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STRUCTURAL BEHAVIOUR OF HIGH PERFORMANCE REINFORCED CONCRETE BEAMS INTERNALLY CURED WITH LOCAL WASTE MATERIALS

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By:

Ahmed Abbas Shareef
B.Sc. Civil Engineering- University of Baghdad
(2009)

Supervised by:

Asst. Prof
Dr. Laith Shakir Rasheed

Asst. Prof
Dr. Ali Talib Jasim

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Certification

We certify that this thesis titled " ***STRUCTURAL BEHAVIOUR OF HIGH PERFORMANCE REINFORCED CONCRETE BEAMS INTERNALLY CURED WITH LOCAL WASTE MATERIALS*** " which is being submitted by "***Ahmed Abbas Shareef*** " was prepared under our supervision at University of Kerbala in partial fulfillment of the requirements for the degree of Master of Science in Infrastructure Engineering.

Signature: L. Sh. Rash

Name: Asst. Prof. Dr. Laith Shakir Rasheed

Date: 23/ 4/ 2018

Signature:

Name: Asst. Prof. Dr. Ali Talib Jasim

Date: / / 2018

Approval of the Civil Engineering department:

Signature: Waqed H. H.

Name: Asst. Prof. Dr. Waqed H. Hassan Al-Muhammed

(Head of Civil Engineering Department)

Date: 29/ 4/ 2018

Examination Committee Certification

We certify, as an examining committee, that we have read the thesis entitled ***"STRUCTURAL BEHAVIOUR OF HIGH PERFORMANCE REINFORCED CONCRETE BEAMS INTERNALLY CURED WITH LOCAL WASTE MATERIALS"****, and examined the student* ***"Ahmed Abbas Shareef"****, in its contents and in our opinion it meets the standards of the thesis for the degree of Master of Science in Civil Engineering (INFRASTRUCTURE).*

....	....
Dr. Ali Talib Jasim	***Dr. Laith Sh. Rasheed***
Asst. Professor	**Asst. Professor**
(Member and Supervisor)	**(Member and Supervisor)**
Date 27/ 4/ 2018	**Date 23 / 4 / 2018**
....	....
Dr. Jawad Talib Abodi	***Zainab Hasan Abduabbas***
Asst. Professor	**Asst. Professor**
(Member)	**(Member)**
Date 23/04/ 2018	**Date 24 / 4 / 2018**

....

Dr. Waleed A. Waryosh
Asst. Professor
(Chairman)
Date 26 / 4 / 2018

Approved by Civil Engineering department:

Waqed H. H
....

Dr. Waqed H. Hassan Al-Muhammed
Asst. Professor
(Head of civil engineering department)
Date 29/ 4/ 2018

Approved by College of Engineering- University of Kerbala:

....

Dr. Basim Khlail Nile
Asst. Professor
(Dean of College of Engineering)
Date 29 / 4 /2018

University Of Kerbala
Dean Office
College Of Engineering

بسم الله الرحمن الرحيم

(ولسوف يعطيك ربك فترضى)

صدق الله العلي العظيم

سورة الضحى– آية ٥

To the three stars of my life :

My dear kind mother ...

My beloved wife ...

And

My sweet daughter ...

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Abstact

The bridge girders are one of the important structural parts in the infrastructure system, which require durability to ensure long service life. Using internal curing materials proved to be useful at enhancing the properties of concrete. This work aimed to examine the effect of internal curing using locally available waste materials as a partial replacement of the fine aggregates on the structural behavior of high performance concrete (HPC) girders. Therefore, two internal curing materials were used in this thesis: crushed brick and limestone, in two percentage of replacement (5% and 10%) for each material. Studying the structural behavior of fifteen reinforced concrete beams (1700*150*100) mm that represents the three testing ages (28,90 and 150) days for each material and replacement percentage as well as the mechanical properties of concrete. Utilizing internal curing showed an increase in ultimate loading capacity for the beams, with 11.5% improvement at 28 days for the 5% brick mix and 23.5% for the mix of 10% limestone at 90 days and 12.6% for the 10 lime stone at 150 days of test. Also, improved the compressive strength by 13.6% for the mix of 10% limestone. Furthermore, improvement in splitting tensile strength of the internally cured mixes was noticed up to 14.8% for the internally cured mix of 5% limestone. Toughness of the beams increased, where the beam of 10% limestone at 90 days had the higher toughness of (0.351) MPa which was higher than the reference mix. Despite that, internal curing materials had negative impact on the ductility of the mixes, since the reference beams showed ductility indices (3.3 to 3.6), but all the internally cured beams had ductility index lower than (3). Using internally cured HPC showed good improvement in both structural behavior and mechanical properties of concrete and the late ages showed high ultimate loads capacity which means internal curing enhanced the durability of concrete structure elements.

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List of abbreviations

Symbols	Description
HPC	High performance concrete
I C	Internal curing
D I	Ductility Index
R1	Reference mix at 28 days
R2	Reference mix at 90 days
R3	Reference mix at 150 days
BA1	5% Brick mix at 28 days

BA2	5% Brick mix at 90 days
BA3	5% Brick mix at 150days
BB1	10% Brick mix at 28 days
BB2	10% Brick mix at 90 days
BB3	10% Brick mix at 150 days
GA1	5% Limestone mix at 28 days
GA2	5% Limestone mix at 90 days
GA3	5%Limestone mix at 150 days
GB1	5% Limestone mix at 90 days
GB2	10% Limestone mix at 90 days
GB3	10% Limestone mix at 150 days
LVDT	Linear Variable Differential Transformer
NWA	Normal Weight Aggregate
W/B	Water to Binder ratio

List of Symbols

Symbols	Description
P. cr	Load of first crack
Δy	Deflection of yeilding load
P. ult	Ultimate load of the beam
Δult	Deflection of ultimate load
P. yld	Yielding load of beam
Δcr	Deflection of first crack load

Introduction

1.1. General

Concrete can be defined according to ACI Committee [1] as: "a composite material that consists essentially of a binding medium within which are embedded particles or fragments of aggregate, usually a combination of fine aggregate and coarse aggregate". The early origin of concrete is back to early ages near Greco-Roman era. However, the cement known these days was first introduced as a prototype in Britain in 1824 by J. Aspdin [2].

Since then the concrete played a major role in human civilization as being the pillar of the construction industry fo it is the most suitable material with less efforts and less price when compared with other building systems. Therefore, numerous studies had been conducted on the behavior of concrete and various types were introduced to comply with the structural needs such as: high strength concrete, high performance concrete, light weight concrete...etc.

High performance concrete among these types is distinguished by its properties that not necessarily of high strength only but besides this it have low permeability and high durability [3]. A major key in producing high performance concrete HPC is minimizing the amount of water in the mix and that would cause further problems in terms of improper hydration and early age cracking. So, to overcome these problems a moderately modern approach had been used and examined to improve HPC , it is called internal curing which involve implementing the inner moisture of light weight aggregate (LWA) or other materials in the hydration process. This method showed improvements on concrete properties when it was used in many infrastructure projects including concrete bridges parts like girders and deck slabs [4].

1.2. High Performance Concrete

As mentioned before, one of the important types of concrete nowadays is High Performance Concrete (HPC). This type of concrete has high strength and durability properties, these properties cannot achieved by normal concrete. A combination of normal and special materials are implemented to produce this type of concrete with its special characteristics. Some of these characteristics are: high strength, high modulus of elasticity, high early strength, high abrasion resistance, high durability and long life under severe environment, low permeability, volume stability, resistance to chemical attack, high resistance to frost, high toughness and impact resistance[5].

High performance concrete is desired in the infrastructure projects, which mainly consists of concrete elements like: bridges and roads with concrete pavements. Therefore, these elements should have long service life and durability which can be achieved by using low permeability[6].

Concrete can be considered of high performance if it has these three properties: high strength, high workability and high durability. HPC in general must be of low porosity and contains discontinuous capillary pore structure of the cement paste, this achieved by implementing low water/binder ratio, superpalstisizer and pozzolanic materials. Many modern concrete mixes showed some advantageous properties like high workability, high strength and low permeability. Despite that, HPC showed more sensitivity to early age cracking than traditional concrete[7]. These problems of shrinkage and cracking are mostly regarded to the conventional curing methods like "spraying", especially that low water/binder is one of the elements in the production of HPC.

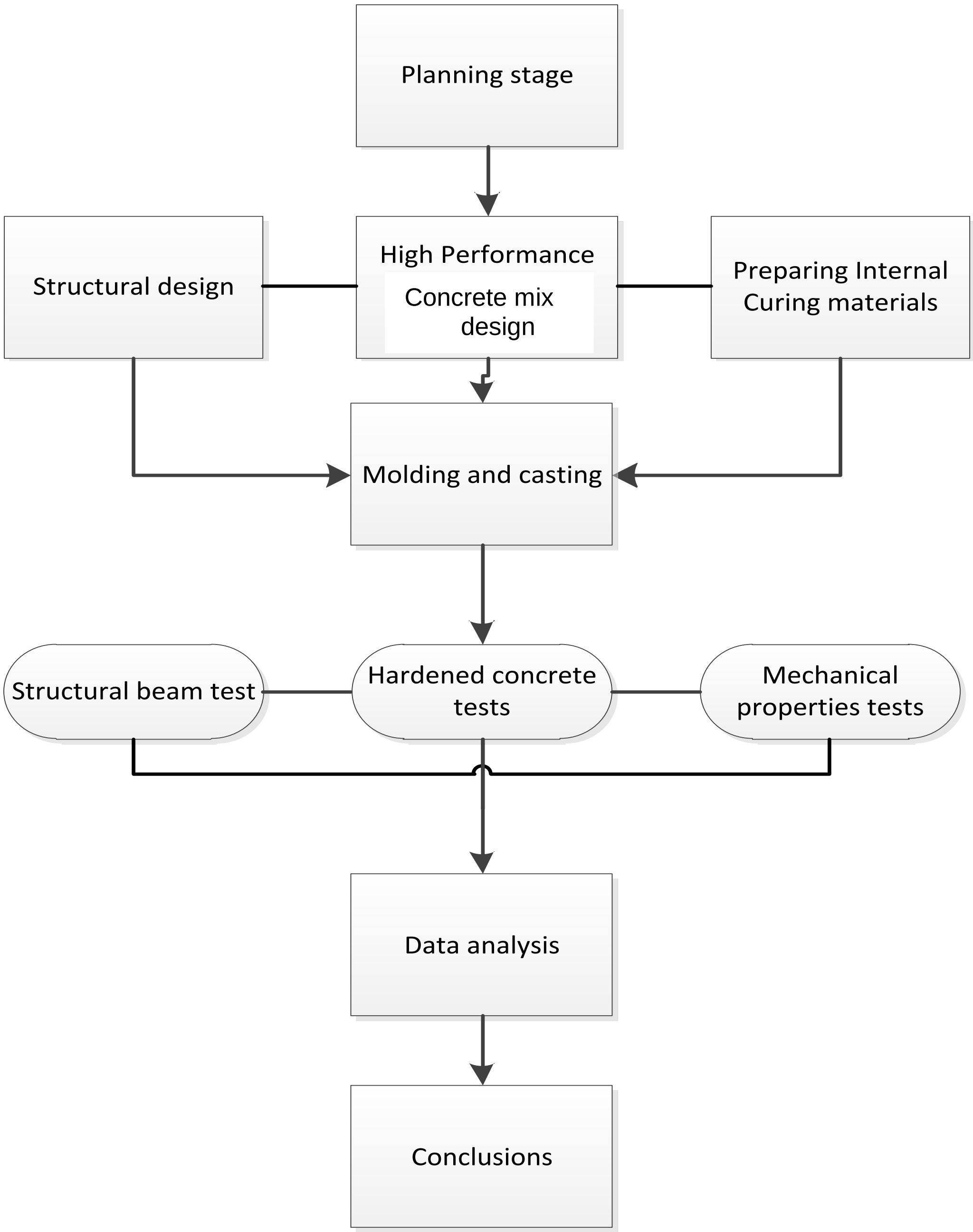
1.3. Internal Curing

Internal curing is: the process by which the hydration of cement continues because of the availability of internal water that is not part of the mixing water. The difference between internal curing and the ordinary curing method is that moisture is applied to the surface of the concrete, while internal curing provides moisture from the internal concrete raw materials and not part of w/b ratio[8].

The external moisture penetration is limited and does not reach to all parts inside the concrete member. Therefore, internal curing is considered efficient to ensure the reach of moisture. Many materials are used to provide the internal moisture, such as light weight aggregate "LWA", and super absorbent polymers (SAP),…etc[9].

1.4. Objective of Research

The main objective of this thesis is to study the effect of internal curing on the structural behavior of HPC beams and mechanical concrete properties. The process of internal curing implemented local waste materials (crushed brick and limestone “Gubbra”) with two percentages (5% and 10%) as partial replacement of fine aggregate having the same grading zones. The study also examines the effect of age in enhancing the properties of the concrete girders by testing the specimens in three ages of (28, 90 and 150) days. The details and strategy of this thesis is illustrated in Figure (1-1).



Figure(1 -1): Thesis Strategy Throughout the Work

1.5. Research Layout

The main variables chosen for this investigation were partial replacement material types, the replacement percentages and the ages of a test.

This thesis includes five chapters, an emphasis of the main lines of the thesis and the objectives of the research and a brief explanation of the internal curing and HPC properties and production are pointed out in chapter one. In chapter two a review in details for the historical and thorough details of concrete curing and internal curing benefits in improving the concrete characteristics and properties, especially when used with HPC and review previous researches in this field.

A presentation of detailed information about the experimental work, including: the material properties, mix design, and the structural and mechanical tests of the specimens was covered in chapter three. Chapter four states the test results in tables and figures, also discussing the results of the experimental program. The last chapter consists of the conclusions of the work results and mention some recommendations for future works.

Review of literature

2.1. General

This chapter reviews information and details about the background of the curing of concrete and the relationship between internal curing (IC) and the properties of (HPC). Moreover, it reviews internal curing materials and mechanism, the effect of internal curing on the production of HPC and its role in overcoming problems of low water content in HPC such as shrinkage at early ages. Finally, this chapter discusses the importance of using HPC in many infrastructure projects that mainly require long service life, like bridge girders and deck slabs. In these projects, the presence of long life durable concrete is essential to prolong the life of structural members. The field of HPC and using IC in structural applications of the girders and material properties and recruiting the waste materials in concrete production as a step towards supporting the sustainability concept.

2.2. High Performance Concrete

High performance concrete can be defined as : “concrete meeting special combinations of performance and uniformity requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing, and curing practices” [10]. The term "HPC" was used by *Mehta* and *Aitcin* [11] for concrete mixtures having high durability, high workability besides high strength. At first according to the high strength of concrete at specific ages it was defined as (High Strength).

After the progress of studies in that field this concrete was referred to by (High Performance Concrete) because it had enhanced properties like durability and abrasion resistance as well as high strength[3].

Moreover, HPC is a type of concrete that has a special characteristics and properties that cannot be obtained by using conventional material and processes of ordinary concrete [12]. High performance concrete distinguishing properties are low porosity and less capillary pores in the binder paste.

That properties gained by low water/binder ratio which requires using super plasticizers and pozzolanic admixtures like silica fume to the mixture. This concrete has several desirable properties such as: high workability in fresh concrete, low permeability and high strength [9].

The exact definition of HPC and high strength concrete was changing through the last century, in the twenties a (21 MPa) is considered high strength concrete while this was raised to (34 MPa) and (52 MPa) in the fifties and sixties, respectively. Nowadays, high strength concrete could be made in strength as high as (138 MPa) [13]. The low w/b ratio and additives make HPC has lower permeability than ordinary concrete. Despites that, the composition of materials is optimal and the compaction is perfect but inappropriate curing can cause problems. Insufficient curing would increase the porosity of concrete which leads to high permeability [14].

Another definition is "concrete that meets the requirements or goes beyond the limit of the normal performance range (the term "limit" applied for durability, density, ductility, strength, etc)[15]. The high strength of concrete had a 25 MPa increment to classify HPC. Class I represents high performance concrete with compressive strength between 50 and 75 MPa, class II is between 75 and 100 MPa, class III have strength between 100 and 125 MPa, class IV is between 125 and 150 MPa and clas V more then 150 MPa, as detailed in Table (2-1) [16].

Table (2-1) : High Performance Concrete Classes [16]

Compressive strength (MPa)	50	75	100	125	150
High performance concrete class	I	II	III	IV	V

2.2.1. High Performance Concrete Materials

1. Cement: To produce concrete with high strength and performance almost any Portland cement type can be used to get compressive high strength (beside using admixtures). To produce high strength concrete mix while keeping high workability, it is found that cement composition, fineness and its compatibility with admixtures is important. Several experimental studies have shown that low C_3A cements in general produce concrete improved rheology [17].

2. Aggregate: Rounded shape fine aggregate particles with smooth texture have been found to require less water in concrete therefore it is preferred in HPC. *Aiticin* recommended using fine aggregate with higher fineness modulus (2.7 to 3.0), his justification for that was because of:

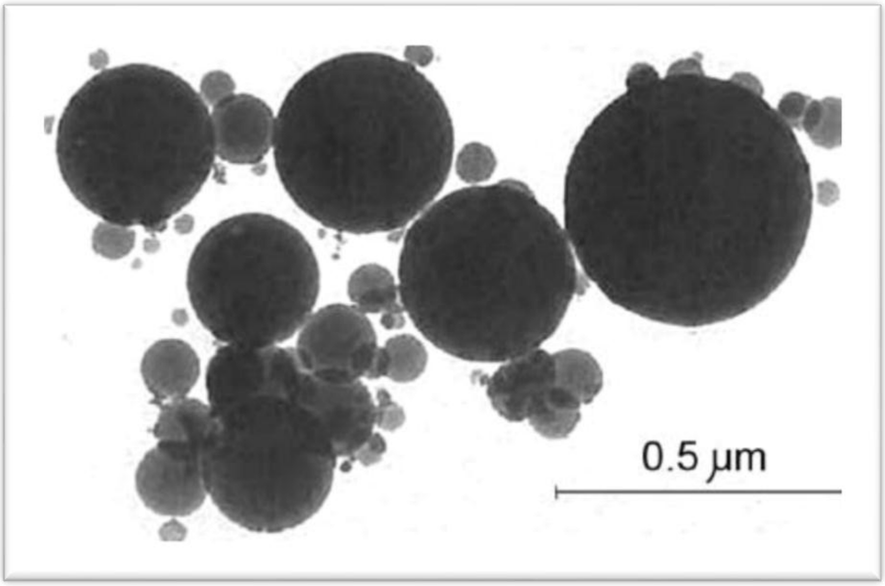
- The higher the modulus of fineness of fine aggregate was, the less water needed to obtain the same good workability.
- The fine aggregate of greater sizes during mixing phase will generate higher shear stress which will prevent any possible flocculation of the paste[16].

3. pozzolanic materials: Four types of fine particles admixtures are mainly used:

- Cementitious materials Ground Granulated Blast Furnace Slag (GGBFS).
- Pozzolanic materials like fly ash and rice husk ash .
- Materials with both cementitious and pozzolanic properties.

These admixtures mainly are of powder state with particles finer than the cement. Therefore, it would affect the properties of fresh concrete in the same manner as the cement affect these properties. Finley mineral admixtures with cementitious, pozzolanic and/or both properties would improve concrete strength. In addition to the improvement of the physical properties of the freshly mixed concrete, it also modifies the physical properties of the final hardened concrete[18].

Here are some details of one of the most widely mineral admixtures used in HPC: (Silica fume) for structural purposes and for application at surface and it is used as repair material in situations where low permeability and abrasion resistance are required. It is a by-product resulting from the reduction of high-purity quartz with coal in electric arc furnaces in the process of production of silicon and ferrosilicon alloys. This by-product fume contains high amounts of amorphous silicon dioxide and consists of very fine spherical particles, which then were collected from the escaping gasses from the furnaces. Silica fume consists of very fine vitreous particles having a surface area about 20,000 m^2/kg The particle size distribution of a typical silica fume shows most of the particles to be smaller than one micrometer (1µm) with an average diameter about 0.1 µm, which make it approximately 100 times smaller than a cement particle as shown in Figure (2-1):



Figure(2-1): Electron Micrograph of SF Microspheres
(Courtesy of *Elkem ASA Materials*)

The specific gravity of the silica fume is typically 2.2, but it may reach as high as 2.5. After collection the bulk density of silica fume is (160 to 320 kg/m^3)[19].

