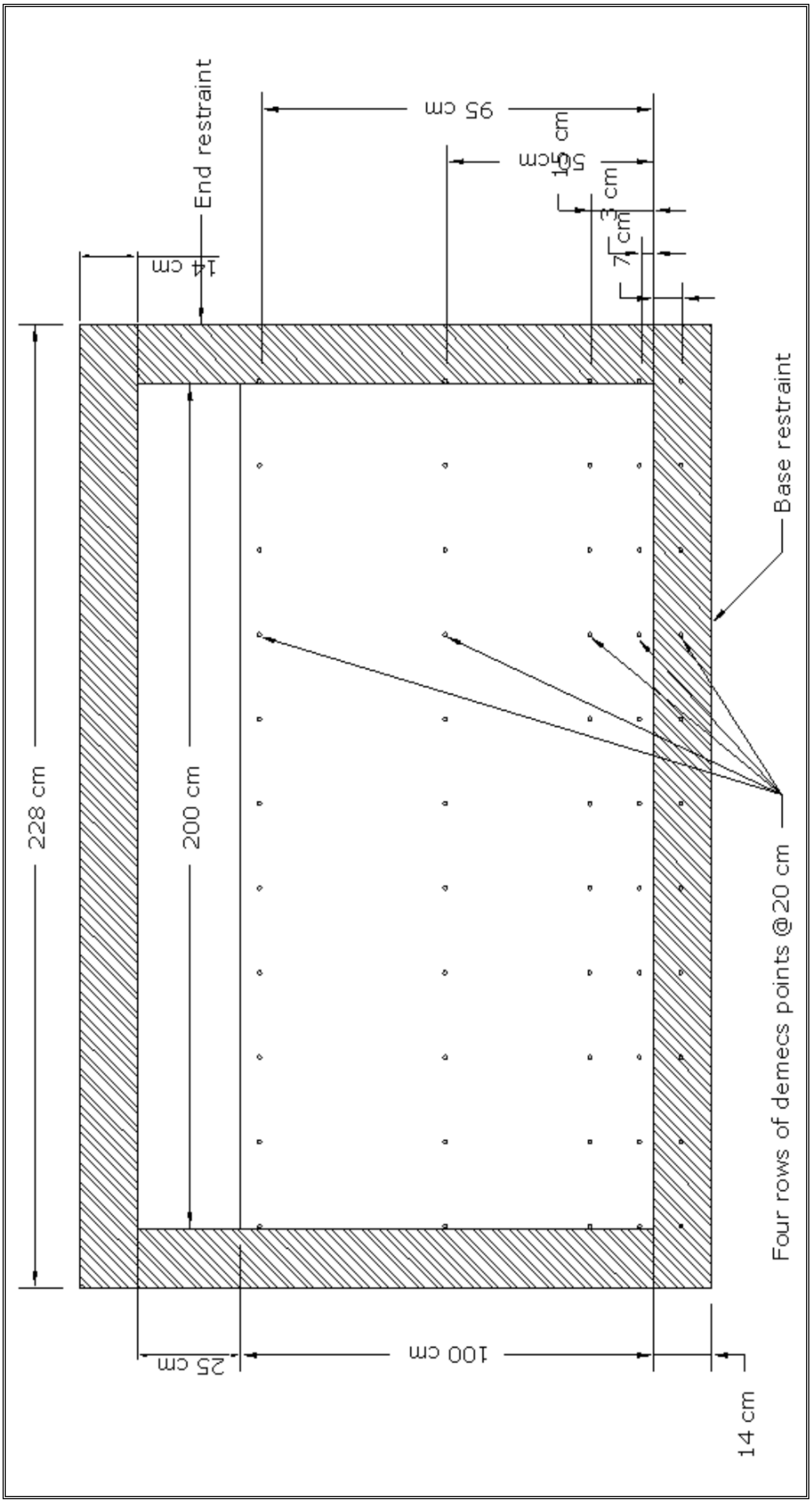


Figure (A-21) Fixing of demecs on wall (2w) and (2S)

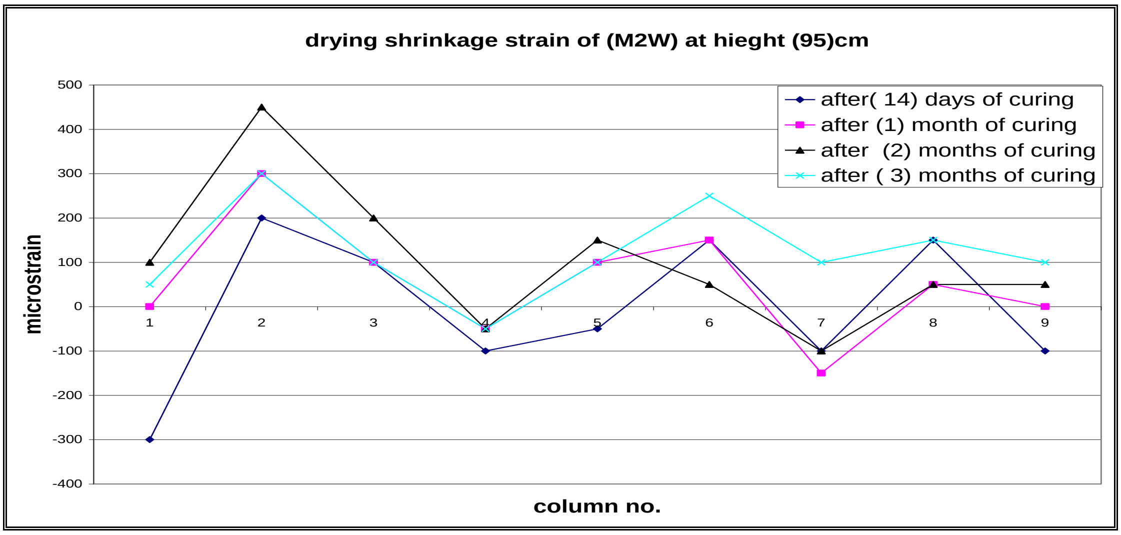


Figure (A-22) drying shrinkage strain of wall (2W) at height (95) cm

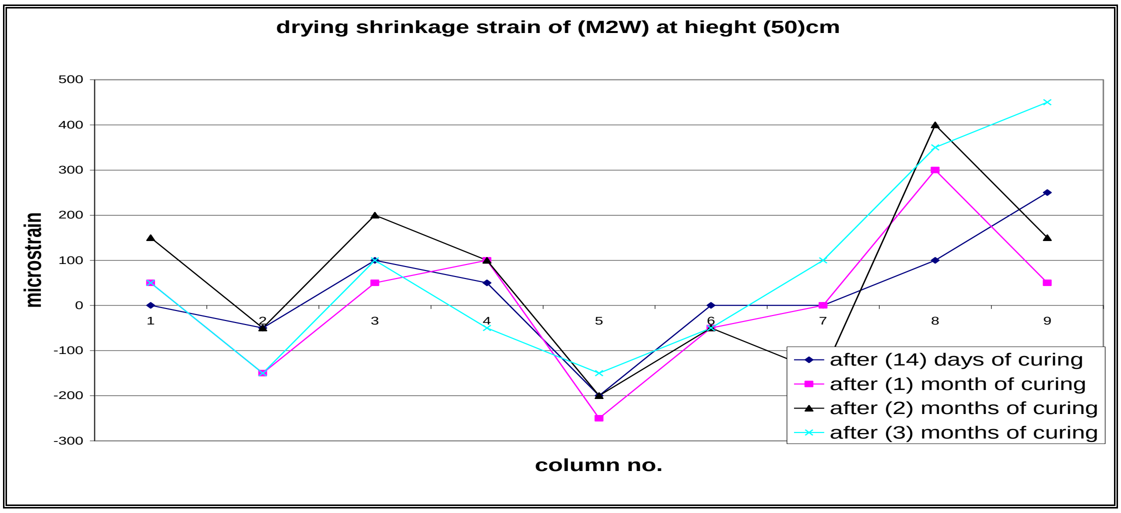


Figure (A-23) drying shrinkage strain of wall (2W) at height (50) cm

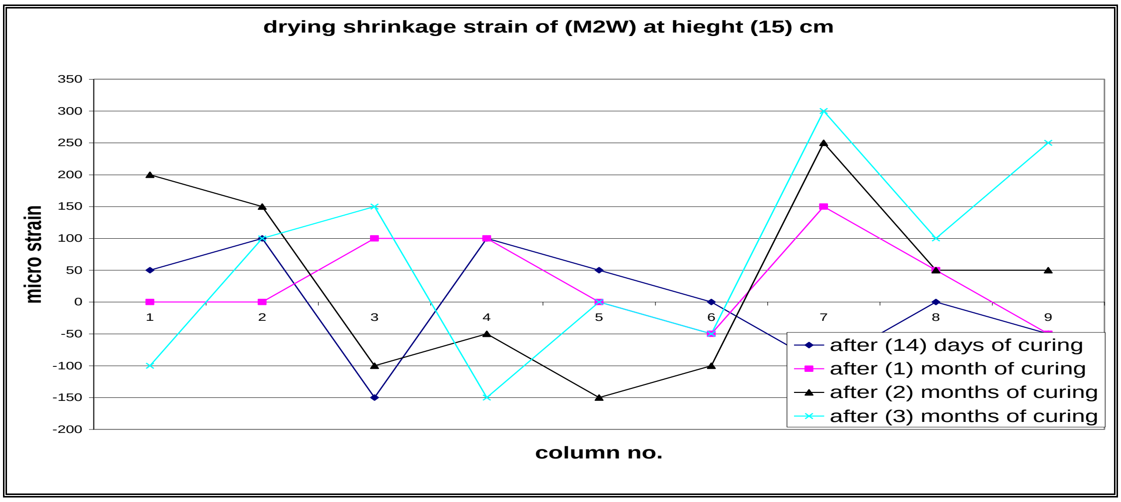


Figure (A-24) drying shrinkage strain of wall (2W) at height (15) cm

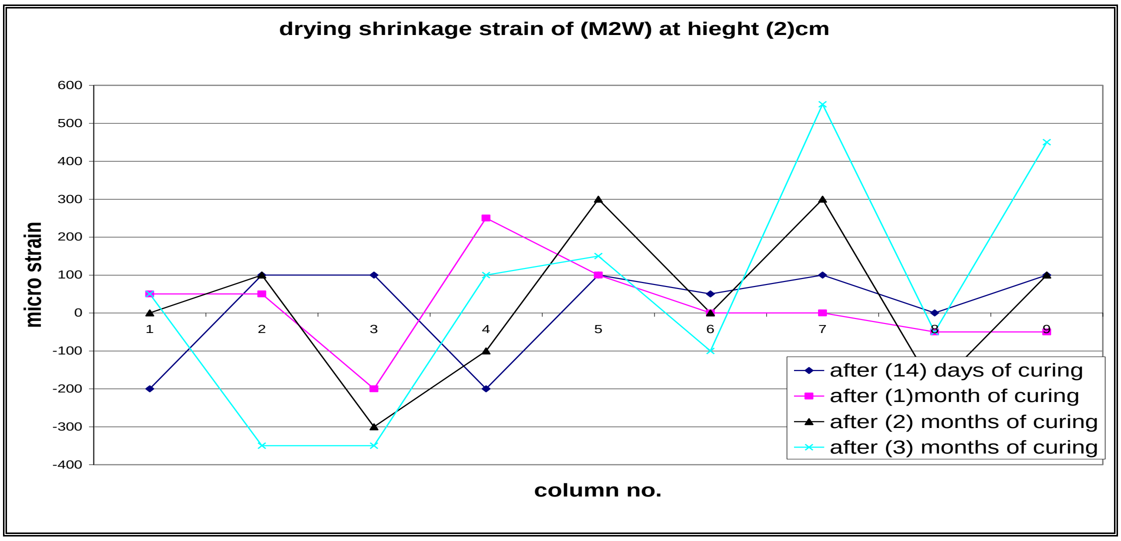


Figure (A-25) drying shrinkage strain of wall (2W) at height (3) cm

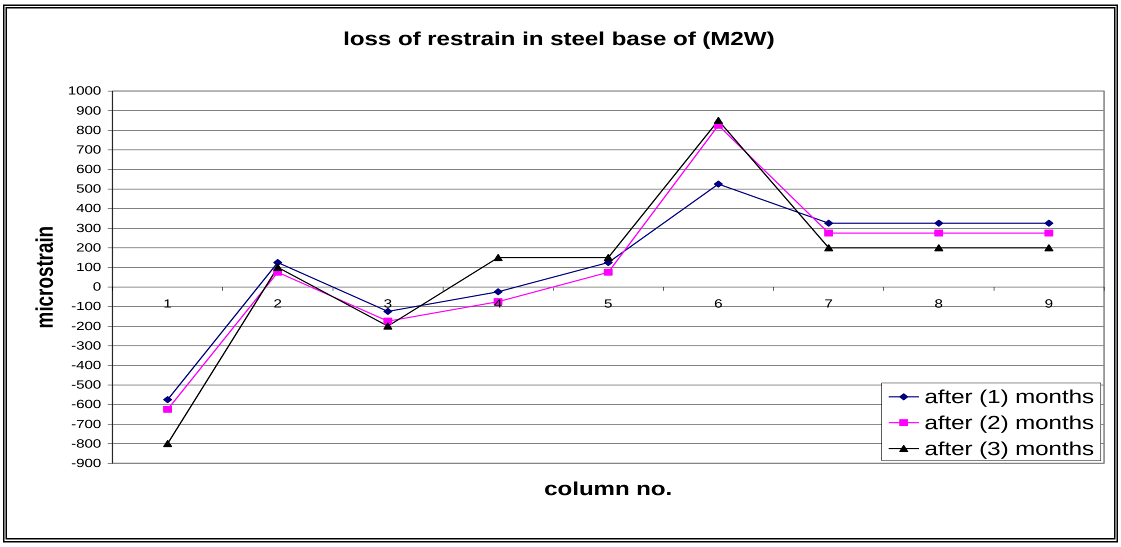


Figure (A-26) Loss of base restraint of wall (2W)

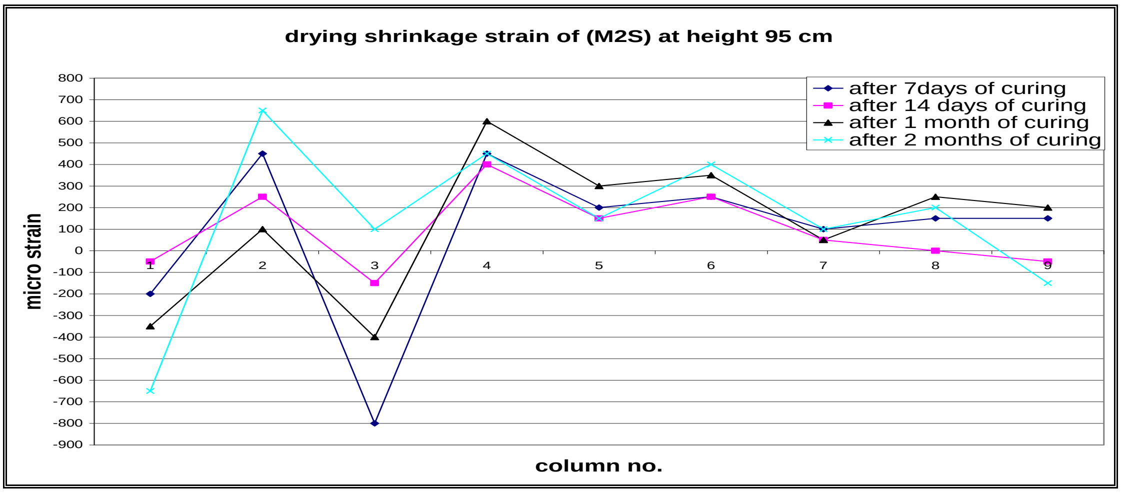


Figure (A-27) drying shrinkage strain of wall (2S) at height (95) cm

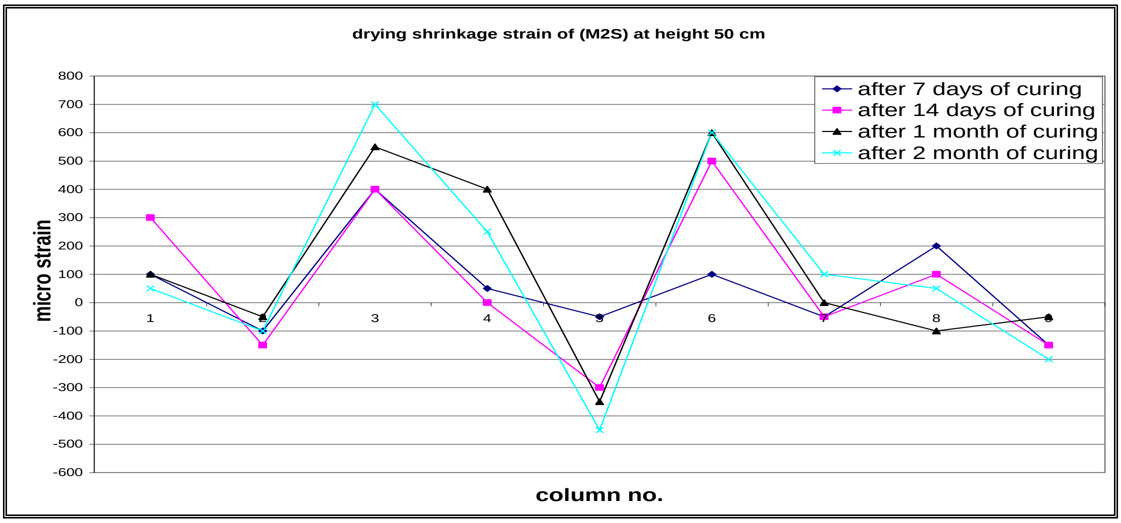


Figure (A-28) drying shrinkage strain of wall (2S) at height (50) cm

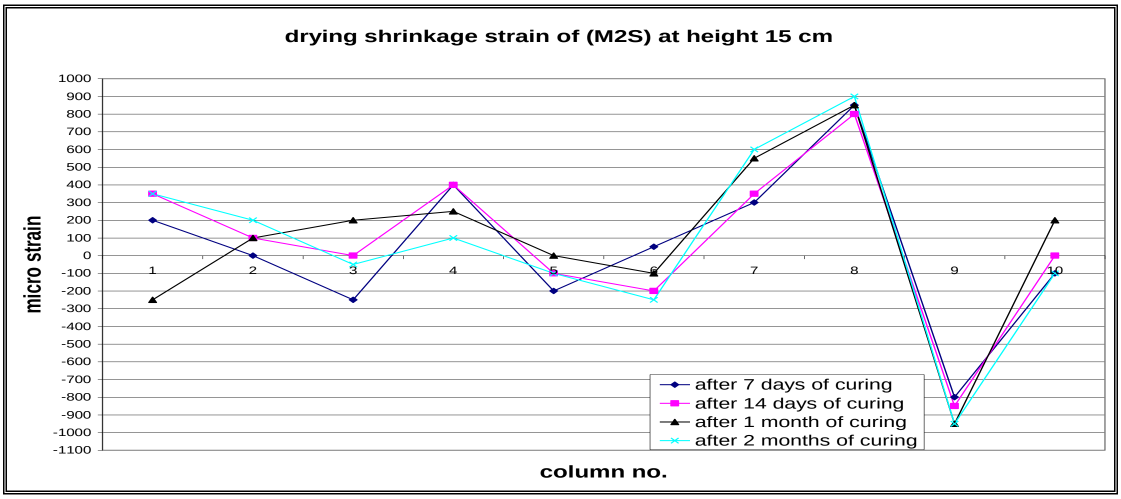


Figure (A-29) drying shrinkage strain of wall (2S) at height (15) cm

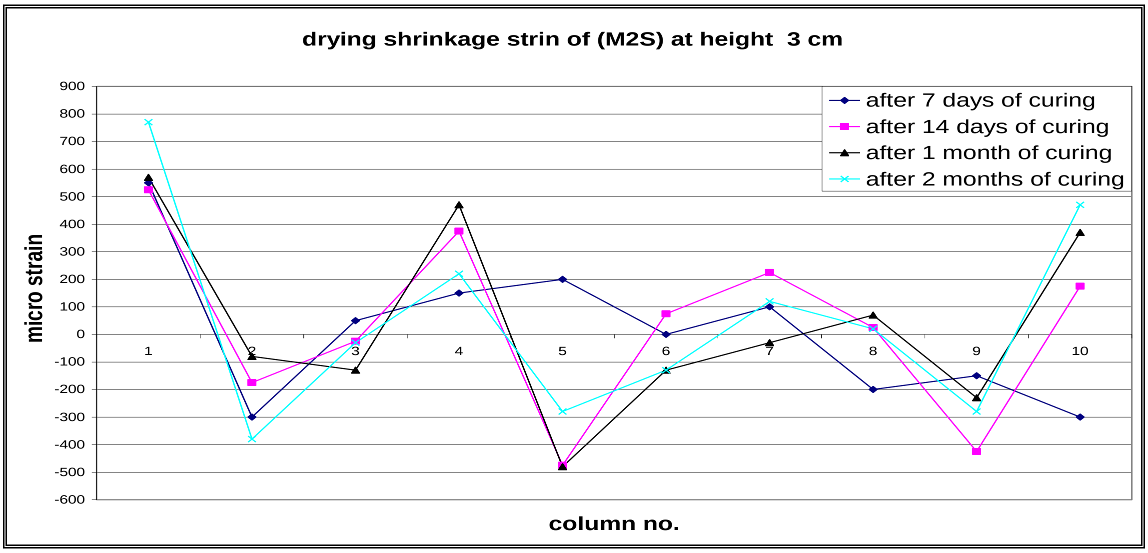


Figure (A-30) drying shrinkage strain of wall (2S) at height (3) cm

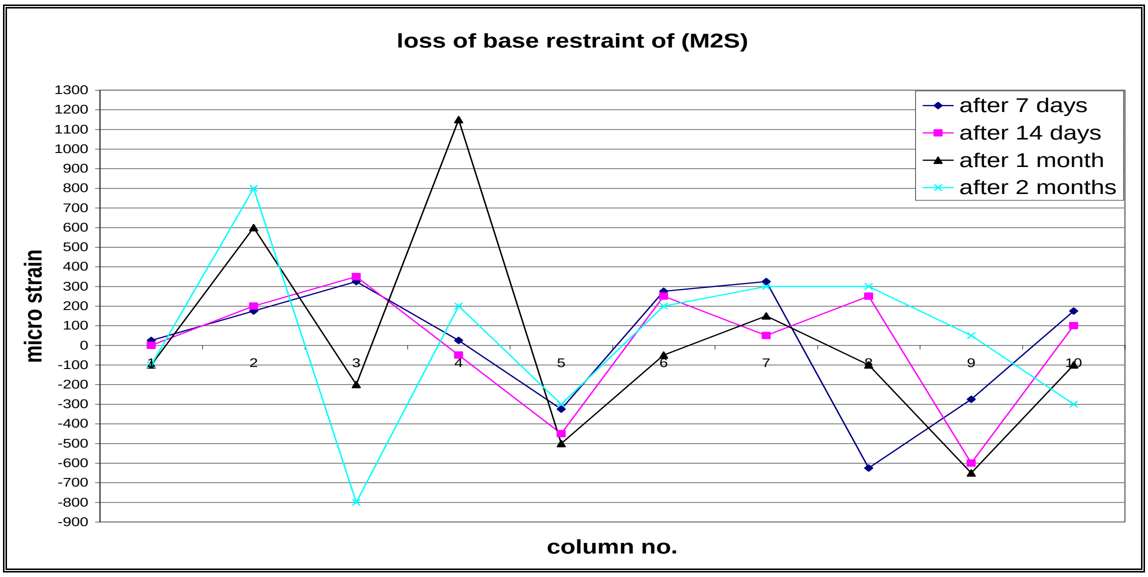


Figure (A-31) Loss of base restraint of wall (2S)

<u>**Appendix (B)**</u>

Cracks width data

Table (B-1) cracks width data of wall (2W)

		Crack width (mm)
Crack no.		1
Distance from the left edge (cm)		55
Interval of crack length (cm)		0-21
Height of crack widths observations (cm)	20	
	15	0.02
	10	0.04
	5	0.1

Table (B-2) cracks width data of wall (4W)

		Crack width (mm)						
Crack no.		1	2	3	4	5	6	7
Distance from the left edge (cm)		30	95	135	175	205	295	335
Interval of crack length (cm)		15-25	45-60	0-25	0-45	0-20	0-30	0-20
Height of crack widths observations (cm)	60							
	55		0.02					
	50		0.03					
	45							
	40							
	35				0.04		0.02	
	30							
	25	0.02		0.04	0.08		0.04	
	20	0.02						
	15			0.08	0.3	0.02	0.08	0.03
	10							
	5			0.03	0.2	0.04	0.02	0.04

Table (B-3) cracks width data of wall (8W)

		Crack width (mm)								
Crack no.		1	2	3	4	5	6	7	8	9
Distance from the left edge (cm)		145	165	200	220	245	265	320	350	385
Interval of crack length (cm)		5-25	10-25	0-15	0-40	0-30	5-25	0-15	10-24	0-23
Height of crack widths observations (cm)	60									
	55									
	50									
	45									
	40									
	35				0.02					
	30									
	25		0.02		0.06	0.02			0.02	
	20	0.02					0.02			
	15		0.03		0.16	0.08			0.03	0.02
	10	0.04					0.08			
	5			0.02	0.1	0.04		0.02		0.04

Table (B-4) cracks width data of wall (2S)

		Crack width (mm)											
Crack no. (according initiation)		2	5	5	3	3	3	1	1	2	4	2	1
Distance from the left edge (cm)		25	32	48	56	60	77	99	116	126	141	165	180
Interval of crack length (cm)		0-36	0-15	10-48	23-47	5-19	0-67	5-60	3-52	0-45	0-22	5-45	5-52
Height of crack widths observations (cm)	70												
	65												
	60												
	55						0.02						
	50							0.03	0.08				0.08
	45						0.04						
	40				0.03			0.03	0.14			0.08	0.04
	35			0.03			0.04			0.06			
	30				0.06			0.02	0.03			0.2	0.06
	25	0.06		0.04			0.04			0.04	0.02		
	20					0.03		0.02	0.01			0.1	0.06
	15	0.08		0.03			0.03			0.1	0.03		
	10					0.02		0.02	0.02			0.03	0.06
	5	0.06	0.03				0.02			0.06	0.02		
	0												

Table (B-5) cracks width data of wall (4S)

		Crack width (mm)													
Crack no. (according initiation)		5	4	2	3	2	4	1	1	2	1	2	3	3	2
Distance from the left edge (cm)		40	80	133	137	148	180	185	193	216	245	264	302	333	355
Interval of crack length (cm)		0-50	0-100	0-32	2-70	0-52	15-80	0-100	0-29	0-30	0-53	5-45	0-70	20-60	13-48
Height of crack widths observations (cm)	70		Pri.				0.02	Pri.							
	65		0.02		0.02			1.0					0.06		
	60						0.04								
	55		0.04		0.06			1.0					0.2	0.01	
	50						006								0.02
	45	0.02	0.05		0.1	0.02		0.6			0.02		0.2	0.04	
	40						0.1					0.04			0.04
	35	0.02	0. 14		0.14	0.03		0.6			0.03		0.28	0.04	
	30						0.1					0.02			0.04
	25	0.06	0.14	0.02	0.14	0.02		0.5	0.03	0.02	0.04		0.24	0.02	
	20						0.1					0.04			0.03
	15	0.03	0.06	0.08	0.14	0.02		0.4	0.1	0.02	0.04		0.14		
	10											0.04			
	5	0.02	0.02	0.02	0.1	0.06		0.3	0.2	0.02	0.02		0.14		

Note: pri. Means primary crack

Table (B-6) cracks width data of wall (8S)

		Crack width (mm)																	
Crack no. (according initiation)		3	3	3	4	2	5	3	3	4	4	1	4	5	4	3	5	2	3
Distance from the left edge (cm)		29	52	72	107	156	200	208	210	233	259	276	287	302	320	331	337	348	364
Interval of crack length (cm)		5-32	0-24	5-27	0-32	0-47	0-30	2-32	11-43	0-19	0-32	0-32	0-47	0-23	5-37	10-46	0-32	8-39	0-45
Height of crack widths observations (cm)	50												0.04						0.03
	45								0.02				0.04						0.08
	40																		
	35					0.02			0.04				0.14			0.02		0.03	0.1
	30	0.02		0.02											0.02				
	25				0.08	0.02	0.04		0.02		0.04	0.04	0.14			0.02	0.04	0.03	0.15
	20	0.02		0.04				0.06							0.02				
	15		0.04		0.14	0.1	0.04		0.02	0.03	0.04	0.03	0.1	0.02		0.03	0.08	0.03	0.1
	10	0.04		0.02				0.1							0.04				
	5		0.02		0.12	0.1	0.02			0.04	0.04	0.04	0.02	0.04			0.08		0.02

Abstract

This investigation is conducted to study the cracking behaviour due to restrained drying shrinkage of end and base restrained concrete walls with different length to height ratios. The choosing of mortar walls with a reduced scale models instead of concrete walls was to compensate the small shrinkage strains in the actual large concrete walls and to study this problem at complete cracking pattern in the short interval time.

Many characteristics of reinforced walls are measured such as restrained wall movements, crack spacing, crack width, crack lengths, drying conditions, different length to height ratios and their relationships after an exposure period of drying shrinkage. These walls are exposed to drying shrinkage through a period of three months in winter and two months in summer.

From the results obtained, it has been found that the number of cracks in summer is greater than that in winter and the number of cracks increased with increasing the length/height ratio. Also the number of cracks for all walls with both end and base restraint of different (L/H) ratios are greater than those restrained at the base only in previous researches, this is due to the presence of end restraint. All cracks that found in walls of (L/H)=(2) are secondary cracks (crack height less than wall height), while the cracks that found in walls of (L/H) more than (2) are secondary and primary cracks (crack height reach to the full wall height). All cracks are vertically aligned except a small number of cracks in wall (2S) are oblique laterally. Crack spacing is increased with increasing wall height. Cracking height increased in the hot and dry weather. It is also increased with increasing length to height ratio. Height of cracks in the walls of (L/H=2, 4, 8) range between (0.2-0.6)H, (0.3-0.8)H, (0.4-0.9)H respectively. The relationship between cracks spacing and wall height :

$S_{ave.}$=(0.86H, 0.85H, 0.72H) for (L/H=2, 4, 8) respectively.

Width of cracks that are induced initially is greater than that of cracks that are formed later. Also the height of the maximum crack width decreases with increasing length to height ratio as follow:

$Y_{w.max}=0.62H_c$ for(L/H=2), and $Y_{w.max}=0.4H_c$ for (L/H≥4).

Through this study, it has been attempt to find out the final cracking pattern of shrinkage cracking to predict the suitable position of horizontal reinforcement to be effective as a means of limiting the width of cracks due to restrained shrinkage.

Chapter One

Introduction

1.1- General:

The volume changes that are considered in this study are those caused by changes in moisture content in the mortar body. These changes result in the contraction of the mortar mass. Shrinkage movement will not induce stresses in the concrete or mortar mass unless this member is restrained, the restrained movement will produce tensile stresses in the body, and may produce cracking if they exceed the tensile strength or strain capacity of the mortar itself.

Free movement of concrete members (unrestrained) is rarely present in practice, in which a rigidly interconnected parts are concreted at various stages. Thus the older sections restrict the newly concreted parts.

Many researchers **(Evans and Hughes - 1968, Kheder - 1986, Al Attar - 1988, and Al Mashhadi - 1989)**, have studied the mechanism and control of shrinkage and thermal cracking in reinforced mortar and concrete walls. They are concerned with different types of cracks (primary and secondary cracks) only in the base restrained walls. These cracks can occur in walls with different length to height ratio. which is usually associated with low and high walls.

1.2- Objective of this work:

Many researchers such as **Kheder - 1986, Al Attar - 1988, and Al Mashhadi - 1989** have been studied the case of base restrained walls where they used concrete and mortar walls. The results of concrete walls have been neglected due to the very poor results. The accumulative shrinkage strains in the actual high and long concrete walls can cause of a considerable cracking width , but in small (reduced scale) concrete wall the shrinkage strains are not appreciable, therefore this research depends on only mortar walls.

In high walls such as tunnels and other retaining structures, with wall heights about 4 to 5m, the length of such walls is usually not longer than about (10-15m), the length to height ratio for these walls is less than 2.5,whereas in long retaining walls, aqueous reservoirs or long tunnels, the length to height ratio for these walls is increased to be greater than 2.5. The long wall casts in sequenced parts for construction purposes. The previous are parts joined with the next parts by horizontal steel reinforcements to create a case of single or two end restraint, so that after drying shrinkage commence, each part will be restrained at both sides by to adjacent panels and its base. It has been tried to study this problem in this research with end and base restrained walls of different (L/H) ratios.

In this research, another case of restraint has been achieved which is the combined end and base restraint walls. To simulate this problem to practical site conditions, reduced scale mortar walls were cast and exposed to outdoor conditions.

This study concentrate on the main cracking characteristics, which are:

1. The main parameter studied is the effect of end restraint in different (length / height) ratios of reinforced mortar walls on the cracking

behaviour (crack width, crack spacing and crack length) of these walls. The obtained results are compared with previous results of related studies.

2. Studying the effect of different outdoor conditions on the rate of cracking and the final cracking pattern in the end restrained walls.
3. Studying the formation of secondary cracks and primary cracks relating with (length / height) ratio of the wall.

1.3- 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 shrinkage and types of shrinkage, and the main factors influencing it, the forms of restraint and their effects on cracking, very early shrinkage and long-term shrinkage, elastic tensile strain capacity, mechanism of cracking, and properties of the induced cracks.

Chapter Three, “Experimental work”, is a description of the constructed steel frames that represent the base and end restraint of the mortar walls. Materials, mixing and casting procedures, and procedure of measurements carried out in this study are discussed.

In chapter Four, "results and discussion", the cracking age, cracking sequence, minimum and maximum crack spacing, maximum crack widths observed and their variations with wall heights are presented. Also it presents different relationships between cracking properties with wall height and shape.

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

Chapter two

Literature review

2.1 Shrinkage of concrete:

Shrinkage is a time dependent volume change exhibited by an unloaded concrete body when placed in an unsaturated air, this phenomenon is caused by changes in moisture content of the concrete and chemical reactions in it .

Concrete is a useful structural material because of its compressive strength, durability, impermeability and mouldability, and because of its competitive cost. However, concrete also exhibits certain unfavorable characteristics, including a relatively low tensile strength, and a propensity towards volume changes which occur with variations in moisture content and temperature. In most applications, because of these characteristics, concrete must be allowed to crack if it is to be used economically. However, it is important that the width of these cracks be limited to some acceptable maximum value so that the serviceability of a structure is not impaired ***Campbell – Allen, 1979***.

2.1.1-Shrinkage cracking of concrete

The cracking of the concrete is undesirable for many reasons . Beside being unsightly , cracks may reduce the safety of a structure by preventing monolithic action and , in some cases , they may affect such properties as durability and water – tightness .

Cracking of concrete due to drying shrinkage is one of the most common and serious problem encountered in concrete structure . As in the case of cracking due to decrease in temperature , the case of cracking due to drying shrinkage is the combination of the tendency to contract and the restraint which prevents it from doing so . When the resulting tensile strength of the concrete is less than tensile stresses , it will crack .

ACI committee 207.2R-95 has reported that there are two different approaches practically used in the control of shrinkage cracking in concrete :

The first , reduction of the restrained movement .

The second used approach is the provision of steel reinforcement to distribute these cracks, so that one large crack is replaced by a number of smaller cracks of acceptable width .

The tensile stress development which may lead to cracking involves the following factors :

1 - Rate and amount of drying shrinkage .

2 - Modulus of elasticity .

3 – Degree of restraint .

4 – Creep or relaxation of tensile stress during shrinkage .

The restraint of concrete comes from several source. For example some degree of restraint is provided by the foundation and another part of the structure. A very common type of restraint is developed by the difference in shrinkage at the surface and in the interior of a concrete member ,especially at early age of exposure to drying, Since the drying shrinkage is largest at the exposure surface, the interior concrete restraints the shrinkage at the surface ,thus developing tensile stresses. This may cause surface cracks, which do not penetrate deep into the concrete. However, as drying of the concrete continues, the surface cracks will frequently penetrate deeper into the concrete and become major structural cracks. In the case of the mass concrete structure or large structural elements, the problem of the surface cracking would be aggravated if drying and cooling of the surface should occur simultaneously, resulting in high tensile stress development. For this reason it is important to protect the surface of large structural elements from drying during the concrete's cooling period. This can be accomplished for example by continued curing or by sealing the surfaces to minimize moisture loss ***(Evans and Hughes, 1968***).

Campbell – Allen, (1979) has shown that the normal range of ultimate shrinkage strains for unrestrained specimens of standard size under Australian conditions is of the order of (200) to (600) microstrain, depending primarily on humidity of the environment. ***David*** and ***Steven***, ***1998*** found that the drying shrinkage strain for rolled compacted concrete (low (w/c) ratio) of (80-330) microstrain.

2.2-Types of shrinkage

2.2.1-Plastic shrinkage

Plastic shrinkage is the reduction of volume in fresh concrete when placed in forms due to settlement of the solids and bleeding of clear water to the surface ***(Troxell, 1968)***. This process continues until the attainment of maximum compaction of the solids particle interface, or the beginning of the set. It usually ends within the first hour after placement. No appreciable stresses result since concrete is in its plastic state. Some surface cracking can take place at this stage .

According to ***ACI 305R-91, 1994*** the risk of plastic shrinkage cracking is identical for the following combinations of temperature and relative humidity: 41^{o}C and 90 percent, 35^{o}C and 70 percent, 24^{o}C and 30 percent. Typical plastic shrinkage cracks are parallel to one another, spaced 0.3-1.0 m apart, and have considerable depth.

Gary Ong and Kyaw ,(2006) reported that When freshly-cast concrete is exposed to a dry environment, plastic shrinkage occurs which is associated with the deformations that arising from a difference in vapor pressure inside the fresh concrete. The cause of which has been attributed to insufficient protection against external drying.

Plastic shrinkage cracking can develop also when a large horizontal area of concrete makes contraction in the horizontal direction more difficult than vertically, deep cracks of an irregular pattern are then formed ***Lerch, 1957***.

2.2.2-Drying shrinkage

Drying shrinkage Is the main type of shrinkage of concrete .Drying shrinkage is attributed to the withdrawal or movement of moisture from the concrete when it is stored in unsaturated atmosphere. Drying shrinkage ranges from less than 200 millionths for low slump lean mixes with good quality aggregate and under high relative humidity environment to over 1000 millionths for rich mortars or some concretes containing poor quality aggregates and an excessive amount of water under low relative humidity environment ***(ACI committee 207.2R-95)***.

According to ***Power*** **and** ***Brownyard, 1946*** there are three types of water occurring in cement paste which are:

1- Non-evaporable water [fixed water].
2- Gel water.
3- 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.

Capillary water is easy to evaporate but its effect on shrinkage is limited by the paste structure.

According to ***Hobbs and Parrot, 1979***, due to progressive loss of water from cement paste, capillary tension that develops in the residual water induces compressive stresses in concrete resulting in concrete shrinkage.

2.2.3-Autogenous shrinkage

Autogenous shrinkage is the chemical shrinkage that arises in which the absolute volume of hydration products is less than the total volume of unhydrated cement and water before hydration. This results in a reduction in the absolute volume of the hydrated cement paste. The net autogenous volume change of most concrete is a shrinkage ranging from an insignificant 10 millionths to somewhat in the excess of 150 millionths ,but the latter value is only about one fourth the shrinkage of an average concrete due to drying ***(Neville,1995 and Troxell (1968))***.

Autogenous shrinkage tends to increase at higher temperatures, with a higher cement content, and possibly with finer cements, and with cements which have a higher C_3A and C_4AF content ***Houk et. Al., 1969***.

Véronique Baroghel-Bouny' and ***Pierre Mounanga***, ***2005***, found through their experimental results that, at a given age, the magnitude of autogenous shrinkage increases linearly as W/C decreases from 0.60 down to 0.25 (an increase of 1025 μm/m has been recorded on the "ultimate", *i.e.* 1-year, value

2.2.4-Carbonation Shrinkage

The presence of carbon dioxide (CO_2) and moisture in air can produce carbonic acid, which reacts with hydrated lime to cause this type of shrinkage. The rate of carbonation depends on the relative humidity of the concrete and the ambient temperature***(Nevill,1995 and Troxell (1968)***) .

The carbonation shrinkage develops only in the layers of concrete exposed to air with relative humidity limits of 30 to 70%. Carbonation shrinkage is caused by the chemical reaction of hydration products with carbon gas from the air. Under the actions of drying and moistening the carbonatation shrinkage is coupled with drying shrinkage and provokes very fine cracks. Effects of carbonatation shrinkage are superficial ***(CEB-FIB, 2006)***

2.3- Factors affecting shrinkage :

Shrinkage of concrete is influenced by many factors , some of which have a significant effect while others are of minor importance . The major factors include :

2.3.1- Degree of restraint:

All concrete elements are restrained to some degree by volume change, because there is always some restraint provided either by the supporting elements or by different parts of the element itself. Restrained volume change can induce tensile, compressive, or flexural stresses in the concrete elements depending on the type of restraint and whether or not the change in volume is an increase or decrease. Normally, restrained conditions which induce compressive stresses in concrete are not considered in design, because of the ability of concrete to withstand compression***(ACI committee 207.2R-95)***. On the other hand, restraint conditions which induce tensile stresses are of main concern, due to the low tensile strength of concrete.

2.3.2- Effect of aggregate :

Other important factors influencing drying shrinkage of concrete the size and rigidity of aggregate provide an internal restraint which significantly reduces the potential contraction of the paste. Depending on the type, size and the amount of aggregate used (***Carlson, 1938 and Carlson et al ,1979)***.

The twin influences of water/cement ratio and aggregate content is combined in one graph shown in Figure (2-1).

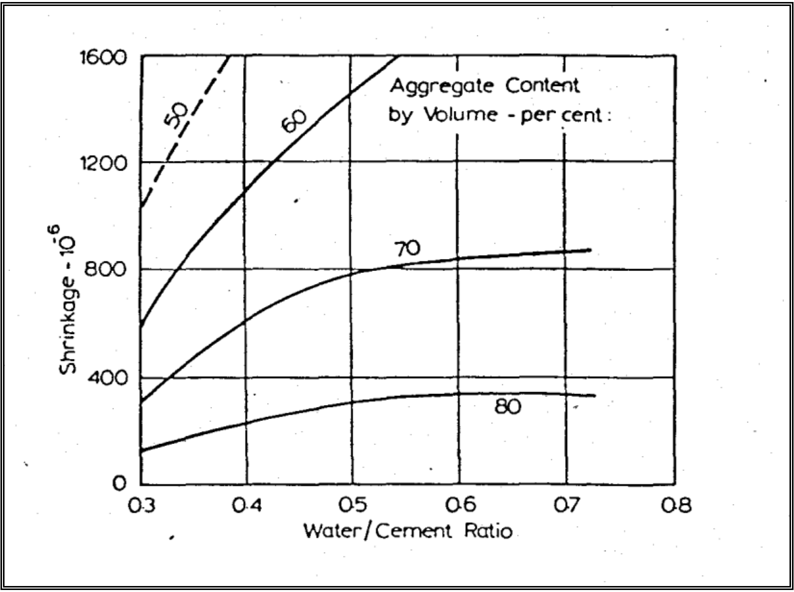


*Figure (2-1) influence of water cement / ratio and aggregate content on shrinkage(**Neville, 1995**)*

Persson et. al., (2005) reported that in the case of externally restrained (in one direction) boundary condition, the maximum residual stress level is not observed to vary significantly with increasing the volume fraction of aggregates.

2.3.3 – Effect of the size and the shape of the member :

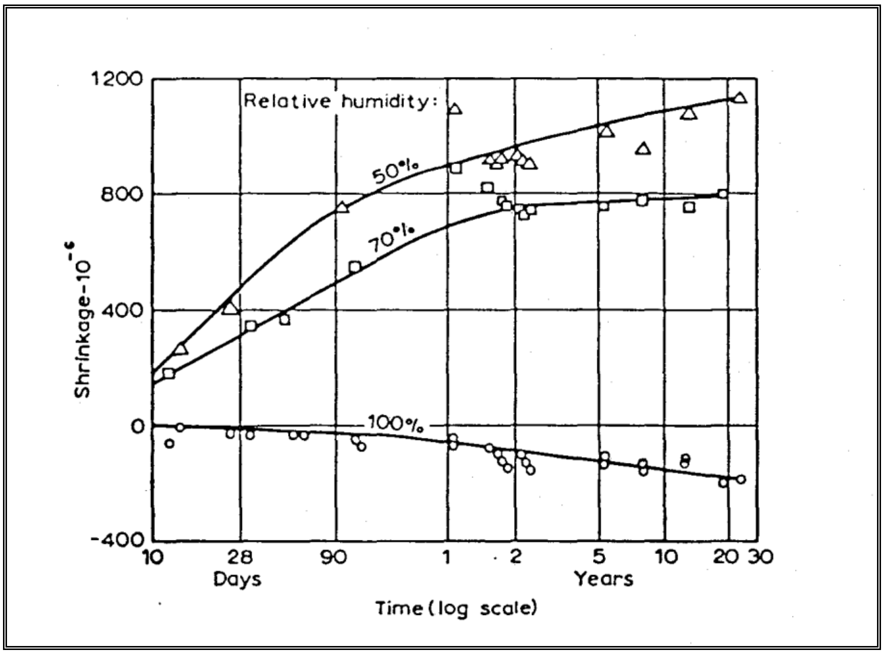
Hansen and Mattock, 1966 concluded through extensive tests, that the volume/surface area ratio of the concrete member is a suitable parameter when estimating its shrinkage and creep deformations. Increasing the volume / surface area decreases the shrinkage.

The size and the shape of the concrete member has a significant influence on shrinkage where both the rate and the final values of shrinkage and creep decrease as the member becomes larger ***(ACI committee 224R-90 and Carlson et al ,1979), Persson et. al., (2005)***.

2.3.4 – Effect of humidity of the environment:

The higher the relative humidity of the environment, the lower the ultimate shrinkage of the concrete. If the average relative humidity is, for example, 7o%, then, as reported by ***Troxell et al*, *1968*** the drying shrinkage would be about one - third lower than if the concrete was exposed to a 50% relative humidity ***(Carlson et al ,1979)*** .

Relative humidity of the medium surrounding the concrete affects the rate and magnitude of shrinkage, as shown in Figure (2-2) ***Neville, 1995***.



*Figure(2-2)Relation between shrinkage and time for concrete stored at different relative humidity(**Neville, 1995**)*

2.3.5- Effect of Curing, Exposure Conditions:

Duration of moist curing of concrete does not seem to have much effect on drying shrinkage, especially for curing times of less than a month ***Hanson, 1968.***

Neville, 1995 reported that the effect of curing on magnitude of shrinkage is small and that well cured concrete shrinks more rapidly.

2.4 -Very early age shrinkage:

Although the properties of concrete in recent decades have improved, it appear that some types of concrete are particularly prone to cracking at an early age caused by early-age shrinkage of concrete.

By using of image analysis technique with a high resolution camera of 6.3 megapixels was used by ***(Gary Ong and Kyaw, 2006)***. The images captured were analyzed using software. The following two tables, (2-1) and (2-2) show the mixture proportion and physical properties of concrete respectively.

Table (2-1) Mixture proportion of concrete ***(Gary Ong and Kyaw ,2006)***.

Designation	w/c	Water, kg/m^3	Cement, kg/m^3	Fine aggregate, kg/m^3	Coarse aggregate, kg/m^3	High-range water-reducing admixture		Density, kg/m^3
						L/m^3	mL/100kg cement	
Mixture 1	0.30	157	540	790	900	7.25*	1350	2400
Mixture 2	0.30	157	540	790	900	7.00†	1300	2410
Mixture 3	0.30	158	540	790	900	5.00*	925	2380

Table (2-2) physical properties of concrete***(Gary Ong and Kyaw ,2006)***.

Designation	Cube strength, MPa				Slump, mm	Initial setting time, h:min	Final setting time, h:min
	1 day	3 days	7 days	28 days			
Mixture 1	15.3	60.1	72.2	87.5	110	16:10	18:00
Mixture 2	42.3	65.5	82.1	86.4	80	3:00	4:25
Mixture 3	39.7	68.4	79.1	90.8	30	5:10	8:35

2.4.1-Moisture loss and temperature rise:

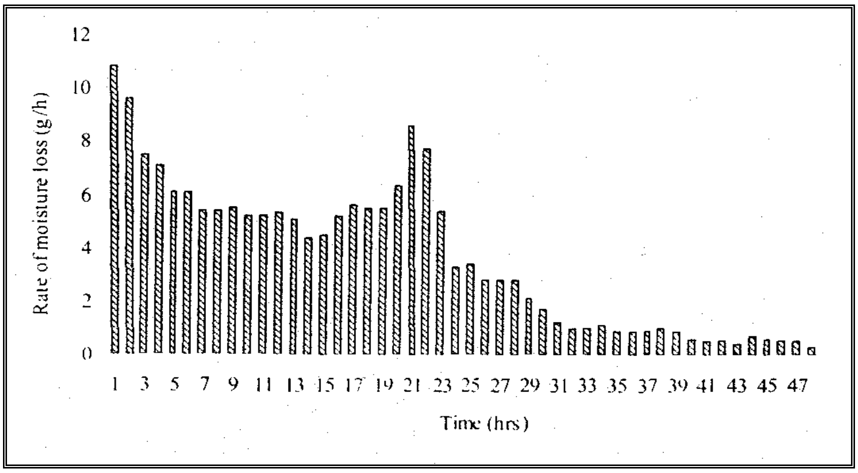
Gary Ong and Kyaw, 2006, found through the use of prisms of (500*100*100)mm with their trowelled surface exposed, in a temperature and relative humidity-controlled room with a temperature and relative humidity of 30 ± 0.5 °C and 65 ± 2%, respectively. The specimens, as mentioned previously, remained in steel molds for the entire duration of the test. Except for the ends, all other mold surfaces were lined with 1 mm-thick Teflon sheets. The results showed that the peak moisture loss and peak temperature rise occurred at approximately the same time, approximately 21 hours after adding water to the mixture. As a result, loss of moisture from the specimen would be primarily from the top exposed surface. During the initial stages of hydration, the temperature of the specimen would first increase as hydration began and then later decreased when the rate of hydration slowed down.

To show the effects of moisture loss, the weight loss with time of an accompanying prism specimen was also measured during the first 48 hours after adding water to the mixture. A digital scale with 0.1 g sensitivity was used to monitor moisture loss with time. The moisture losses over successive hour-long periods in g/hour are plotted in Figure (2-3).

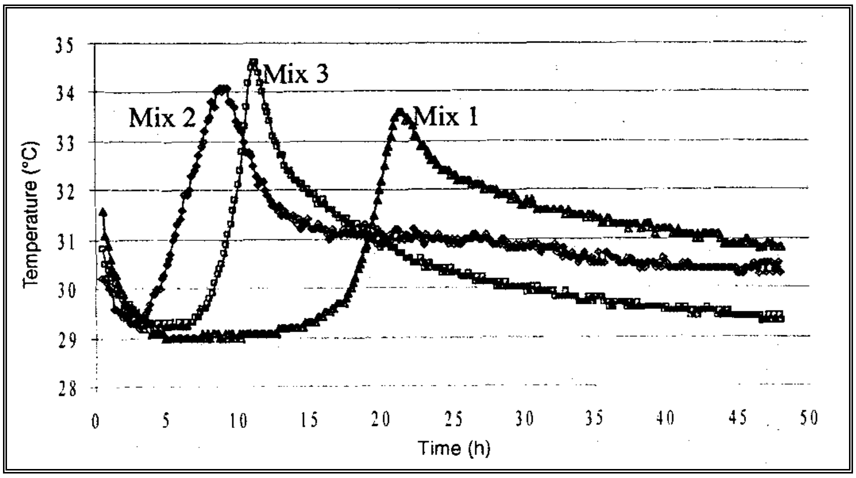
To monitor the temperature rise due to the hydration reaction, thermocouples were placed at a depth of approximately 50 mm at the center of the prism specimens that were used for shrinkage monitoring immediately after casting. The embedded thermocouples were connected to a data-logger that recorded the temperature of the specimen every 10 minutes during the first 48 hours.

The results showed that the peak moisture loss and peak temperature rise occurred at approximately the same time, approximately 21 hours after adding water to the mixture for Mixture 1. Figure (2-3) also shows that the moisture loss per hour decreased initially from that at one hour after adding water to the mixture. The moisture loss per hour started to increase again at the onset of initial setting, reaching a peak when the peak temperature was monitored Figure(2-4). This may be due to thermal effects caused by the heat of hydration in the specimen.

Another possible reason is that a major portion of the evaporable water in the concrete may have been lost during the first few hours. Together with the fact that some of the water became chemically bound, very little water was available to be lost thereafter. Thus, the water loss decreased rapidly with time. After peaking, the moisture loss, together with the temperature, started to reduce, reaching almost zero and a temperature of 31°C, respectively, after approximately 48 hours after adding water to the mixture.

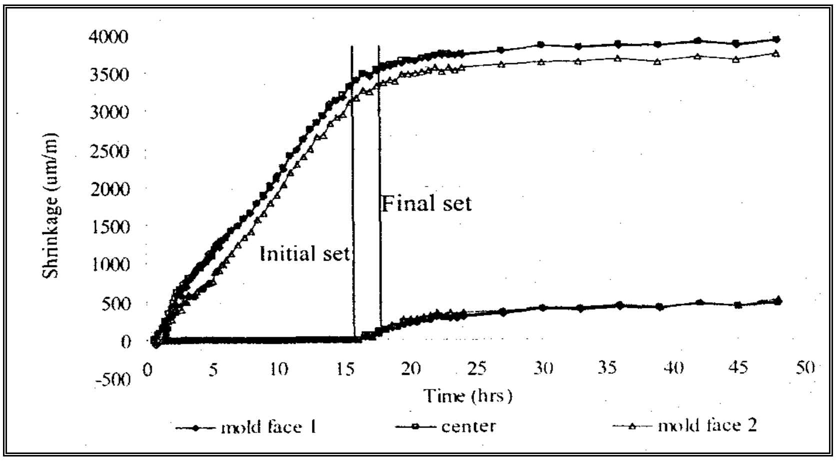


Figure(2-3) rate of moisture loss (g/h) after adding water to mixture (1) ***(Gary Ong and Kyaw ,2006)***.



Figure(2-4) – temperature of specimen during(48)hour after adding water to mixture ***(Gary Ong and Kyaw ,2006)***.

Gary Ong and Kyaw, 2006, also found that the shrinkage of concrete was very high within the first few hours before final set. This is probably due to moisture lost from the exposed trowelled surface.



Figure(2-5)- very early shrinkage of concrete specimen during first 48 hour after adding water to mixture I ***(Gary Ong and Kyaw , 2006)***

2.5- Long-term creep and shrinkage:

Gardner and Zhao, 1993 report that it is impossible to consider early-age effects on creep and shrinkage without considering the other characteristics of long-term deformations. If very accurate prediction of deformations is required, the long-term behavior must be extrapolated, using an assumed time function, from creep and shrinkage tests performed on the prototype concrete. However, relatively simple prediction equations are required for design when the only factors known to the design engineer are specified concrete strength, age of loading, probable ambient humidity, and member volume-surface ratio. The equations presented in this work are proposed as an alternative to the ACI 209-82 recommendations. A concrete structure may have a service life of 50 years, but tests on materials made prior to construction can typically extend over a time span of only 1 to 2 years.

ACI 209-82 and CEB 1990 Models recommend different types of equations represent the deformation

ACI 209-82

$$total\ strain = shrinkage\ strain + \frac{stress}{\mathrm{E}_{cmto}} \times \left[1 + creep\ coefficient\right]$$

CEB Model Code1

$$Total\ strain = shrinkage\ strain + \frac{stress}{E_{cmto}} + \frac{stress}{E_{cm28}} \times creep\ coefficient$$

Where

E_{cmto} = mean modulus of elasticity at loading

E_{cm28} = mean modulus of elasticity at 28 days

The reported moduli may not be appropriate for use in calculating deformations.

The experimental data are reduced by determining, first, a relationship between the modulus of elasticity and compressive strength; secondly, expressions for strength development with time; then an expression to calculate shrinkage; and, finally, an expression for the creep strain.

Gardner and Zhao ,1993 proposed the following equations for design purposes

$$E_c = 3500 + 4300\ (f'_{cm})^{1/2} \quad ....(2\text{-}1)$$

Also ***Zhao, 1993*** suggested that the early-age concrete strength developed can be estimated from the following equation

$$f'_{cmt} = f'_{cm28} \times \frac{t^{3/4}}{a + bt^{3/4}} \qquad .....(2\text{-}2a)$$

for

Type I cement concrete a=2.8 and b = 0.77 …………..(2b)

Type II a = 3.4 and b = 0.72 …………..(2c)

Type III a = 1.0 and b = 0.92 …………..(2d)

Where

f'_{cmt} = mean concrete strength at age t

Using the proposed time expression, the following equation was developed by ***Gardner and Zhao ,1993*** to calculate the shrinkage, at time *t*, from the concrete shrinkage at 40 percent relative humidity with correction factors for (a) age of loading, (b) strength of concrete, (c) duration of loading, and (d) member size and relative humidity

$$\varepsilon_{sh} = \varepsilon_{shu} \times \beta(h) \times \beta(t) \qquad ....(2\text{-}3)$$

$$\varepsilon_{shu} = 900 \times k \times \left(\frac{f'_{cm28}}{f'_{cmtc}}\right)^{1/2} \times \left(\frac{25}{f'_{cmtc}}\right)^{1/2} \times 10^{-6}$$

$$\beta(t) = \left[\frac{7.27 + \ln(t - t_c)}{17.18}\right] \times \left(\frac{(t - t_c)}{t - t_c + 0.0125 \times (v/s)^2}\right)$$

And

$\beta(h) = (1 - h^4) \quad for \quad h < 0.99$

$\beta(h) = -0.20 \quad for \quad h = 1.00$ swelling only when concrete not stressed

Where

ε_{shu} = ultimate drying shrinkage

h = humidity expressed as a decimal

t = age of concrete, days

t_c = age drying commence, days

t_o = age concrete loading , days

k = 1 Type I cement, k = 0.7 Type II cement , k = 1.33 Type III cement

v/s = volume surface ratio

f'_{cm28} = concrete mean compressive strength at 28 days

f'_{cmtc} = concrete mean compressive strength when drying commence

f'_{cmto} = concrete mean compressive strength when loading commence

β(h) = correction term for effect of humidity

β(t) = correction term for effect of time on shrinkage

Gardner and Zhao, 1993 have found that the basic creep coefficient is insensitive to age of loading. However, the drying creep coefficient will include the effects of relative humidity, member size, developed concrete strength, and 28-day compressive strength. There is a need for experimental data on the effects of loading age and relative humidity on creep coefficient.

$$\text{Creep coefficient} = \left[\frac{7.27 + \ln(t - t_o)}{17.18}\right] \times \left[1.57 + 2.98 \times \left(\frac{f'_{cm28}}{f'_{cmto}}\right) \times \left(\frac{25}{f'_{cm28}}\right)^{1/2} \times \right.$$

$$\left. (1 - h^2) \times \left(\frac{(t - t_o)}{(t - t_o) \times 0.1 \times (v/s)^2}\right)\right] \quad \text{.....(2-4)}$$

Proposed equations give better agreement than the two codes expressions. In some cases, the percentage error on total deformation is less than the percentage error on shrinkage or creep individually, due to the errors offsetting each other.

Table(2-3) Ultimate (20,000-day) shrinkage $(\times 10^{-6})$, must multiply by $\beta(h) = 1 - h^4$ (***Gardner and Zhao, 1993)***

28 – day specified strength,Mpa	Development strength at end of wet-curing		
	57 percent,4days	70 percent,7days	100 percent,28days
25	1039	933	783

35	910	818	686
50	784	704	591
75	655	588	494

Table (2-4) Ultimate (20,000-day)creep coefficient with relative humidity, concrete strength, and age of loading(***Gardner and Zhao, 1993)***

28 – day specified strength,Mpa	Development strength at end of wet-curing								
	57 percent,4days			70 percent,7days			100 percent,28days		
Relative humidity, percent / 28 – day specified strength,Mpa	40	70	90	40	70	90	40	70	90
25	5.4	3.9	2.44	4.66	3.45	2.27	3.37	2.89	2.06
35	4.93	3.61	2.33	4.28	3.21	2.18	3.48	2.73	2.00
50	4.46	3.32	2.22	3.90	2.99	2.10	3.21	2.57	1.94
75	3.99	3.04	2.12	3.52	2.75	2.01	2.94	2.40	1.88

2.6 -Free and restrained volume change of concrete:

Restraint to volume change can be divided into two types: external and internal restraint. External restraint can further be subdivided into: continuous edge (base) restraint, end restraint, and intermittent restraint, whereas internal restraint arises when one part of the concrete expands or contracts differentially to another part of the same section. In many cases, one of these types of restraint is dominant, but there are few situations where both types of restraint need to be considered***(Harrison, 91-1981)***.

The degree of restraint depends largely on relative dimensions, strength, creep or relaxation, modulus of elasticity of concrete and the restraining object. In practice, the degree of restraint of any concrete section within the member depends largely on its location and the type of restraint ***Gilberr, 2001***.

2.6.1-External restraint:

External restraint are subdivided into:

2.6.1.1-Continuous external restraint:

Continuous edge restraint exists along the contact surface of concrete and any material against which the concrete has been cast. The degree of restraint depends primarily on the relative dimensions, strength and modulus of elasticity of the concrete and the restraining material. After casting of a concrete wall onto a base, a temperature rise in the wall above that of the base will take place. this stage, contraction of the wall starts to take place , after this contraction is attributed to the drop in temperature of the wall and to the shrinkage which starts developing as the concrete of the wall begins to dry after the end of the water curing period. This contraction is restrained by the base Figure (2-6 a). without restraint, the concrete in the wall freely contracts to the shape indicated on Figure (2-6 b), but because the wall remains compatible with its base, the actual finished shape is as in Figure (2-6 c,d). Restraint in concrete walls with continuous base restraint varies from point to point through out the wall. Also the restraint in the wall depends on its size (length to height ratio)***Harrison, 1981***.