The silica fume shows a pozzolanic reaction with the lime during the hydration of cement and forms a stable cementitious compound called: calcium silicate hydrate (CSH). The availability of high range water reducing admixtures made it possible to use silica fume as part of the cementing material in concrete to produce high performance concrete, normal silica fume content usually ranges between (5% to 15%) of Portland cementious material [17].

4. admixtures: For decades it was known that some certain organic molecules have dispersing properties and also could be used to neutralize the electrical charges on the surface of cement particles so it reduces their tendency to flocculate. Nowadays these molecules are used as water reducer, dispersing and superpalstisizer agents [20].

As mentioned earlier, one of the requirements of HPC is low w/b ratio. To solve this problem a water reducing admixtures must be implemented in the process. These admixtures should meet applicable requirements of the (specification of chemical admixtures for concrete) according to the ASTM (C 494-10)which classify these admixtures to the following[18] :

- Type A Water-reducing
- Type B Retarding
- Type C Accelerating
- Type D Water-reducing and retarding
- Type E Water-reducing and accelerating
- Type F High-range water-reducing or super plasticizing, and
- Type G High-range water-reducing, or super plasticizing and retarding.

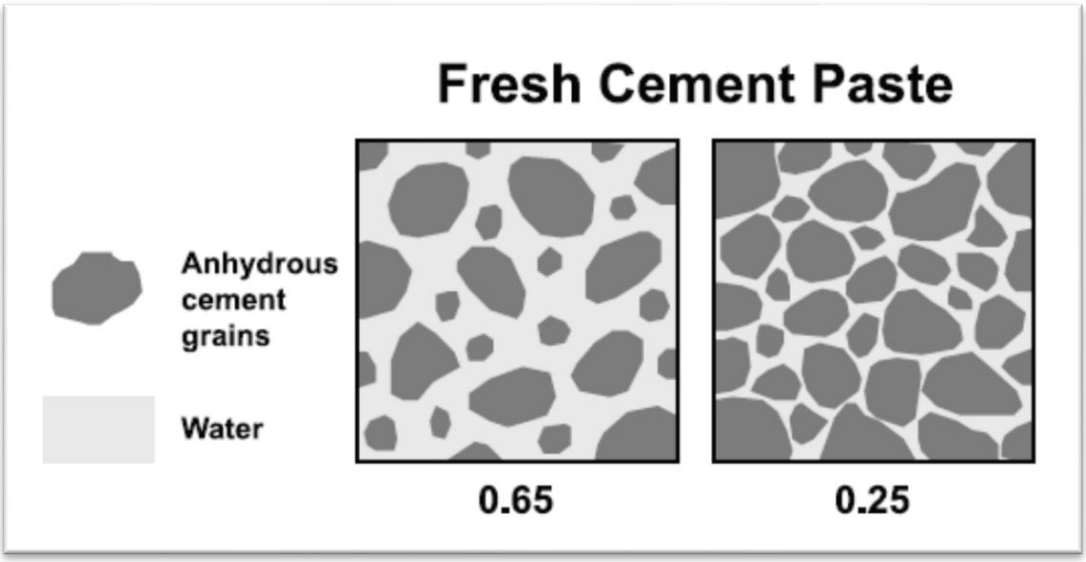
The main action of the long molecules is to wrap themselves around the cement particles and charge it with negative charge so that they repel each other. This results in deflocculating and dispersion of cement particles. The resulting improvement in workability can be exploited in two ways[21]:

a) By producing concrete with a very high workability

b) Concrete with a very high strength.

2.3. Durability of High Performance Concrete Structures

As HPC became so popular in the construction to extend the service life of the concrete structures due to its special properties, like low permeability which reduces the effect of chlorides and its effect on protecting the reinforcement from corrosion [3]. HPC is showing high durability in comparison with conventional concrete, not only because of its lower porosity but its capillary and pores networks are not connected, therefore the internal stresess due to self-desiccation are prevented [22]. The main key to provide durable concrete is to decrease the voids in the cement paste, these voids are caused by a high w/b ratio as shown in Figure (2-2) which illustrates the schematic difference between 0.25 and 0.65 w/b ratio concretes [22].



Figure(2-2): Schematically View of the Microstructure of Two Cement Pastes 0.25 and 0.65 w/b Rice Husk Ash (22)

The pozzolanic materials in general improved concrete properties when used with HPC in bridge girders, a research by *Zhang* showed that using fly ash as supplementary material can restrain creep of high performance concrete, the results of concrete creep mixed with fly ash at a different w/b ratio of 0.28, 0.32 and 0.40 is less than standard concrete[23].

A research by *Jonson* on the durability of bridge girders examined the durability of HPC based on measuring the resistance of an electrical charge passed through concrete, the unit to measure this electrical charge is (coulombs) during six hours, both silica fume and furnace slag increased the resistance of concrete to chloride ion penetration. Using a binder system of cement, silica fume, and slag would provide high results at resisting chloride ion penetration in concrete [24]. This improvement in concrete durability is important to the overall performance of the structures, a work by *Mehta* indicates one of the advantages of producing a durable concrete is to prevent and minimize the amount and size of the cracks to protect the steel reinforcement from salts attack[25].

The concrete bridge decks is a member that relies on the durability, since it considered a part of the infrastructure and like other concrete structures it deteriorates with time. Therefore, it is economically more efficient to use durable concrete with less cracks to assure longer life as a research showed in a case study in "Oklahoma" that using HPC decreases the cracks when compared to conventional concrete after one year significantly[26].

In addition, the use of internal curing alongside with HPC is of a great benefit to the structures as a work by *Cusson* showed an increase in the bridge decks service life about 10 years and reducing the life cycle cost of the decks, that achieved by the ability of internal curing to reduce the initial cracking of concrete[6].

A case study of internal curing showed improvement in service life about 23 year when compared to conventional curing methods by using analytical predictive models to estimate shrinkage cracking, chloride diffusion, corrosion initiation and corrosion propagation leading to cracking [27].

2.4. Waste Materials Utilization in the Structural Field

It is suitable to mention some of the structural researches that studied the behavior of structural members made of HPC. The major studied properties in this field besides the concrete properties is studying the relationship between load and deflection, stress-strain relationship of the concrete member, crack pattern at failure... etc.

The use of waste materials as a factor towards improving the sustainability of the environment by reducing the effect of waste material was also studied. The most common material of the demolition materials that used to produce concrete made of recycled aggregate which does not act as good as the ordinary concrete but could be reliable in some cases.

A study by *Won-Chang* studied the structural behavior of concrete beams made entirely of recycled aggregates which showed nearly the same behavior of the ordinary concrete because similar crack patterns are observed in both aggregate types used in the work, even though numerous cracks were presented in the beams composed of recycled aggregate [28].

Another study by a researchers from "Ghana" used phyllite aggregates obtained from mining waste in a production of reinforced concrete beams, The results showed that the load-deflection curves of the reinforced concrete beams are similar to structural responses of other reinforced concrete beams made of conventional aggregates [29]. Deflections compared reasonably well with BS 8110 requirements. However, the average crack widths under service loads were greater than the limit permitted by the code indicating that the beams may encounter durability problems with time [30]. Another work on T section beams made entirely of fly ash aggregates (coarse and fine) showed that these beams gives a satisfactory structural performance according to ACI [31].

It can be concluded from what mentioned earlier that there is orientation of using a recycled or waste materials in the concrete industry and in some cases combining them with HPC. Using HPC in some of bridge parts like girders is useful due to the incorporation of the cheap admixtures, the cement content decreased significantly and reduces the production cost. Adding pozzolanic materials like slag and fly ash significantly affect the workability of concrete. High performance concrete can save the cement consumption of about 70%, which can effectively reduce the energy consumption of cement production and reduce the pollution of the environment. The use of fly ash and slag can also reduce the damage to the environment, these measures for saving energy and reducing consumption plays a big role [32].

2.5. High Performance Concrete and Internal Curing Effect

The effect of IC on concrete properties and durability had been under study in many researches. The most important property in evaluating concrete is the compressive strength. Thus, many researches focused on evaluating and improving the compressive strength. An experimental work conducted by *Ackay* showed that using pumice materials as LWA presoaked in water can efficiently diminish the autogenous shrinkage in concrete [33].

Byard and *Schindler* used different LWA like shale, clay and slates to produce internally cured concrete mixtures that presented similar or higher compressive strengths when compared to ordinary concrete. These results were produced using a w/b ratio of 0.42 [34].

A structural behavior of concrete made of sand stone also had been investigated[35]. The New York State Department of Transportation (NYSDOT) constructed a series of bridges from 2009 to 2011 using LWA for internal curing The mix designs for the bridges specified a 30% light weight aggregates LWA replacement (by volume) and a w/b ratio of 0.40. Using data from the report shows that internal curing either had a negligible effect on strength or resulted in higher strengths [36].

Test data from *Cusson and Hoogeveen* showed that internally cured concrete by variable saturated LWA reduced autogenous shrinkage significantly without affecting the strength or elastic modulus of concrete, This result was accomplished by using a total w/b ratio of 0.34 for four different mixtures and reducing the mix water by the quantity of water provided by the LWA to reduce the effective w/b ratio [37].

Compressive strength is the major properties to evaluate concrete class and in general, but other properties of concrete should be taken into consideration especially when dealing with HPC, because tensile strength is related to the propagation of cracks in concrete which affect concrete durability[21].

The effect of internal curing on concrete tensile strength had been investigated in many thesises and researchs which showed improvement in tensile strength[38]. Some researchers investigated the effect of silica fume replacement percentage on the tensile strength of concrete, it shows improvement of tensile strength when using silica fume[39],[40].

Also, the addition of ground granulated blast furnace slag and SiO_2 nanoparticles as a binder in a research by *Nazari* showed an improvement in tensile strength capacity [41].

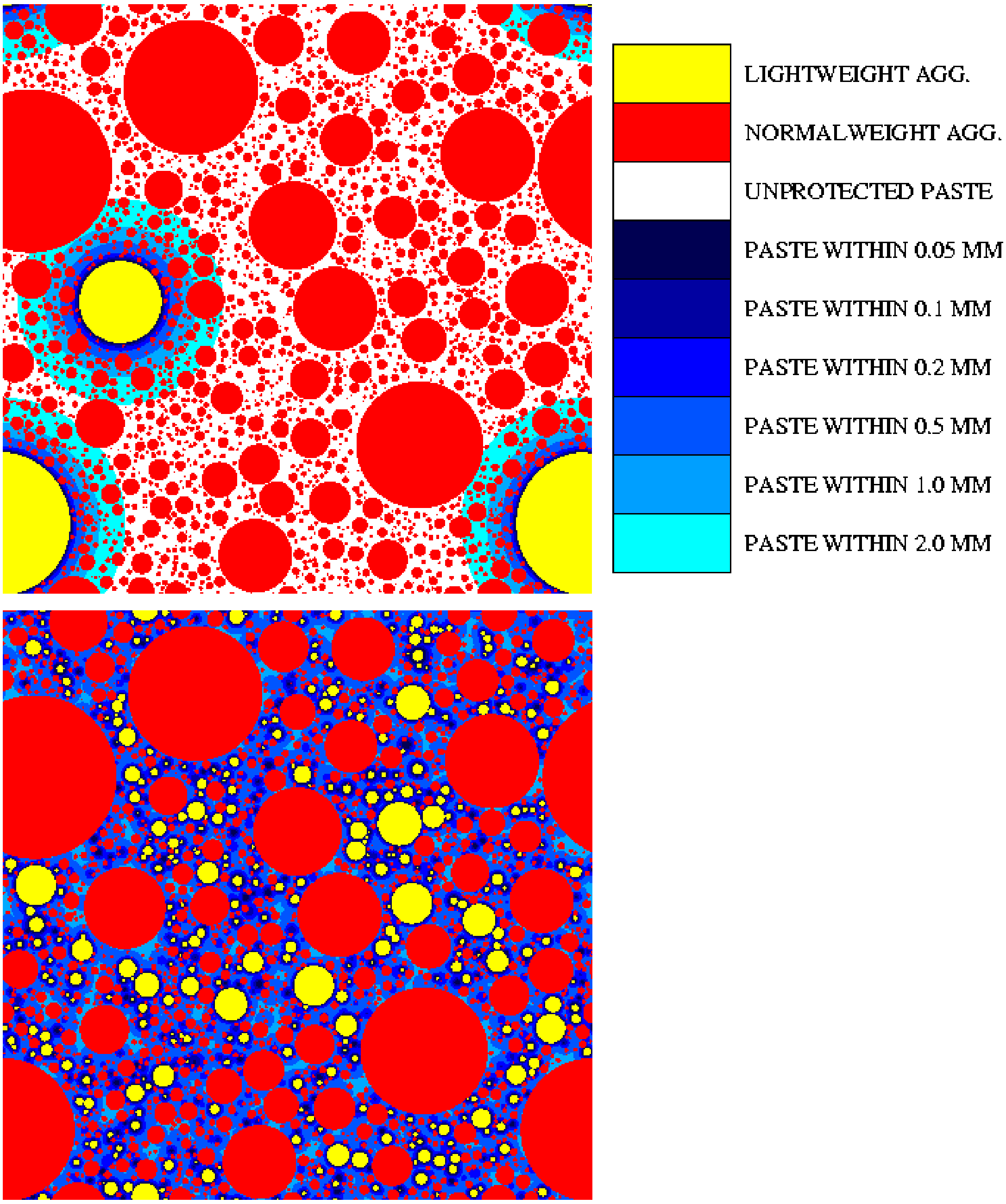
Concrete porosity is the relative volume of pores or voids in the cement paste, it is considered a weakness point in the concrete. It is known that The major factor affecting the porosity is (w/b)[42]. In low and medium strength concrete made with normal weight concrete (NWA), both the interfacial transition zone between the matrix and aggregate as well as the porosity of the cement paste matrix determine its strength, the strength of interfacial transition zone decreases with lower w/b. Thus, the parameter w/b is important in determining the porosity of both the matrix and the interfacial transition zone.

Low w/b decreases porosity of the matrix and the interfacial transition and improve both the compressive and tensile strengths of concrete [43].

2.6. Internal Curing Mechanism and Materials

Internal curing can be defined as the process by which the hydration of cement continues because of the availability of internal water that is not part of the mixing water. The difference between internal curing and the ordinary curing method is that moisture is applied to the surface of the concrete, while internal curing provides moisture from the lightweight aggregate and not part of the w/b ratio[8].

One of the problems of LWA is its low strength compared to the normal aggregate, especially the coarse LWA which is considered a weak point of the concrete more than fine aggregate, also its tendency to defuse internal curing water to the cement paste is not efficient as the fine aggregate which blended efficiently within the cement matrix[44]. As shown in Figure (2-3) :



Figure(2-3): Effect of LWA Size in Internal Curing

Bentz suggested a formula to estimate the amount of internal curing water and the light weight aggregate to deliver this which is given in the following formula[45]:

$$M_{LWA} = \frac{Cf * CS * \alpha max}{SLWA * \varphi LWA} \quad \text{.. Equation (2-1)}$$

Where:

M $_{LWA}$ = LW A dry mass needed in concrete (kg/m3);

Cf = Cement factor (cement content) in the mix (kg/m3);

CS = Chemical shrinkage of cement (gm. Water/gm. Cement);

α ***max*** = maximum expected degree of cement hydration ;

S_{LWA} degree of saturation (0 to 1)

(Note: this equation is only valid for non-zero values of *S*, otherwise the amount of aggregate will diverge to infinity)

ϕ ***LWA*** = LWA Absorption (kg water/kg dry LWA).

For *w/b* below 0.36, the maximum expected degree of hydration of the cement under saturated conditions can be estimated as ([*w/b*m]/0.36) and should not vary significantly with curing temperature. For *w/b* higher than 0.36, the maximum expected degree of hydration of the cement can be estimated as 1. Because the densities of the dry lightweight aggregates and the conventional aggregates are substantially different, the ultimate substitution in the concrete mixture should be performed on a volume. Internal water curing is conducted using one of the following materials [46]:

- Natural or Synthetic Light Weight Aggregate Fines.
- expanded shale with higher water absorption capacity.
- Super Absorbent Polymers (SAP) (60-300 mm size) - Sodium salts of poly-acrylic acid, polyacrylamide copol., ethylene maleic anhydride copol, cross-linked carboxymethyl- cellulose and polyvinyl alcohol copol, etc.
- Coarse aggregate of nominal maximum size and gradation (Water absorption = 20%)
- Shrinkage Reducing Admixture - SRA polyethyleneglycol.
- Saturated Wood powder / fibres.

Internal curing is especially helpful when used in conjunction with HPC, because of low w/b ratios in HPC, there is typically insufficient water to fully hydrate the cement. Internal curing with LWA provides the additional water needed as well as improving the following properties[47]:

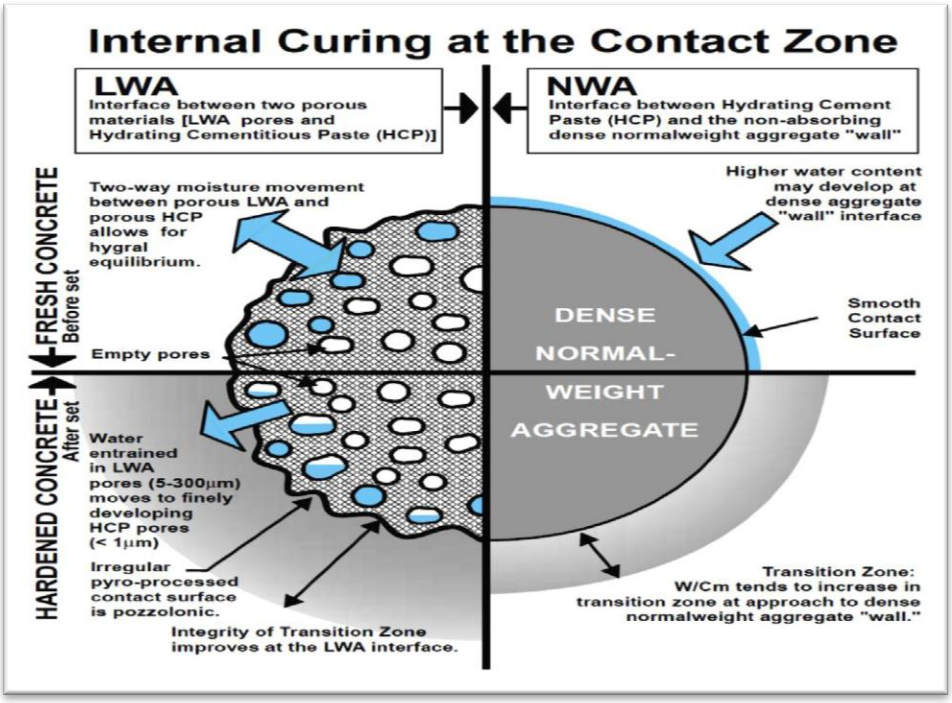
- Reduces autogenous shrinkage
- Reduces cracking

- Hydrates more of the cement
- Increases strength from the first 24 hours and beyond
- Keeps internal relative humidity high
- Reduces self-desiccation
- Reduces chloride permeability
- Improves durability

The porous structure of LWA provides the ability to retain water in the grains of aggregates, such water could be useful later in the hydration of cement. Figure (2-4) shows a brief illustration of how the water improves the transition zone of aggregate in comparison with ordinary aggregates. One important act of internal curing is its improvement of (contact zone) shown in Figure (2-4).

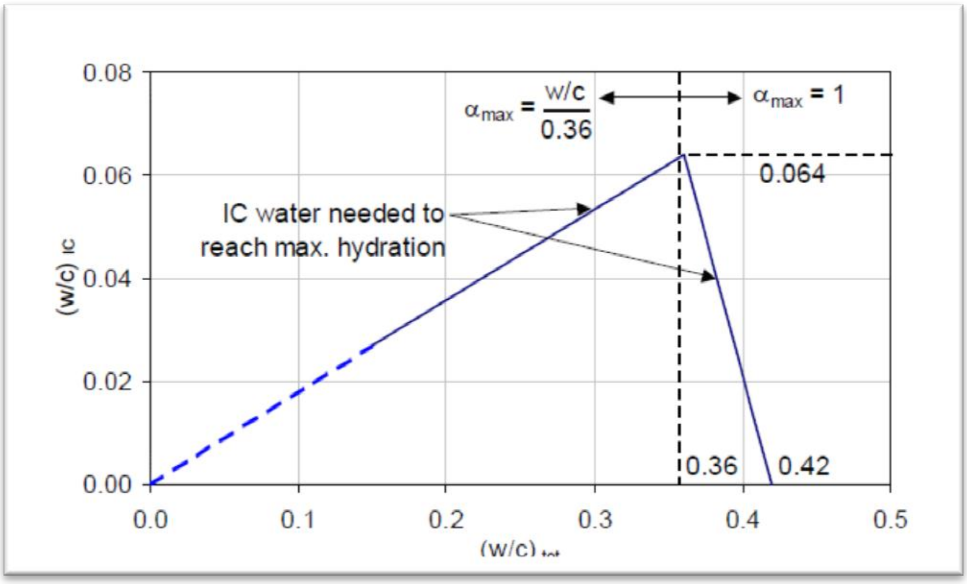
Contact zone represents: (1) the mechanical adhesion of the cementitious matrix to the surface of the aggregate and (2) the variation of physical and chemical characteristics of the transition layer of the cementitious matrix close to the aggregate particle.