Stoffers, 1978 showed that the restraint in concrete walls with continuous base restraint, varies from point to point through out the wall. Also the restraint in the wall depends on its size (length to height ratio) as given in Figure (2-7) .

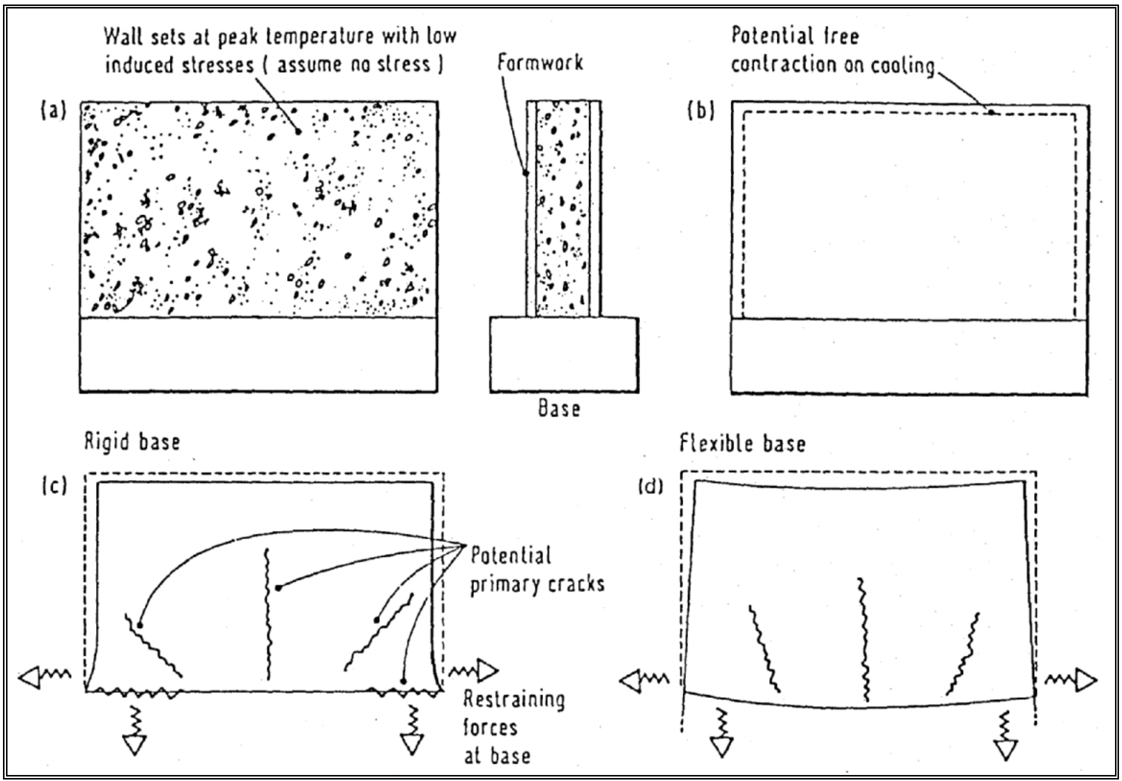


*Figure (2-6)continuous edge restraint of the wall cast on base,**(Harrison,1981)***

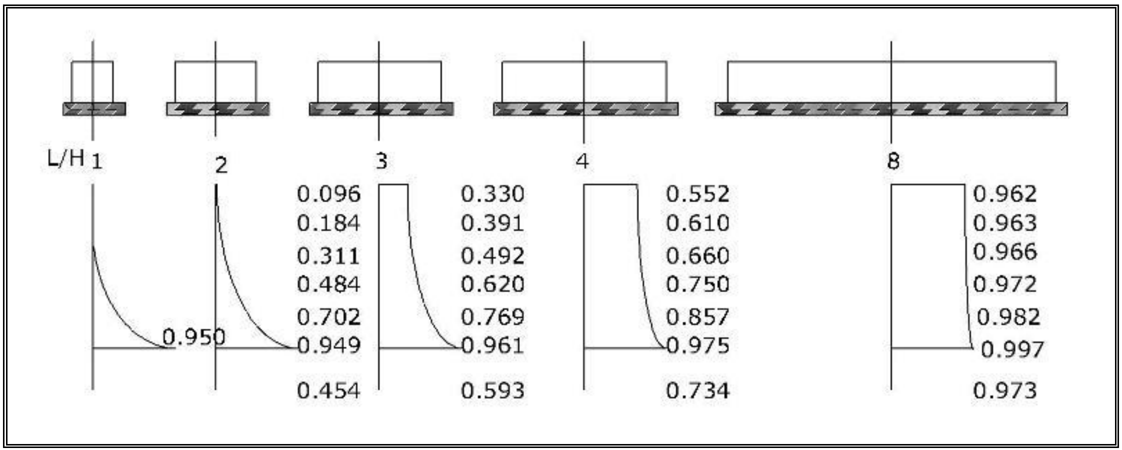
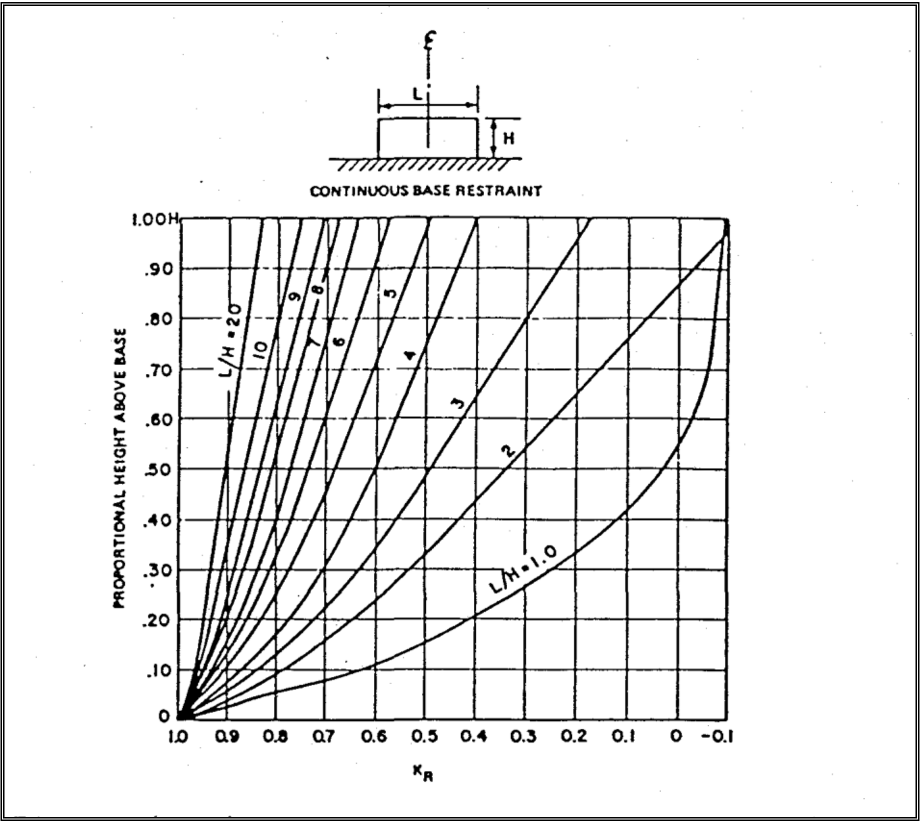


*Figure (2-7)stress distribution in the cross section in the middle of the wall in the elastic range for various values of length /height, **(Stoffer, 1978)***

Similar investigations were conducted by ***Carlson and Reading (1950, 1965)*** as reported by the ***ACI committee 207.2R-95***, the results obtained are given in Figure (2-8).



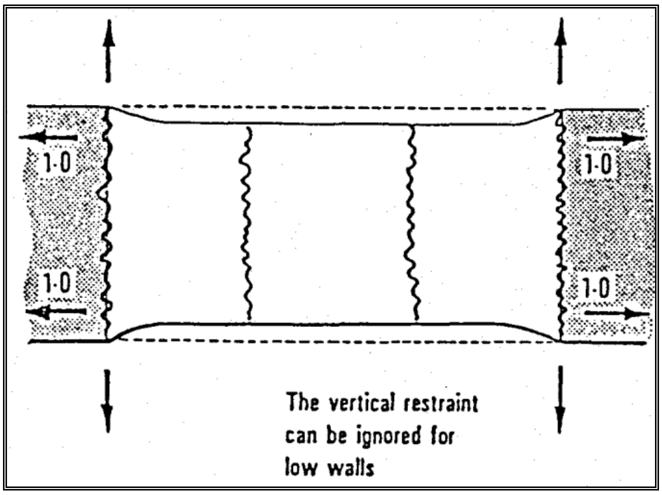
*Figure(2-8) degree of tensile restraint at center section with continuous base restraint, **(ACI committee 207.2R-95)**.*

2.6.1.2-End restraint (thin sections) :

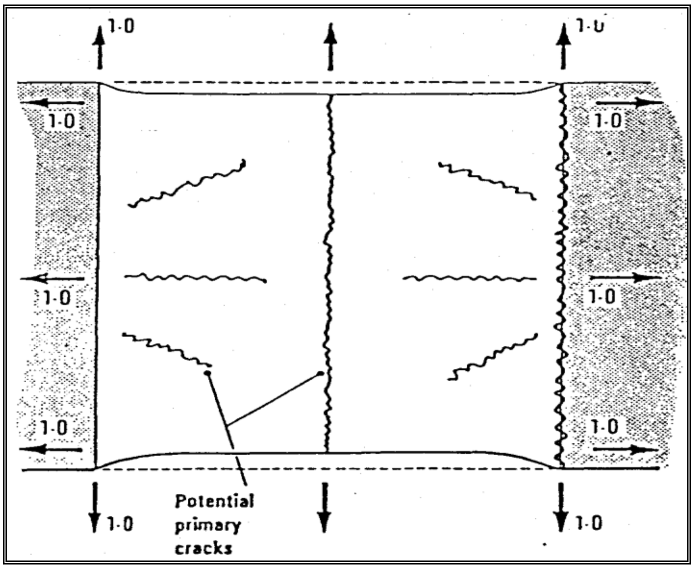
A continuous circular tank wall with a free to slide base is an example of pure end restraint ***.Harrison,1981*** assumes that there is a full horizontal restraint for the full height of the wall. As the wall contracts, uniform tension develops along its length. If the wall is low, vertical restraint could be ignored as on Figure (2-9)a. When the restraint edge exceeds about 5 meters in length as in Figure (2-9)b, the contraction parallel to the rigid supports need to be considered.

2.6.1.3- combined end and edge restraint :

According to ***Harrison, 1981***, when just the panel ends are fixed, full horizontal restraint exists, and fixing the base in addition to the ends does not increase the horizontal restraint, although it modifies the cracking pattern, compare Figure (2-10)a and (2-9)b. In this assumption ***Harrison*** also ignores the elongation of the steel passing through the construction joints by assuming full end restraint (R=1.0) at the joint.

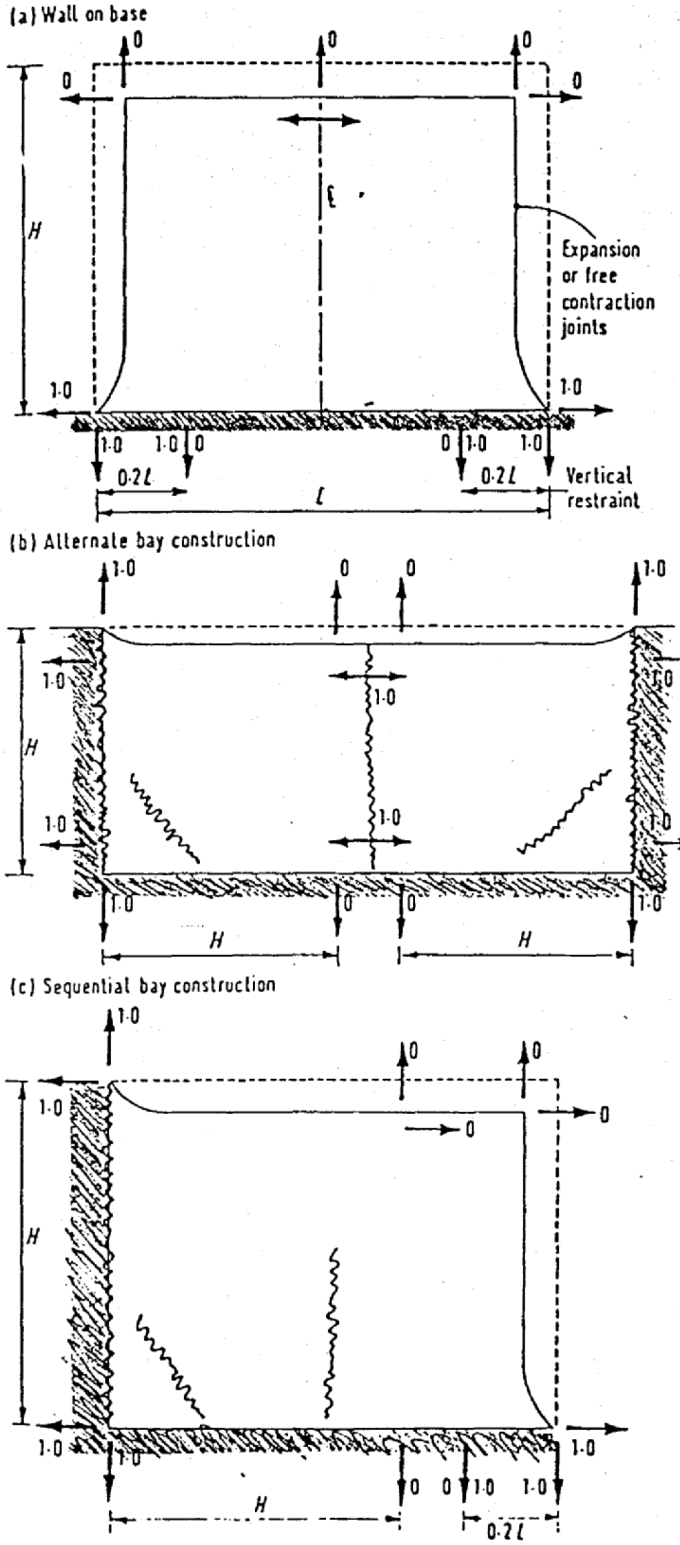


a) low wall free to slide on base



b)slab between rigid restraint

*Figure(2-9) End restraint factors in walls or slabs**(Harrison,1981)**,*



L/H ratio	Centreline horizontal restraint factor	
	Base of section	Top of section
1	1·0	0
2	1·0	0
3	1·0	0·1
4	1·0	0·6
≥8	1·0	1·0

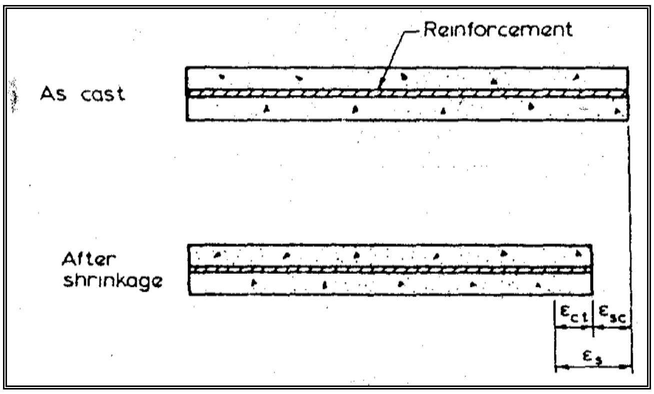
*Figure(2-10)restraint for various slabs or wall poured in sequence**(Harrison,1981)***

2.6.2- Internal restraint:

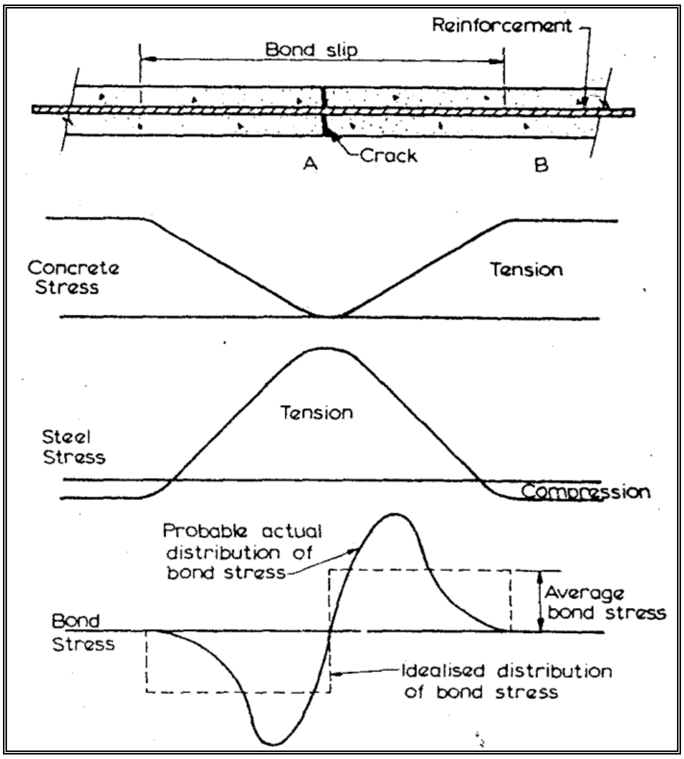
Internal restraint exists in members with non uniform volume change in a cross section. This occurs for example with walls, slabs, or masses with interior temperatures greater than surface temperatures. It also occurs in slabs projecting through the walls of buildings with cold outside edges and warm interiors and in walls with the base or lower portions covered and the upper portion exposed to the air. ***Jae, 2006***, found by using finite element analysis that the increase in aggregate volume reduces the overall free shrinkage of a cementitious composite due to the addition of aggregate results in the generation of internal residual stresses.

2.7-Unrestrained shrinkage:

Consider the cracking potential of a reinforced concrete element of unit length , which is subjected to shrinkage of concrete and is restrained only by the reinforcement as shown in Figure (2-11). If the shrinkage strain of plain concrete is ε_c, then after shrinkage of element, to satisfy compatibility of deformations there is a tensile strain ε_{ct} in the concrete and a compressive strain ε_{sc} in the steel , such that,



*Figure(2-11) shrinkage of a reinforced concrete member without external restrained, **(Campbell-Allen,1979)**.*



Figure(2-12) diagrammatic stress distribution in an externally restrained reinforced concrete member following cracking, ***(Campbell-Allen,1979)***.

$$\varepsilon_{sh} = \varepsilon_{sc} + \varepsilon_{ct} = \frac{f_{sc}}{\mathrm{E}_s} + \frac{f_{ct}}{\mathrm{E}_c}$$

Where ε_{sh} shrinkage strain

ε_{sc} compressive strain in the steel reinforcement

ε_{ct} tensile strain in the concrete

f_{sc} compressive stress in the steel reinforcement

f_{ct} tensile stress in the concrete

E_s modulus of elasticity of steel

E_c modulus of elasticity of concrete

Since there is no external restraint, internal forces may be equated. If f_{sc} is then eliminated we obtain,

$$f_{ct} = \frac{\varepsilon_{sh}}{\left(\frac{1}{\rho E_s} + \frac{1}{E_c}\right)} \quad \text{.........(2-5)}$$

Where $\rho =$ ratio of steel area to the area of concrete

Typical values of these variables lead to concrete tensile stresses in unrestrained members which are sufficiently small for cracking not to results ***(Campbell-Allen,1979)***.

2.8-Fully Restrained shrinkage:

Cracks of restrained concrete members under the action of thermal and shrinkage volume changes is virtually inevitable.

Consider a long, fully restrained reinforced concrete element which has cracked under the action of shrinkage, as shown in Figure (2-11). Subsequent increases in shrinkage strain can only be accommodated by:

1- yielding of the steel reinforcement at the crack, or
2- Further cracking of the concrete, as the tensile stress elsewhere reaches the concrete tensile strength***(Campbell-Allen, 1979)***.

As reported by ***Kheder, 1986*** the effect of creep has been neglected by most researchers. Thus the net stress in the concrete will be:

$$f_{ct} = (\varepsilon_{sh} - D - C)E_c \quad \text{......(2-6)}$$

Where D = loss of restraint before cracking.

C = creep strain before cracking.

The cracking of the concrete will take place only when the value of (ε_{sh} - D - C) will reach the elastic tensile strain capacity of the concrete (e_t).

$$(\varepsilon_{sh} - D - C) \geq \varepsilon_{ct}$$

2.9-Mechanism of shrinkage cracking:

Vetter,1933 is the first to suggest the occurrence of a slip between the steel and the concrete adjacent to the crack. His theory assumed a uniform spacing of shrinkage cracking which decreases with the increase in shrinkage until a minimum crack spacing is reached. Also he defined this minimum crack spacing to be twice the bond slip distance.

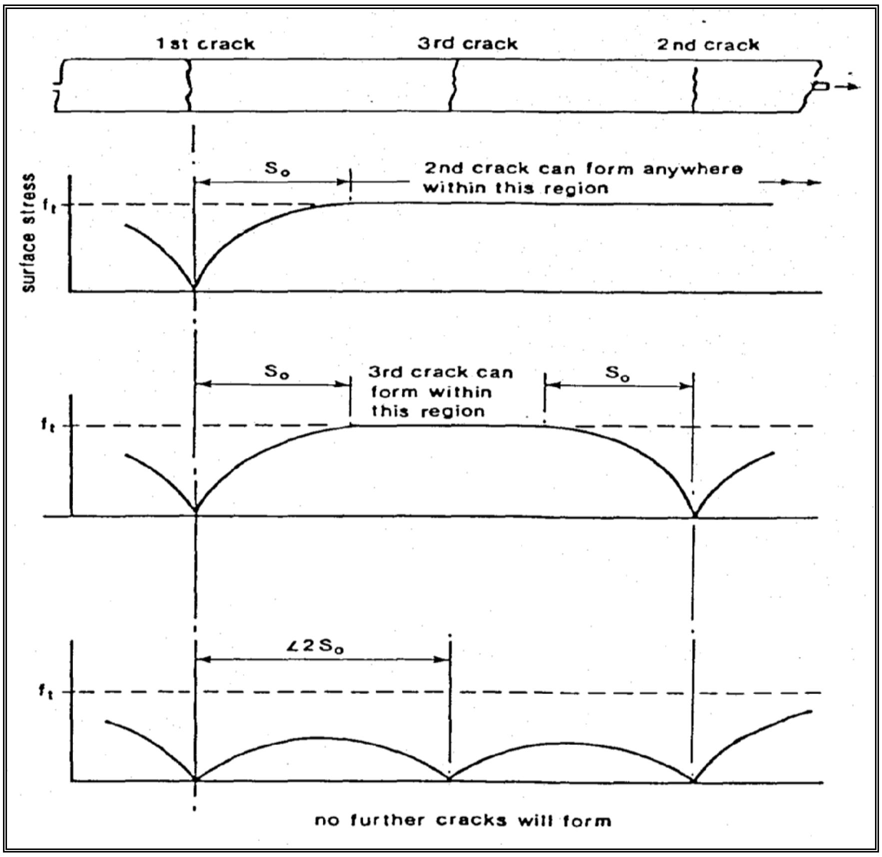
Beeby, 1979 found that all theories from ***Saliger's*** theory ***,1936*** up to the (1979), deal with the cracking of hardened concrete; plastic cracking, for example lies outside their scope. A further condition is that sections should contain sufficient reinforcement to ensure that the steel remains elastic after cracking under the loading.

All theories start from the following basic considerations .

1- Consider the situation when the first crack forms in a member, on the surface of the member, the stress in the concrete must be zero at the edge of the crack . With increase distance away from the crack, the surface stress will increase until, at some distance, S_o, the stress distribution remains unaffected by crack (i.e. the crack affects the stresses only within a distance $\pm$ S_o from the crack). Since the crack has reduced the concrete surface stress to below the tensile strength of the concrete within $\pm$ S_o of the crack, the next crack to form must form out side this region. The minimum distance between cracks is thus S_o. If two cracks form at a distance apart greater than $2S_o$, there will be an area between the cracks where the stress is not affected by either of the cracks and so another crack can form, whereas, if cracks form at a lesser spacing than $2S_o$, the concrete stresses will be reduced over the whole length between the two cracks and another crack will not form When all the cracks have developed, the maximum spacing will thus be $2S_o$ and the final crack pattern will consist of cracks having some distribution of spacing within the range:

$$S_o \le S \le 2S_o$$

This argument is illustrated in Figure (2- 13)



*Figure(2-13) conditions on the surface of an axially reinforced tension member during the development of cracking ,**(Beeby, 1979)**.*

2-The average crack width is given by the average final crack spacing multiplied by the average strain minus the average residual surface strain in the concrete between the cracks:

$$w = s_m(\varepsilon_m - \varepsilon_{cm})$$

Where w crack width

s_m mean crack spacing

ε_m average strain

ε_{cm} average residual surface strain

Commonly, the strain in the concrete between the cracks ε_{cm} is ignored. This assumption will normally will be reasonable and results in the relationship,

$$w_m = s_m \varepsilon_m \qquad \ldots\ldots(2\text{-}7)$$

The mean final crack spacing S_m has commonly been assumed to be given by ($1.5S_o$)but there are theoretical reasons for believing that a value of ($1.33\ S_o$)is more correct. The problem facing theorists is to develop a means of prediction S_o.

Campbell-Allen, 1979 found that the Crack spacing and widths are dependent upon the bond characteristics of the concrete – steel interface. Consider the concrete element shown in Figure (2-11) provided that the critical reinforcement ratio is exceeded, an increase in strain will lead to the formation of further cracks until the stage is reached that no more cracks can form because of there being insufficient available anchorage length.

Further strain then relieved by bond slip and widening of existing cracks. the formation of the complete crack pattern is finally achieved when the bond force between the steel and concrete is equal to the tensile strength of the concrete, that is,

$$f_b S \sum(\pi\, d) = f_t' A_c$$

Whence

$$S_{\min} = \frac{f_t'\, d}{4 f_b\, \rho} \qquad \ldots\ldots\ldots(2\text{-}8)$$

Where f_b average bond strength between concrete and reinforcement

$S_{\min}$ minimum crack spacing

d diameter of reinforcement bar

f_t' tensile strength of concrete

A_c cross sectional area of concrete

ρ ratio of steel area to the area of concrete

Once a fully developed crack pattern has been achieved, the minimum crack spacing, S_{min}, is given by Expression (2-8) and the maximum crack spacing, S_{max}, is twice this value ,since an intermediate crack can form as soon as the available development length between existing cracks is equal to or greater than twice the minimum value of S_o Hence ,

$$S_{max} = \frac{f_t' d}{2 f_b \rho} \rangle S \rangle \frac{f_t' d}{4 f_b \rho} \rangle = S_{min} \qquad \text{........(2-9)}$$

It has been found that equation (2-8) yields to crack spacing smaller than that really existing in practice.

In spite of the shortcomings in Equation (2-8), it has been adopted by the British code of practice for the structural use of concrete for retaining aqueous liquids ***BS 8007:1987***.

ACI committee 207.2R-95 took the effect of the degree of restraint in the member before cracking into consideration, The committee found that unreinforced concrete members, subjected to base restraint, will ultimately attain cracks through the full block height spaced in the neighborhood of 1.0 to 2.0 times the height of the wall block.

ACI committee 207.2R-95 also considered the effect of horizontal reinforcement in the wall in reducing the above mentioned crack spacing relationship, and gave the equation for the crack spacing in reinforced concrete walls as:

$$S_y = \frac{w_{max}}{18\left(R C_T T_E - f_t / E_c\right)} \qquad \text{..............(2-10)}$$

Where S_y average crack spacing at level y above the base

R degree of restraint

C_T coefficient of thermal expansion

T_E design temperature change

A schematized crack pattern suggested by the committee is given in Figure (2.14).

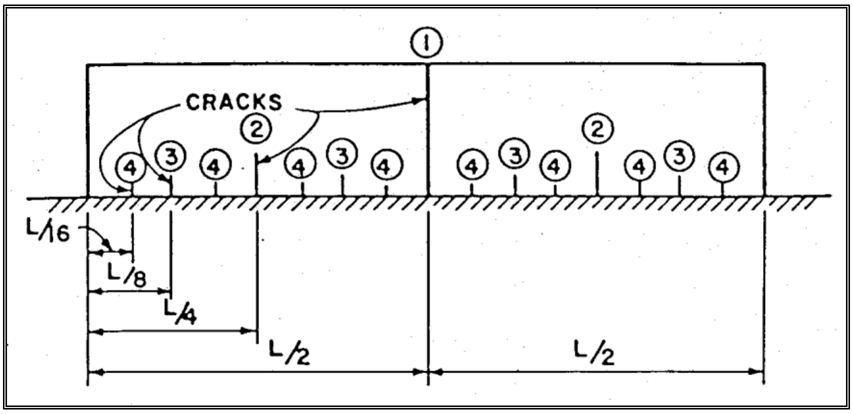


*Figure (2-14)continuous base restrained sequence of cracks propagation,(**ACI committee 207.2R-95**).*

Stoffers, 1978, observed according to the Figure (2-17) that the average crack spacing due to base restraint varies linearly with the height above the base Y,i.e.

$$S_y = (1.0\_1.5)Y \qquad ....(2\text{-}11)$$

Where Y crack height

Also he observed that for low horizontal steel reinforcing ratios (below approximately 0.5%), the effect of reinforcement on the crack spacing was very small or indeed absent. He found that the effect of reinforcement on crack spacing was clearly manifested in wall undergoing no curvature (rigid base) and with steel ratios of above approximately 0.5%. ***Clark*** as reported by ***Harrison, 1981*** found that the primary cracking patterns are completely independent of the amount of horizontal reinforcement and are due solely to base restraint.