A collapse of the structural integrity of the concrete conglomerate may come from the failure of either the aggregate or cementitious matrix, or from a breakdown in the contact zone causing a separation of the still intact phases. The various mechanisms that act to maintain continuity, or that cause separation, have not received the same attention as has the air void system necessary to protect the matrix.



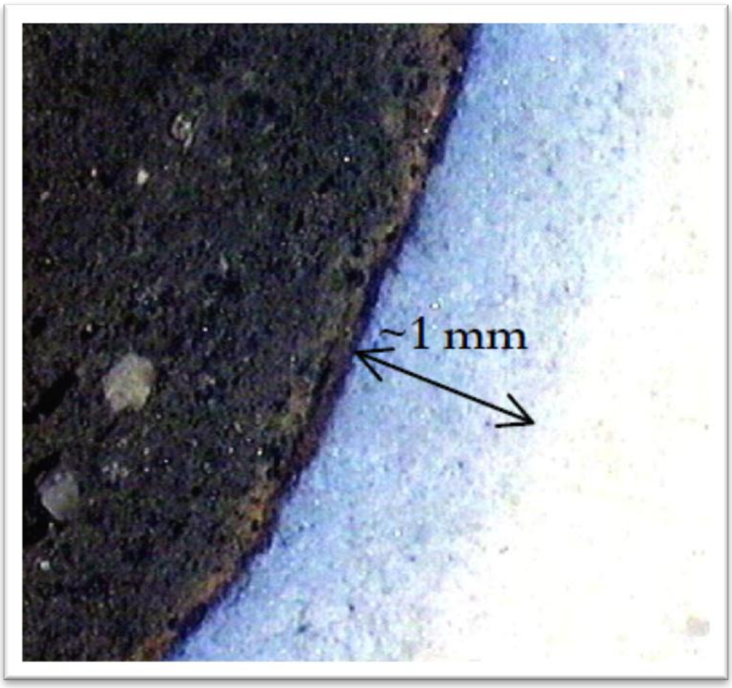
Figure(2-4): Improving the Transition Zone by Using Light Weight Aggregate [48]

Based on the early work of *Powers* (1948) on the chemical shrinkage of cement pastes, A figure has been proposed later to estimate the amount of internal curing water that is required to prevent self-desiccation in concrete and also the resulting autogenous deformations. In Figure (2-5), two solid lines represent the minimum quantity of internal curing water, which is required to gain the maximum degree of hydration, that can be 1.0 (i.e. 100%) for water-cement ratios (w/b) more than (0.36). Below this value, cement hydration would be achieved partially, and can be estimated as (w/b)/0.36. The results showed that conditions of fully saturation for hydrating cement pastes would be possible with a mass of internal curing water of 0.064 (kg w/ kg of cement). The latter value does represent the chemical shrinkage which is typically measured on ordinary Portland cement paste[6].



Figure(2-5): Minimum Amount of IC Water Needed to Achieve Maximum Hydration [45].

Also an experiment made by *Lura* to show the behavior of moisture within LWA by coloring the transition zone, a "liapor grain" had been saturated in ink, then the surface was dried and it was mixed with cement paste. This method enabled to see the ink diffusion into the cement paste, also progression in the ink diffusion was noticed till two weeks after casting as shown in Figure (2-6) [9].

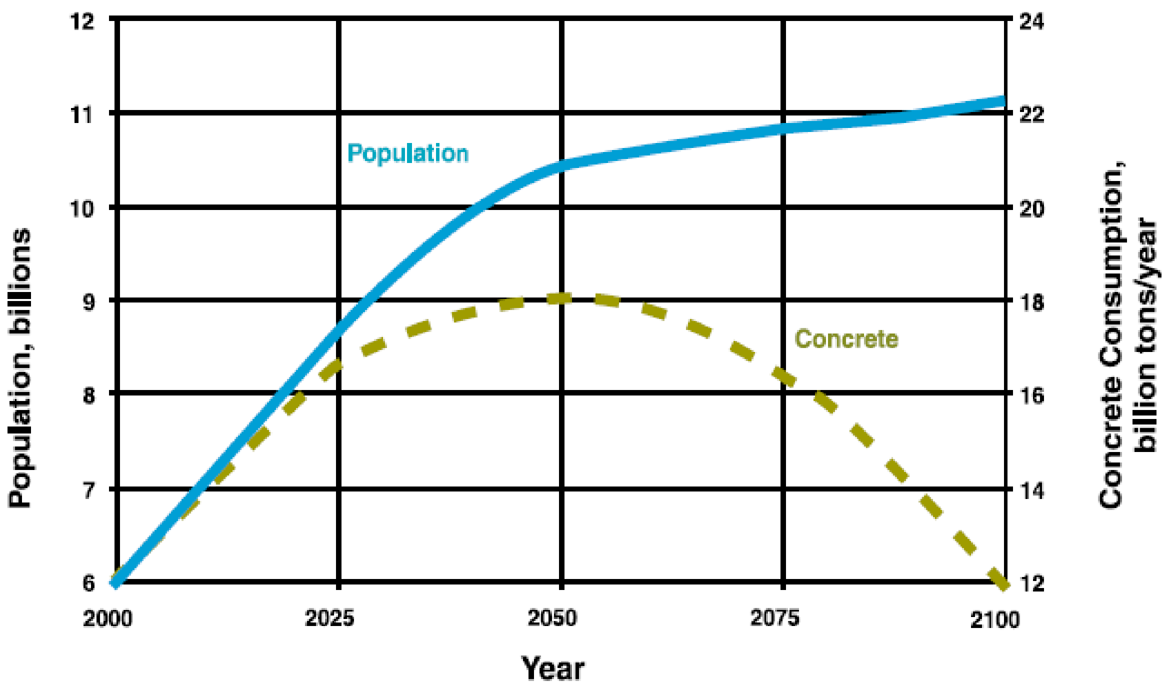


Figure(2-6): Colored Corona Expanding Around a LWA Saturated With Ink [9]

2.7. Sustainability Aspect

The term "sustainable" is a general concept. The concept of sustainable development is nearly an old concept. The first person who used this term was *Hans Carl von Carlowitz* (1645 – 1714), a forester of Saxony. Who was the supervisor of the King's forests he realized that trees should not be cut at a rate higher than which they might regenerate or sustain themselves [49].

Probably the best definition of Sustainable Development is "the development that meets the needs of the present without compromising the ability of future generations to meet their own needs" [50]. Since concrete is considered as one of the sources of the greenhouse gases emission due to its consumption of fuel in the manufacturing of the cement, also the production of concrete is one of the world's most fuel and resources consuming process[51]. In Figure (2-7), it is obvious that the forecasted concrete consumption keeps growing for the next few decades.



Figure(2-7): Forecast of Future Population Growth and Concrete Consumption[51]

Therefore, rises the need of instant solutions to reduce the effect concrete on greenhouse gasses to the least level. Many materials have been used to improve environment and lessen the impact of using concrete such as [52]:

- Fly Ash
- Ground Granulated Blast Furnace Slag (GGBFS)
- Condensed Silica Fume.
- Solid Waste Incinerator Ash.
- Other Potential Supplementary Cementitious Materials.
- Recycled Concrete Aggregate.
- Crushed Post-Consumer Glass
- Plastics
- Dredged Material
- Rock Spoils
- Tires.

Therefore, many researchers examined the effect of using these materials on the mechanical properties of concrete and the structural behavior; such materials like phyllite aggregates from mining waste[29], recycled concrete aggregate[53] and high-volume fly ash on [54] and others.

2.8. Concluding Remarks

- High performance concrete is one of the important types of concrete that not featuring high strength only, but consists of enhanced properties like high durability which make it more durable and can have a longer service life than conventional concretes.
- The production of HPC requires additional materials beyond conventional materials that should be added to the mix, these materials include chemical and mineral admixtures.

- HPC requires a reduction in w/b ratio and that results in early age shrinkage which would possibly propagate cracks. Thus, it is important to provide extra moisture like the moisture of (Internal curing) to reduce such effect.
- Local waste materials which have the act of internal curing agent can be used as water reservoirs in concrete mixes.
- Besides its availability and moderate cost, the use of recycled building waste materials and by-products are considered environmental friendly and a contribution in the sustainability.
- Using HPC is important in the structural elements that requires long service life, especially in the infrastructure field like: bridge girders, deck slabs, rigid pavement, tunnels… etc.

Experimental program

3.1. General

This chapter is dedicated to explain the vast details of the experimental work. The procedures for selecting the materials that needed for the research, choosing mix design, structural design of the concrete beams and the steps of preparing and testing the final specimens also covered in this chapter. Fifteen reinforced concrete beams had been prepared with dimensions of (1700*150*100) mm. All beams had the same cross-section (150*100) mm.

A sum of 72 (100*100*100) mm concrete cube was casted to estimate the compressive strength, hardened properties and fresh concrete properties tests as well as selecting the mix design. Also, 45 cylinders (100*200) mm to estimate tensile strength.

All specimens were cured in water at 24 C^o. Two materials used in this work "crushed brick" and crushed limestone locally known as "gubbra". The volume replacement percentage was (5% and 10 %) of fine aggregate for each material as well as reference mix with high performance concrete to hold a comparison. The test was on three stages (28, 90 and 150) days for each group. All tests had been done at the concrete laboratory of the " Civil Engineering Department – University of Kerbala".

3.2. Materials

This section provides detailed description and test results of the materials that was used in the experimental work of the thesis :

3.2.1. Cement

Ordinary Portland cement (TYPE I) was used in the work from (Karbala cement plant) and was packed in another plastic bags sealed to prevent humidity and environment effects on the cement. The chemical and physical tests of cement were made in the construction materials laboratory of Karbala according to the Iraqi specifications [55]. The chemical and physical properties of the cement are listed in Tables (3-1) and (3-2), respectively:

Table(3-1): Chemical Properties of Cement

Chemical analysis of compounds		
Oxides	**Percentage %**	**I.Q.S. 5 :1984:Limits**
CaO	57.5	/
SiO_2	18.9	/
Al_2O_3	3.7	/
Fe_2O_3	3.6	/
SO_3	2.75	2.8 (max)
Mgo	3.72	5.0 (max)
Na_2O	0.25	/
K_2O	0.67	/
Insoluble Residue (I.R)	0.82	1.5 (max)
Loss On Ignition (L.O.I)	3.5	4.0 (max)
L.S.F	0.95	0.66 – 1.02
Bogue Potential Compound Composition %		
C3S	51.3	/
C2S	17.2	/
C3A	3.9	/
C4AF	10.8	/

Table(3-2): Physical Properties of Cement

PHYSICAL PROPERTIES		
PROPERTIES	RESULTS	**I.Q.S. 5 :1984: Limits**
Fineness (Blaine Specific surface(m^2/Kg))	285	230 (minimum)
Time of Setting (Vicat test) Initial Set (hrs : min) Final Set (hrs : min)	 1:35 4:35	 0:45 (minimum) 10:00 (maximum)
Compressive Strength (MPa) 3 Days 7 Days	 18.9 30.5	 15.00 (minimum) 23.00 (minimum)

3.2.2. Steel Reinforcement

The steel bars used as reinforcement was Ukrainian steel (Grade 60) with diameters of (10, 6) mm for main reinforcement and 6 mm for stirrups. The steel was conforming to ASTM – A615 Specifications [56]. The geometrical and mechanical properties are shown in Table (3-3):

Table(3-3): Reinforcement Steel Properties

Nominal diameter (mm)	Actual diameter (mm)	Cross section area (mm^2)	Yield stress (MPa)	Ultimate stress (MPa)	Elongation (%)	Nominal weight (Kg/m)
6	5.6	24.6	520	540.4	7.3	0.195
10	9.8	75.4	540	580	12.3	0.616

3.2.3. Coarse Aggregate

Black crushed aggregate from (Niba'ai) region was used as coarse aggregate in the mix. The gading and properties tests made to the aggregate were compared to the Iraqi specifications[57], see Table (3-4). The laboratory tests were made in the concrete laboratory of the civil engineering department- University of Karbala.

Table (3-4) Coarse Aggregate Properties

Sieve size (mm)	Accumulated percentage passing	Limits of Iraqi spec. No. 45/1984
37.5	100	100
20	98	95-100
10	35	30-60
5	0	0-5
Sulphate content	0.09 %	max.:0.1%
Specific gravity	2.65	/
Dry rodded density	1650 kg/m^3	/

3.2.4. Fine Aggregate

Sand used in the work was washed sand from (AL-Akhaidir) region. The tests were compared to Iraqi specifications [57]. The tests were made in the concrete laboratory of the civil engineering department- University of Kerbala the test resuts is shown in tables (3-5) and (3-6) :

Table(3-5) : Fine Aggregate sieve Analysis

Sieve size	Percentage passing	I.Q.S. 45 -1984 (ZONE 1)
10	100	100
4.75	90.5	90-100
2.36	68.45	60-95
1.18	54.7	30-70
0.6	32	15-34
0.3	12.8	5-20
0.15	2	0-10

Table (3-6): Fine Aggregate Physical Properties

Properties	result	Limitation
Fine materials passing from sieve (75 μm)	4.1%	Max. = 5%
Specific gravity	2.60	-
Sulfate Content (SO_3) %	0.34	≤ 0.5

3.2.5. Water

The water used in the concrete mixes was ordinary potable tap water from the water network.

3.2.6. Superplasticizers

The admixture used as superplasticizer is **GLENIUM® 51** which had been primarily developed for use with ready mixed and precast concrete. It is free from chlorides and complies with ASTM C494 Types A and F. The admixture also is compatible with all Portland cements that meet recognized international standards. Table (3-7) list the properties of the superplasticizer:

Table (3-7): Superplasticizer Properties

form	viscous liquid
Color	light brown
Density	1.1 gm/cm^3
Relative Density	1.1 @ 20°C
pH	6.6
viscosity	128 +/ - 30 cps @ 20°C

According to the manufacturer the dosage range is about (0.5-0.8) liter per 100 Kg and any value beyond this range must be restricted and authenticated by trial mixes. The add values were chosen to be (1.4 liters per 100 Kg) which produced workable concrete with the targeted compressive strength at (0.3) w/b ratio.

3.2.7. Silica Fume

Silica fume used in the mixes was **MegaAdd MS (D)** densified microsilica. Silica fume is a byproduct of reduction of high-purity quartz with coal in electric furnaces in the production of silicon and ferrosilicon alloys.

The properties of the silica fume are listed in Table (3-8) :

Table 3-8 : Silica fume properties

Properties *	**value**
State	Sub-micron powder
Color	Grey to medium grey powder
Specific density	2.10 to 2.40
Bulk density	500 to 700 kg/m^3
Silicon dioxide (SiO_2)	Minimum 85%
Moisture content	Maximum 3%
Specific surface area	Minimum 15 m^2/g

* Properties are according to the manufacturer

3.2.8. Crushed Brick

Crushed brick used in this study was a waste of a construction, crushed to the desirable size of the fine aggregate and then sieved on a multi sieves to match required grain size distribution. Table (3-9) shows the grading of crushed brick. Figure (3-1) illustrates the steps of preparing the material.

Table 3-9: Crushed brick grading

Sieve size	Percentage passing	Fine aggregate grading
10	100	100
4.75	90	90.5
2.36	70	68.45
1.18	55	54.7
0.6	30	32
0.3	13	12.8
0.15	2	2

The crushed brick was saturated with water for 48 hours to examine its absorption capability which was nearly (25%) of weight.

Figure(3-1): Crushed Brick

3.2.9. Crushed Limestone (Ghubbra)

This material is locally named "Ghubra", which had been brought from local market. It was used as a fine aggregate with the same gradation zones as the sand. The limestone was saturated in water for 48 hours to examine its water absorption capability which was (22%) of weight.

3.3. Mix Design

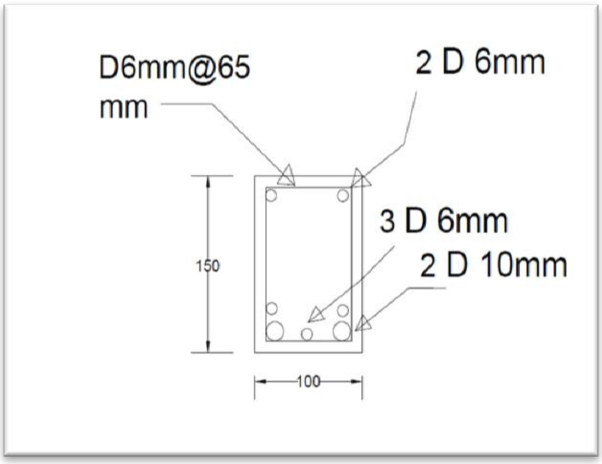
To obtain the suitable mix design to produce a high performance concrete with the desired compressive strength, four concrete trial mixes were made according to the ACI recommendations[58]. There is no specific mix design approach for design high performance concrete mixes. Therefore, ACI 211 suggests starting with reference mix from previous project and obtain new mixes by changing the ingredients to reach the desired strength. The targeted compressive strength for this experimental work was (50 – 55) MPa. All HPC mix proportions are listed in Table (3- 10) which indicate that (mix 3) had the highest strength:

Table 3-10 : Trial Mixes Design

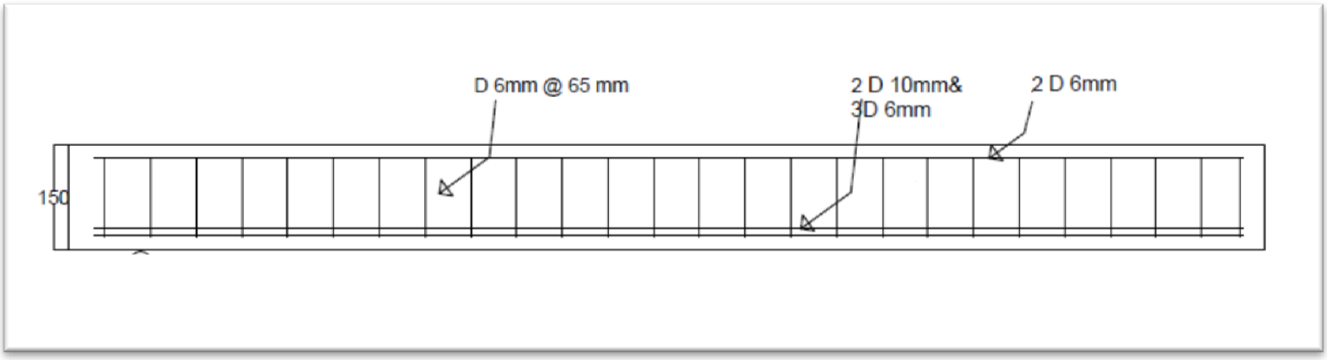
Mix No.	Sand (Kg)	Gravel (Kg)	Cement (Kg)	Silica fume (Kg)	Superplastisizer (L/100 Kg)	w/b	Strength at 28 (MPa)
Mix 1	710	975	470	35	1.5	0.32	53
Mix2	675	1000	500	40	1.3	0.27	50
Mix3	**700**	**950**	**450**	**50**	**1.4**	**0.3**	**55**
Mix4	730	920	425	75	1.7	0.35	49

3.4. Beam Design

A total of 15 reinforced concrete beams had been designed to study the behavior of reinforced concrete beams under two points loads. All beams have the same dimensions of (1700*150*100) mm and the same reinforcement. The concrete section was singly reinforced and the main reinforcement was (2Ø10 mm and 3Ø6 mm) at the tension zone and (2Ø6 mm) at the upper zone of the section . For the shear resistance the beam was reinforced by steel stirrups (Ø6 mm @ 65 mm) as shear reinforcement as shown in the Figure (3-2):



(a)



(b)

Figure(3-2): Reinforced Concrete Beam Details
(a) : Beam cross section
(b) : Longitudinal profile

The beams were designed as a singly reinforced concrete beam by ultimate load method (ULM) according to the (ACI 318-14) code [59]. The beam was designed for bending failure and resist shear so that the failure would occur by bending , either steel yielding or concrete crushing. The cross section and steel design are listed in (APPENDIX) .