Al Rawi, 1986 observed that the minimum crack spacing is about (3.4 times) that obtained from Equation (2-8). The formula suggested by

Al Rawi was based on experimentally measured bond slip distance. A similar trend to that of ***Al Rawi*** has been observed by ***Al Ali, 1985.***

$$S_{\min} = 0.85K\frac{d}{\rho} \;\;.........(2\text{-}12)$$

AL Rawi and Kheder, 1990 modified the equation of minimum crack spacing in end restrained members which was adopted by ***AL Rawi, 1986*** to take into consideration base restraint effect. They expressed the minimum crack spacing by the following formula:

$$S_Y = \frac{KDY}{KD + \rho Y} \;\;.........(2\text{-}13)$$

Where S_y =crack spacing at height Y

K = 0.67, 0.8, and 1.0 for deformed, indented, and plain Reinforcing bars respectively

D = bar diameter

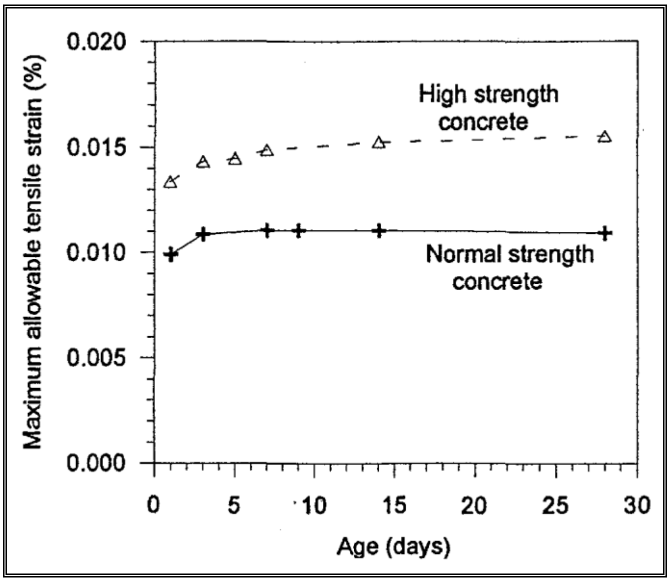
Y = height above base of wall

ρ = steel reinforcement ratio

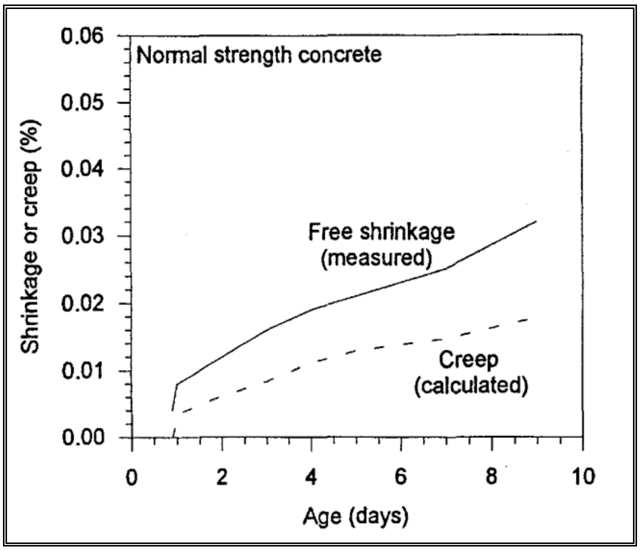
2.10-Elastic tensile strain capacity:

Mann, 1971 reported a value for ultimate strain capacity is approximately 100 microstrain at a variety of ages and strength. Other researchers quoted experimental values of strain capacity ranging between (100-200) microstrain, however higher values can be used safely before cracking and this was confirmed by ***Al-Rawi,1985***. ***Hughes and Ghunaim, 1982*** reported that tensile that strain capacity increases with increase of concrete age. ***Hobbs, 1974*** concluded that the strain capacity will be improved with age and strength of concrete. An increase in strain capacity from (80-100) microstrain to (125-160) microstrain was obtained with increase in age from 7 to 180 days.

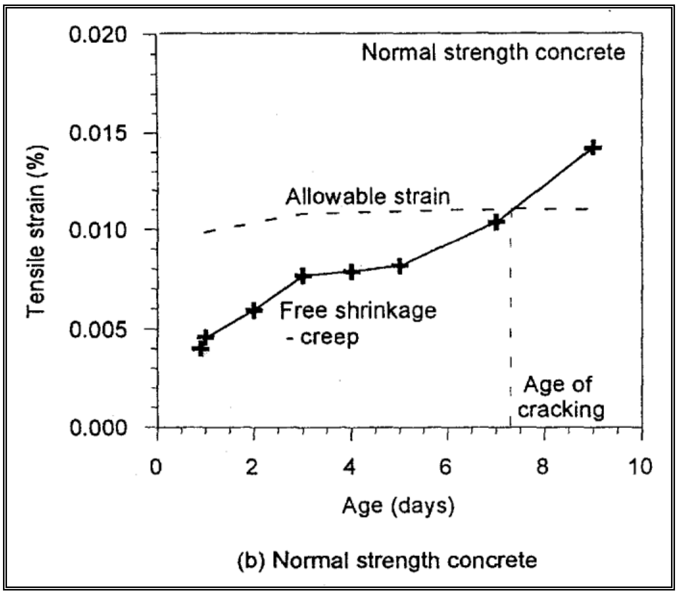
Surendra ,et al ,1998, showed through a ring test to test the cracking shrinkage tendency of concrete based on free shrinkage and material fracture parameters. The maximum allowable tensile strain can be clearly identified. This procedure was applied to both the high and normal strength concretes for ages up to 28 days, and the obtained results are shown in Figure(2-15). Since the maximum allowable tensile strain was obtained from this curve, represents fracture resistance of the ring specimen. When total tensile strain, which is the difference between free shrinkage and creep as in Figure (2-16), exceeds the maximum allowable tensile strain as shown in Figure(2-17), the ring specimen cracks. The maximum allowable tensile strains initially increase with increasing specimen age for both the concretes, but remain almost constants after age of 7 days. The high strength concrete has greater maximum allowable strain compared to the normal strength concrete.



*Figure(2-15)predicted maximum tensile strain(**surendra et al**,**1998**).*



*Figure(2-16) Measured free shrinkage and calculated creep of normal strength concrete(**surendra et al,1998**).*



*Figure(2-17) Theoretical prediction of age of cracking for ring specimen for normal strength concrete (**surendra et al,1998**).*

Jae, 2006, found that the specimen of mortar of fine aggregate volume (55%) does not reach the maximum tensile strength of (5)MGP, but cracked with stresses of (1.8)MGP due to drying shrinkage.

2.11-Cracking due to temperature change:

The tensile stress due to temperature change is merely the product of four quantities, as follows :

(1) Temperature chang in degrees,

(2) Coefficient of thermal expansion in millionths per degree.

(3) The average modulus of elasticity during the temperature change in Mpa.

(4) the degree of the restraint as a fractional part of full restraint.

$$f_t = K_R \Delta_c E_c \quad ....(2\text{-}14)$$

$$\Delta_c = L C_T T_E$$

Where K_R degree of restraint

C_T coefficient of thermal expansion

T_E design temperature change

L length of the member

Since the temperature drop and resulting stress never occur instantaneously, a correction needs to be applied to allow for relaxation .There is now an easy way of determining the relaxation of stress due to slow loading .The method makes use of relaxation coefficients as derived from creep data. Suffice it to say here that it is beneficial to have the temperature drop occur as slowly as possible to reduce the restraint against contraction whenever practicable***(Carlson et al,1979)*** and ***ACI committee 207.2R-95.***

2.11.1-Coefficient of thermal expansion:

The magnitude of the coefficient of thermal expansion depends largely upon the mineral composition of the aggregate. Low coefficient of thermal expansion are usually found when the aggregate are Limestone or Basalts, while high values are found with Quartzite. The range of values for concretes from various projects runs from a low of approximately (7millionths per C°) to high of over (13 millionths per C°) ***(Carlson et al,1979)***.

The thermal expansion of concrete usually reflects the weighted average of the various constituents, ***ACI committee 207.2R-95*** reported a method in ASTM C531 for determination of this coefficient.

$$c_{th} = c_{mc} + 3.1 + 0.72c_a \quad \ldots(2\text{-}15)$$

Where c_{th} = thermal coefficient of concrete

c_{mc} =the degree of saturation of concrete

c_a =the average thermal coefficient of total aggregate

3.1 =the hydrated cement paste component

2.12-Crack Width:

The equation suggested by ***Evans and Hughes,1968*** :

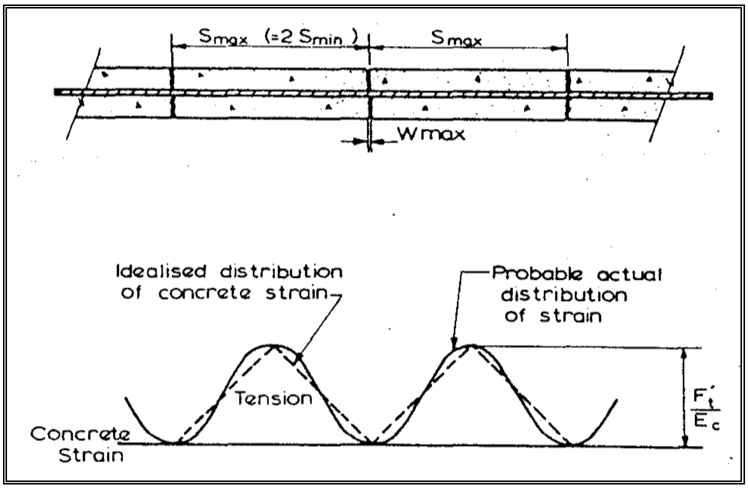
$$w_{\max} = s_{\max}\left(\varepsilon_{sh} - 1/2\varepsilon_t\right) \quad \ldots..(2\text{-}16)$$

Harrison, 1981 considered the effect of the degree of restraint before cracking.

$$w_{\max} = s_{\max}\left(R\varepsilon_{sh} - 1/2\varepsilon_t\right) \quad ..(2\text{-}17)$$

This equation is not applicable in case of walls with continuous edge restraint. According to equation (2-17), the maximum crack width will occur directly above the base, because the value of R is the largest at this position before cracking.

Campbell-Allen,1979 Considered a reinforced member, as shown in Figure (2-18), with a fully developed crack pattern, and subjected to a combined thermal and shrinkage strains, ε. In order to calculate the probable maximum crack width, assume that the cracks are spaced as widely as possible, i.e. at S_{max}. The assumed average concrete strain between cracks is f_t /2E_c. The maximum crack width, W_{max}, is therefore given by,



Figure(2-18) concrete stress distribution in a specimen with cracks at maximum spacing ,S_{max},(**Campbell-Allen,1979**).

$$w_{\max} = s_{\max}\left(\varepsilon - \frac{f_t'}{2E_c}\right) = \frac{f_t' d}{2 f_b \rho}\left(\varepsilon - \frac{f_t'}{2E_c}\right) \quad ....(2\text{-}18)$$

Where ε is the total concrete strain

The width of fully developed crack due to drying shrinkage and heat of hydration in restrained slabs and walls may be obtained ***B.S. code 8007, 1987.***

$$W_{\max} = S_{\max}\left[\varepsilon_{cs} + \varepsilon_{te} - \left(100 \times 10^{-6}\right)\right] \quad ......(2\text{-}19)$$

where;

w_{max} : maximum crack width,

S_{max} : maximum crack spacing,

ε_{cs} : shrinkage strain, and

ε_{te} : total thermal contraction after peak temperature due to heat of hydration.

Al-Rawi**, **1986 investigated the combining effects of the base restraint and the reinforcing steel in distributing volume change cracking.

He suggested the following equations for prediction maximum crack width (w_{max}) in base – restrained concrete members:

$$w_{\max} = S_{\max}\left[K\left(R_b - 0.8R_a\right)e_{sh} - e_{ult}/2\right] \ldots..(2\text{-}20)$$

K=1-c'

where;

R_a : degree of restraint after cracking,

R_b : degree of restraint before cracking,

S_{max} : maximum crack spacing,

e_{sh} : free shrinkage, and

e_{ult} : elastic tensile strain capacity of concrete.

Assuming the sum of the creep strain before cracking C_b and the average value of the creep strain after cracking C_a to be c' (constant) of the total net shrinkage, where c' equals 0.4 for reinforced concrete walls and 0.25 for plain concrete walls.

If it is assumed that the average crack spacing is 3/4 S_{max} , the corresponding average crack width is equal to 3/4W_{max}. Prior to the establishment of the complete crack pattern, When crack spacing are generally greater than S_{max}, the average crack width is closer to W_{max} .
Equation (2-18) has been derived assuming elastic behavior of the concrete. However, creep would to some extent relieve the bond and tensile stresses in the concrete adjacent to cracks, and thus reduce the crack width. The effect of creep has been allowed for in the values of strain suggested.

The equation derived by ***(Base and Murray, 1981***) the prediction of crack width is given below:

$$w = 2a\left(\frac{f_s}{E_s} + \frac{\varepsilon_{sh}}{3}\right) \quad \text{.......(2-21)}$$

Where a is the no-bond length for the reinforcement near the crack

Equation (2-21) was developed for end restraint members, and does not apply for walls with continuous base restraint.

The equation for the limiting mean crack width is

$$w_{\lim} = \frac{2a\varepsilon_t'}{3n\rho}(n\rho + 3)$$

Where n is the modular ratio, E_s / E_c

ε_t' is the tensile cracking strain

And this may be simplified with little loss of accuracy to

$$w_{\lim} = \frac{2a\varepsilon_t'}{3n\rho} \quad \text{......(2-22)}$$

As reported by previous research, ***Murray (1977)and Lim (1982)*** ,it suggested that for deformed bar reinforcement a reasonable estimate for the no – bond length is given by

$$a = 0.8 \times \frac{bar\ diameter(d)}{re\inf orcement\ ratio(\rho)}$$

An approximate expression for the critical reinforcement ratio is

$$\rho_{crit} = \frac{f_t'}{f_{sy}}$$

Where f_{sy} yielding tensile strength of reinforcement

2.12.1-Crack width limitations:

It is usual to apply the flexural crack width restrictions to cases of thermal and drying shrinkage, and require that maximum crack widths should be limited to (0.1)mm to (0.3)mm depending on exposure conditions. The maximum allowable crack width applicable to a particular condition of exposure need to be reduced for either an aesthetic or a functional reasons, such as in the case of a prestige buildings or water retaining structures***(Carlson et al,1979)*** . supercede

2.13-Minimum reinforcement ratio:

As reported by ***Kheder, 1986***, ***Vetter, 1933*** defined the critical steel ratio as the minimum amount of reinforcement, below which the yielding of steel reinforcement will take place. Vetter's equation which was derived for end restrained members was modified by ***(Evans end Hughes, 1968)*** to include the effect of base restraint.

$$f_{ct} = \rho\left(f_{st} + f_{sc}\right)$$

The equilibrium of the portion of the member adjacent to the crack is given by equating forces, which leads to:

$$\rho_{crt} = \frac{f_t' - F / Ac}{\left(f_{sy} + f_{sc}\right)} \qquad \text{.......(2-23)}$$

The tensile stresses reach their maximum values,

Where F = friction force between the wall and the base

A_c=contact area

Evans and Hughes.1968 suggested to neglect the effect of the base restraint, and recommended the use of equation (2-24)) instead of (2-23) due to the amount of reinforcement required in the equation (2.18) greater than a member subjected to the base restrained.

$$\rho_{crt} = \frac{f_t'}{\left(f_{sy} + f_{sc}\right)} \quad ....(2\text{-}24)$$

The British code of Practice for the structural Use of concrete for retaining aqueous liquids ***(BS 8007: 1987)*** adopted equation (2.24) instead of (2.23) .

A critical steel ratio , $\rho_{crt} = \frac{f_t'}{\left(f_{sy} + f_{sc}\right)}$

According to ***Campbell-Allen,1979*** ,the typical values of these variables are such that f_{sc} is very much smaller than f_{sy} , and may be approximated to zero in determining ρ_{crt} .And since thermal and shrinkage cracking is commonly observed to occur within a period of (3) days or so after casting, it is convenient and realistic to substitute the (3)days value of concrete tensile strength in above expression:

$$\rho_{crt} = \frac{f_{t(3)}'}{f_{sy}} \quad ......(2\text{-}25)$$

Both references ***Evens and Hughes,1968*** and ***BS 8007***: 1987 did not consider any reduction 'in the amounts of horizontal reinforcement over the height of the wall.

ACI Building code 318-2002, limited the minimum amount of horizontal steel reinforcement in concrete walls at 0.2% times gross area of the wall for the bars of diameter less than no.16, which is somewhat smaller than 0.3 times gross area that specified by ***BS 8007:1987***.

Finally, as reported by ***(Kheder,1986)*** considering the formula suggested by ***(Base and Murray,1981)***, their equation neglecting the effect of the base, as their experimental work was based on end restrained wall. Their equation was:

$$\rho = \left(\frac{\varepsilon_{sh} - \varepsilon_t}{3} + \varepsilon_t\right)\frac{E_s}{nf_s l}\left(l - 2ma\right) - \frac{2ma}{nl} \quad .....(2\text{-}26)$$

Where l is the length of the member

m is the number of the cracks at any time

They also gave the opinion that the steel ratio obtained from the equation above should not be less than the critical steel ratio of equation(2-24).

2.14-Thick sections:

As reported by (***Campbell-Allen,1979)*** ,the above considerations are based on the assumption that the distribution of strain over the cross section of the member is uniform . While this assumption is reasonable for the usual slender structural member , ***Hughes (1972)*** suggests that for massive sections, where drying shrinkage and thermal strains are primarily surface effects, the reinforcement ratio should be based on an effective “surface zone” having a thickness assumed to be approximately (250)mm at each cooling or shrinkage face, as reported by ***ACI 318,2002***, . Hence the quantity of reinforcement required in a wall exceeding (500)mm in thickness should be the same as that for a thickness of (500)mm. ***Leonhardt (1977)*** suggests a similar figure for the effective surface zone at each face of (200)mm to (300)mm .

Chapter Three

Experimental work

3.1- Introduction:

The main aim of this research is to study the effect of drying shrinkage on the end and base restrained concrete wall through the study the strains of concrete (cracks number, crack width, crack height, crack spacing, and sequence of cracking)

This research has been concerned with some variables such as (length/height ratio) of wall, environment conditions (winter and summer). Other variables that have not been considered such as the amount of reinforcement, and type of mixes, where these variables have been examined by many previous researches (**Kheder - 1986, Al Attar - 1988, and Al Mashhadi** – **1989)** and studied in detail.

The use of mortar walls instead of concrete walls is to amplify the effect of drying shrinkage on the crack generation. Also the most previous researches have not come out with a good results in the concrete specimens due to low shrinkage tendency of concrete in the limited period of time.

3.2-Program of the work:

Three different (length/height ratios) mortar walls had been cast in winter and another three mortar walls cast in summer. The three shapes are of the dimensions (2000 * 1000 * 100) mm , (4000 * 1000 * 100) mm , and (4000 * 500 * 100) mm, (length * height * thickness) respectively.

The (L/H) ratios of these three shapes are (2,4,8) respectively have been chosen so that one of them less than 2.5 and the others, more than

2.5 to study the two types of cracks (secondary cracks in walls of (L/H) ratio less than 2.5 and primary cracks in walls of (L/H) more than 2.5.

To reduce the time interval that is required to form a rigid lateral and base restraint, a rectangular steel frame are constructed of W-shape steel beams with a base of semi natural roughness performed by welding a (4) deformed bars φ (10)mm along the top surface of the (I) section to simulate the natural roughness of the base.

All sets of mortar walls were of mix proportions (1:2:0.5), (cement, sand, w/c ratio) respectively. Table (3-1) shows the wall shapes and notation for both sets of Summer and Winter.

Table (3-1) Wall characteristics

Wall notation	Wall dimension (length*height)m	Time of casting	Mix proportion Cement:sand:w/c	P% (Horizontal)
2W	2*1	Winter (January)	1 : 2 : 0.5	0.3
4W	4*1			
8W	4*0.5			
2S	2*1	Summer (June)	1 : 2 : 0.5	0.3
4S	4*1			
8S	4*0.5			

The reinforcement used was of deformed steel bars of grade 425 MPa with 10 mm-diameter for both vertical and longitudinal reinforcement, and the steel ratio adopted in this work was (0.3%) which is the minimum ratio allowed for shrinkage according to previous research as shown in Figure (3-1).

After casting, each wall remained two days in Winter and one day in Summer without dismantling the molds and with covering the upper surface of the wall to prevent the plastic shrinkage occurrence. The curing of walls began directly after removing the form work. The curing period continued for (7)days in Winter and Summer.

For each set of walls, (12) cubes of (100)mm for compressive, (12) cylinders of (100*200)mm for splitting, and (12) cylinders of (150*300)mm for modulus of elasticity were cast for two ages of strength (7) and (28) days and for both, water curing and outdoor curing. The specimens in the outdoor condition cured by the same way of walls curing. All specimens were of mortar of the same mix proportion of walls.

The measurements of free shrinkage of the walls were performed by using extensometer to measure the strains horizontally at different levels of wall height. Cracks width were measured by using crack meter microscope as shown in plate (3-1). These measurements continued for (60)days.

The free volume change of mortar only was measured by using I-shape mortar beam restrained at both ends and with mid length diaphragm according to model of ***Al Rawi, 1985*** . The measurement continued for the same period of wall measurements.

The elastic tensile strain of mortar was measured by using the same I-shape mortar beam restrained at both ends, but without mid length diaphragm. The measurement continued until the initiation of the first crack in the beam.

3.3-Experimental work

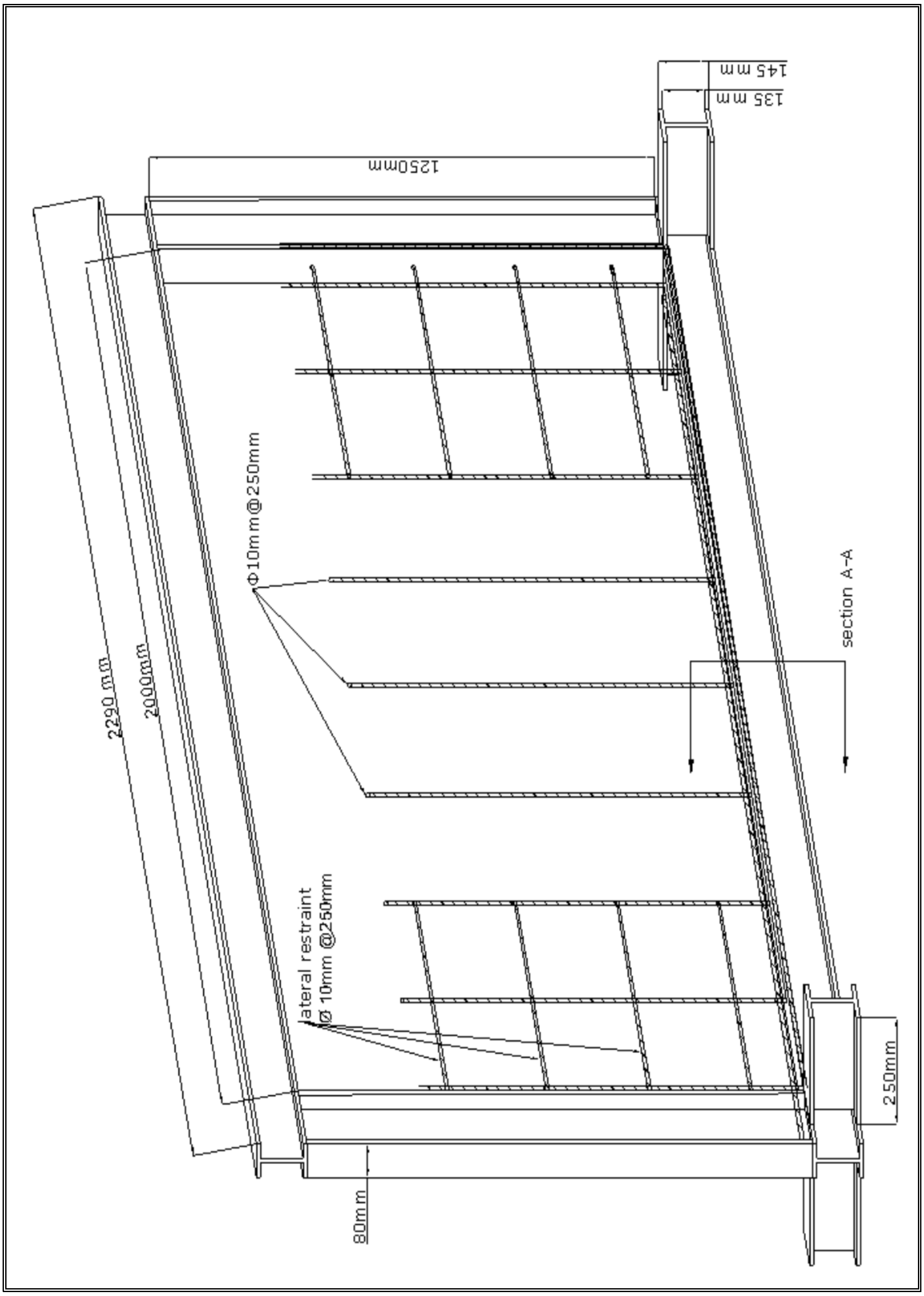
3.3.1Edge and base restraint of walls:

The restraint of the walls was made by constructing a rectangular steel frames. The properties of the steel beams used were W-shape ($W_{6\times12}$) of cross sectional area of (1758)mm^2, web height (145)mm, and flange width (80)mm. The frames were checked structurally for the restraining purposes.

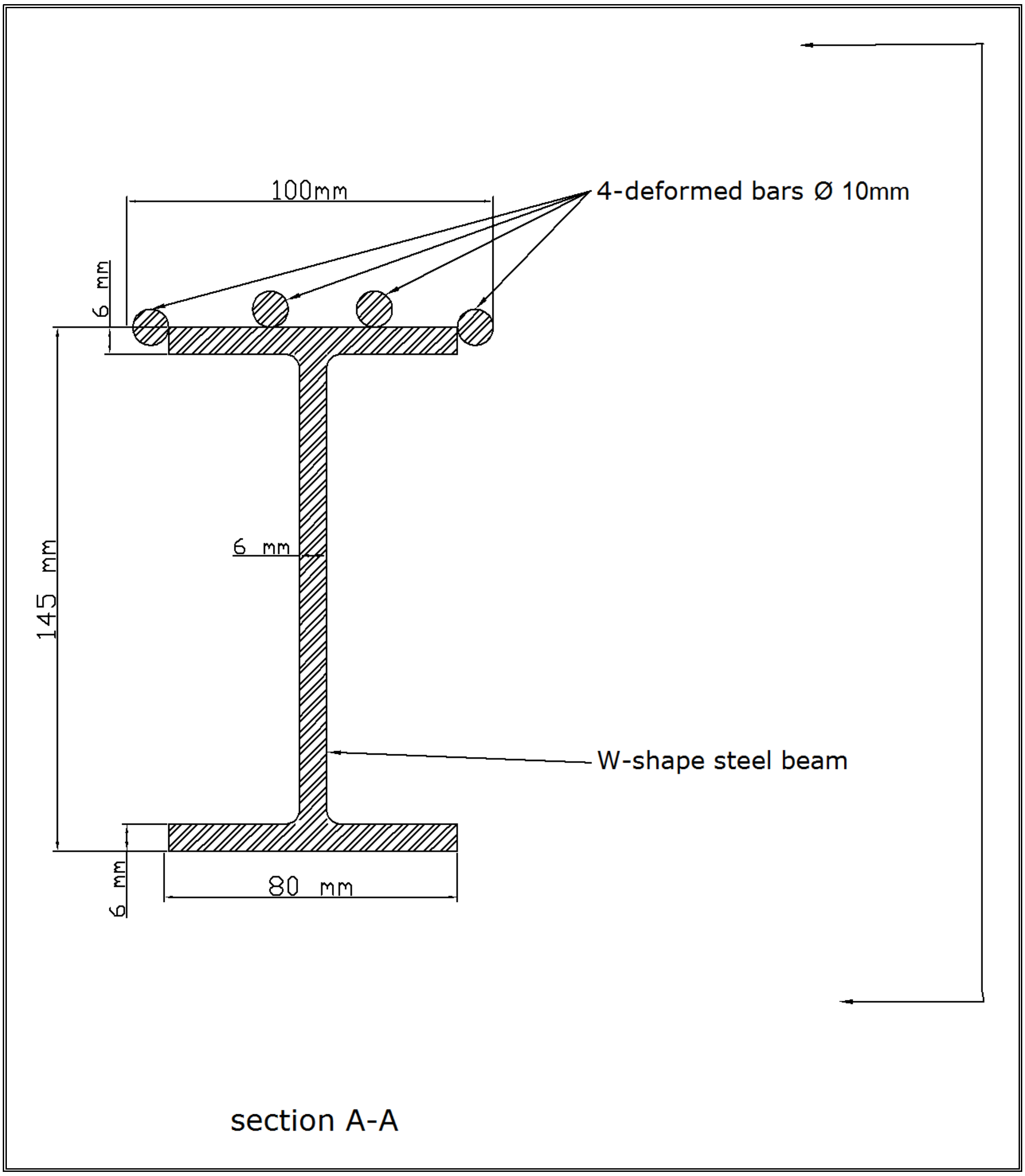
Because of the large sizes and heavy weights of these three frames, they were constructed inside the laboratory. Each frame was composed of lateral two steel column (end restraint), lower steel beam (base restraint), and the upper steel beam, fixing all the parts to each other at ends by welding to form a monolithic single frame as shown in figure (3-1).

The vertical and lateral dowels were fixed to frame by welding in the same plane of casting to give the efficient restraint. To increase the roughness of the base, (4) steel deformed bars φ (10)mm were welded along the upper surface of the base.

The choice of this type of restraint is for two reasons: first to reduce the time required to exhaust the shrinkage strain of the base which might extend to several months. second, the thermal expansion of steel is similar to that of concrete as reported by ***ACI committee 207.2R-95*** and ***209R-92***.



Figure(3-1)Edge and base restraining steel frame of (Length/Height)ratio equal(2)



Figure(3-2)cross section of the restraining steel base of three different (Length/Height) mortar walls

Plates (3-2) and (3-3) show three types of frames with different L/H ratios.

3.3.2-Molds of the Walls :

The form work used in the casting of the walls is composed of four ply woods of dimension (1000*2400)mm and thickness (16)mm and two steel plates of (2000*1000)mm with thickness of (0.9)mm, were fixed on the internal face of the frontal 2-ply woods to give a smooth surface. The surfaces of the steel plates were greased (with oil) before using to facilitate the dismantling of the molds.

The ply woods of both sides were supported by timber joists of length (1500)mm and cross sectional area of (50*100)mm on each sides with spacing of (500)mm as shown in plate(3-4).The molds were removed after 2-days in Winter and 1-day in Summer to avoid the disturbance of the measuring faces.

Six wall models were cast in the above mentioned steel frames. These walls were grouped in two sets ,the first set in winter and the other in Summer .

3.3.3 -Materials

3.3.3.1 –Reinforcement

Deformed steel bars of 10 mm diameter were used. The average yield strength of three samples is 425 MPa. The test was coducted in the material engineering laboratory University of Babylon. The bars conform to ***ASTM-A615*** specifications.

3.3.3.2 –Cement

The cement used in this study was ordinary Portland cement manufactured by the New Cement Plant of Kufa. The cement was properly stored in the laboratory. This cement complied with the ***Iraqi specification No.5/1984***. Tables (3-2) and (3-3) show the chemical and physical properties of this cement respectively. The test was made in the environmental laboratory University of Babylon.

Table (3-2): Chemical Composition of Cement

Oxide	%	I.O.S. 5:1984 Limits
CaO	61.89	–
SiO_2	20.0	–
Fe_2O_3	3.24	–
Al_2O_3	5.96	–
MgO	4.15	≤ 5%
SO_3	2.35	≤ 2.8
Free lime	1.12	
L.O.I	1.47	≤ 4.0
L.S.F	0.88	0.66-1.02
Compound Composition	**%**	**I.O.S. 5:1984 Limits**
C_3S	39.86	----
C_2S	28.76	----
C_3A	10.31	----
C_4AF	9.86	----

Table (3-3): Physical Properties of Cement

Physical Properties	Test Results	I.O.S. 5:1984 Limits
Setting Time:		
Initial min	130	≥ 45 min
Final min	200	≤ 600 min
Compressive Strength MPa		
3-days	18.2	≥ 15
7-days	27.7	≥ 23
Fineness (Blaine),in m^2/Kg	325	≥ 230

3.3.3.3 –Fine aggregate

Natural sand from AL-Najaf Sea region was used. Table (3-4) shows the grading of the fine aggregate and the limits of the Iraqi specification NO.45/1984. Table (3-5) shows the physical and chemical properties of fine aggregate.