3.5. Casting of Specimens

For the casting of the reinforced beams the following procedures were made :

- The materials of the concrete (cement, sand , silica fume …etc.) were prepared and weighed according to the specified ratios of the mix design.

- The internal curing materials (crushed brick and limestone) had been submerged in water 48 hour before batching , then it had been rubbed by clean towels to remove the surface moisture.
- Wooden molds had been manufactured whith the proper dimensions as designated in the structural design (1700*150*100) mm.
- The molds were strengthened by steel clippers during the casting to prevent any displacement and changes in dimensions due to the pouring of concrete.
- The steel reinforcement were detailed and bonded together by wires to form the designed reinforcement steel cage.
- The mixing were made by a (0.12 m^3) rotary batching machine at the concrete laboratory of the Civil Engineering Department- University of Kerbala.
- The batching machine was cleaned and moistened before every mix and the mix ingredients were added and mixed as well as the internal curing materials in the exact ratios.
- The ingredients then mixed in the batching machine for 3 to 5 minutes at a stable rotating rate then poured on clean steel sheet before casting.
- Every beam had been cast in three layers and every layer was vibrated by vibration rod to make sure that the concrete is condensed properly and no segregation would occur.
- The concrete beams had been put in a large tanks in shade and immersed completely in water at (25 C^o) degree for 28 days.
- Every beam was painted by white water-base paint to make the observation of the cracks easier while testing the beam.

Figure (3-3) shows some of the beam casting steps.

(a) Preparing the molds for casting

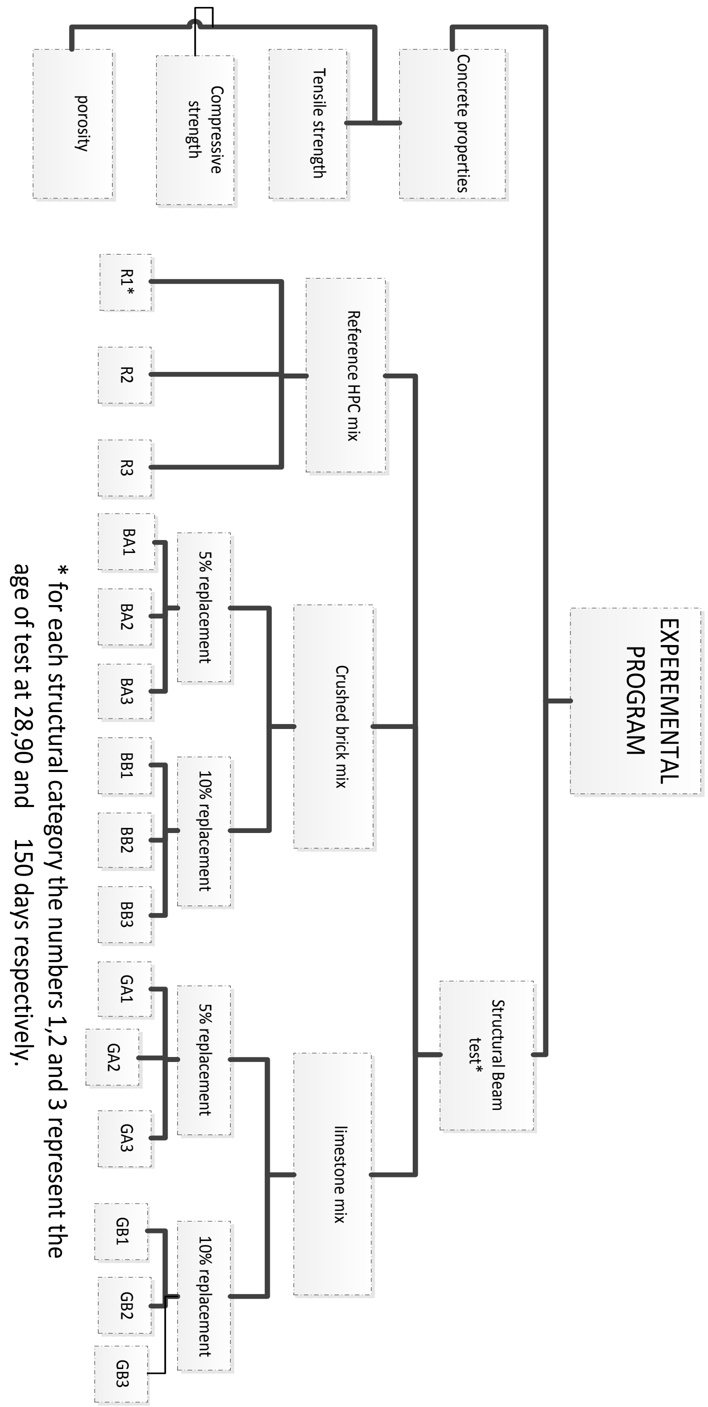
(d) : Beam painting before test

Figure(3-3): Reinforced Concrete Beam Casting Steps

3.6. Testing Program

This section provides detailed description about testing methods implemented in this work. To differentiate among the different concrete mix proportions, each mix was given a certain designated symbol. The reference ordinary mixes were given the symbol (R), the crushed brick mixes were given the symbol (B) while the limestone (Gubbra) mixes were given the symbol (G).

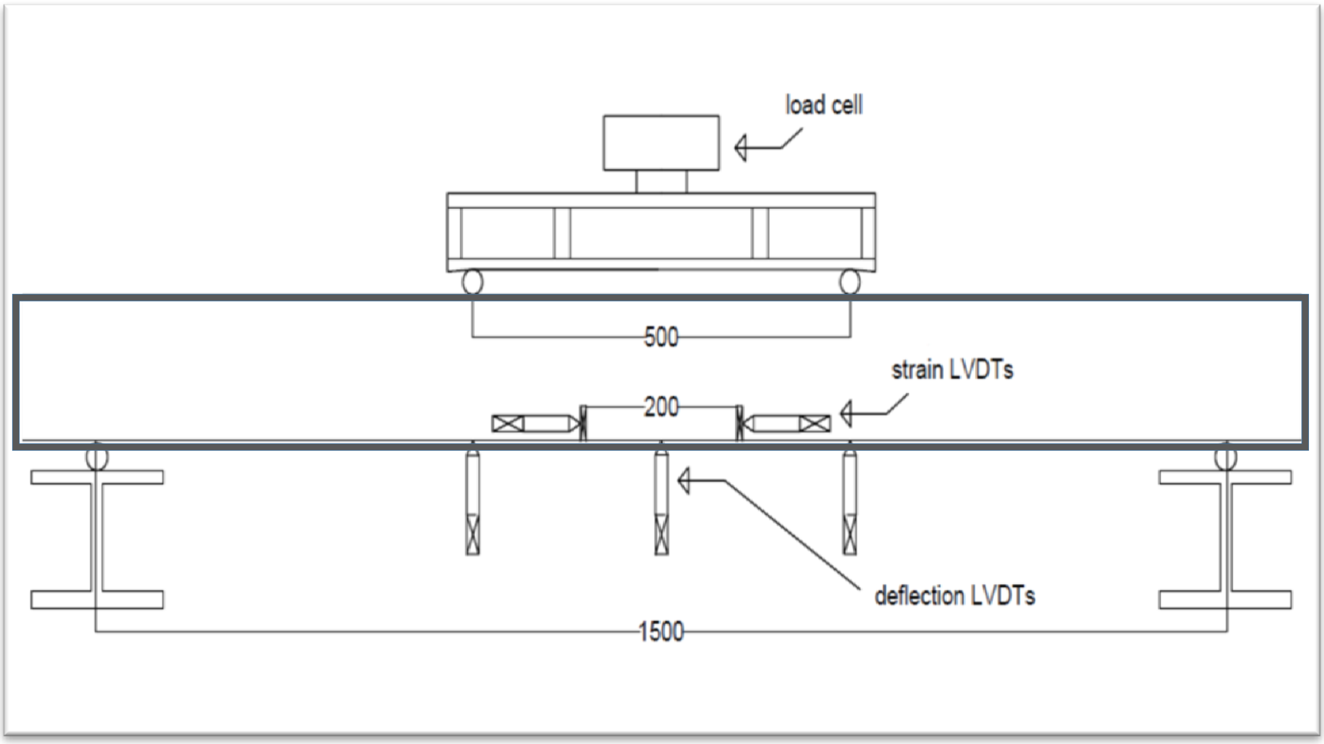
To indicate the percentage of replacement in every mix the symbols (A and B) were used after the mix material symbol to represent the replacement percentage of (5% and 10 %), respectively. Additionally, the age of testing specimens is added to the end of the name like (1, 2 and 3) to represent (28, 90 and 150) days, respectively. For example: the mix (**GA1**) stands for limestone mix with 5% replacement percentage at 28 days, and so on for the rest of the mixes. Figure (3-4) shows the detailed test program.



Figure(3-4): Schematic Test Program Chart

3.6.1. Structural Testing of R.C Beams

Each beam had been tested by applying load up to failure. The test approach was four points bending test, the beam span was divided into three thirds 500 mm each.
The load divided into two point loads by using steel girder on the beam wich has a rigid cross section to make sure no defrmation would happen to it that could affect the test. Figure (3-5) illustrates the beam and LVDTs positions. Figure (3-6) illustrate the final setting of the beam and LVDTs in the testing machine to conduct the test of the beam



Figure(3-5): Structural Beam Test Details

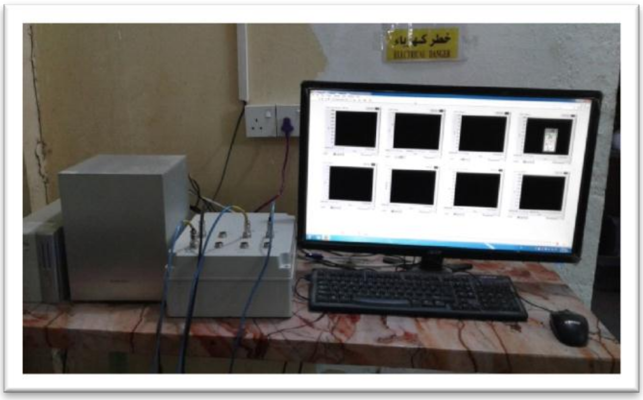
.

(a)

(b)

Figure(3-6): Beam Loading Position

The test was conducted in the concrete laboratory of the civil engineering department – University of Kerbela. A universal testing machine with (2000 kN) loading capacity designed and manufactured for the structural loading test was used during the experimental testing program. The machine is equipped with electronically controlled gages (linear variable differential transformer -LVDT) and loading cells which collects load, deflection and strain data using a computerized system programmed by (LABVIEW) software as shown in Figure (3-7).



Figure(3-7): Computerized Data Collection

To measure the strain in the beams the following procedures were followed: two steel plate were fixed at a constant distance of 100 mm from each side of the beam center and two dial gages (LVDT) were positioned in the opposite direction at the same level as shown in figure (3-6) . Later the difference of reading of the dial gages was used to measure the strain of the beam.

3.6.2. Compressive Strength

The compressive strength was done with (100*100*100) mm cubes and had been carried out for the sum of 72 cubes and had been tested according to (BS 1881 part 116) [60]. Testing machine was a hydraulic compression machine of (2000) kN capacity "ELE digital machine" with a constant force applying rate. The average compressive value of three cubes was taken at each test, and the testing periods were for (28,90 and 150) days for each mix. Each cube was compressed to failure and the compressive strength, then calculated by the following equation:

$f_{cu} = P/A$ ……………………………………………….. (3-1)

Where:

f_{cu}: Compressive strength (MPa).

P: Compressive load at failure (N).

A: Cross-section area (mm^3)**.**

Figure(3-8): Compressive Strength Testing Machine

3.6.3. Splitting Tensile Strength

Splitting tensile strength was conducted in accordance with the (ASTM C496-04)[61] . Cylinders of (100×200) mm were used. After that, two thin plywood strips were placed at the top and bottom edge of a specimen, a hydraulic compression machine of (2000 Kn) is used as shown in Figure (3-9). The test consists of applying a compressive force along the length of a cylindrical concrete specimen until the failure occurs. The average value of three cylinders was calculated at each test. The equation used to determine the splitting strength was:

$$\boldsymbol{fsp = \frac{2P}{\pi d l}} \text{ ....equation (3-2)}$$

Where:

f_{sp}: Splitting tensile strength, (MPa)
P: Max. applied load by the machine, (N)
d: Diameter of cylinder, (mm)
l: Length of cylinder, (mm)

Figure(3-9): Splitting test arrangement

3.6.4. Hardened Concrete Properties

Some physical properties of hardened concrete specimens were measured like specific gravity, absorption and percentage of voids. This was done according to ASTM: C642-82[62]. In this test, four weights were used:

1. Oven dry weight = A
2. Saturated weight after immersion = B
3. Saturated weight after boiling = C

This method is useful in collecting the data required for mass/volume conversions for concrete. It can be used to determine conformity with concrete specifications and to show variability from place to place within a mass of concrete. To determine the mentioned properties for each mix by taking the weights of three samples of (100*100*100) mm cube to get the necessary data :

1.Absorption after immersion, % = $[(B - A) / A] \times 100$ …………….. Eq (3-3)

2.Absorption after immersion and boiling, % = $[(C - A) / A] \times 100$…….Eq(3-4)

3.6.5 Hardened Density Test

Cubes of (100*100*100) mm were used to determine density. The specimens were immersed in tap water for (24) hours, and the immersed mass was determined. Then the specimens were taken to the oven at a temperature of (100-110) C° for (24) hours, then the dried mass was determined. This test was carried out according to (ASTM C 138-01). density was calculated using the following equations:

$\gamma wet = W\ wet / V$ …………. (3-5)

Where:

γ wet: wet density of HPC, (gm/mm^3).

W wet: immersed mass, (gm).

V : volume (mm^3).

Results and Analysis

4.1. General

This chapter devoted to introduce and discuss the experimental test results which have been collected through various structural and mechanical tests as mentioned earlier in the previous chapter. This research studied the effect of internal curing in two main sides : the first one studied the structural behavior of fifteen reinforced concrete beams by evaluating three parameters: ultimate load capacity, toughness and ductility index.

Then, compare the improvement of these values according to three variables: the internal curing materials, the percentage of replacement and the effect of age. The second side of the tests was for the mechanical properties of the concrete which studied the effect of internal curing on compressive strength, splitting tensile strength and density and absorption of the concrete.

Three variables used to discuss and study the effect on the concrete properties mentioned above these variables were: the effect of certain internal curing material where two materials used as partial replacement of fine aggregates (crushed brick and limestone), the percentage replacement of internal curing materials where two percentages used for each material (5% and 10%) and the effect of time on these properties by conducting all the mentioned tests at three different ages (28,90 and 150) days.

4.2. Structural Behavior of Internally Cured Concrete Beams

Loads that are applied to beams may either cause the beam to fail in shear, flexural tension or by concrete crushing. To prepare the reinforced concrete beam to reach the full flexural strength, shear stirrups were provided to support the shear capacity of the concrete. The compatibility of the concrete and the reinforcement causes them to deflect together. In a beam designed to fail in flexure, flexural cracks begin to develop within concrete at a point of high moments (middle third of simply supported beam) when loading exceeds the flexural strength of concrete. From that point, the tensile reinforcement proceeds with load carrying. Fifteen beams (1700*150*100) mm were tested by four points load test, each beam was loaded to failure with recording the deflection at midspan and strain with each loading step as mentioned in chapter three.

The collected data of the structural test for each beam were used to obtain three structural parameters, these parameters used to assess the structural behavior of beams for all the mix groups. The parameters and the approach of calculating them are illustrated in the following subsections :

The Ultimate and Cracking load capacity To examine the behavior of the beam at failure for each beam of the mixes groups at ages of (28,90 and 150) days, the loads were measured by loading cell connected to a computer program. LVDTs were used to measure the deflection at midspan and under the loads and another LVDTs to measure the strain also. Each beam had been loaded symmetrically at the mid span and the applied load was divided by load spreading steel girder to spread the load equally at one third from the support.

The second parameter was **Ductility Index** which is a measure of the element capacity to endure inelastic behavior and absorb energy. Several forms of ductility are available. These include curvature, rotational, and displacement ductility. In this research, displacement ductility is investigated. Displacement ductility, ductility index **D.I** is defined as the ratio of deflection at ultimate moment of the specimen to the deflection at first yielding of the tensile reinforcement [63].

The higher the ductility index indicates a high ability of the member to absorb loads before failure. The deflection of the beams was measured at midspan of the beam using LVDT for every loading step of the beam to the stage of maximum failure load. It is obvious that the values of ductility index for the present work ranges between (3.6 and 2.1) as shown ib Table(4-2). The ductility index is strongly affected by steel ratio of the member and since all beams have the same steel ratio then the differences may be referred to the concrete mixes only. *Ashour* [64] mentioned that the displacement ductility, in the range of 3 to 5 is considered imperative for adequate ductility, especially in the areas of seismic design and the redistribution of moments. Therefore, assuming that a D.I value of 3 represents an acceptable lower limit to ensuring the ductile behavior of flexural members.

The third parameter was **Toughness**. The toughness of a material can be found by calculating the total area limited below the stress–strain curve. This area gives an insight of the amount of energy per unit volume that the material can support up to rupture. Strain of the beams had been measured by means of two LVDTs fixed at (100 mm) of the center line of the beam at the bottom of the beam from each side of the beam centerline. Thus, the total distance between the two LVDTs was 200 mm.

The readings of the these LVDTs were divided by the distance between them at each loading step to find the strain in the concrete and drawing the stress-strain curve. The stress was calculated from the applied load and by applying the gross moment of inertia (I_g) for loads before first crack and using the actual moment of inertia (I_e) after the cracking load which is given by the following formula from (ACI 318-14) [59] :

$$Ie = \left(\frac{Mcr}{Ma}\right) Ig + \left[1 - \left(\frac{Mcr}{Ma}\right)^3\right] Icr \quad \text{....................................Eq. (4-1)}$$

Where:

I_e : effective secant moment of inertia of the entire beam at any load level (mm^4).

I_g : gross transformed section moment of inertia (mm^4).

I_{cr} : cracked section moment of inertia (mm^4).

M_{cr} : cracking moment (N.mm).

M_a : maximum moment in the beam (N.mm).

4.3. Effect of Internal Curing Materials on Reinforced Concrete Beams

In this part of the thesis all of the data from structural tests were presented as shown in Table (4-1) and (4-2) and Figures (4-1) to (4-3) in details.