Table (3-4): Grading of Fine Aggregate

Sieve No.	Sieve Size (mm)	Passing %	I.O.S. 45:1984 Limits Zone (2)
3/8 in	9.5	100	100
3/16 in	5	97	90-100
8	2.36	83	75-100
16	1.18	70	55-90
30	0.60	57	35-59
50	0.30	25	8-30
100	0.15	4	0-10
	Fineness modulus	2.64	

Table (3-5): Physical and Chemical Properties of Fine Aggregate

Physical Properties	Test Results	I.O.S. 45:1984 Limits
Specific gravity (S.G)	2.65	–
Absorption %	1.6	–
Sulfate content (SO3)%	0.432	≤ 0.5
Clay % (< 75μm)	2.3	≤ 3.0

3.3.3.4 Water

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

Note: all tests of material were conducted in the laboratory University of Babylon.

3.3.4 –Mortar mixes

All sets of walls were cast with a single mortar mix. The mix proportion was (1:2) (cement : sand) by weight and effective w/c ratio of 0.5.

The compressive strength of 7 and 28 days were 24 MPa and 30 MPa respectively whereas the 7 and 28 days splitting strength were 5.31 and 6.46 MPa respectively .

3.3.5 –Mixing procedure and Castig

The mixing procedure is important to obtain the required homogeneity of the mortar mix. Mortar was mixed in the electrical tilting - type laboratory mixer of size 0.05 m^3 cement according to ***ASTM-C192*** and ***C305*** specifications. The sequence of mixing steps are shown in Table (3-6).

Table(3-6) Steps of mixing of mortar

sand + 1/2 of water	Mixing 1 minute
1/2 of cement	Mixing 1 minute
1/2 of cement	Mixing 1 minute
1/2 of water gradually	Mixing 2 minute

The mixes were placed in the molds in four lifts, each lift was compacted internally by using electrical vibrator.

Plate (3-5), (3-6) and (3-7) show the three types of end and base restrained wall with different (L/H) ratio.

3.3.6 –Curing and Exposure

To prevent plastic shrinkage cracking due to rapid evaporation from the upper surface of the walls and beams of elastic tensile strain and free shrinkage cured according ***ASTM C192***, polythene sheets were used to cover the upper surface of the beams and walls after 30 minute from the casting. The walls and beams were cured by covering them with wetted hessian and polythene sheets and wetted once every day for first 7-days, then air dried in uncontrolled laboratory and outdoor conditions for 60-days.

3.3.7-Testing of Mortar Specimens

3.3.7.1- Compressive Strength Test

For the hardened mortar, the compressive strength test was carried out according to ***BS.1881 part 116:1983***, using a machine of (2000 kN) maximum capacity. The load was applied and increased gradually at a constant rate (200-400)KN per second. Each compressive strength value was average of three cubes of (100)mm.

3.3.7.2- Splitting Tensile strength

The splitting tensile strength was determined according to the procedure of ***ASTM C-496***. Each splitting tensile strength value was an average of three specimens.

3.3.7.3- Modulus of Elasticity Test

The static modulus of elasticity were determined according to ***ASTM C-469*** specification.. The top surface of cylinders was well finished and capped with soft plastic pad to avoid any loss of strength. The load was applied gradually and increased continuously at constant rate until the ultimate load. The dial gauge used has a gauge length of 200 mm, with an accuracy of 0.001 mm. The specimen were reloaded for 40% of ultimate load for at least two cycle before final loading. The recorded results were the average readings of two cylinders. The modulus of elasticity was calculated from the relation:

$$E = (R_2 - R_1)/(\delta_2 - 0.00005)$$

Where:

E : the static modulus of elasticity, MPa .

R_2 : stress corresponding to 40% of ultimate load, MPa.

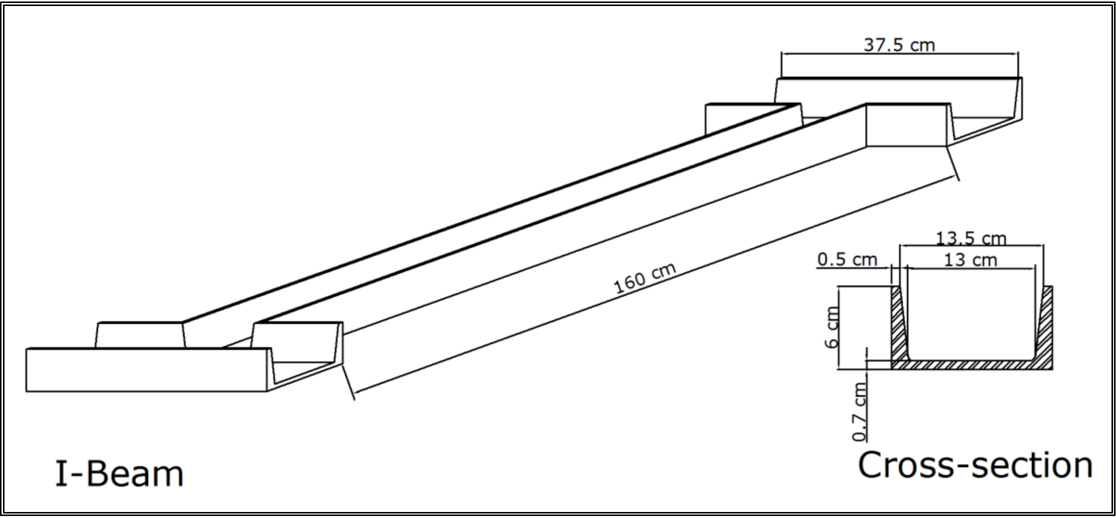
R_1 :stress corresponding to a longitudinal strain (0.00005), MPa.

δ_2 : longitudinal strain produced by stress R_2.

Note: All tests of specimens were conducted in the laboratory of Babylon Technical institute.

3.3.8-Elastic tensile strain capacity tests

Elastic tensile strain capacity of mortar measured according to the direct method by using the same I-shaped steel mold ***(Al-Rawi, 1985)*** as shown in the Figure (3-3). Plain mortar beam was cast 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 mortar, crack width was measured by microscope divided by the total length which was assumed to represent elastic tensile stain capacity of mortar.



Figure(3-3) Steel I-Beam mold for elastic tensile strain and free shrinkage measurement

3.3.9 –Free volume change

In order to measure the free volume change of the mortar walls, beam of plain mortar was cast in the same I-shaped mold as described in the section (3.3.8) with an artificial crack (gap) made in the mid-span of the beam by using a 4 mm plastic diaphragm, and exposed to the same conditions of restrained mortar walls . The friction in the contact surfaces between the mortar and the mold was minimized by applying two layers of greased polythene sheets, and covered with polythene sheet after casting of the beam to avoid the rapid plastic shrinkage and cured daily for 7 days and then subjected to exposure conditions similar to those of the walls . this beam was left free to shrink and move. The movements were measured by two demec points fixed adjacent to the gap sides and an extensometer. The measuring continued for the same period of walls measurement (about 60 days) to represent free volume change with time.

Surface strain measurements of the walls and of the restraining base were carried out by using 4 rows of demec points fixed on the walls and one row fixed on the steel base. The rows were at heights 30, 150, 500 and 950mm from the base. The spacing between demec points in the same row was 200mm. A two-pack of epoxy resin was used to glue the demec points to the wall and base. An extensometer with an accuracy of (0.002mm/division) was used to measure strain. Measurements were taken in the morning (about 8 o'clock) every 2 days until of the first crack, then measurements prolonged for three months in winter and for two months in Summer.

The measurements of temperature ($T_{av.}$) and relative humidity (RH) were taken from weather forecasting center in Babylon through the period of measurements as shown in table (3-7).

Table (3-7) Average temperature and relative humidity

Month	Babylon Forecasting Centre ($T_{av.}$) c^o	Babylon Forecasting Centre (RH)%
February	27	44
March	30	48
April	35.5	36
May	36.5	31
June	39.5	26
July	48	28
August	45.5	31
September	41.8	35

Crack widths were measured for each (100)mm of height according to the crack height from the base. The measurements were carried out by using a portable measuring microscope with 40X magnification and a measuring field of 3.5mm

Plate (3-1) measuring devices

Plate(3-2)Restraining frames of walls of L/H 2 and 4 respectively and their reinforcement

Plate(3-3)Restraining frame of wall of L/H (8) and its reinforcement

*Plate (3-4) Form work of wall (4*1) m*

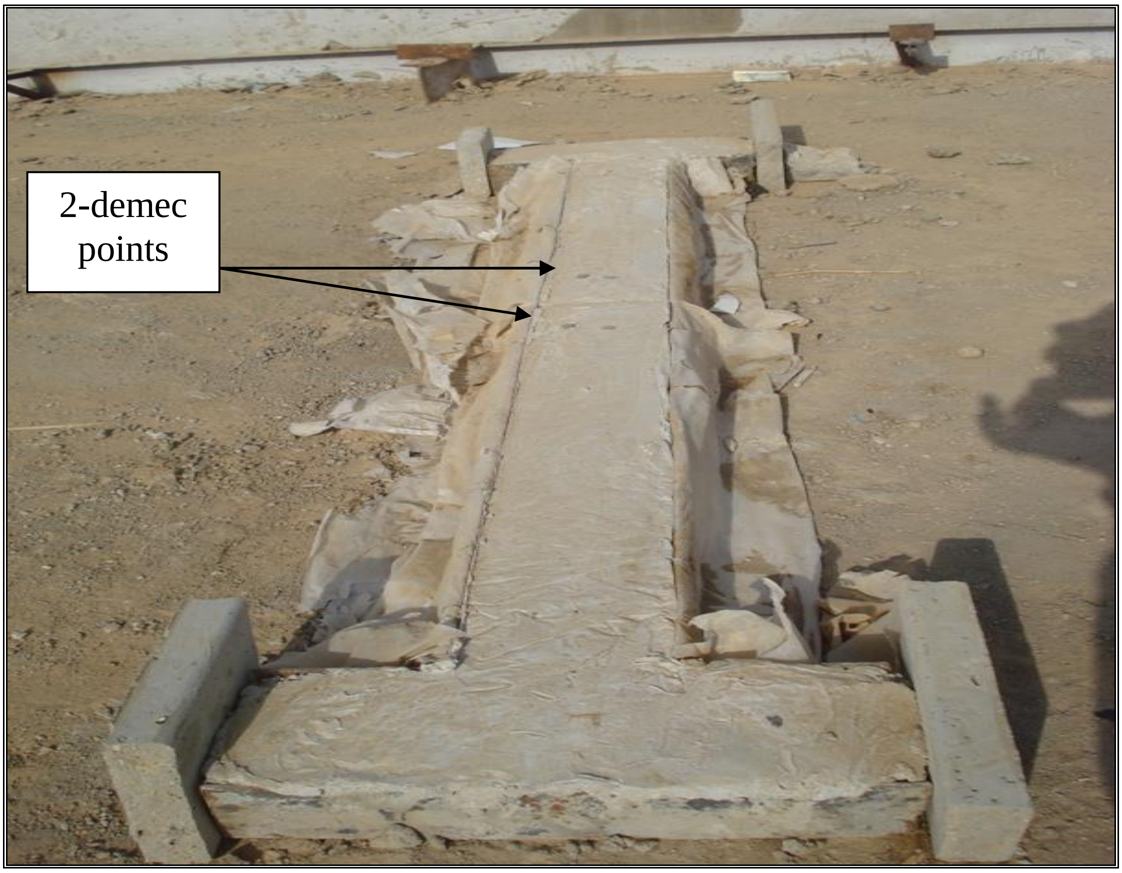
Plate(3-5) Curing of wall (2W)

Plate(3-6) wall (4w)

Plate(3-7) wall (8W)

Plate(3-8) Three walls (2S) (4S) (8S).

Plate(3-9) Elastic tensile strain measurement (I-shape mold).



Plate(3-10) Free shrinkage measurement (I-shape mold)

Chapter Four

Results and discussion

4.1- Introduction:

In this chapter, the results obtained from the experimental work are presented and discussed. These results include the following characteristics of mortar such as free volume change, elastic tensile strain, elastic tensile stresses of restrained wall movement, cracking data of walls and their relationships.

Most previous researches depend on mortar walls results, because of developing a complete cracking pattern greater than concrete walls, therefore mortar walls are chosen instead of concrete walls in this study.

4.2- Free volume change:

To investigate the behavior of the free shrinkage of mortar individually, the same model of ***Al Rawi, 1985*** of I-shape beam was used as described in chapter (3). The model was cast at the same time and exposed to the same outdoor conditions of the walls to observe the mortar shrinkage individually without effect of base and reinforcement restraint.

The first measurement for this model was taken after (7)days of casting immediately after finish of curing period to avoid more losses of shrinkage strain. Figure (4.1) shows the free shrinkage of I-shape mortar model. The maximum value of free shrinkage of I-shape beam obtained in this work was (1235) microstrain through a period of (45) days.

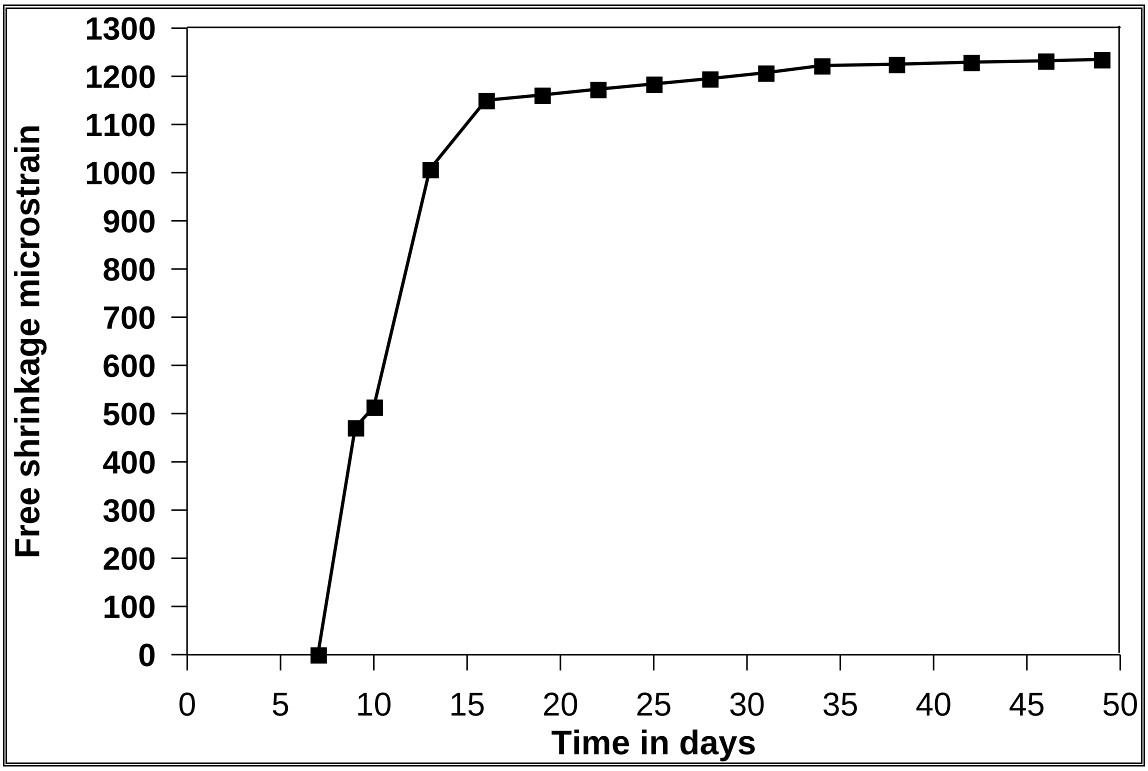


Figure (4-1) Free shrinkage strain of I-shape plain mortar

4.3- Elastic tensile strain capacity:

Elastic tensile strain capacity of mortar was obtained by measuring the crack width by crackmeter immediately after cracking of I-shape plain mortar beam and divided by the total restrained length of the beam.

$$\varepsilon_{ult} = \frac{crack\ width}{total\ restrained\ length} \quad \text{...........(4-1)}$$

This beam was cured for 7 days and then exposed to the same outdoor conditions of walls. The value of (ε_{ult}) of mortar is (175) microstrain. It was obtained after (12) days of casting where the first crack initiated.

4.4- Restrained Wall Movement:

The horizontal wall movements during the observation period were measured at four different levels (30, 150, 500 and 950 mm above the base) in the walls of height 1000mm, whereas in the walls of height 500mm the levels of measurements were (30, 150, and 450mm, by using extensometer to investigate the effect of base and end restraint with different Length/Height ratios on the final cracking pattern.

Through the observations of Figures (A-1) to (A-31), the strains of the upper rows were more than the strains of those near the base, although the full height was subjected to the same degree of the end restraint (R=1), this

due to the effect of base restraint that minimize the strains gradually downward. **Harrison, 1981** states that "the effect of base restraint decreased toward the upper rows". Also this phenomenon were approved by the ***ACI committee 207.2R-95, Stoffers, 1978, and AL Mashhadi, 1989***. The second reason was the slippage that occurred in the lateral restraining dowels where this was observed clearly in the oblique and horizontal cracks that occurred near the lateral dowels in each wall.

In general the readings of free shrinkage of each adjacent two column were fluctuating from maximum to minimum values, this is attributed to the confining field of movement, where this field is approximately limited by the lateral two edges while the whole length tend to contract centrally, therefore the amount of contraction (shrinkage) for full length of any row equals the amount of expansion (creep or cracking) of the same row.

The positive values represent contraction of this column at that level, whereas the negative values represent the expansion of this column at the same level.

The abrupt and extreme changes at such positions in walls represented by the maximum and minimum values represent initiation of a crack inside or beside that column at this level.

The contraction of the base during the exposure period of the wall was called loss of restraint (L_r). The difference in the measured contraction between the base and the wall at their contact surface before the occurrence of the cracking, represent the slippage between them (sl_b)whereas the difference in the measured strain between the base and the wall at their contact surface at the end of the exposure period represent the slippage after cracking (sl_a). The average (l_r, sl_b, sl_a) measured for all walls (8W, 4W, 2W, 8S, 4S, 2S) which are illustrated in table (4-1).

Table (4-1) Losses of base restraint, slippage before, and slippage after cracking in the contact surface

Set order	Wall notation	L_r *10^{-6} (microstrain)	Sl_b *10^{-6} (microstrain)	Sl_a *10^{-6} (microstrain)
winter set	8W	62.5	0	107.5
	4W	57.5	75	65
	2W	41.5	8.5	9
Summer set	8S	70	30	40
	4S	75	32.5	67.5
	2S	35	0	15

Through the observations of Figures (A-1) to (A-31), it has been found that:

1. Most of the absolute positive values are greater than the absolute negative values, this means that the amount of total drying shrinkage

of each row is greater than the total amount of expansion and this means the end restraint has a partial effect on the members of end restraint.

2. The absolute positive and negative values for each wall decrease toward the base, this means that the effect of base restraint appears gradually toward the base to reduce the induced strains of end restraint.
3. The summation of strains of each row produces positive value, this means that the walls tend to shrink, despite of the end restraint, this indicates an occurrence of a slippage in the end restraint or loss of restraint in steel frame.
4. The amount of drying shrinkage strain of each wall in Summer is greater than in Winter for the same wall shape.
5. The negative values of strains indicate that there is an expansion in this column and the repeated negative values in the same position represent initiating a crack in this column.
6. The amount of slippage after cracking was less than that obtained in the previous researches due to the presence of end restraint as well as the high base restraint where the restraining surface of base per unit length is equal to wall thickness plus perimeter of four welded bars on the base whereas in the previous researches the restraining surface per unit length is equal to wall thickness only.
7. Most final measurements of strains near the ends prone to abrupt high positive value, this indicates occurrence of slippage in the end restraint.

4.5- Cracking of walls:

Tables (4-2) and (4-3) summarize the cracking data of two sets of mortar walls. As shown in the tables, the effect of environment conditions (temperature and humidity) were very large on the drying shrinkage and cracking of walls, where this effect is clear in the number and the age of crack appearance which were faster in Summer.

The first reason for these results was the presence of walls in moist environment in Winter reduces the drying shrinkage and then the mortar will have a slight increase in the tensile strain capacity before drying onset whereas in Summer the mortar stresses before acquiring the sufficient hardening, so that the mortar can withstand the cracking if they are cast in Winter. The second reason was that the creep of mortar in Winter can take its role to relief the induced stresses before cracking initiation.

As shown in Table (4-2) and (4-3) and Figure (4-2), the number of cracks and the average cracking height increase with the increase of (L/H) ratio. These results, according to ***ACI committee 207.2R-95,*** where the wall of base restrained, the degree of restraint increased upward with increase of (L/H) ratio, therefore the height and number of cracks will increase consequently.

Table (4-4) shows a comparison in the number of cracks with previous research. In this study the number of cracks that induced in wall of (L/H=2) (2S) is greater than that found by ***Kheder, 1986*** and ***AL Mashhadi, 1989*** of base restrained walls of (L/H=2). Also for walls of (L/H) greater than (2.5), it has been found that the number of cracks in wall of (L/H=4) is greater than the number of cracks found by ***Kheder, 1986*** of wall of (L/H=5). This means that the case of end restraint has a significant effect on the final cracking pattern of walls with different (L/H) ratio.

Table (4-2) cracking data of walls (Winter set)

1st set. (Winter)						
Wall name	Wall Dime-sions	Distance from left edge cm	Crack height cm	Maximum crack width mm	Height of max. crack width cm	Crack no.
2W	(2*1) m	55	21	0.1	6	1
		150	15	0.05	5	2
4W	(4*1) m	30	15-25	0.02	5	1
		95	45-60	0.03	5	2
		135	0-25	0.08	10	3
		175	0-45	0.3	35	4
		205	0-20	0.04	5	5
		295	0-30	0.08	10	6
		335	0-20	0.04	5	7
8W	(4*0.5) m	145	5-23	0.04	8	1
		165	10-25	0.03	5	2
		200	0-15	0.02	5	3
		220	0-40	0.16	5	4
		245	0-30	0.08	10	5
		265	5-25	0.08	15	6
		320	0-15	0.02	5	7
		350	10-24	0.03	10	8
		385	0-23	0.03	8	9

Table (4-3) cracking data of walls, (Summer set)

2nd set. (Summer)						
Wall name	Wall Dime-sions	Distance from left edge cm	Crack height cm	Maximum crack width mm	Height of max. crack width cm	Crack no.
2S	(2*1) m	25	0-36	0.08	21	2
		32	0-15	0.03	5	5
		48	10-48	0.04	20	3
		50	23-47	0.06	9	3
		60	5-19	0.03	10	3
		77	0-67	0.04	40	2
		99	5-60	0.03	40	2
		116	3-52	0.14	34	1
		126	0-45	0.1	20	2
		141	0-22	0.03	7	4
		165	5-45	0.2	25	3
		180	5-52	0.08	40	1
4S	(4*1) m	40	0-50	0.06	20	5
		80	0-100	0.14	40	4
		133	0-32	0.2	12	2
		137	2-70	0.14	25	3
		148	0-52	0.06	7	2
		180	15-80	1.0	25	4
		185	0-100	1.0	50	1
		193	0-29	0.2	4	1
		216	3-30	0.02	12	2
		245	7-41	0.04	14	1
		264	5-45	0.04	10	2
		302	0-70	0.28	34	3
		333	20-60	0.04	20	3
		355	13-48	0.04	15	2
8S	(4*0.5) m	29	5-32	0.04	5	3
		52	0-24	0.04	20	3
		72	5-27	0.04	7	3
		107	0-32	0.14	17	4
		156	0-47	0.10	17	2
		200	0-30	0.04	15	5
		210	11-43	0.04	17	3
		233	0-19	0.04	4	4
		259	0-32	0.04	17	4
		276	0-32	0.04	7	1
		287	0-47	0.14	17	4
		302	0-23	0.04	8	5
		320	5-37	0.04	7	4
		331	10-46	0.03	5	3
		337	0-32	0.08	7	5
		348	8-39	0.03	6	2
		364	0-45	0.15	10	3

Table (4-4) comparison of minimum crack spacing with other researcher

Wall shape (L/H)ratio	Mix proportion	Ratio of steel reinforcement	No. of cracks	Min. crack spacing	researcher
2	1:2	0.3 %	12	0.1H	Present work
2	1:2	0.4 %	4	0.28H	ALMashhadi
2.5	1:3	0.4 %	2	0.8H	Kheder
4	1:2	0.3 %	14	0.25H	Present work
5	1:3	0.4 %	9	0.6H	Kheder

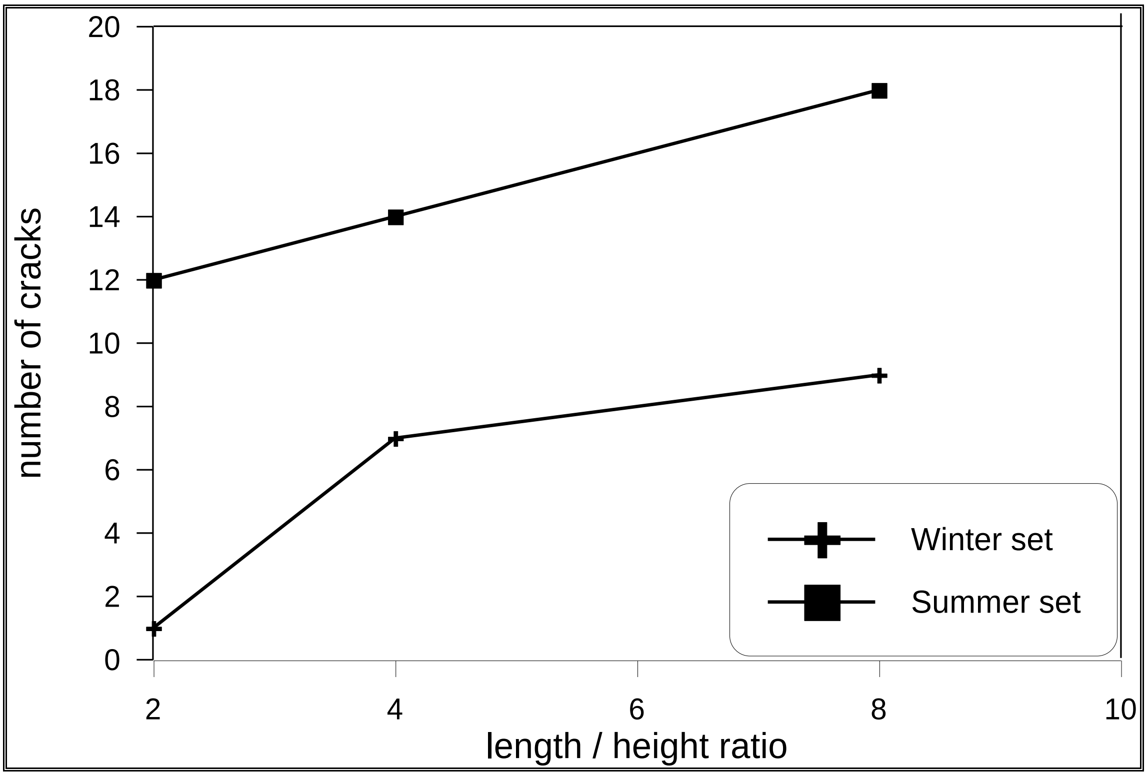


Figure (4-2) Relationship of number of cracks with different (L/H) ratios of walls in Winter and Summer.

4.6- Cracking age and sequence

In this study, it has been observed that more than one of initial cracks were induced in the same time at different spacing of the wall length. In each intermediate spacing of initial cracks there were another group of cracks formed. With progress of initial cracks age, these cracks will develop in length and width and another cracks will initiate. The crack initiation of most walls was in the lower third of wall height and then developed upward and downward.

The initiation of cracks of base and end restrained walls with (L/H) less than (4) was complied to the ***ACI committee 207.2R-95*** in the cracking sequence of the continuous base restrained walls as shown in Figure (2-14) with few differences. The ***ACI committee 207.2R-95*** states that the highest degree of restraint in the bottom of wall centerline; therefore, the first crack induced in this position. After formation of first crack, the degree of restraint will decrease to zero and new distribution of restraint will form, the new highest degree of restraint in the centerline of new two halves of wall, therefore another crack will form in this position. This can be seen clearly in walls of ($L/H \leq 4$) as shown in Figures (4-6) and (4-7).

In walls of ($L/H=8$) the cracks initiate in the upper third and then descend toward the base. After that new (L/H) will form, therefore the cracks will initiate in the lower third of wall to comply to the same effect of walls of ($L/H \leq 4$) that mentioned previously, as shown in the Figure (4-8), this attributed to the end restraint, where the upper part of the wall restrained in both ends only, therefore the total drying shrinkage of this line will induce small number of cracks with large cracking width, this will lead to quick formation of cracks, while the lower part of the wall restrained by end and base, therefore the total drying shrinkage of this line will induce a large number of cracks with very small cracking width (effect of base restraint) and this will lead to delay the formation of cracks after that of upper part of wall.

All cracks formed in the three walls of Winter set were secondary cracks, while the cracks of Summer set were composed of two types of cracks, secondary cracks in wall of (L/H=2) and (secondary and primary) cracks in walls of (L/H=4 and 8) to confirm the theory of base restrained walls of ***ACI committee 207.2R-95*** and the results of previous researches.

Due to the high rate of drying shrinkage in hot and dry weather (Summer) and consequently the development of the tensile stresses is greater than the development of tensile strength in the early ages, therefore the walls of (L/H) greater than (2.5) can produce primary cracks while the walls of Winter set of the same (L/H) can not produce that, this due to the creep effect which can take its role to relief the slowly induced stresses consequently the primary cracks can not form.

As shown in Figures (4-3) to (4-8) all cracks that formed along the wall length were vertically aligned in comparison with that of base restraint walls. This case of vertical cracking differ from the cracks that induced in the base restrained walls, where the cracks that formed near the ends splayed laterally. The reason of this difference were the effect of end restraint that let the wall of end and base restraint near the ends to behave as a continuous wall subjected to horizontal stresses that produce vertical cracking, while in the walls of base restraint only, the plane stresses that act in right angle, horizontally by adjacent part of wall which induce horizontal force toward the wall center and vertically by base restraint, therefore the resultant of these two forces will produce an oblique cracks perpendicular to their opposite action.

Many plastic shrinkage cracks were initiated in the upper edge of each wall and descended downward about (10-15)cm as shown in Figures (4-3) to (4-8). This type of cracks had not given any importance in this study.

Figures (4-6) to (4-8) show the development of cracking with time for each wall of Summer set.

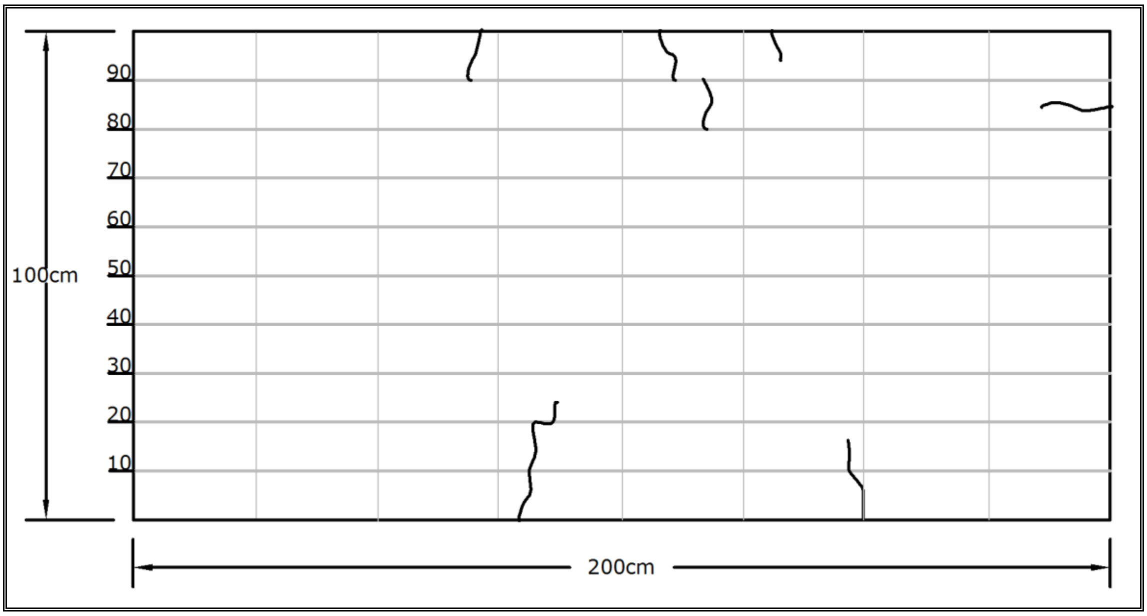


Figure (4-3) Final cracking pattern of wall (2W) after age of (90) days

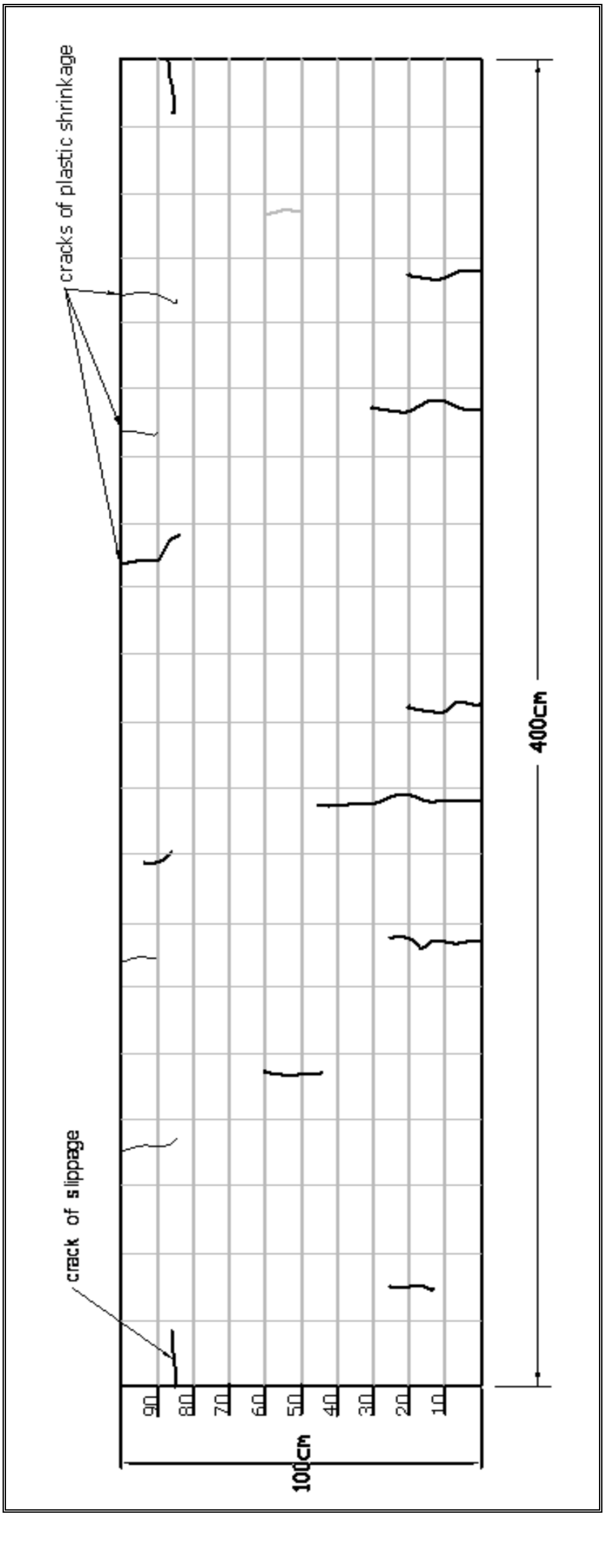


Figure (4-4) Final cracking pattern of wall (4W) after age of (90) days

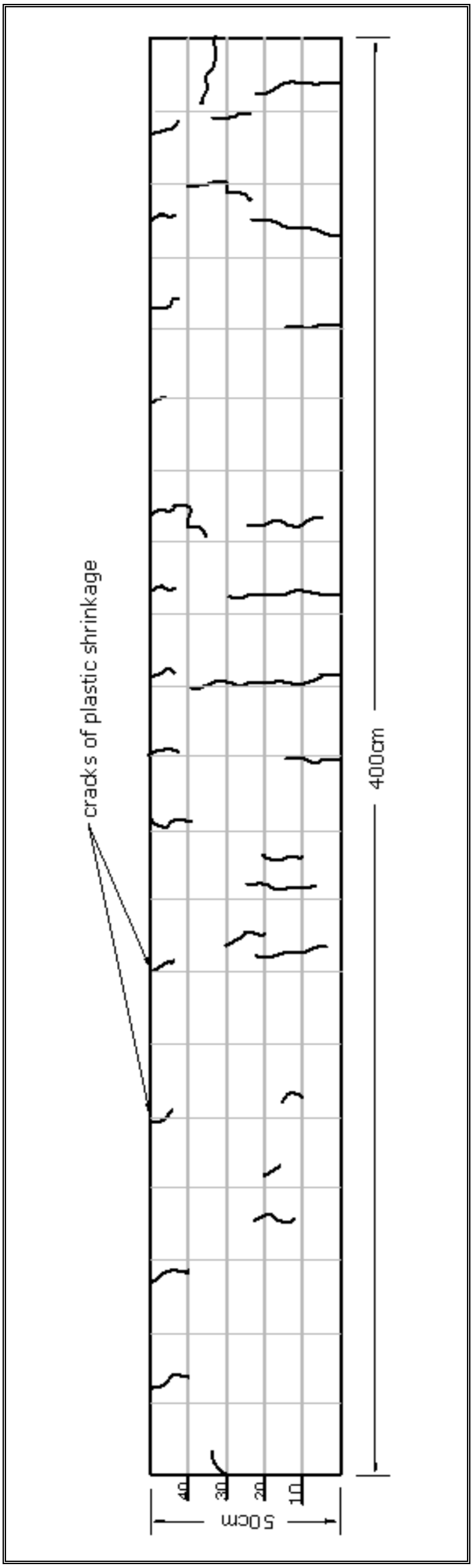
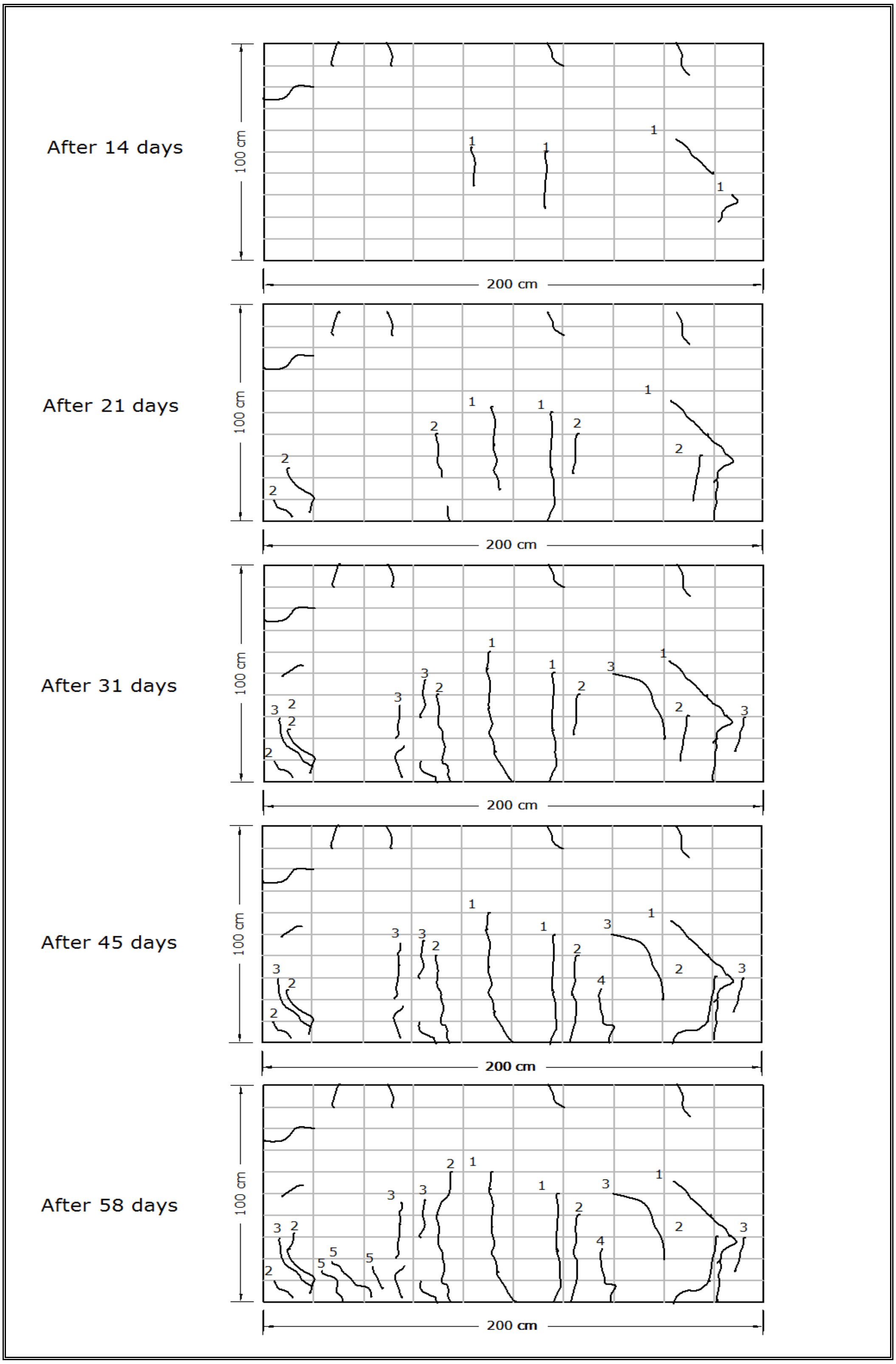
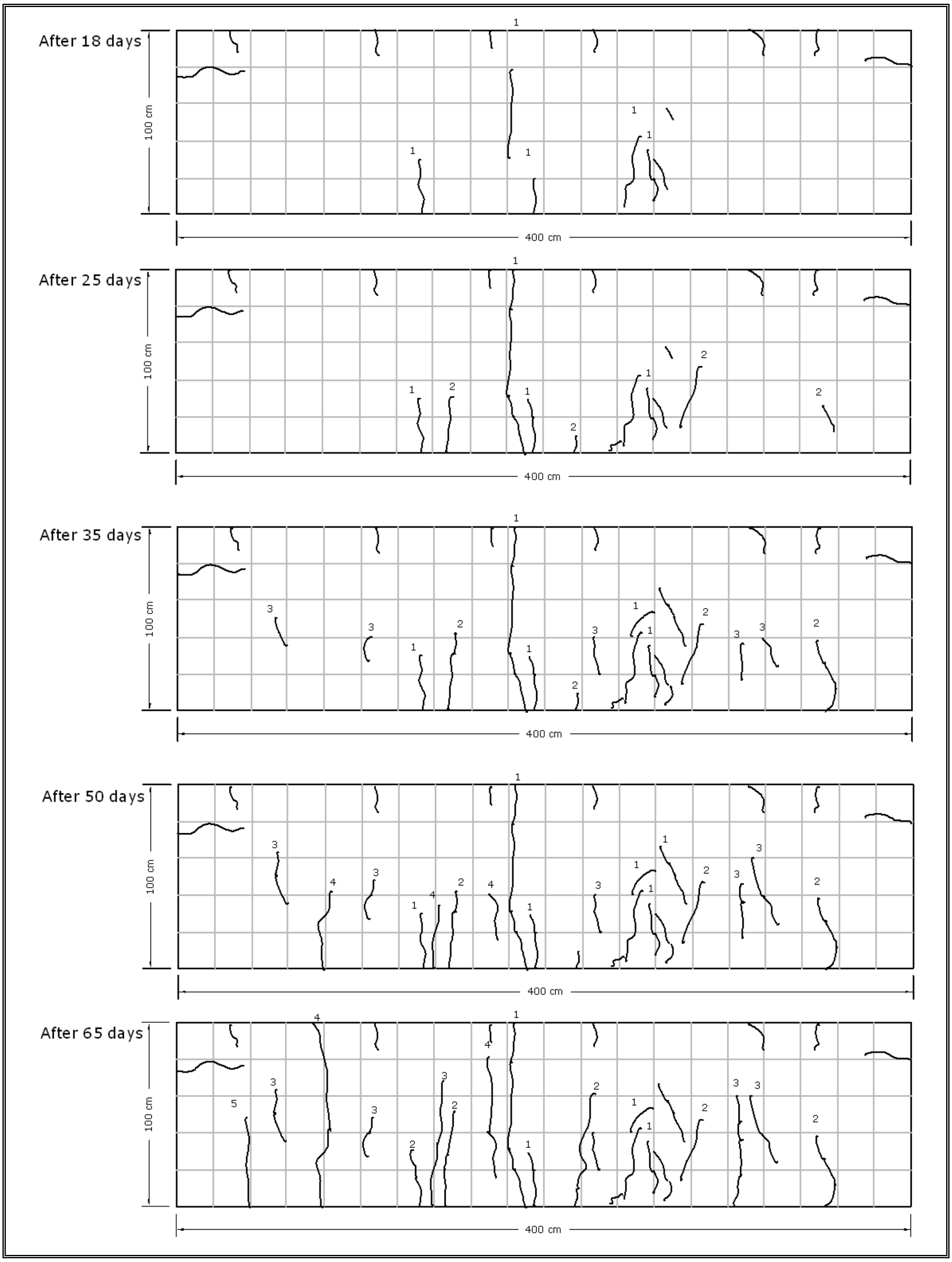


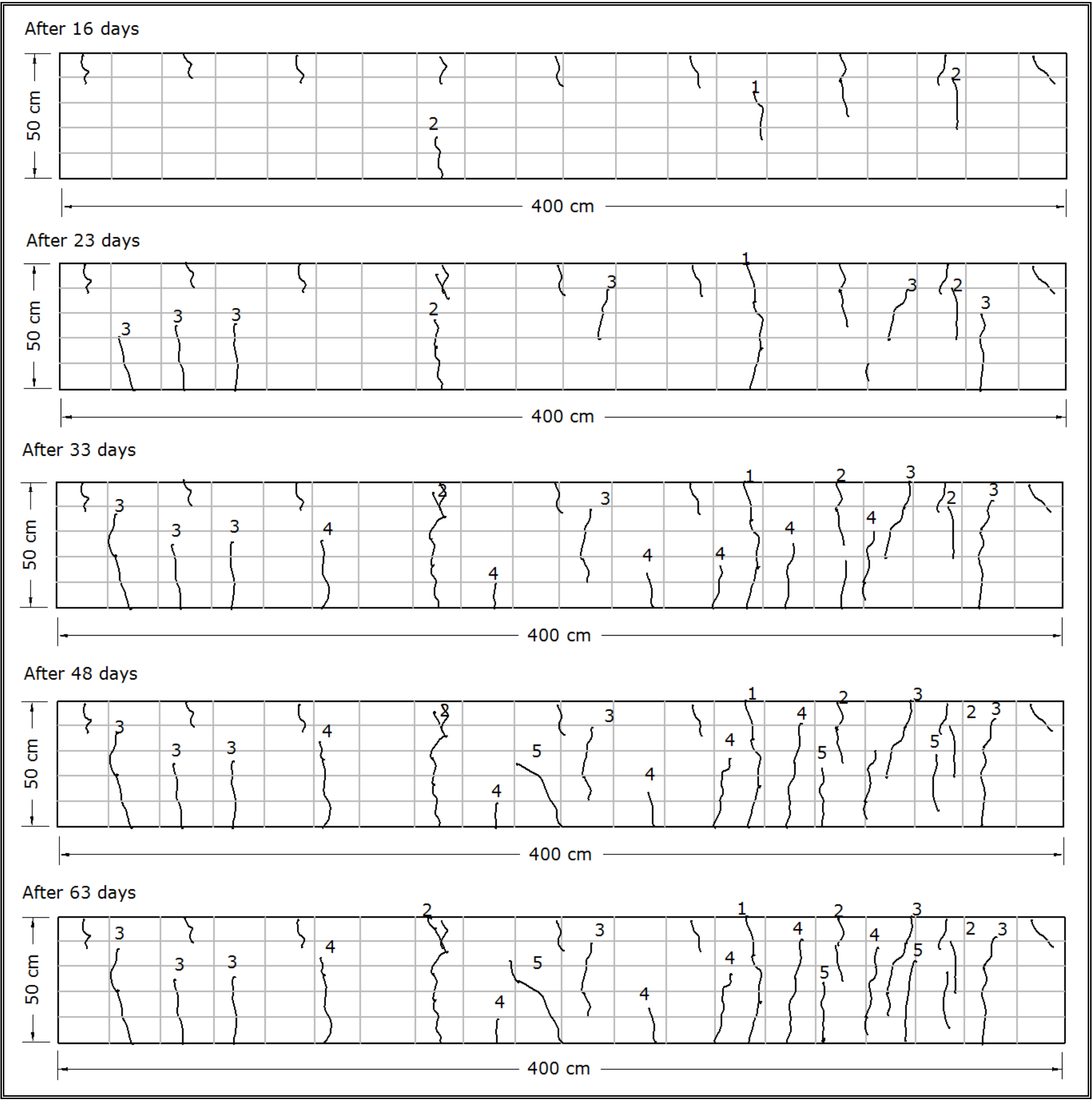
Figure (4-5) Final cracking pattern of wall (8W) after age of (90) days



Figure(4-6) Final cracking pattern of wall (2S) after ages of 14, 21, 31, 45 and 58 days of drying commence , (Summer set)



Figure(4-7) Final cracking pattern of wall (4S) after ages of 18, 25, 35, 50 and 65 days of drying commence, (Summer set)



Figure(4-8) Final cracking pattern of wall (8S) after ages of 16, 23, 33, 48 and 63 days of drying commence (Summer set)

4.7- Crack spacing:

Table (4-5) and (4-6) summarize the minimum, maximum, and average crack spacing in comparison to ***Stoffer's, 1978*** results. As shown in Figures (4-3) to (4-8) of the final cracking pattern of six walls in two sets (winter and Summer), the first set of walls (Winter set) showed a very poor cracking, the main reason of this results was the very low temperature (16-

22) C^o with high relative humidity about (40-48)% in the midday where the casting was in January. The second set of summer appeared to have a large number of cracks in each wall due to hot and dry weather that increase the rate of drying shrinkage where the casting were in the end of June, with degree of temperature (40) C^o and relative humidity (26)% according the data of temperature and relative humidity that obtained from the weather forecasting center in Babylon.

The information of crack spacing in winter deviate widely corresponding to other results due to incomplete cracking pattern, therefore we can not adopted these results in comparison, whereas the information of crack spacing in summer correspond to the trend of ***Stoffers, 1978***.

Stoffers, 1978 found for (18) model walls with (L/H) ratios of (6.67-8) which was under compressive prestress and which represented the floor. He observed that the spacing of cracks propagating through the full wall height ranged between (1-1.5)H and this valid for secondary cracks.

According to the equation of ***stoffers*** the theoretical crack spacing:

$S_{ave.}$=(0.1 – 1.2)H for H_c=(0.1-0.8)H

In this study through the Summer results, it has been found that the result of average crack spacing:

$S_{ave.}$=(0.16-1.0)H for H_c=(0.1- 0.8)H

Where $S_{ave.}$ = average crack spacing

H_c = crack height

H = wall height

Table (4-5) Minimum, maximum and average crack spacing of Winter walls

Wall name	Height Y/H	$S_{\max}/H$	$S_{\min}/H$	S_{ave}/H	Primary cracks		$S_{ave.Y}/H$
					$S_{\max}/H$	$S_{\min}/H$	***Stoffers***
2W	0-0.1	0.7	---	0.7	---	---	0.1-0.15
4W	0-0.2	0.85	0.3	0.48	---	---	0.2-0.3
8W	0-0.2	1.1	0.4	0.6	---	---	0.2-0.3

Table (4-6) Minimum, maximum and average crack spacing of Summer walls

Wall name	Height Y/H	$S_{\max}/H$	$S_{\min}/H$	S_{ave}/H	Primary cracks		$S_{ave.Y}/H$
					$S_{\max}/H$	$S_{\min}/H$	***Stoffers***
2S	0.0	0.25	0.1	0.16			
	0.1	0.3	0.08	0.2			0.1-0.15
	0.2	0.61	0.09	0.25			0.2-0.3
	0.3	0.4	0.08	0.25			0.3-0.45
	0.4	0.4	0.15	0.25			0.4-0.6
	0.5	0.7	0.2	0.45			0.5-0.75
4S	0.0	0.6	0.25	0.29	1.0		
	0.2	0.54	0.1	0.29			0.2-0.3
	0.4	0.44	0.1	0.28			0.4-0.6
	0.6	0.85	0.1	0.46			0.6-0.9
	0.8	-	-	1.0			1.20
8S	0.0	1.0	0.22	0.42	2.4	1.1	
	0.2	0.46	0.1	0.46			0.2-0.3
	0.4	0.57	0.1	0.46			0.4-0.6
	0.6	1.6	0.28	0.86			0.6-0.9
	0.8	1.3	0.3	0.72			0.8-1.2

Figure (4-9)shows the relation between wall height and the average crack spacing. These relationships were

$$S_{ave} = 0.86\ H \qquad \text{for (L/H=2)}$$

$$S_{ave} = 0.85\ H \qquad \text{for (L/H=4)}$$

$$S_{ave} = 0.72\ H \qquad \text{for (L/H=8)}$$

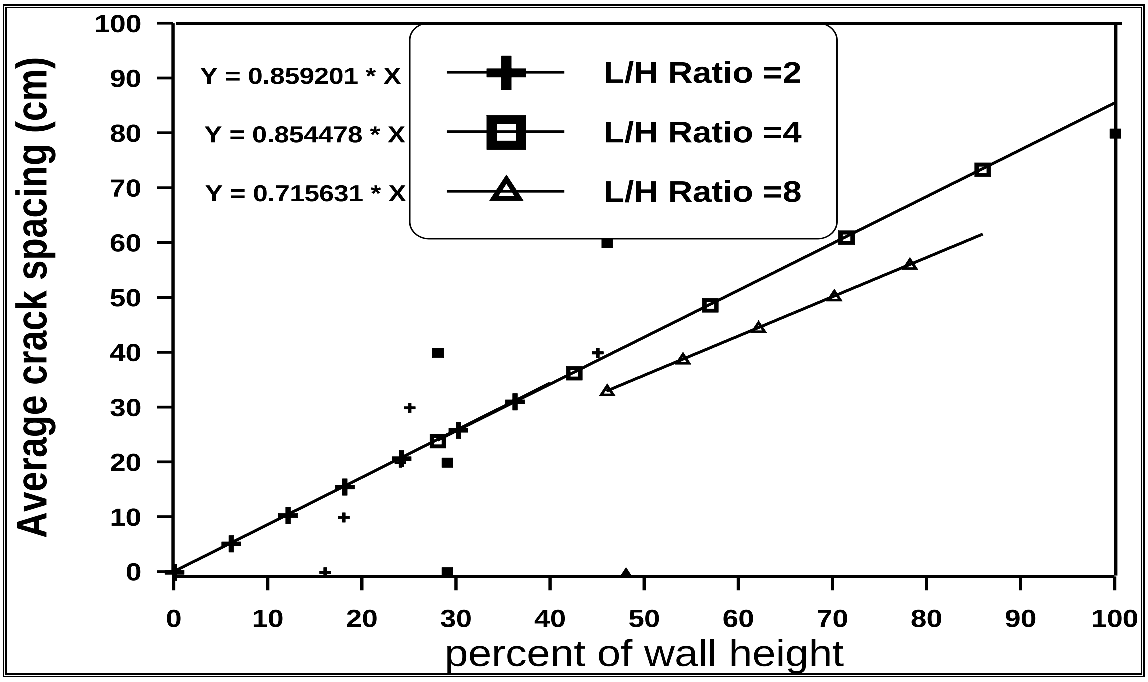


Figure (4-9) Relation between percent of wall height and the average crack spacing.

In this study, primary crack spacing ranged between (1.0-2.4)H**.** ***ACI committee 207.R-95*** found that for unreinforced concrete member, subjected to base restraint, will ultimately attain cracks through the full block height spaced in the neighborhood of (1-2)times the height of wall block.

The high roughness of base surface ensure small crack spacing subsequently small crack width and high crack number, this is evident when making comparison with that of ***Kheder, 1986*** and ***AL Mashhadi, 1989*** as shown in Table (4-4).

Through the obtained results of crack spacing, the end restraint increase the cracking number also it increase the crack spacing for walls of (L/H=4 and 8), where these results compared with the results of ***ALTamimi, 1987*** and ***Khedher, 1986*** as shown in Figures (4-11), and (4-12). This is due to the effect of base restraint where it is the maximum value in the wall centerline and decreased laterally in the walls of base restraint, therefore the cracks are crowded in the center of wall and decreased toward the ends subsequently the average crack spacing is low in spite of small crack number, whereas in the walls of end and base restraint the cracks distributed along the wall length subsequently the average crack spacing is high in spite of large crack number. The obtained results of crack spacing for walls of (L/H=2) were inversed because of the effect of end restraint increases with decreasing of (L/H) ratio as seems, so that a large number of cracks were formed per unit length in wall of end and base restraint with respect to walls of only base restraint as shown in Figure (4-10).

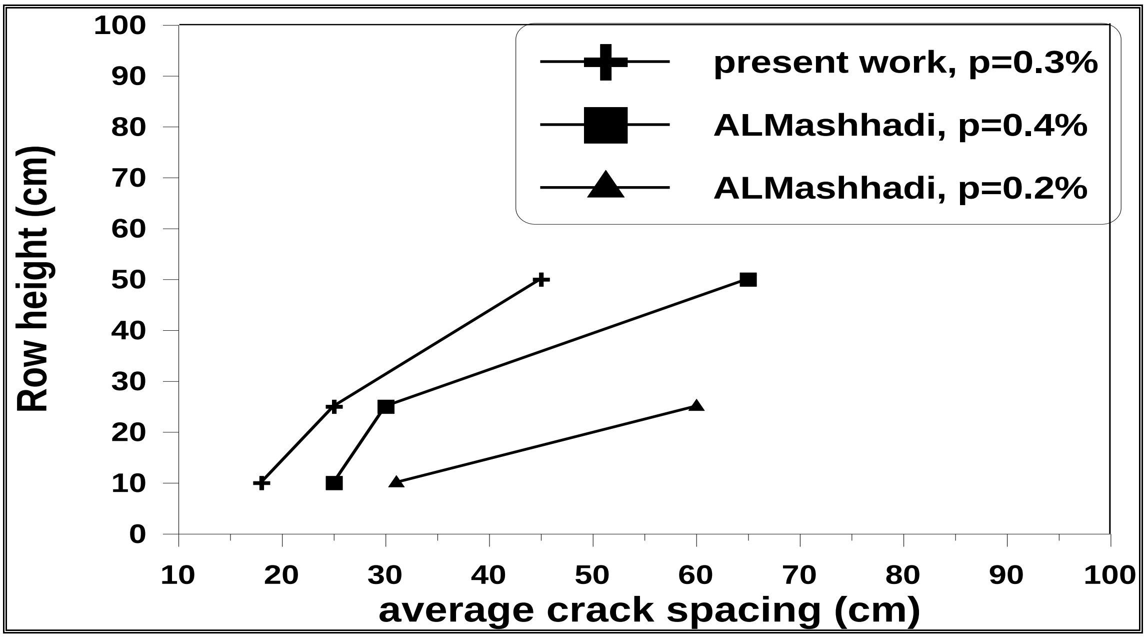


Figure (4-10)Variation of average crack spacing for end and base restrained walls (observed) and only base restrained walls(ALMashhadi,(L/H)=2)

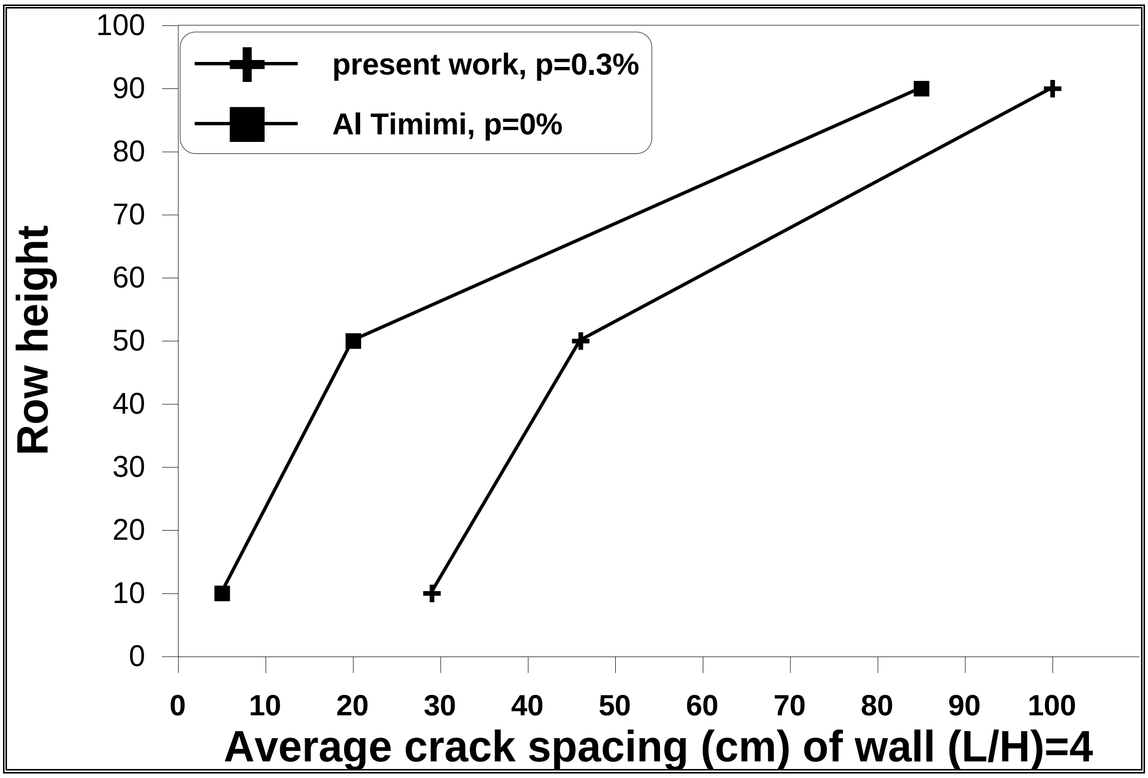


Figure (4-11)Variation of average crack spacing for end and base restrained walls (observed) and only base restrained walls(ALTamimi,(L/H)=4)