Table (4-1) : Results of Deflection and Load Capacity of Tested Beams

Beam groups	beam	Age (day)	P. cr kN	Δcr *mm*	P. yld kN	Δy *mm*	P. ult kN	Δult *mm*
Reference	R1	28	23.8	2.09	44	6	56.2	21.77
	R2	90	30.9	2.98	52	6.4	58.3	21.02
	R3	150	36.4	1.65	55	5.4	62.6	18
Crushed Brick (5%)	BA1	28	28.1	2.25	42	6	50.4	15
	BA2	90	15.8	2.13	42	6.6	52.5	19
	BA3	150	21.9	2.3	48	5.5	64.8	14.36
Crushed Brick (10%)	BB1	28	34.5	3.5	53	5.5	62.6	15.5
	BB2	90	34.9	3	59	6.1	69.1	17
	BB3	150	28.8	1.63	58	5.2	70.2	12.96
Limestone (5%)	GA1	28	35.3	3.23	56	5.9	62.6	15.5
	GA2	90	27	3.75	62	6.5	63	15
	GA3	150	36	3.01	64	6	68.4	13.22
Limestone (10%)	GB1	28	27.12	1.71	55	6.5	60.12	17
	GB2	90	18	2.95	68	5.9	72	16.69
	GB3	150	26.3	2	63	6.1	70.5	15.7

Table (4-2) : Structural Results of Tested Beams

Beam groups	beam	P.Ultimate (kN)	D.I	Toughness (*MPa*)	Ult. load variation (%)
Reference	R1	56.2	3.6	0.256	-
	R2	58.3	3.3	0.311	-
	R3	62.6	3.3	0.304	-
Crushed Brick (5%)	BA1	50.4	2.5	0.201	-10.3
	BA2	52.5	2.9	0.271	-9.9
	BA3	64.8	2.6	0.285	3.4
Crushed Brick (10%)	BB1	62.6	2.8	0.263	11.5
	BB2	69.1	2.8	0.261	18.5
	BB3	70.2	2.5	0.348	12.1
Limestone (5%)	GA1	62.6	2.6	0.321	11.5
	GA2	63	2.3	0.310	8
	GA3	68.4	2.2	0.320	9.2
Limestone (10%)	GB1	60.12	2.6	0.278	7.1
	GB2	72	2.8	0.351	23.5
	GB3	70.5	2.6	0.338	12.6

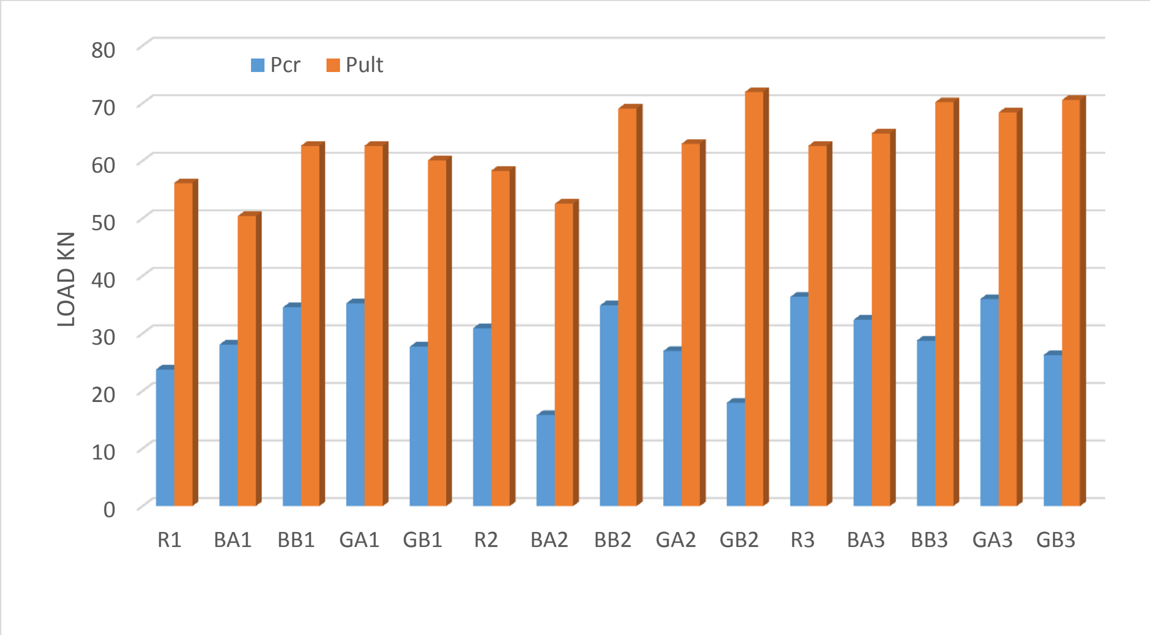


Figure (4-1): Ultimate Load Capacity and Cracking Load for All Beams

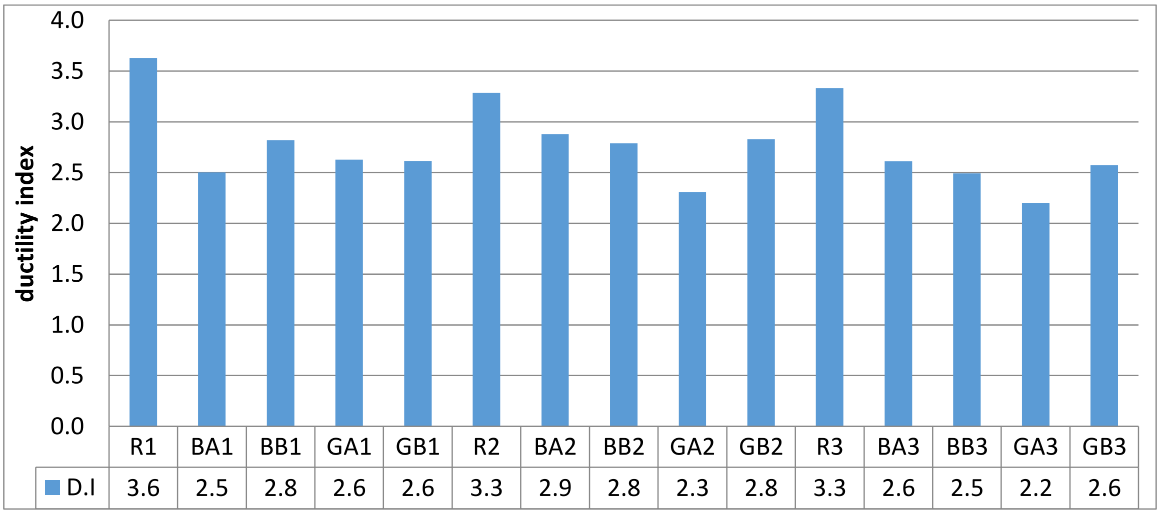


Figure (4-2): Ductility Index for All Beams

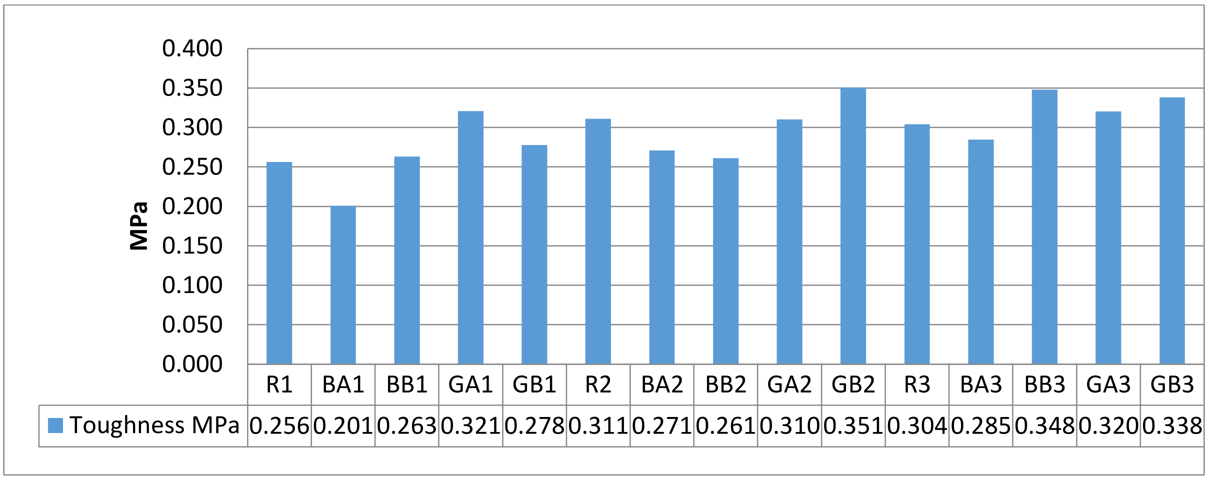


Figure (4-3): Toughness for All Beams (MPa)

Figure (4-1) shows the initial crack load and ultimate cracking load until failure for each beam that represent the mixes at various ages. As shown in table (4-1) beam GB3 had the highest ultimate load capacity of 70.6 kN.

The lowest load capacity was 50.4 for the beam BA1. The beam R3 had the highest cracking load of 36.4 kN. The lowest cracking load was of the beam BA2 by 15.8 kN. It is shown from the results that there is a normal increase in the maximum load for the beams that represent each mix with respect to the age of test and that was expected due to the fact that the concrete gains strength with time, as it is obvious for the reference mix which had (56.2,60.1 and 62.6) kN at test ages of (28, 90 and 150) days, respectively.

The results showed that mixes of 5% brick exhibits lower load capacity than reference mix at early ages but showed higher strength at 150 days. This behavior might be attributed to the effect of LWA replacement, which is not as strong as ordinary aggregate. However, at late ages the effect of internal curing showed improvement of the mixes which overcome the weakness of LWA.

The other mixes (10% brick, 5% and 10% limestone) showed noticeable improvement compared to the reference mix at each age. The ductility results of the beams shown in Figure (4-2) indicates that the reference mix showed higher values compared to other mixes (3.6,3.3 and 3.3) while other mixes had ductility index less than 3, which means that the internal curing materials affect negatively on the ductility of the concrete, though BA2 showed a relatively high ductility index of 2.9.

As for the toughness of the mixes the reference mixes had (0.311) MPa and the highest toughness was for beam GB2 of (0.351) MPa.

4.3.1. Effect of Crushed Brick

Two mixes of crushed brick replacement percentage were tested (5% and 10%) each mix was divided into three beams, which represent the age of the test at (28,90 and 150) days.

Figure (4-4) through (4-11) shows the load – deflection curves of all the brick mixes which is useful to obtain some structural properties like: maximum load capacity and ductility index of the beam.

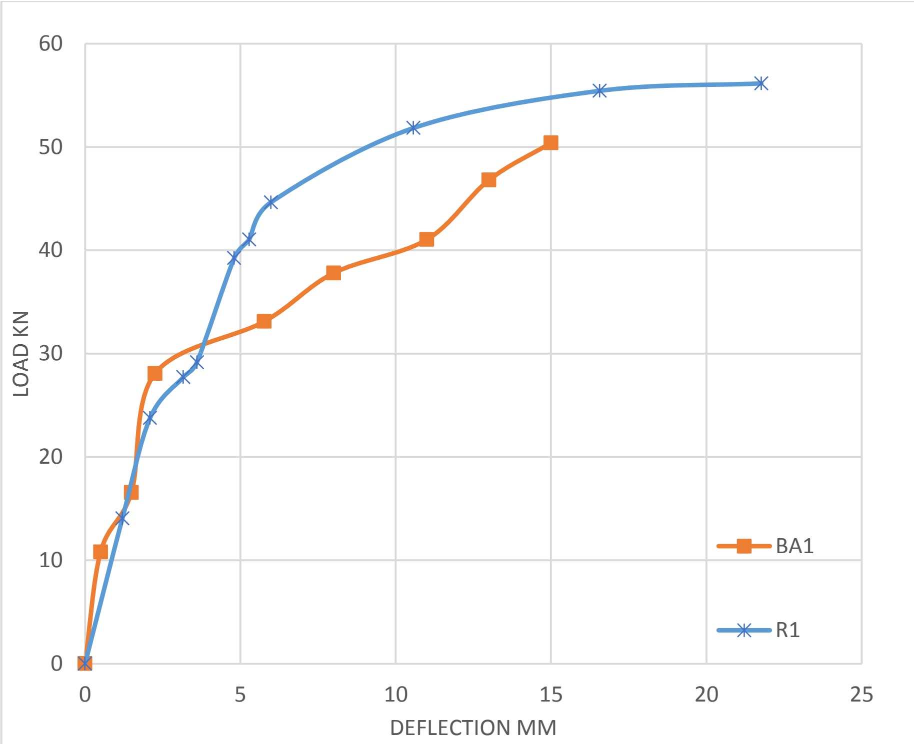


Figure (4-4): Deflection of 5% Brick Mix Meam at 28 Days

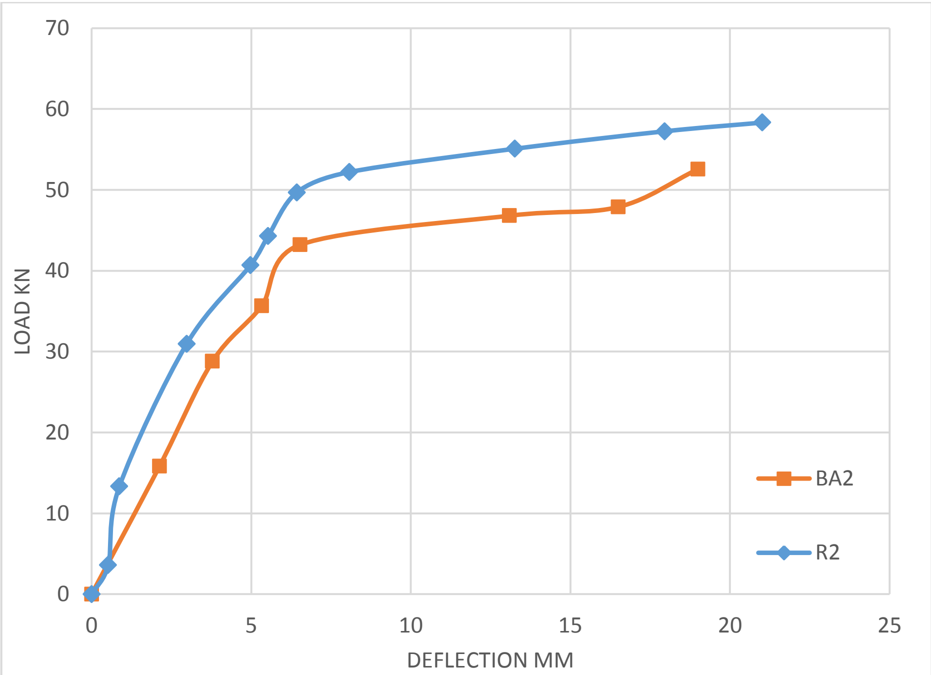


Figure (4-5): Deflection of 5% Brick Mix Beam at 90 Days

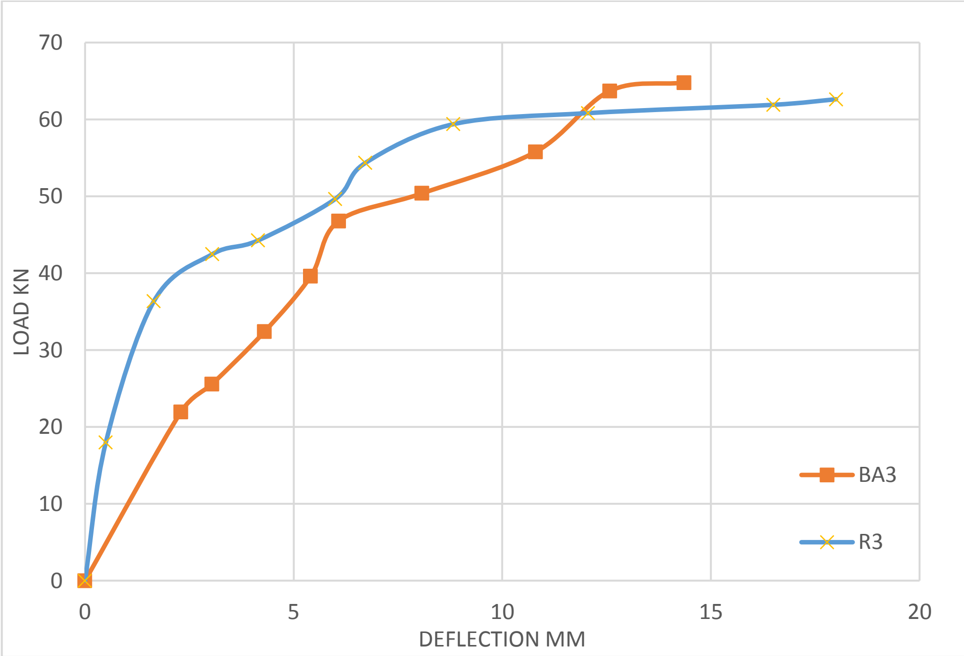


Figure (4-6): Deflection of 5% Brick Mix Beam at 150 Days

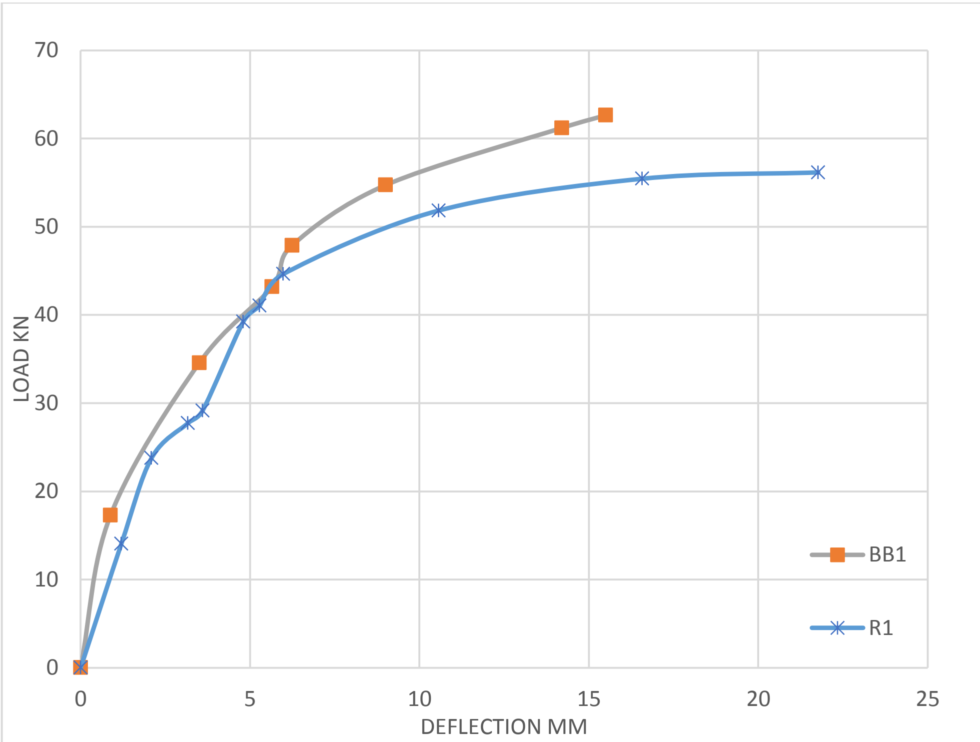


Figure (4-7): Deflection of 10% Brick Mix Beam at 28 Days

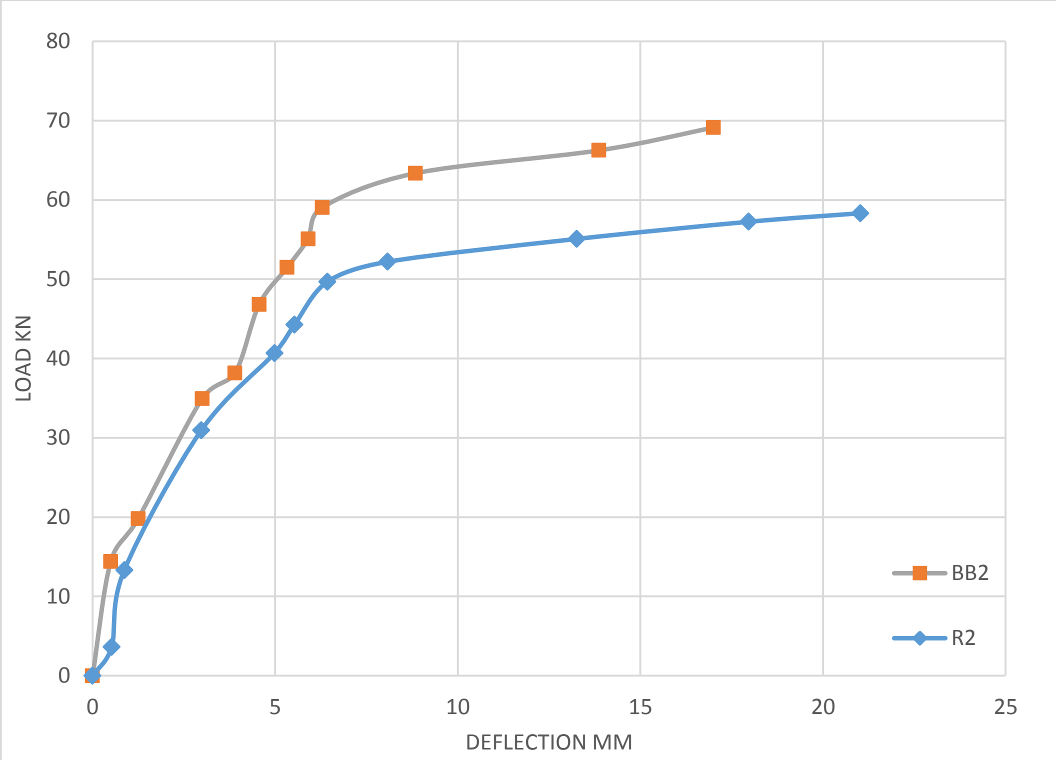


Figure (4-8): Deflection of 10% Brick Mix Beam at 90 Days

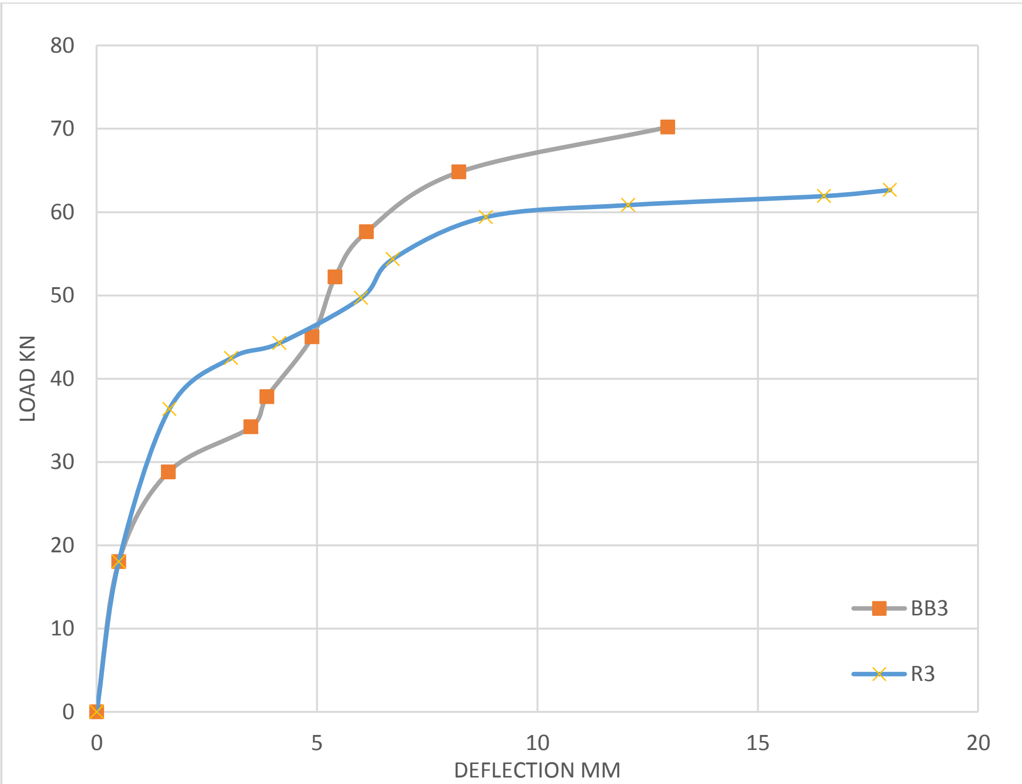


Figure (4-9): Deflection of 10% Brick Mix Beam at 150 Days

The ultimate load capacity for all the specimens internally cured with crushed brick were ranging between (50.2 and 70.2) kN. The minimum value was for the beam BA1 while the greatest one had been gained by BB3. In general the 5% brick mixes exhibited a lower load capacity than 10% brick mixes which were (50.4,52.5 and 64.8) kN for the test ages (28,90 and 150) days respectively . While 10% mixes had (62.64, 69.1 and 70.2) kN for the test ages (28,90 and 150) days, respectively.