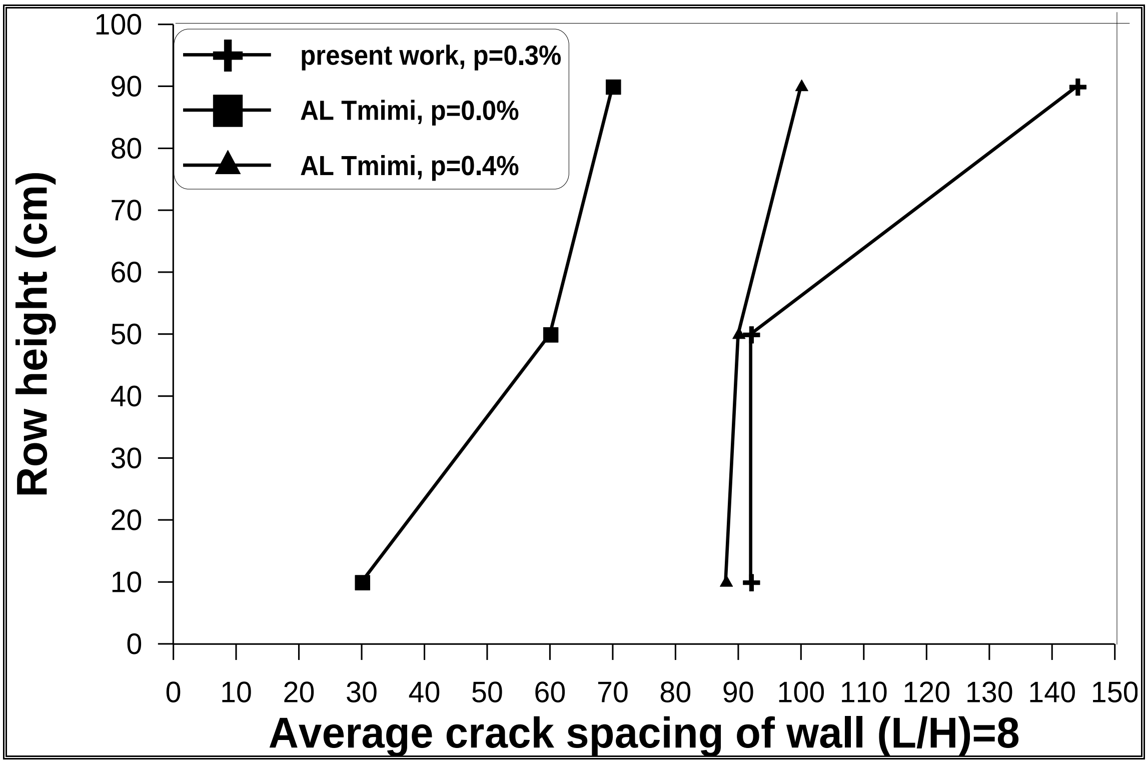


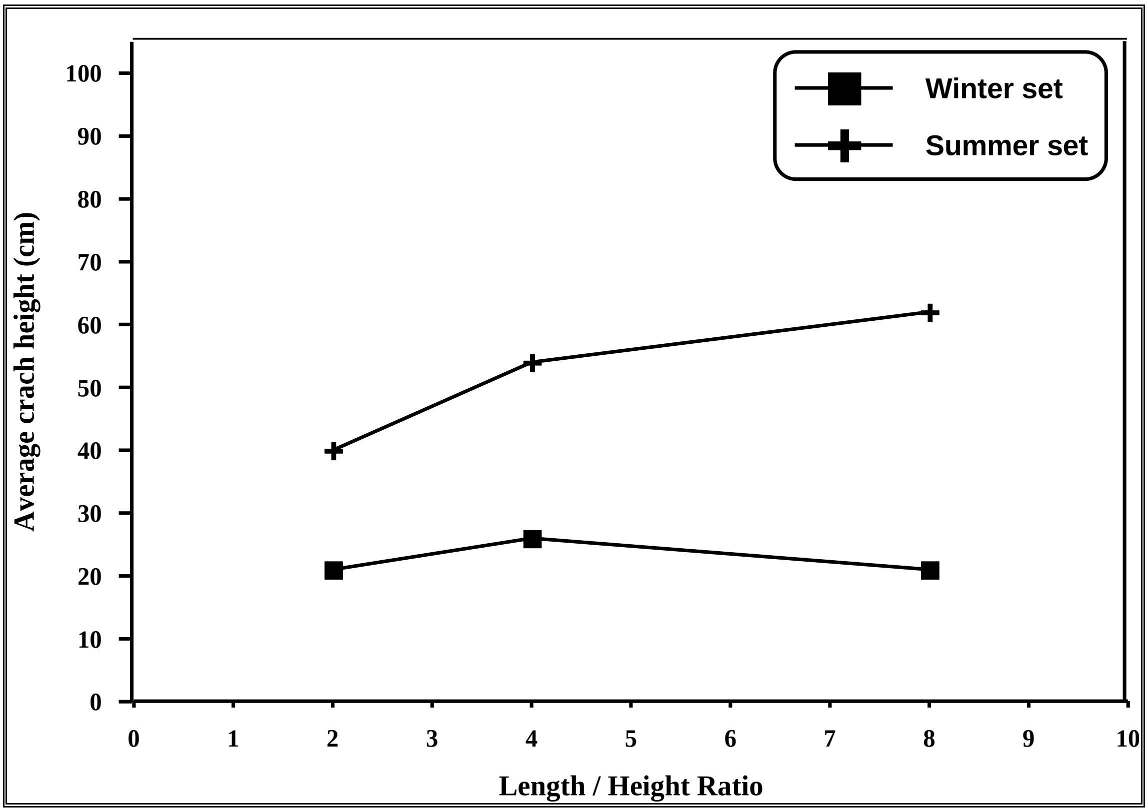
Figure (4-12)Variation of average crack spacing for end and base restrained walls (observed) and only base restrained walls(ALTamimi,(L/H)=8)

4.8- Cracking height

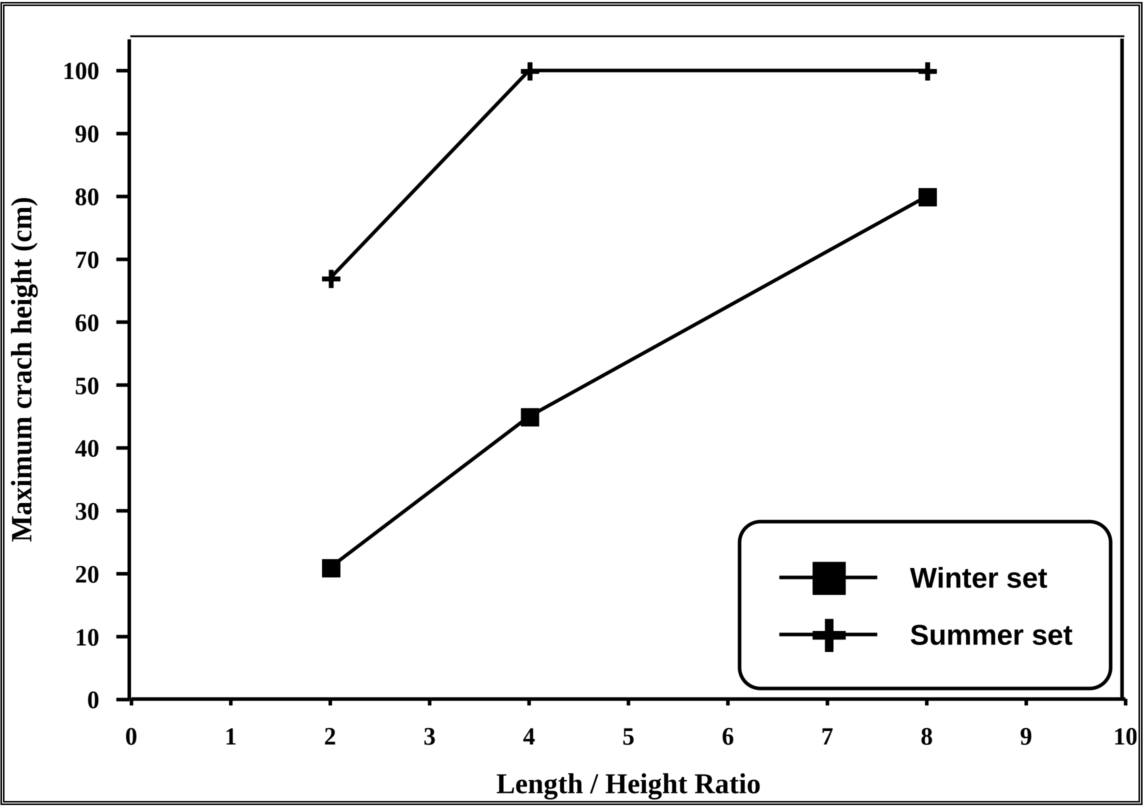
From the experimental results of all walls of (L/H) ratio greater than (2.5) induced secondary and primary cracks, while the walls of (L/H) ratio less than (2.5), (2S) induced only secondary cracks. According to the ***ACI committee 207.R-95***, the variation of restraint with height, decreased with increasing of (L/H) ratio, therefore the propagation of cracks height will increase with increasing of (L/H) ratio.

AL Mashhadi, 1989 through his research found the same results for the walls of (L/H) less than (2.5). ***ACI committee 207.R-95*** states, governing only secondary shrinkage cracks in walls with length to height ratios less than 2.66.

Figures (4-13) and (4-14) show the relationship of average cracking height and maximum cracking height with (L/H) ratio respectively.



Figure(4-13)Relationship between the average crack height and L/H ratio



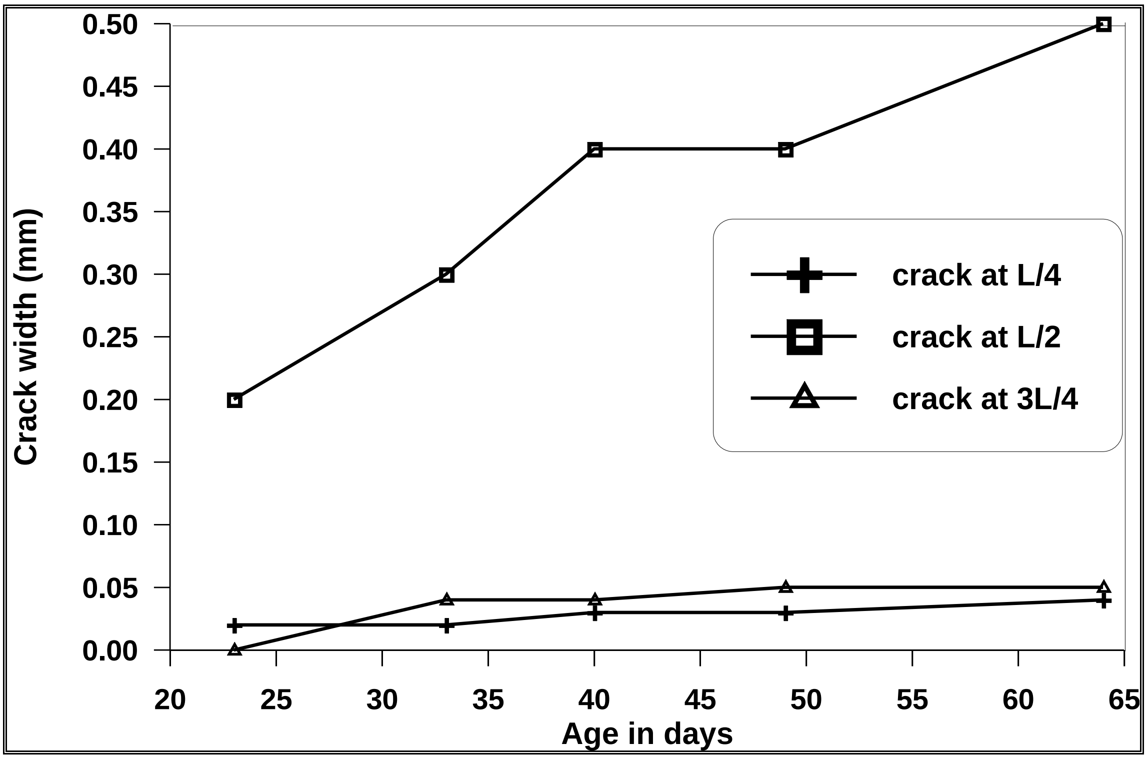
Figure(4-14)Relationship between the maximum crack height and L/H ratio

4.9 - crack width :

Initiation of cracks width with a beginning value which is about (0.02)mm in the lateral cracks and (0.2)mm in the centerline of the wall according to the degree of base restraint. These cracks widened progressively until a certain constant width in about 60 days occurred. The first crack in each walls observed at the age (30) days in Winter and (14) days in Summer. There are many factors affecting cracking age such as, environmental conditions, roughness of base surface and (L/H) ratio. The high roughness of base surface on which the walls where casted, plays an important role in increasing the number and decreasing the width of cracks. This clear through the results that obtained, where the number of crack is greater and the crack width is smaller in comparison with results that obtained by ***Kheder, 1986*** and ***AL Mashhadi, 1989***, where the contact surface between wall and the restraining base per unit length represented by wall width plus perimeter of four deformed steel bars that welded above the flange, whereas in the previous researches the contact surface per unit length represented by only wall width.

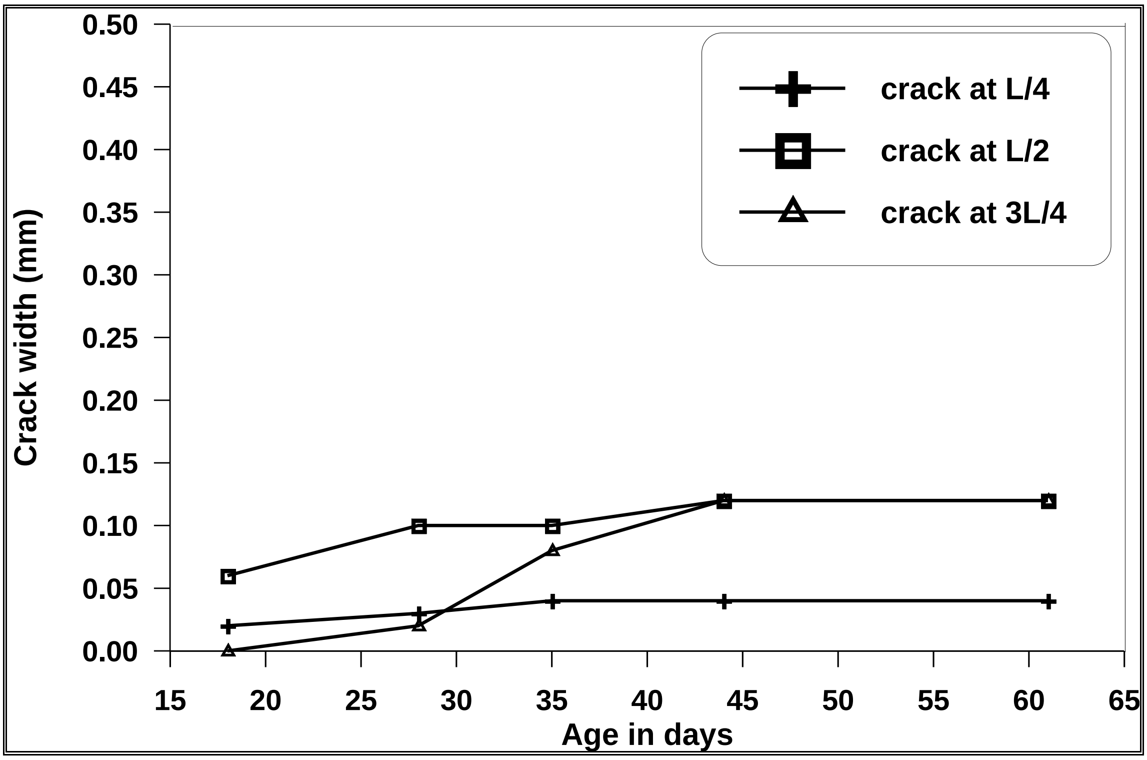
For all investigated walls, the maximum crack width in each wall was that of the earliest crack occurrence. Tables (4-2) and (4-3) summarize the final crack widths and heights of maximum crack width, measured for all cracks in each wall.

Figures (4-15) and (4-16) show the propagation of crack width with time for a selected cracks of two walls. The crack width and rate of development of each crack for the same wall depends on its position in the wall and consequently the degree of restraint, where the cracks in the centerline of wall larger and developed faster than that near the ends



Figure(4-15) Crack width propagation of wall (4S)

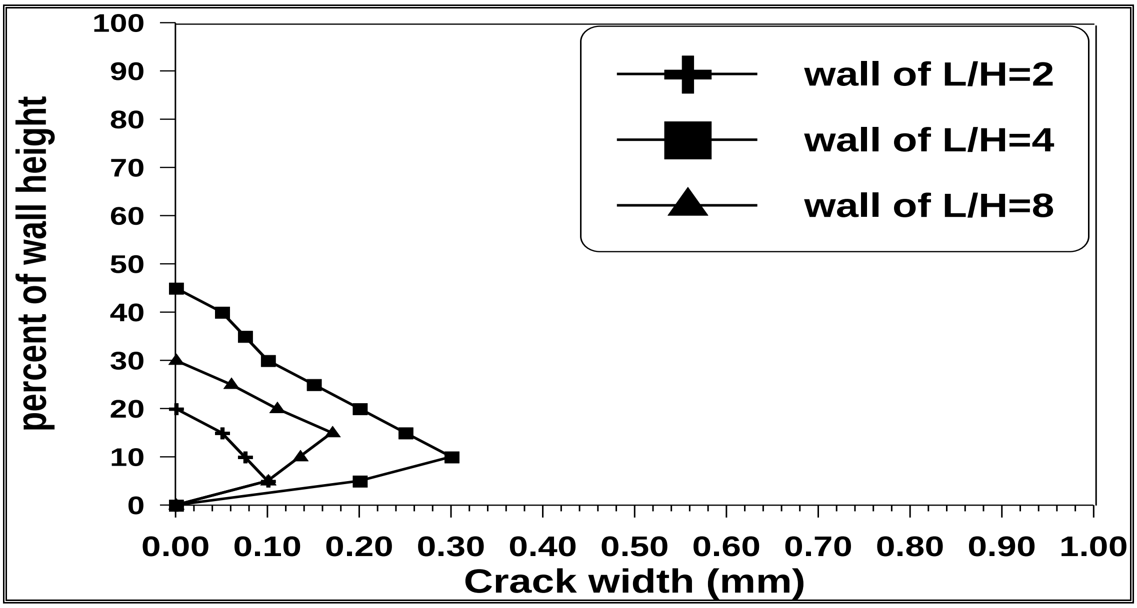
(Distance taken from the left edge)



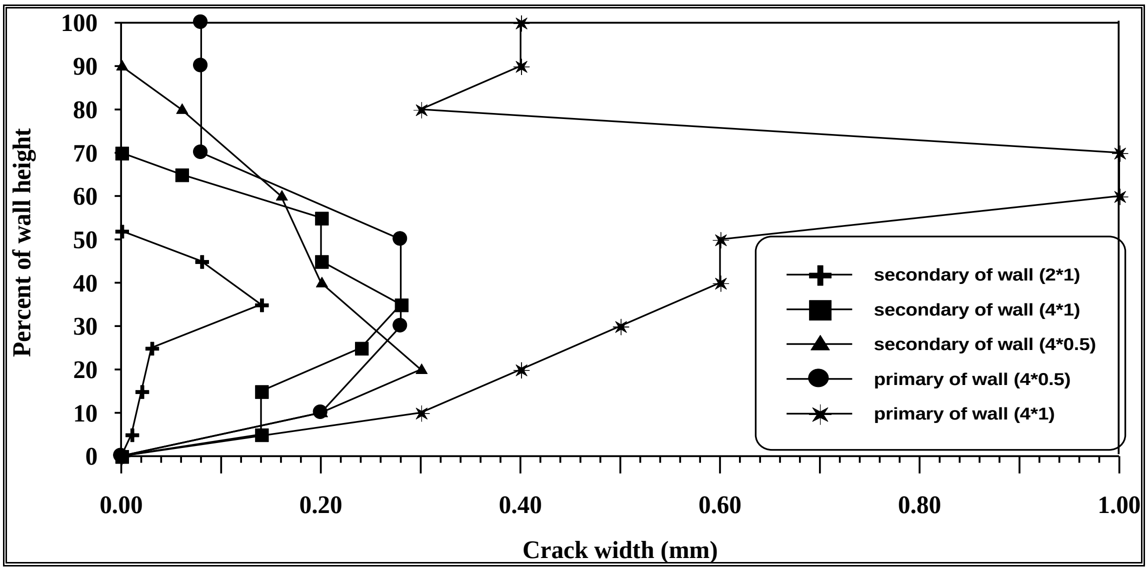
Figure(4-16) Crack width propagation of wall (2S)

(Distance taken from the left edge)

The variation of crack width through the wall height in each wall are shown in Figures (4-17) and (4-18). All cracks formed in the walls of Winter set were secondary cracks, while the walls in Summer are of (L/H) more than (2.5) induced primary cracks in addition to secondary cracks, this attributed to the hot and dry weather that lead to increase the rate of drying shrinkage that increase the induced stresses greater than tensile strength development of mortar so that crack developed to the full wall height.



Figure(4-17) Variation of crack width with crack height, winter set (secondary cracks only)



Figure(4-18) Variation of crack width with crack height, Summer set (secondary and primary cracks)

Figure (4-19) shows the level of maximum crack width ($Y_{w.max}$) with respect to crack height for three (L/H) ratios. It has been found that ($Y_{w.max}$=0.62H_c, 0.4H_c, 0.4H_c) for ((L/H)= 2, 4, 8) respectively. This indicate the height of maximum crack width proportion inversely with

(L/H) ratio to a certain limit, when (L/H=4), beyond that the relation become constant. It can be say that:

$Y_{w.max}=0.62H_c$ for (L/H=2)

$Y_{w.max}=0.4H_c$ for (L/H>2)

Al Mashhadi, 1989 found that the relation of height of maximum crack width with crack height is :

$Y_{w.max}=0.43H_c-5.5$ for (L/H=2)

The results of the height of maximum crack width in this study seem greater than those of ***Al Mashhadi***, this attributed to the effect of end restraint where the degree of restraint in the end restrained walls approaches to (100%)(R=1) that rising the position of maximum crack width.

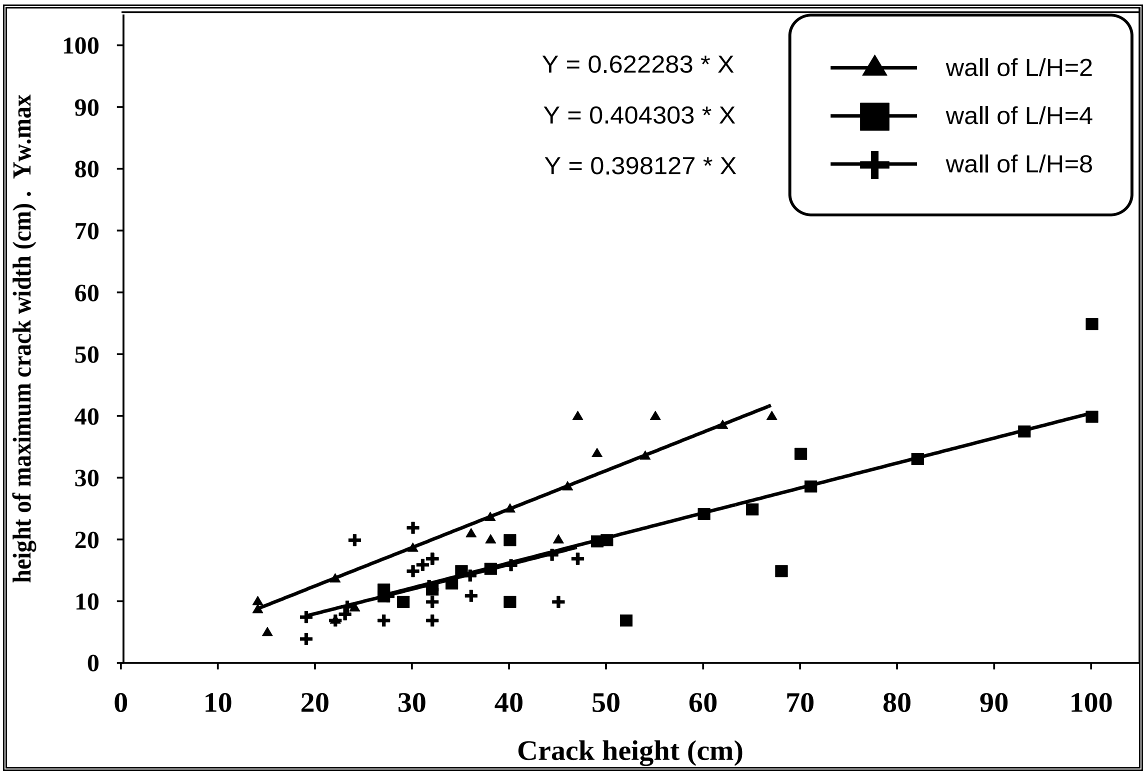
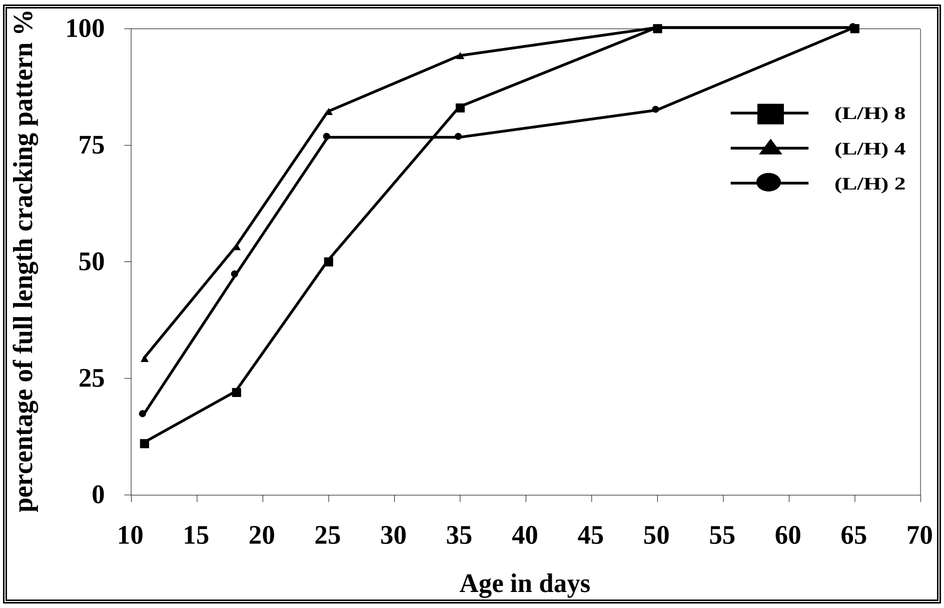


Figure (4-19) Relationship between Y $_{w.max}$. and the whole crack height

Figure(4-20) shows the cracking development of these walls, where the propagation of the cracks proportion with (L/H) of the wall.



Figure(4-20) Cracking development of three different length / height ratios(Summer set)

Through the obtained results of the average cracking width, the end restraint increase the number of cracks, so that the average cracking width will decrease, where these results are compared with the results of ***ALTamimi, 1987***, ***Khedher, 1986, AL Mashhadi, 1989,*** and ***AL Attar, 1988*** as shown in Figures (4-21), (4-22) and (4-23).

This is due to the effect of base restraint where it is the maximum value in the wall centerline and decreased laterally in the walls of base restraint, therefore the cracks are crowded in the center of wall and decreased toward the ends subsequently the average crack spacing is low in spite of small crack number, whereas in the walls of end and base restraint the cracks distributed along the wall length subsequently the average crack spacing is high in spite of large crack number.

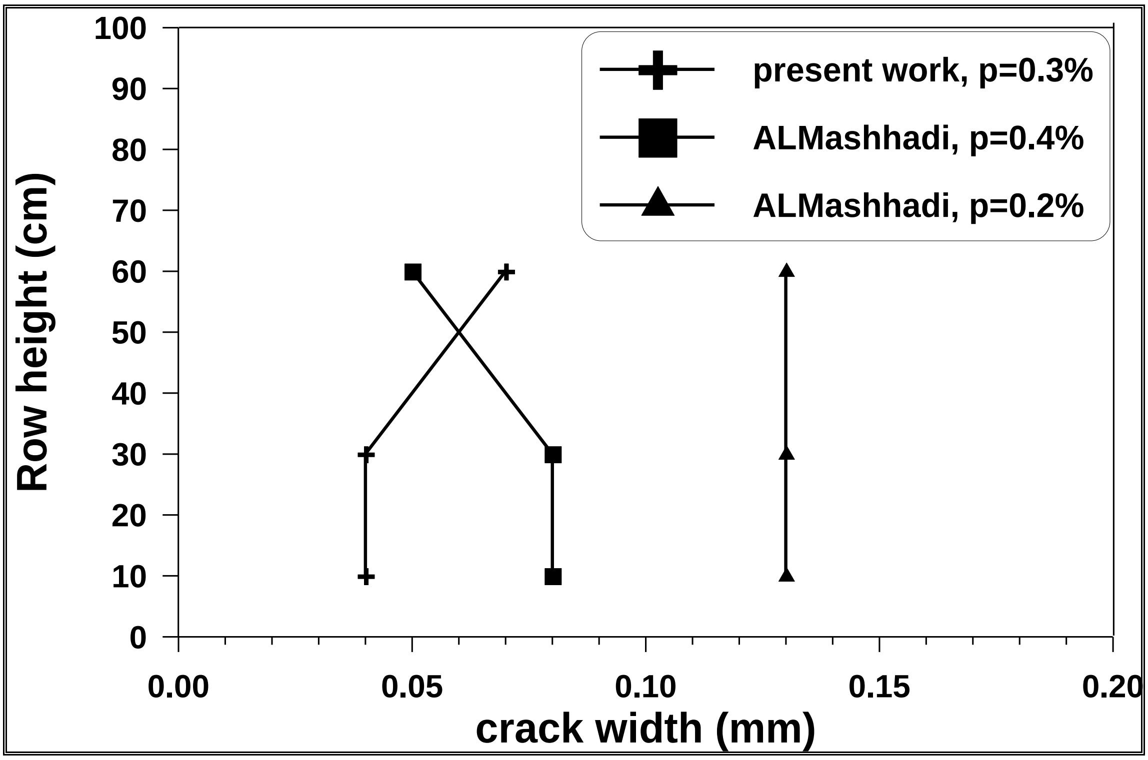


Figure (4-21)Variation of average crack width for end and base restrained walls (observed) and only base restrained walls(ALMashhadi),(L/H)=2

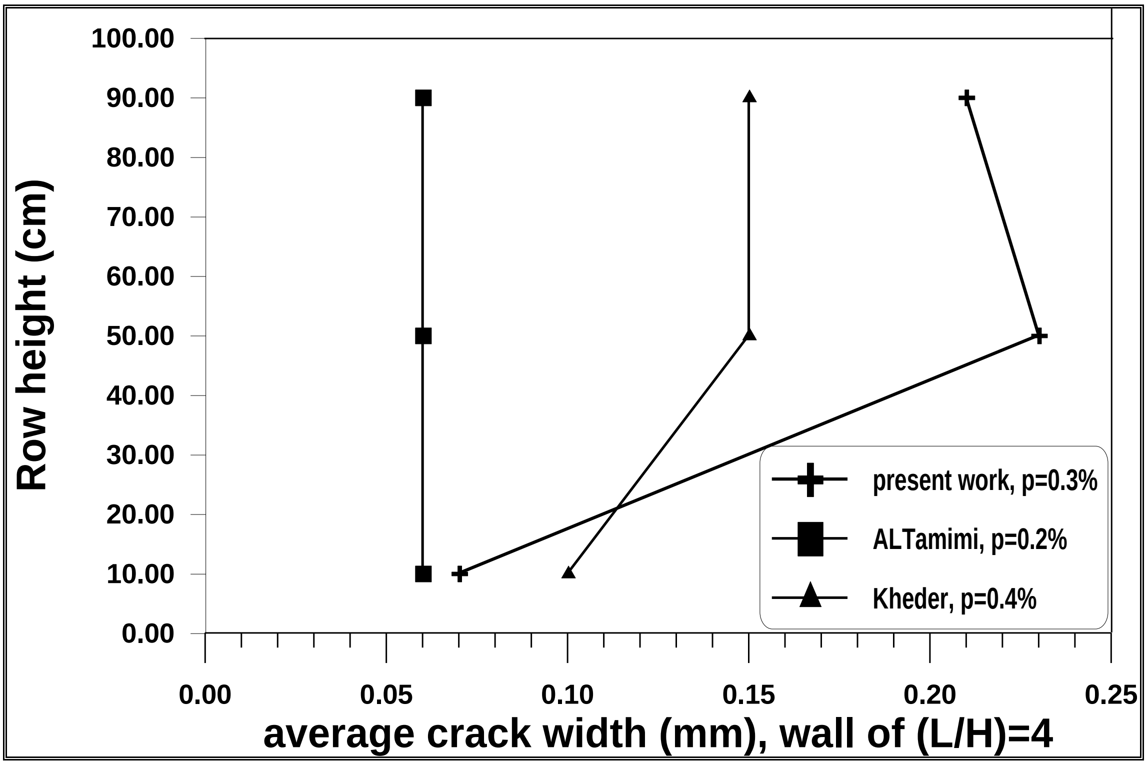


Figure (4-22)Variation of average crack width for end and base restrained walls (observed) and only base restrained walls(other researchers),(L/H)=4

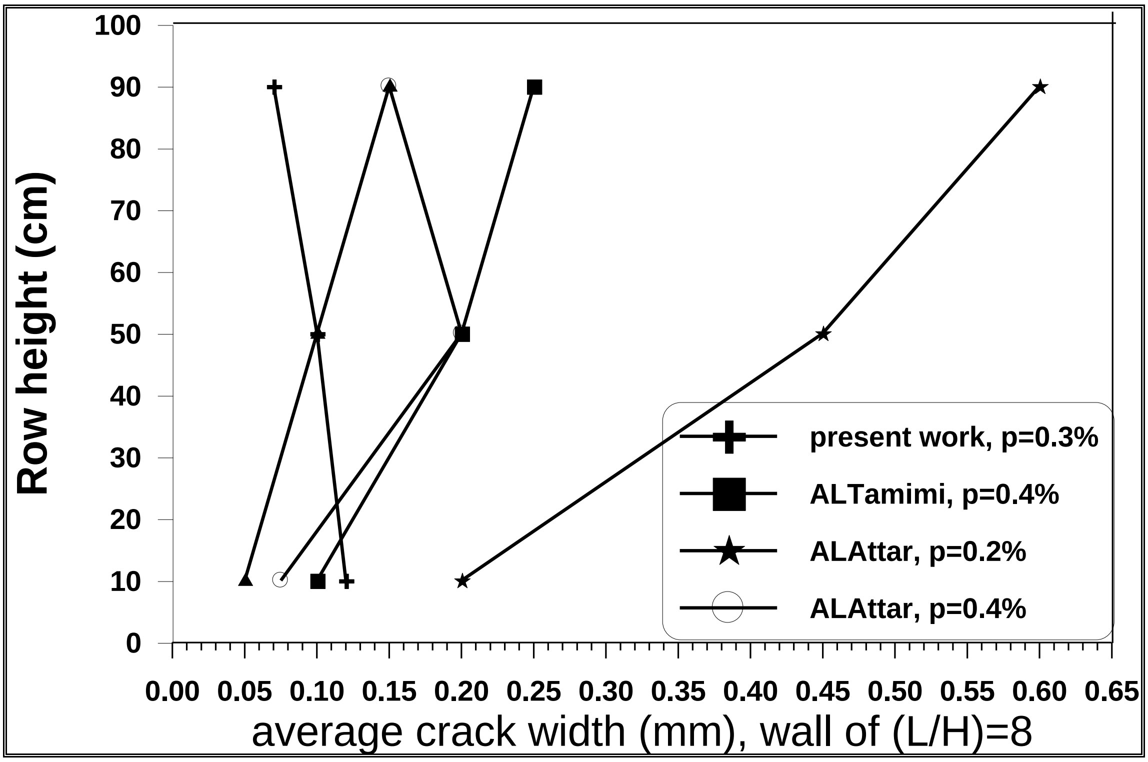


Figure (4-23)Variation of average crack width for end and base restrained walls (observed) and only base restrained walls of other researchers,(L/H)=8, except ALAttar,L/H=7.5

Kadhum M.M., 2003 found that the maximum cracks width of two end restrained concrete slabs of (ρ=0.42%) and (L/H=1) were ranged between (0.2-0.275)mm. Also ***Kubba, H.Z.A., 2007*** found that the maximum cracks width of two end restrained concrete slabs of (ρ=0.57%) and (L/H=1) were ranged between (1-1.35)mm. The crack width that obtained by the previous two researcher were greater than that observed in this study, although they were used concrete in stead of mortar, this due to there is no base restraint in their work that act to increase the crack number and decreased the cracks width.

Chapter Five

Conclusions and Recommendations

5.1- Conclusions

Based on the experimental results of base and end restrained walls and the observations presented in the preceding chapters, the following conclusions are arrived at:

1. Greater than 90% of free shrinkage strains in plain mortar beams are exhausted in (10) days after the end of curing, whereas in walls the free shrinkage strains continue approximately for two months in Summer and three months in Winter, this is due to the internal restraint (reinforcement) that distribute the stresses to make the mortar in each position of the member in the range of elastic tensile strain capacity, so that longer time be needed to disperse the stored energy.
2. Most of the cracks initiate in the lower third in the walls of ($L/H \leq 4$), but in the walls of ($L/H=8$)it has been found the cracks initiate firstly in the upper third of wall height after that the next cracks initiate above the base.
3. All walls of ($L/H=2$) induce only secondary cracks whereas all walls of ($L/H>2$) induce secondary and primary cracks.
4. In winter, all walls of different (L/H) ratios, induce only secondary cracks, this is due to cold and moist weather that reduce the drying shrinkage. Also the number of cracks in Winter were (2, 7, 9), while in summer were (12, 14, 17) for walls of ($L/H=2$, 4, 8) respectively.

5. The number of cracks in the walls of summer and winter sets is greater than that in the only base restrained walls of the same (L/H) ratio and mix proportion in the previous researches. This attributed to the effect of end restraint.
6. The height of cracks in the walls of (L/H=2, 4, 8) range between (0.2-0.6)H, (0.3-0.8)H, (0.4-0.9)H respectively, where (H) represent wall height.
7. Cracking height for all end and base restrained walls of different (L/H) ratios are similar to those of only base restrained walls of the same (L/H) ratios and mix proportion in the previous researches.
8. The effect of combined end and base restraint on average crack spacing with respect to L/H ratio is $S_{ave.}$=(0.92H, 0.96H, 0.88H) for (L/H=2, 4, 8) respectively.
9. Crack width in the walls of only secondary cracks, varies from zero at the base of wall to the maximum value at about (0.2-0.6)Hc and then decrease to zero at the tip of the cracks.
10. Cracks of the maximum width lies in the centreline of the wall, whereas cracking width of other cracks decrease laterally toward the ends.
11. The maximum crack width of primary cracks lies in the range (0.3-0.7) of the walls height.
12. The relationship between height of maximum crack width with respect to cracking height is $(Y_{w.\max.} = 0.4Hc,\ \ 0.4Hc,\ \ 0.62Hc)$ for walls with (L/H=8, 4, 2) respectively, where Hc represent height of crack.
13. Cracks width and their variation along the crack length in each wall proportion to their cracks height. Also sizes of cracks associated with their occurrence time where the cracks that induced firstly are greater than those induced later.

14. For economical purposes, maximum crack width, for walls of any (L/H) ratio, can be controlled by concentrating the horizontal steel reinforcement in the zone of (0.12-0.36) or in general in the lower third of wall height depending on conclusions number 6 and 12 above.

Recommendations:

1. Finding a test method to assess the early age shrinkage occurring immediately after mixing the concrete or mortar for the first 24-hours.
2. Casting mortar walls of one end and base restraint and of the same (L/H) ratios to investigate the behavior of cracking.
3. Studying the effect of expansive additives to reduce the induced crack's number and size.
4. There is no need for casting specimens of drying shrinkage measurements in winter because its effect is not critical, where the extreme cases appear in summer.

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Notations

Symbol	Description
a	No bond length for the reinforcement near the crack
A_c	Cross sectional area of concrete
C	Creep strain before cracking.
C_T	Coefficient of thermal expansion
d	Bar diameter.
Ec	Modulus of elasticity of concrete
ε_{sh}	Free shrinkage strain
ε_{shn}	Net shrinkage strain.
ε_{th}	Thermal strain.
ε_{ult}	Elastic tensile strain capacity of concrete.
E_{sc}	compressive strain of steel
F_b	Average bond strength between concrete and steel
.f_t	Tensile strength of concrete
.fsy	yielding tensile strength of steel
H	Height of wall
H_c	Crack height
K	Constant
Lr	Loss of restraint
.n	Modular ratio
R_a	Degree of restraint after cracking
R_b	Degree of restraint before cracking
S_{max}	Maximum crack spacing
S_{min}	Minimum crack spacing
S_y	Crack spacing at level y above the base
Sl_b	Slippage before cracking
Sl_a	Slippage after cracking
T_E	design temperature change
W	Crack width
W_{max}	Maximum crack width
ρ	Steel ratio
ρ_{crt}	Critical steel reinforcement

Plate (4-1) Final cracking pattern of first quarter of wall (8S)

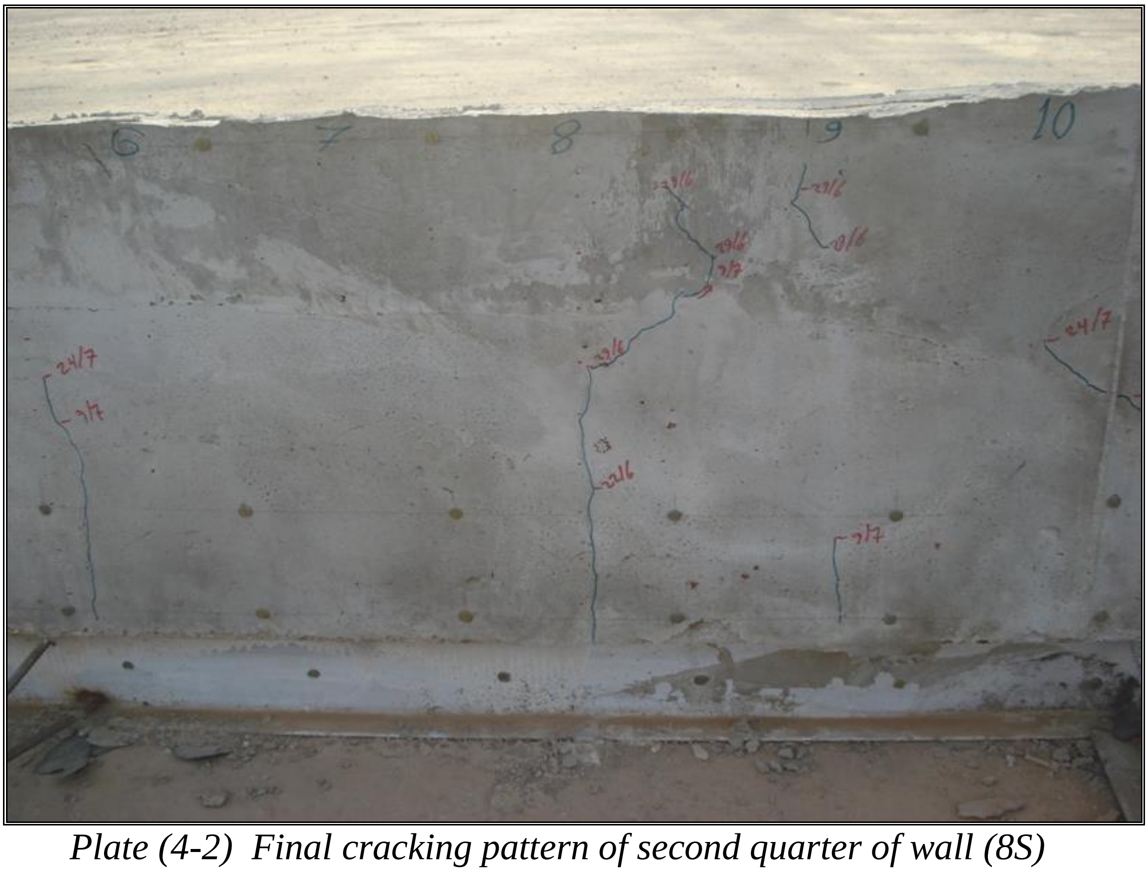

Plate (4-2) Final cracking pattern of second quarter of wall (8S)

Plate (4-3) Final cracking pattern of third quarter of wall (8S)

Plate (4-4) Final cracking pattern of fourth quarter of wall (8S)

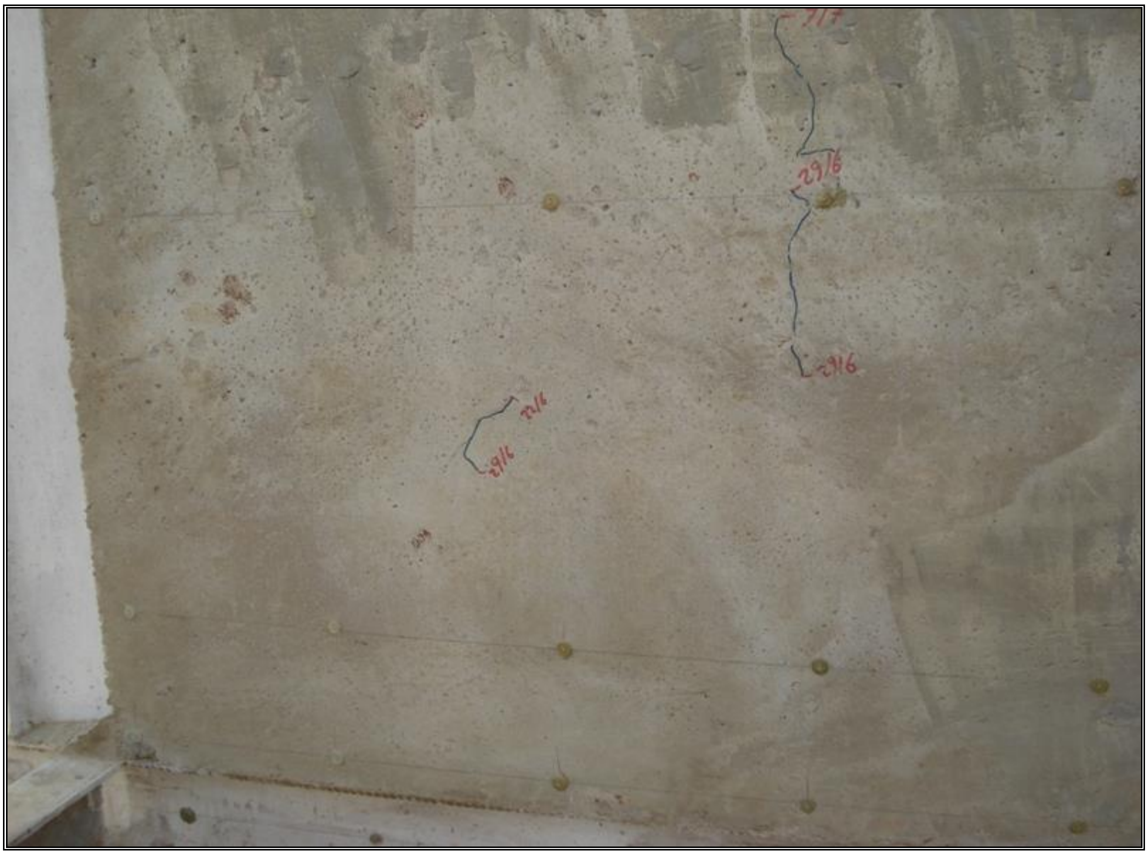

Plate (4-5) Final cracking pattern of first quarter of wall (4S)

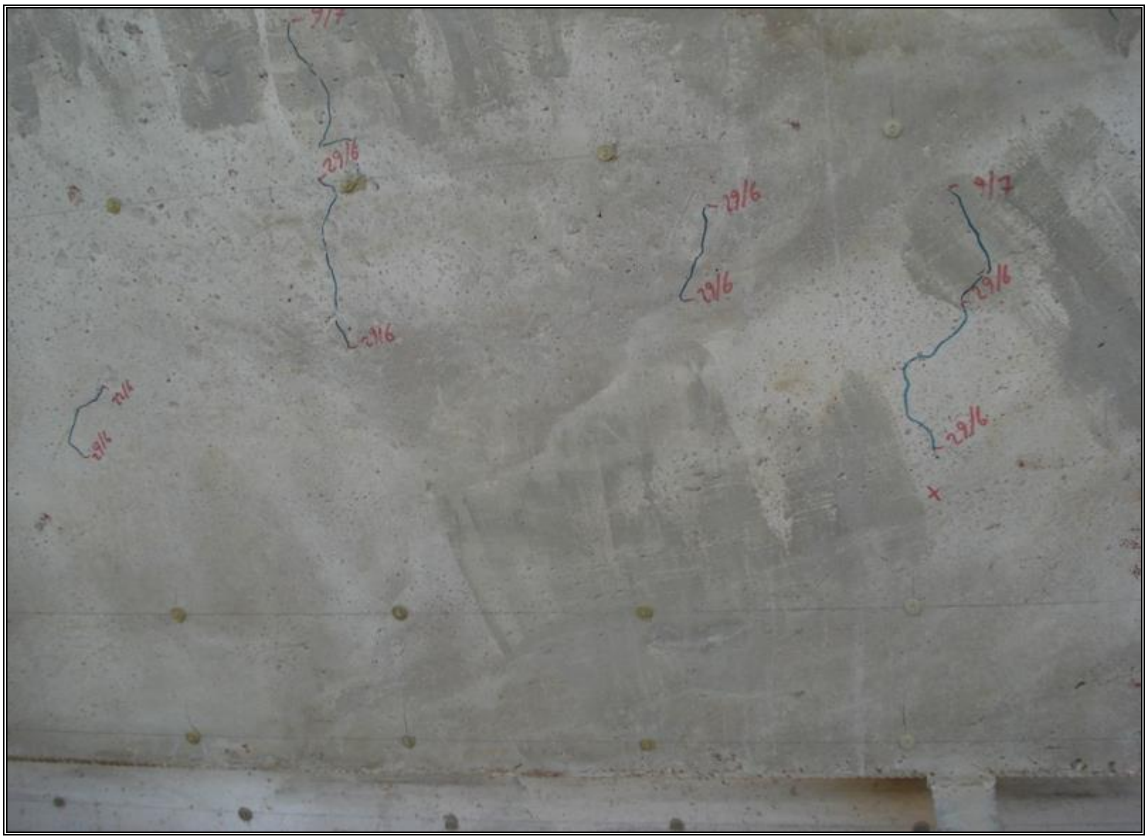

Plate (4-6) Final cracking pattern of second quarter of wall (4S)

Plate (4-7) Final cracking pattern of third quarter of wall (4S)

Plate (4-8) Final cracking pattern of fourth quarter of wall (4S)

Plate (4-9) Final cracking pattern of first third of wall (2S)

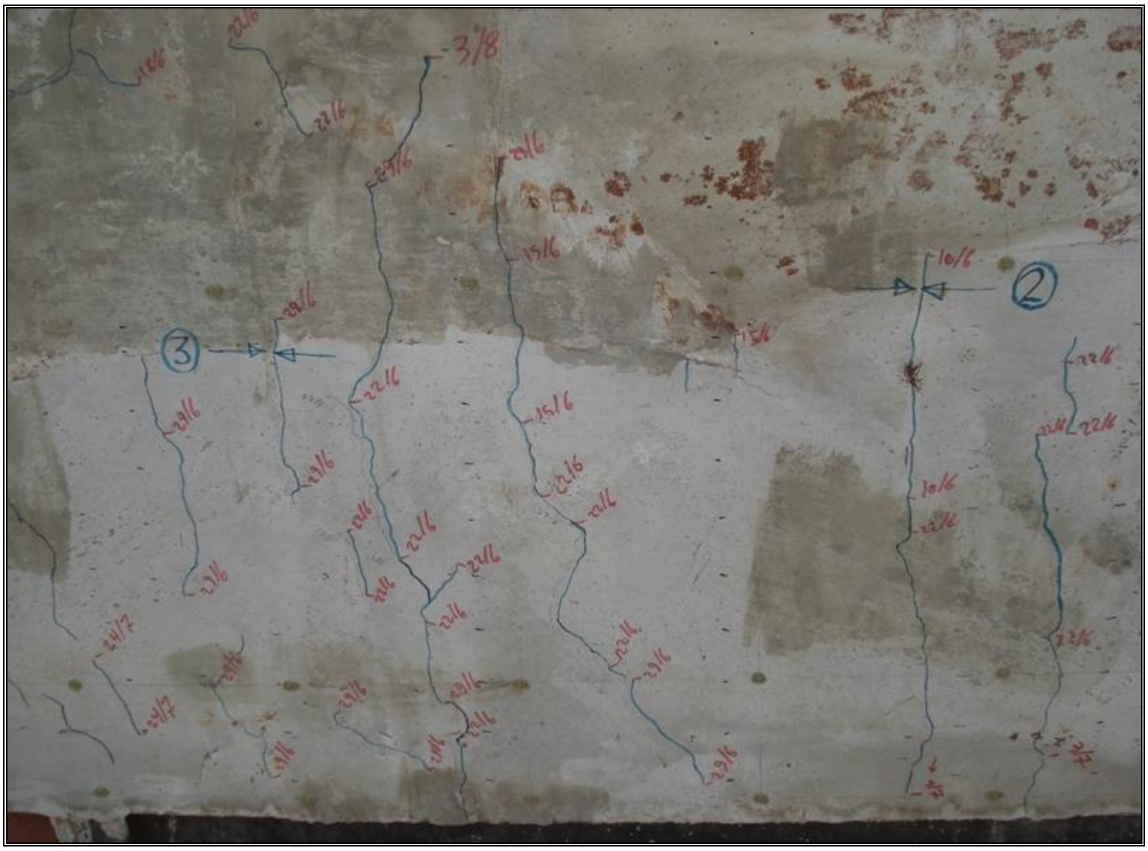

Plate (4-10) Final cracking pattern of second third of wall (2S)

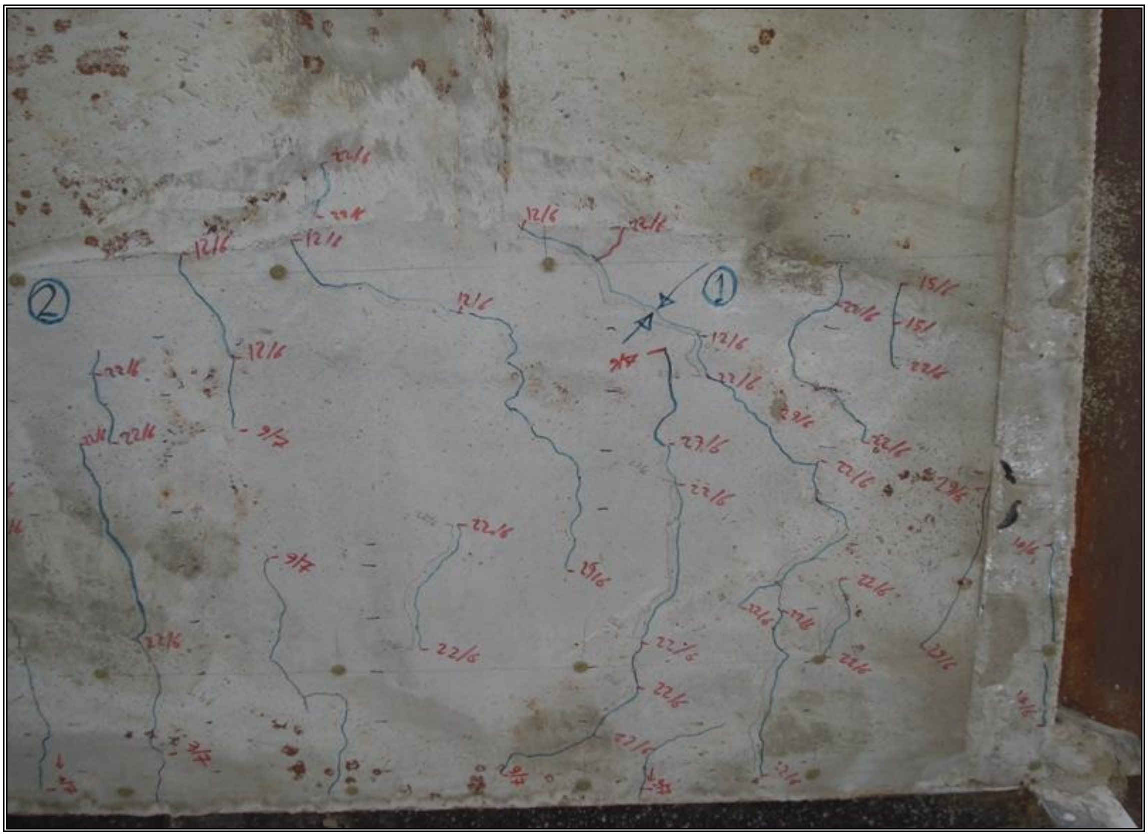

Plate (4-11) Final cracking pattern of last third of wall (2S)

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الخلاصـــــة

أختص هذا البحث لدراسة سلوك التشققات الناتجة من انكماش الخرسانة في الجدران المقيدة من الجوانب بالاضافة الى القاعدة في عدة حالات بالنسبة لشكل الجدار (الطول / الارتفاع). تم اختيار مونة السمنت (سمنت + رمل) بدلا" من الخرسانة (سمنت + رمل + حصى) في انشاء هذه الجدران وذلك لدراسة ظاهرة التشققات التي تظهر في الجدران على ألأمد البعيد خلال فترة قصيرة وهي مدة البحث.

خلال هذه الدراسة تم استخدام نماذج مصغرة من الجدران المسلحة. تم دراسة العديد من الخواص لظاهرة الانكماش في الجدران, منها قياس ألاجهادات التي تحدث في الجدران بشكل أفقي بسبب ألانكماش وعلى عدة ارتفاعات فوق مستوى القاعدة من الجدار كذلك قياس المسافات البينية للتشققات التي تحدث وعرض هذه التشققات وأطوالها, كذلك تم دراسة تأثير ألظروف الجوية (درجات الحرارة والرطوبة) على مدى تطور هذه التشققات وكذلك تأثير شكل الجدار (نسبة الطول / العرض) على الصيغة النهائية للتشققات. بالنسبة لمدة تعرض الجدران للظروف الخارجية كانت ثلاثة أشهر في الشتاء و شهرين في الصيف حيث كانت هذه الفترات كافية لاستنفاذ معظم ألانكماش.

بالنسبة للنتائج التي تم الحصول عليها وجد ان عدد التشققات التي تحدث خلال فترة الصيف أكثر من تلك التي تحدث خلال فترة الشتاء بالنسبة لنفس الجدار كذلك ان عدد التشققات التي تظهر في أي مجموعة (شتوية أو صيفية)تتناسب طرديا" مع نسبة (الطول / الارتفاع). عدد التشققات التي تم الحصول عليها لكل جدار خلال هذا البحث كانت أكثر من تلك التي تم الحصول عليها في البحوث السابقة لنفس الجدران (بالنسبة للشكل والخواص ألأخرى) حيث كان تأثير التقييد الجانبي واضح على الصيغة النهائية للتشققات. كل التشققات التي حدثت في الجدران ذات نسبة (الطول / العرض=2) كانت ثانوية (لم تصل الى ارتفاع الجدار الكامل) بينما كانت كل الجدران ذات نسبة (الطول / ألارتفاع) الأكثر من(2) كانت أولية وثانوية (التشققات ألأولية هي التي تصل الى حافة الجدار العليا). معظم التشققات كانت تمتد بشكل عمودي على القاعدة بالنسبة للتشققات الثانوية عدا نسبة قليلة كانت تميل للخارج في الجدار(2S) والسبب يعزى أيضا" للتقييد الجانبي. المسافات البينية للتشققات كانت صغيرة نسبيا" بسبب حالة التقييد الجانبي, كذلك تزداد هذه المسافات كلما زاد الارتفاع عن القاعدة. بالنسبة لارتفاع التشققات وعددها كان يزداد في ألأجواء الحارة والجافة كذلك يزداد بزيادة نسبة (الطول / العرض) للجدار. ارتفاع التشققات في الجدران ذات (الطول/ألارتفاع=2,4,8) كانت تتراح بين (0.2-0.6), (0.3-0.8), (0.4-0.9) من ارتفاع الجدار على التوالي. علاقة المسافات البينية بارتفاع الجدار كانت كألتالي:

$S_{ave.}=(0.86H, 0.85H, 0.72H)$ حيث $(L/H=2, 4, 8)$ على التوالي.

اما بالنسبة لعرض التشققات حيث كان عرض التشققات المتولدة أولا" أكثر من عرض التشققات المتولدة لاحقا" كذلك ان ارتفاع أكثر عرض للشق الواحد كان يتناسب عكسيا" مع نسبة (الطول / العرض) للجدار حيث يقل كلما زادت هذه النسبة الى حد معين ((طول الجدار / العرض)=4) بعدها يكون ارتفاع أعرض مكان للشق ثابت مع زيادة نسبة (الطول / العرض) والعلاقات كانت كالتالي :

$(Y_{w.max}=0.62H_c)$ بالنسبة للجدران $(L/H=2)$, $(Y_{w.max}=0.4H_c)$ بالنسبة للجدران $(L/H\geq4)$

الهدف الرئيسي من هذا البحث هو معرفة الصيغة النهائية للتشققات الناتجة بسبب ألانكماش في الجدران المقيدة من الجوانب بألاضافة الى القاعدة وذات ألأبعاد المختلفة وذلك لمعرفة أنسب مكان للتسليح ألافقي لتحديد عرض التشققات بأقل كمية.