It is clear from the figures (4-4) to (4-9) that all the tested beams of crushed brick materials had low ductility index than the reference beams with a range of (2.5 to 2.9). Moreover, the 5% mix at age 90 days had the highest D.I of all the brick mixes (2.9) which is close to the ductility of the reference beams , while 10% brick beam at 150 day had a value of 2.5.

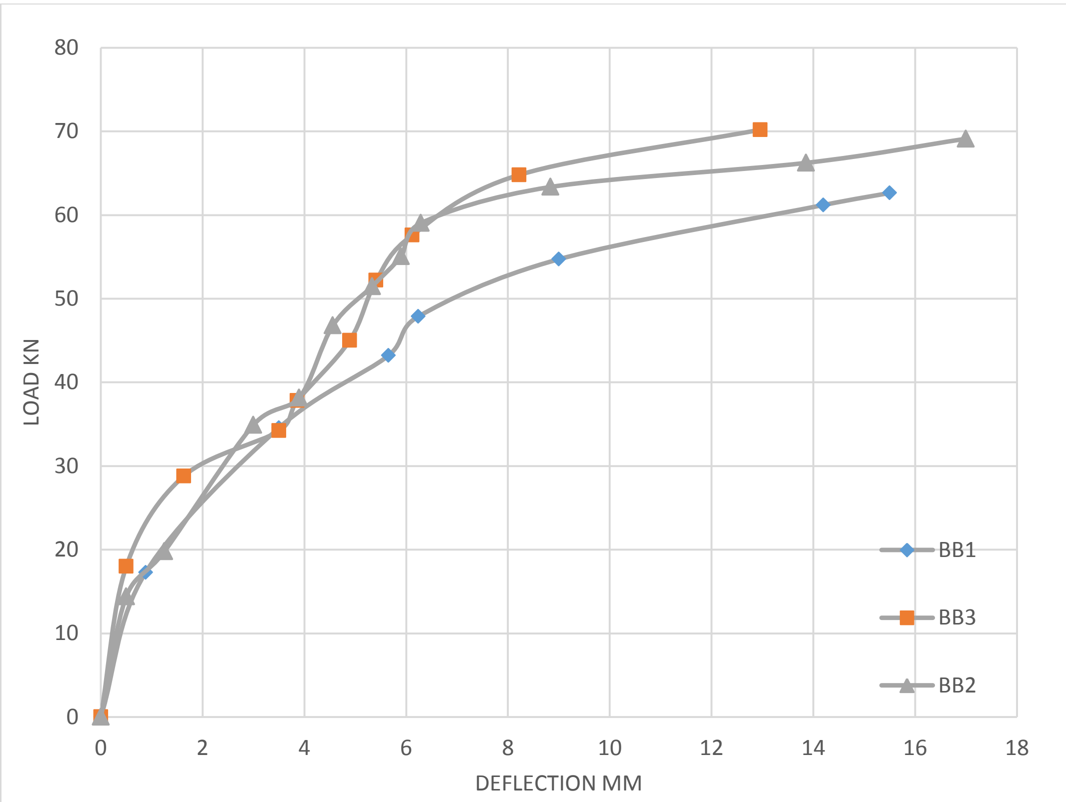


Figure (4-10): Deflection of 10% Brick Mix Beams Comparison

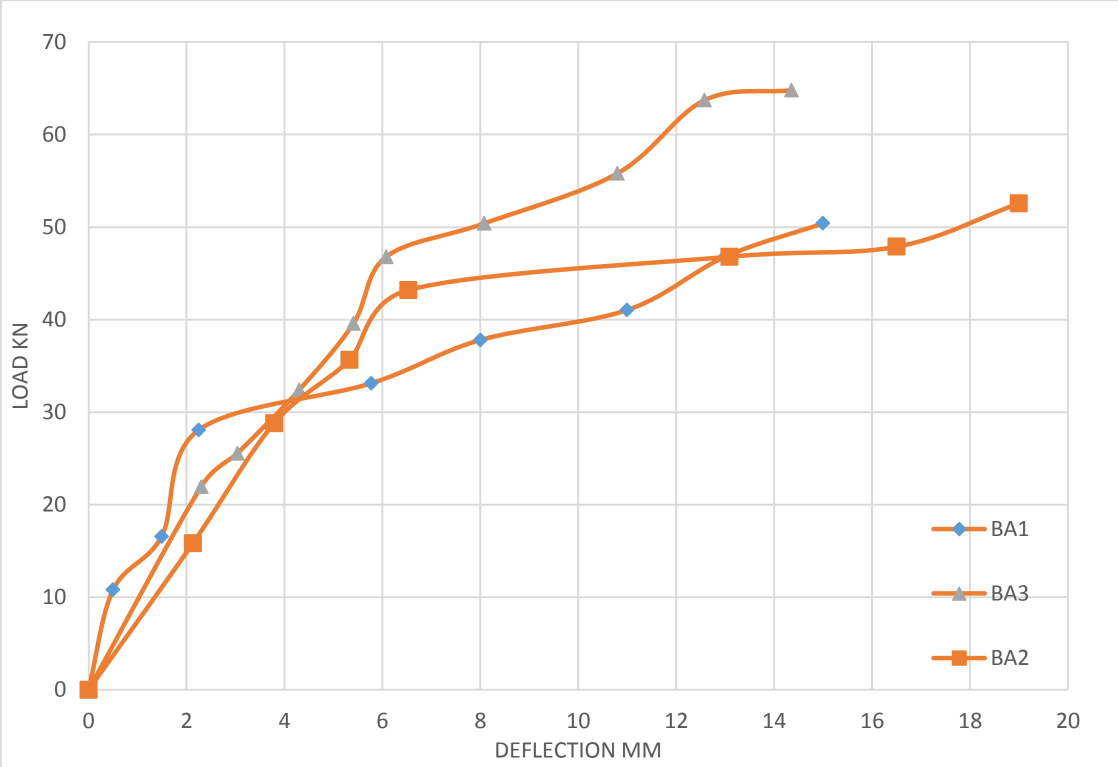


Figure (4-11): Deflection of 5% Brick Mix Beams Comparison

The stress and strain data were used to draw the stress- strain curve for every beam to calculate the toughness. Figures (4-12) to (4-19) show the stress-strain curve for all the brick mixes and the toughness were calculated by finding the area under the curve for each beam by using Auto-CAD software.

The toughness was measured in MPa and it was varied for the brick mixes between 0.201 and 0.348. The mixes of 10% brick had the highest values of (0.263, 0.261 and 3.48) MPa for the testing ages (28,90 and 150) days, respectively. While the 5 % brick mixes had (0.201, 0.271 and 0.320) MPa for the three testing ages (28,90 and 150) days, respectively. The results showed that the 10% replacement provides better structural responce.

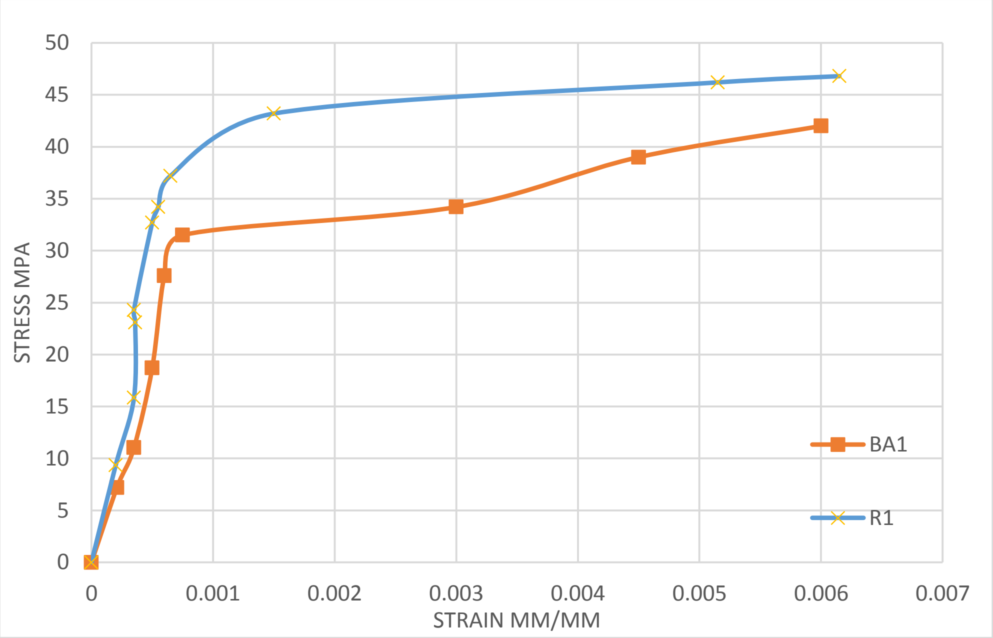


Figure (4-12): Stress-Strain of 5% Brick Mix Beam at 28 Days

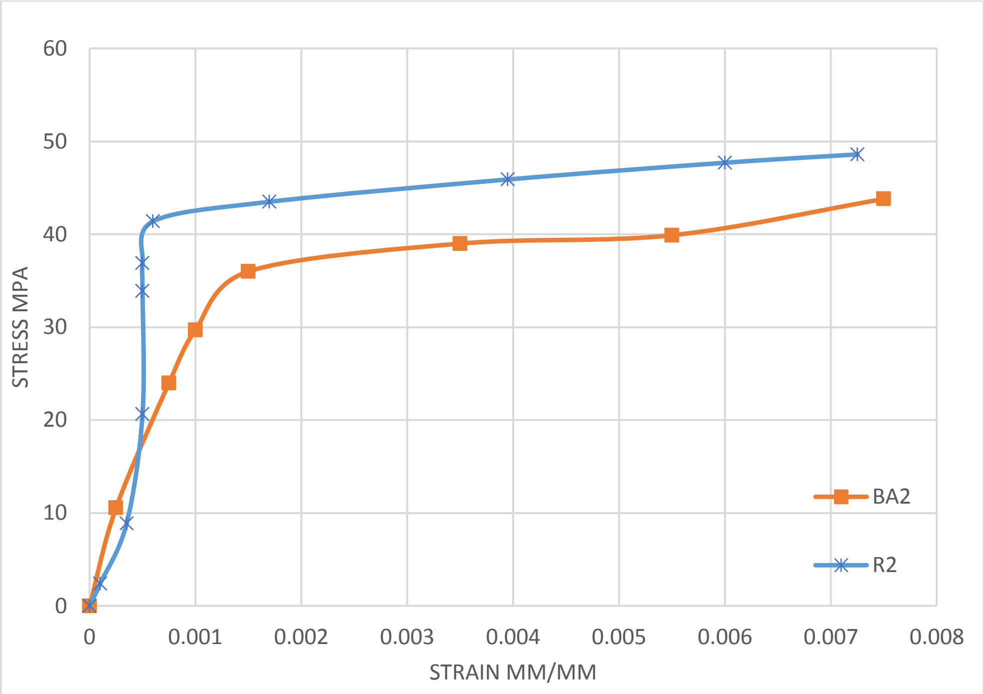


Figure (4-13): Stress-Strain of 5% Brick Mix Beam at 90 Days

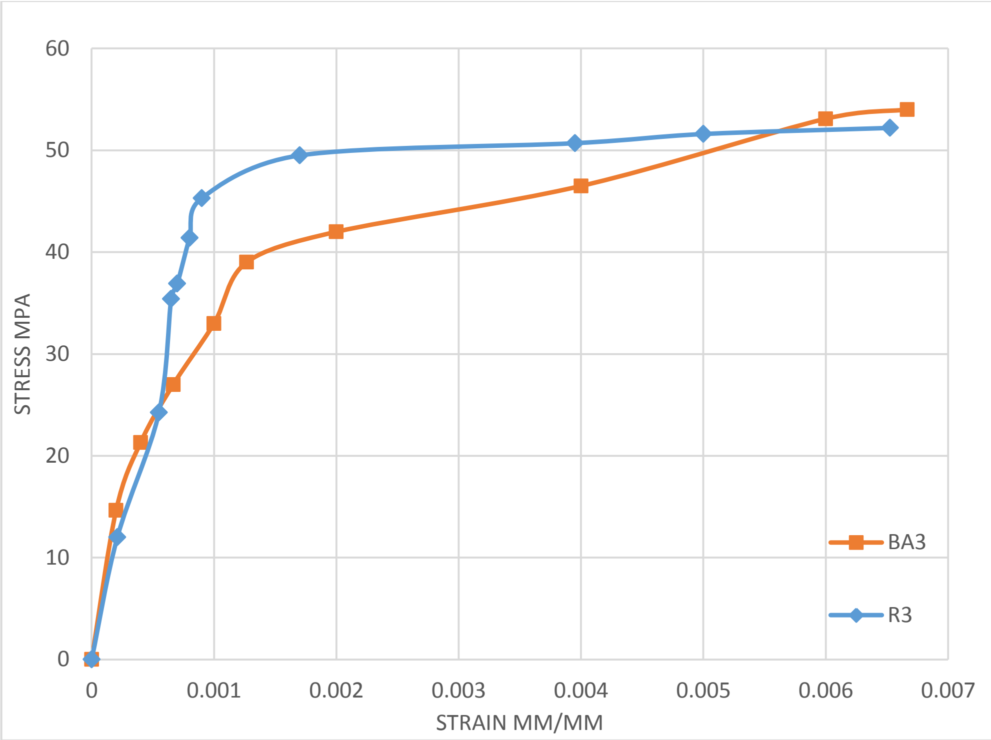


Figure (4-14): Stress-Strain of 5% brick mix beam at 150 days

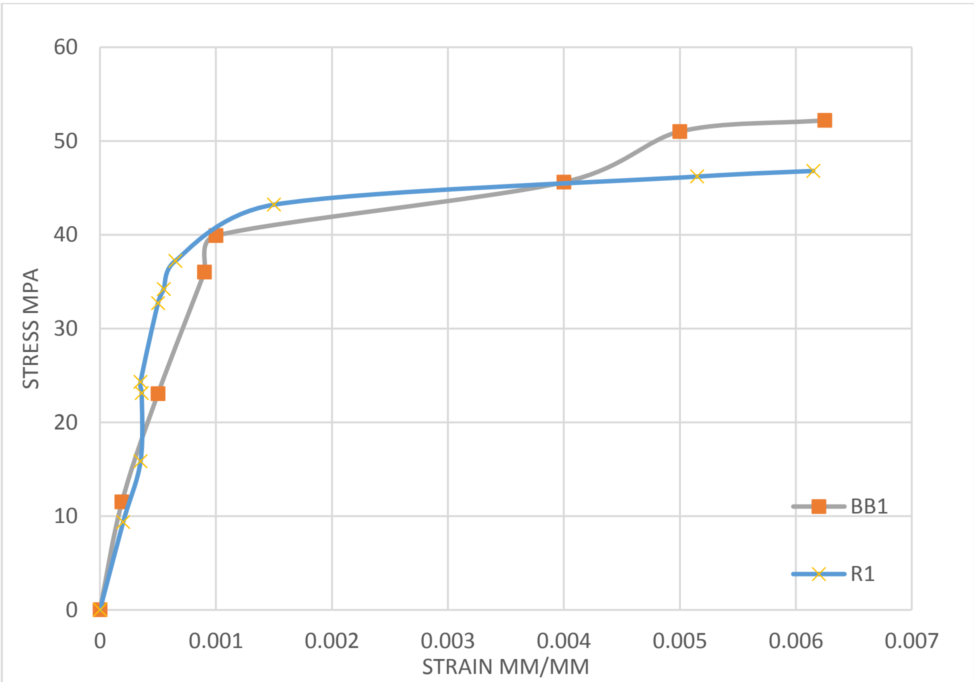


Figure (4-15): Stress-Strain of 10% brick mix beam at 28 days

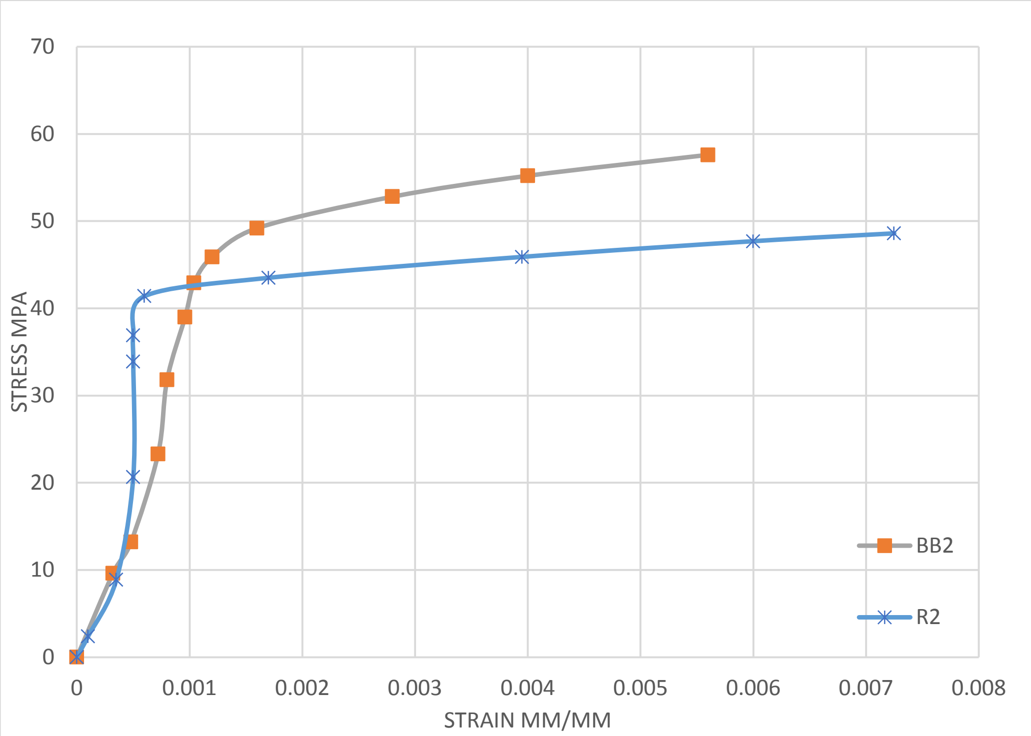


Figure (4-16): Stress-Strain of 10% brick mix beam at 90 days

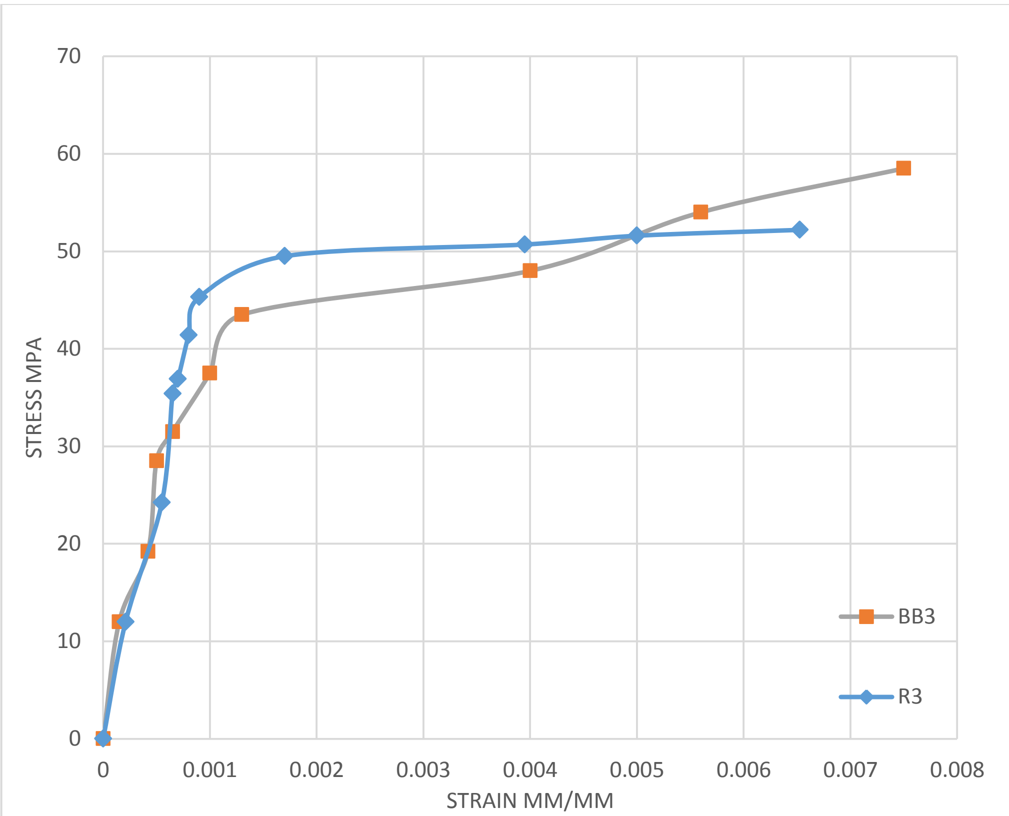


Figure (4-17): Stress-Strain of 10% brick mix beam at 150 days

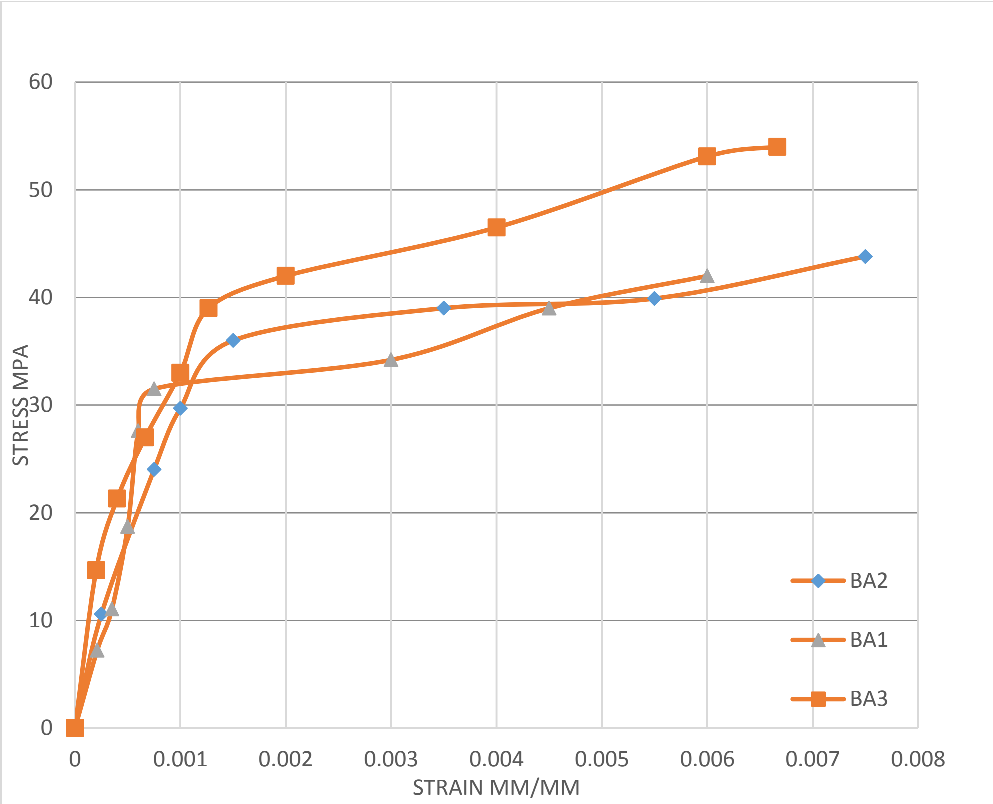


Figure (4-18): Stress-Strain of 5% brick mix beams comparison

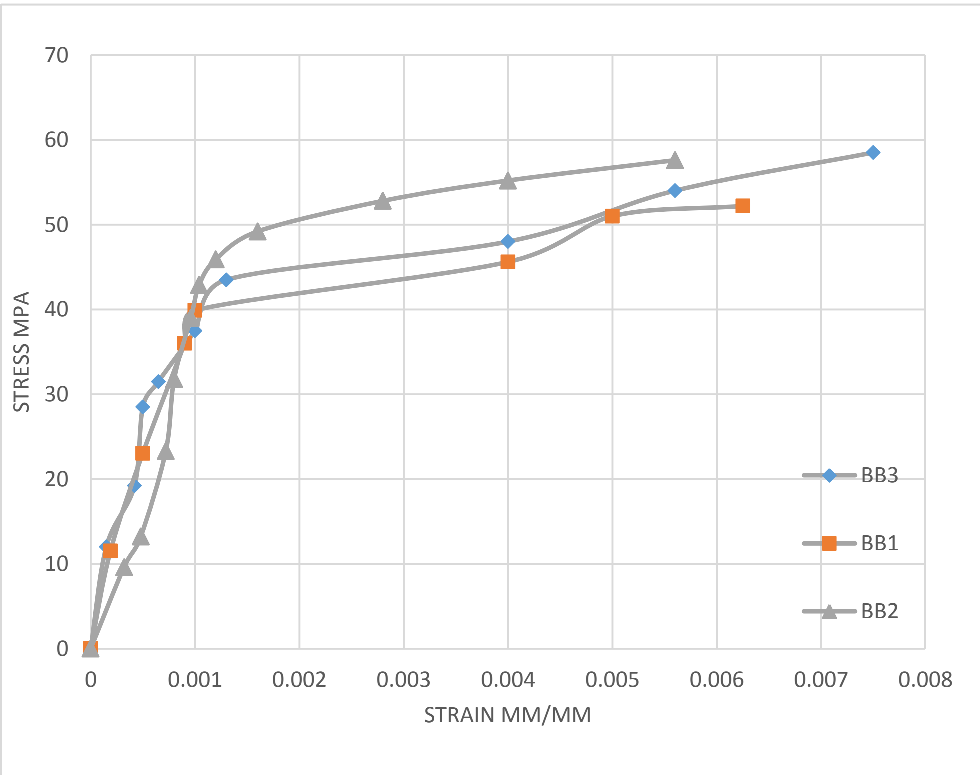


Figure (4-19): Stress-Strain of 10% brick mix beams comparison

4.3.2. Effect of Limestone

The two limestone replacement percentages of 5% and 10% were mixed through this work. Each group consists of three beams tested at 28,90 and 150 days. Figure (4-16) to (4-23) show the load-deflection of the mixes internally cured with limestone. The maximum loading capacity was between (60 and 72 kN) and the beam GA2 had the highest ultimate load capacity in this group of 72 kN.

From the comparison analysis between the ultimate loads results of the two groups of limestone beams, it can be noticed that the 5% limestone mix GA1 had higher load capacity of 62.6 kN compared to 60.1 kN for GB1 at 28 days. Despite that, at the ages of 90 and 150 days the 10% limestone mix had higher loading capacity of 72 and 70.6 respectively. This trend gives this group an advantage in the durability.

From Table (4-1), it is shown that the ductility index of the 10% limestone was higher than the 5% limestone group, it had ductility index of (2.6, 2.8 and 2.6) for the (28,90 and 150) days ages, respectively. While the other mix (5% limestone) was (2.6,2.1 and 2.2) for the same three ages.

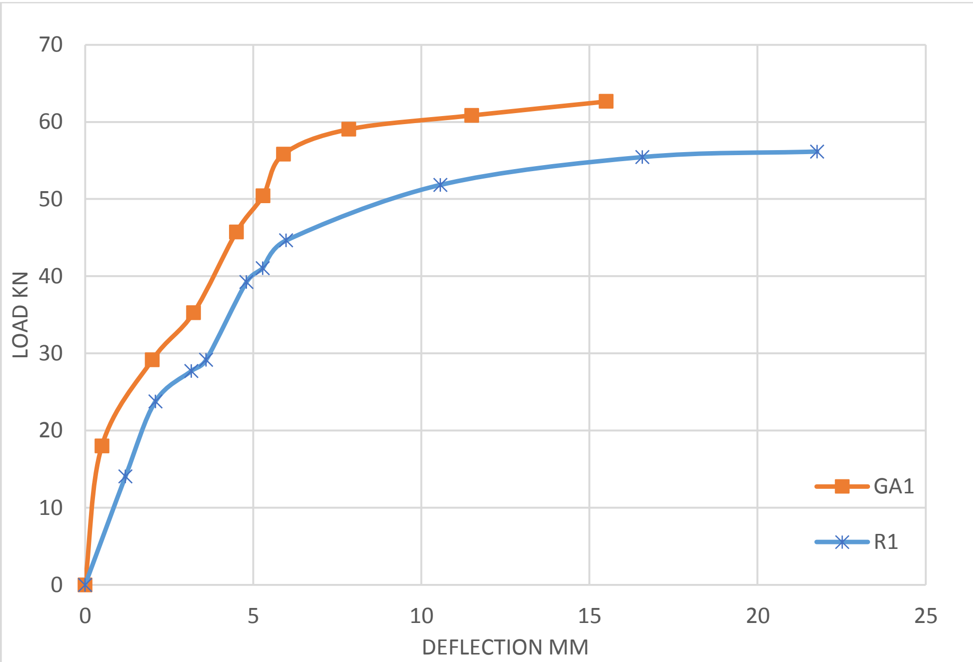


Figure (4-20): Deflection of 5% limestone mix beam at 28 days

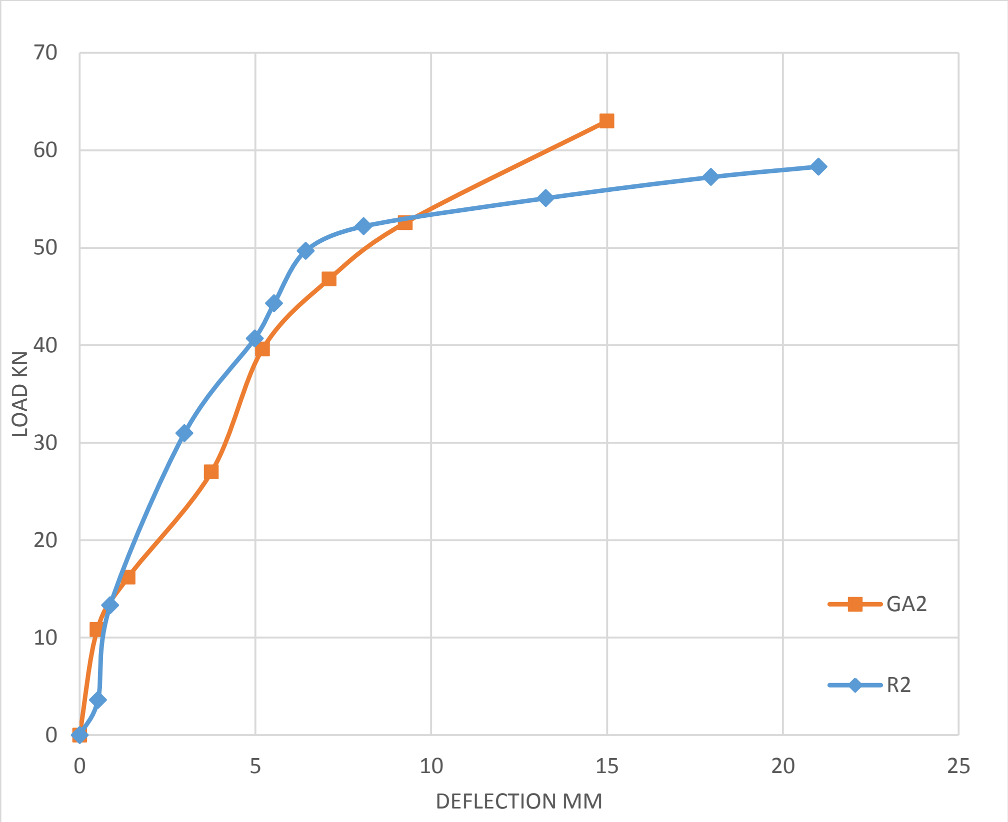


Figure (4-21): Deflection of 5% limestone mix beam at 90

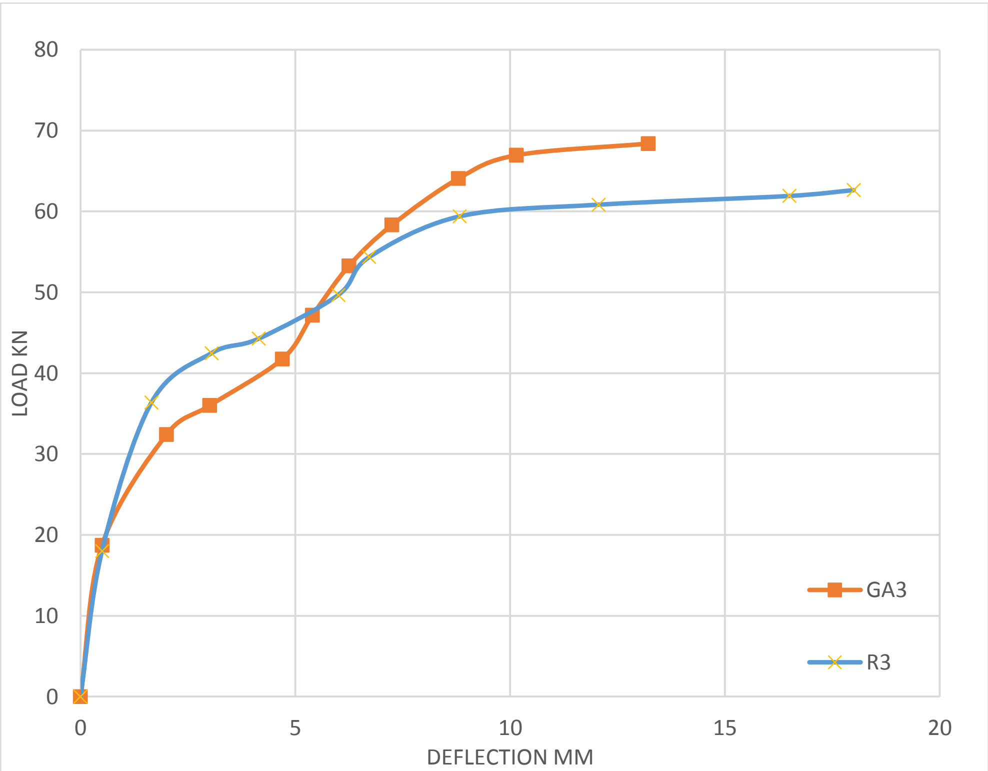


Figure (4-22): Deflection of 5% limestone mix beam at 150 days

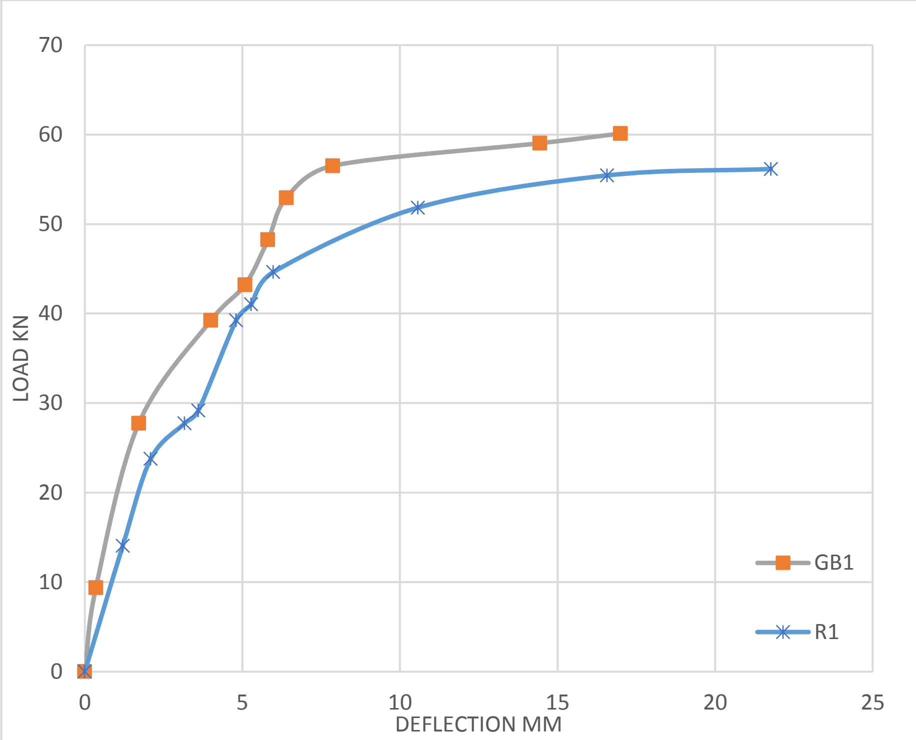


Figure (4-23): Deflection of 10% limestone mix beam at 28 days

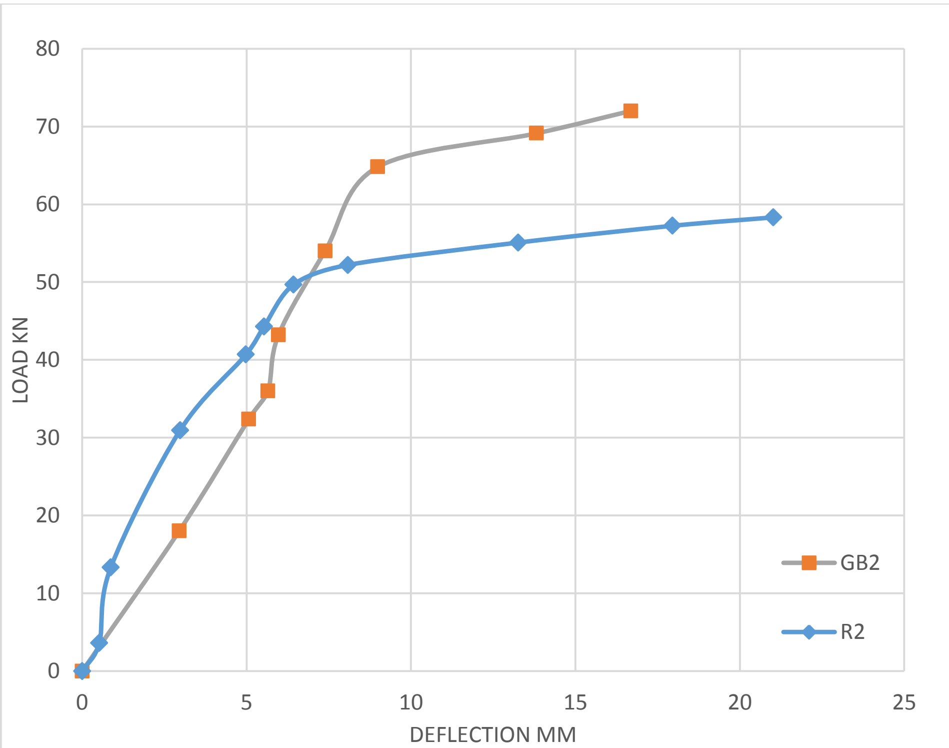


Figure (4-24): Deflection of 10% limestone mix beam at 90 days

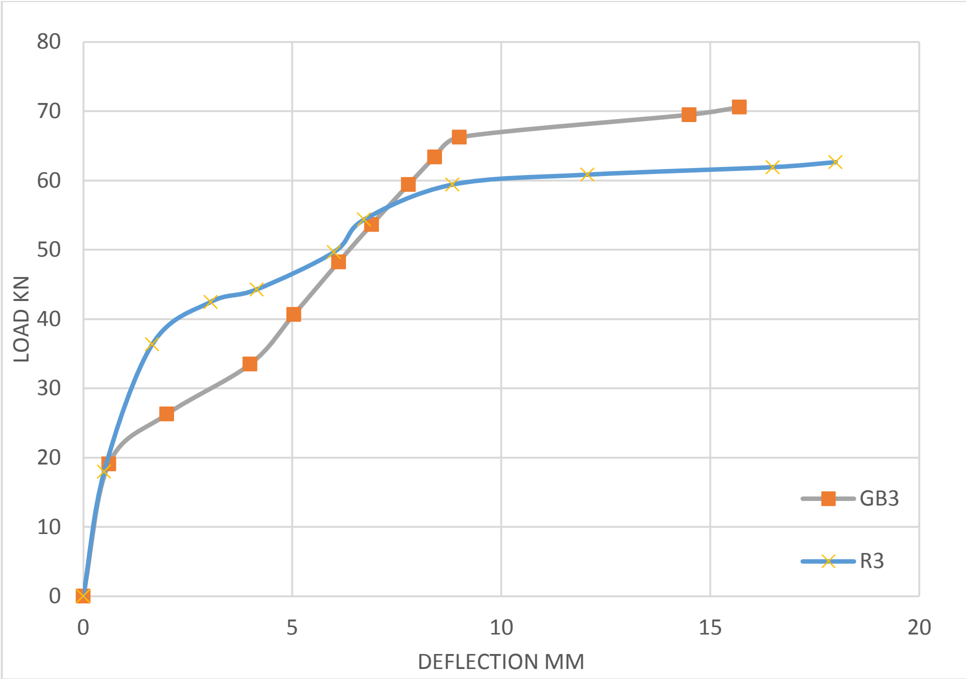


Figure (4-25): Deflection of 10% limestone mix beam at 150 days

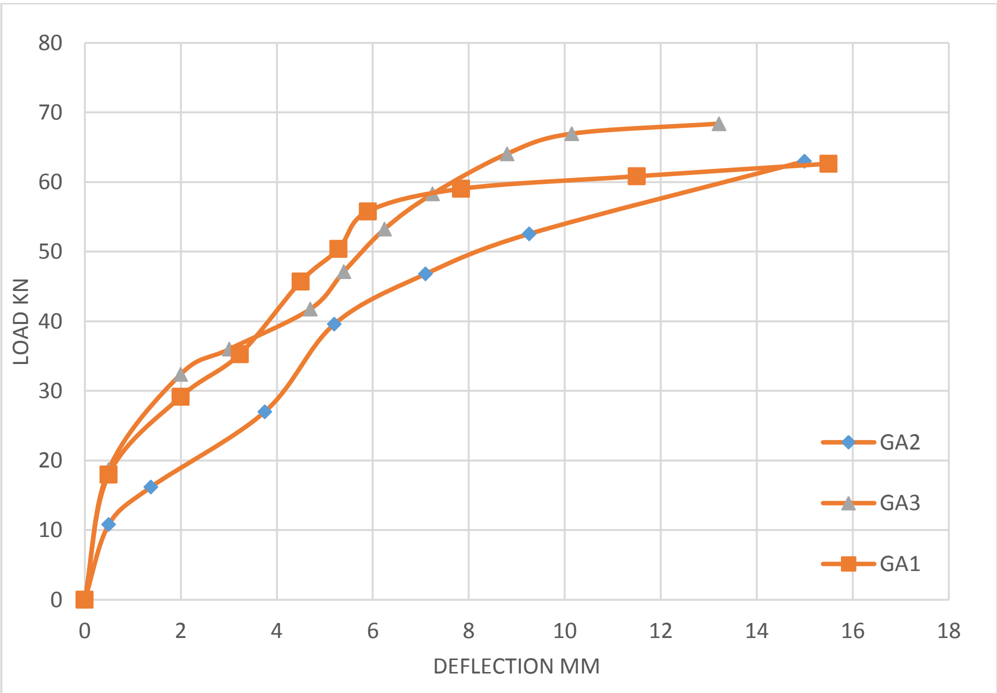


Figure (4-26): Deflection of 5% limestone mix beams comparison

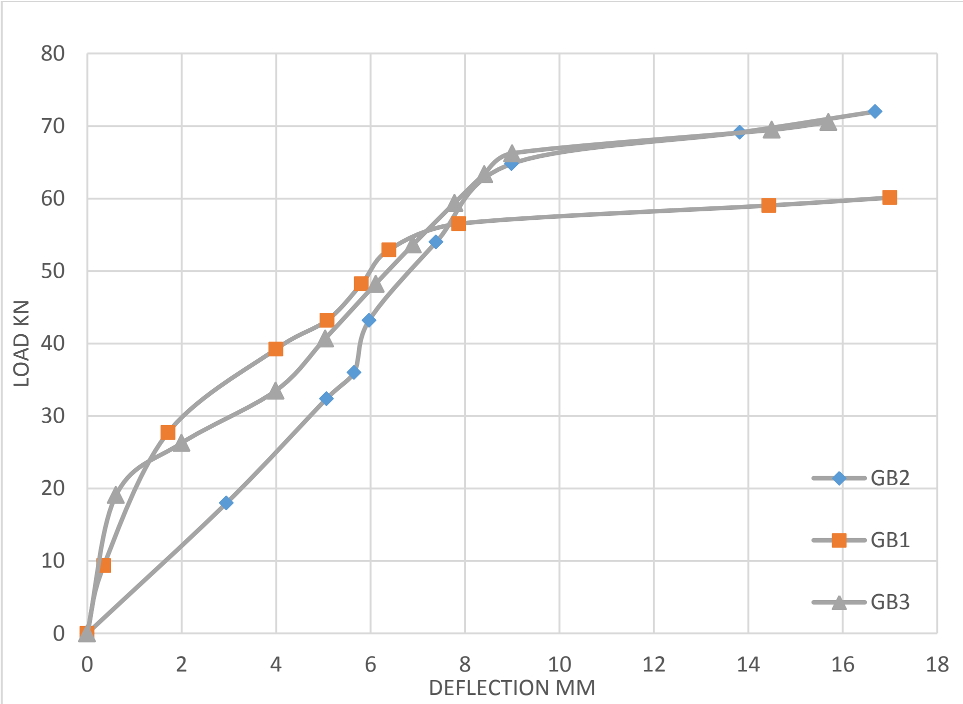


Figure (4-27): Deflection of 10% limestone mix beams comparison

The stress-strain curves of the internally cured limestone beams are shown in figures (4-24) to (4-31). As mentioned before this diagrams is used to calculate toughness of the beams.

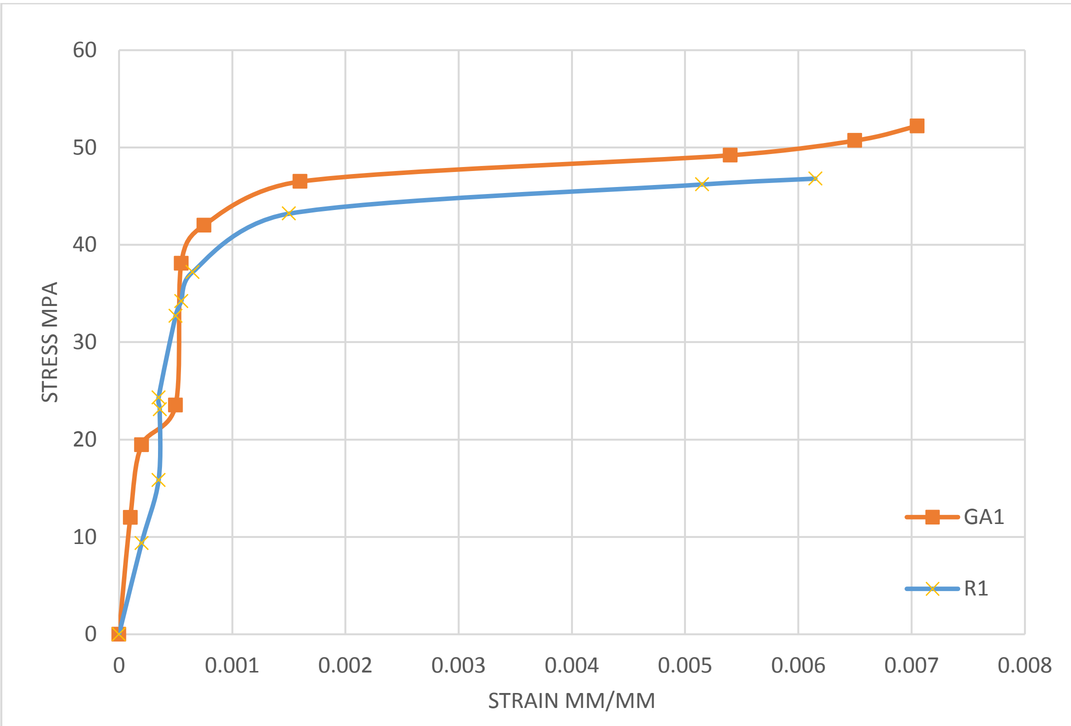


Figure (4-28): Stress-Strain of 5% limestone mix beam at 28 days

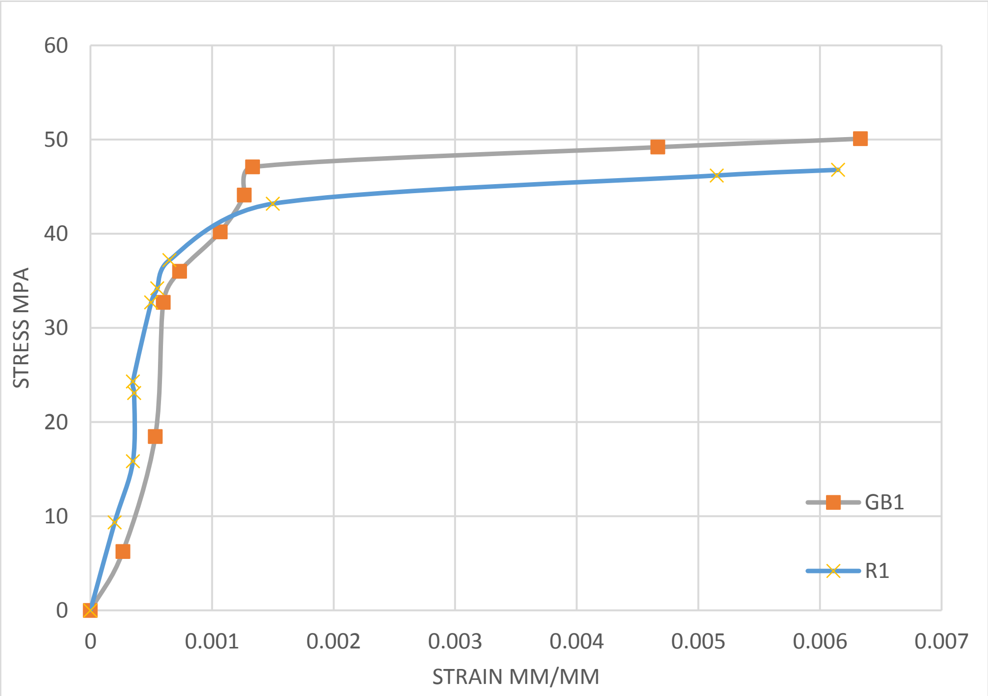


Figure (4-29): Stress-Strain of 10% limestone mix beam at 28 at 28 days

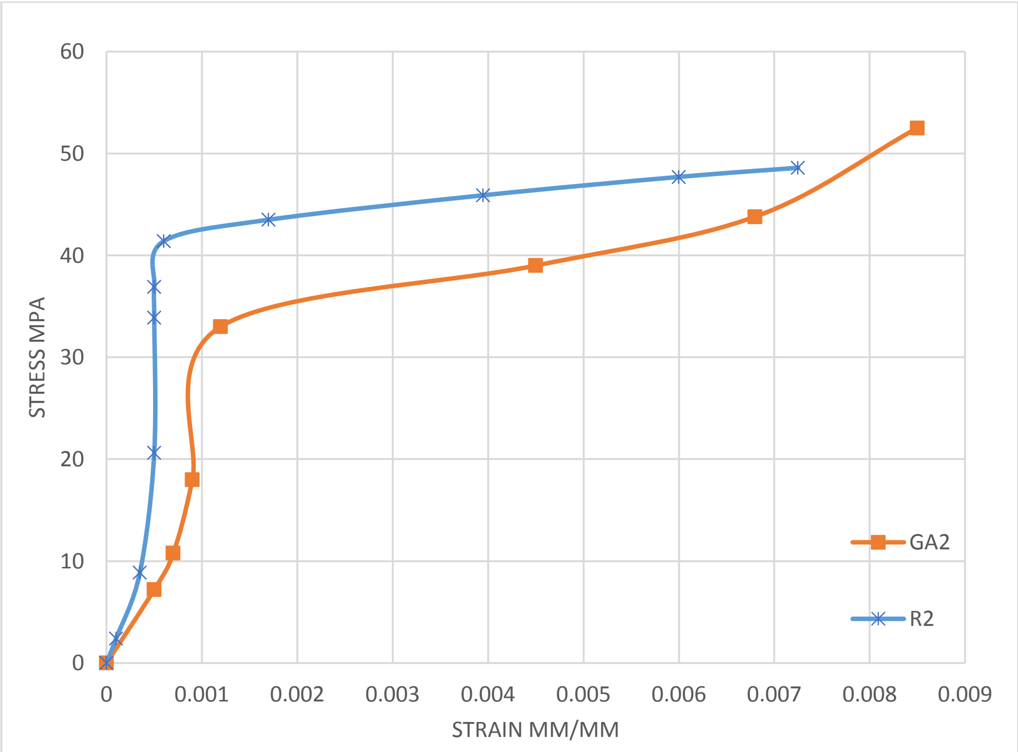


Figure (4-30): Stress-Strain of 5% limestone mix beam at 90 days

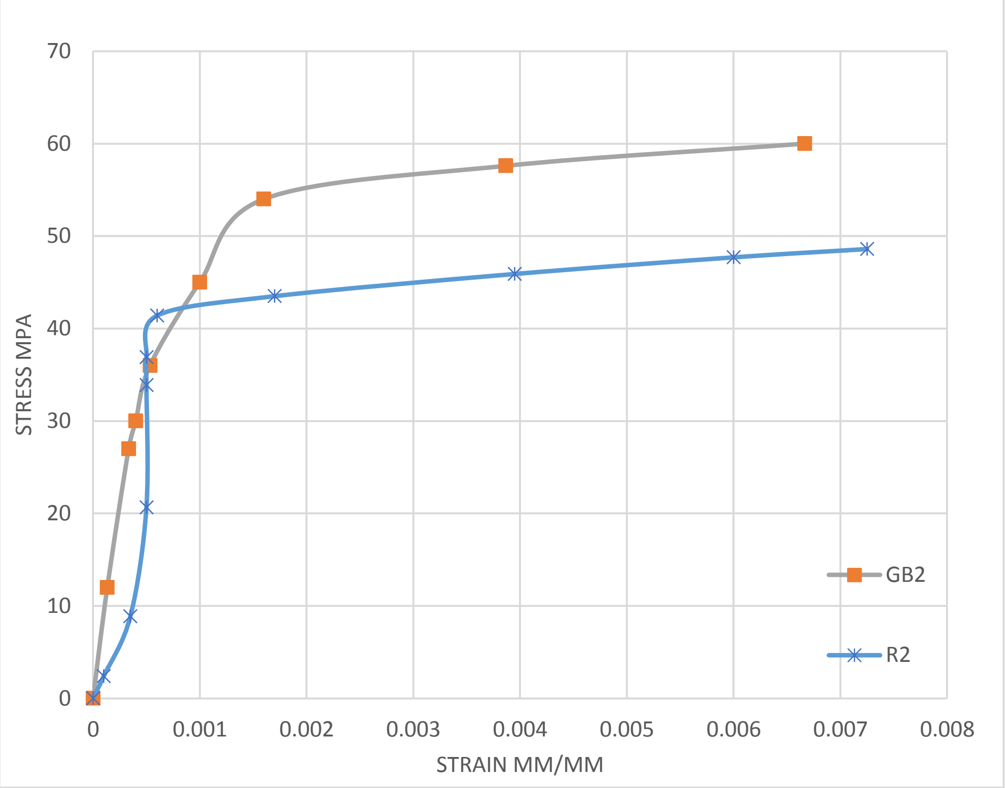


Figure (4-31): Stress-Strain of 10% limestone mix beam at 90 days

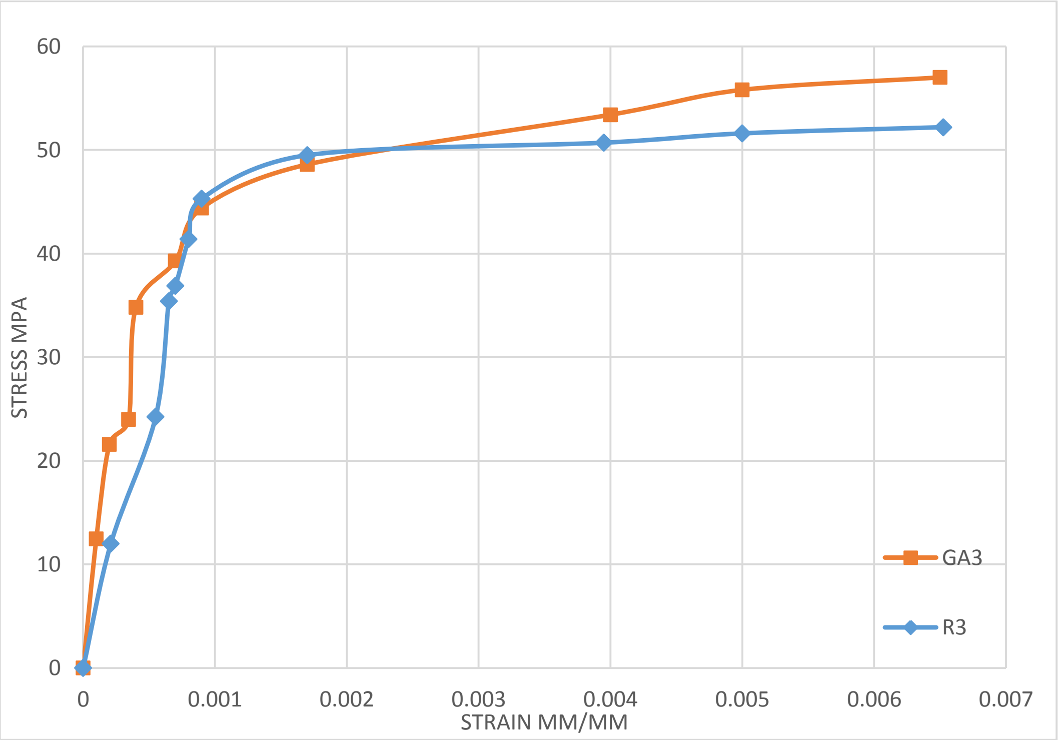


Figure (4-32): Stress-Strain of 5% limestone mix beam at 150 days

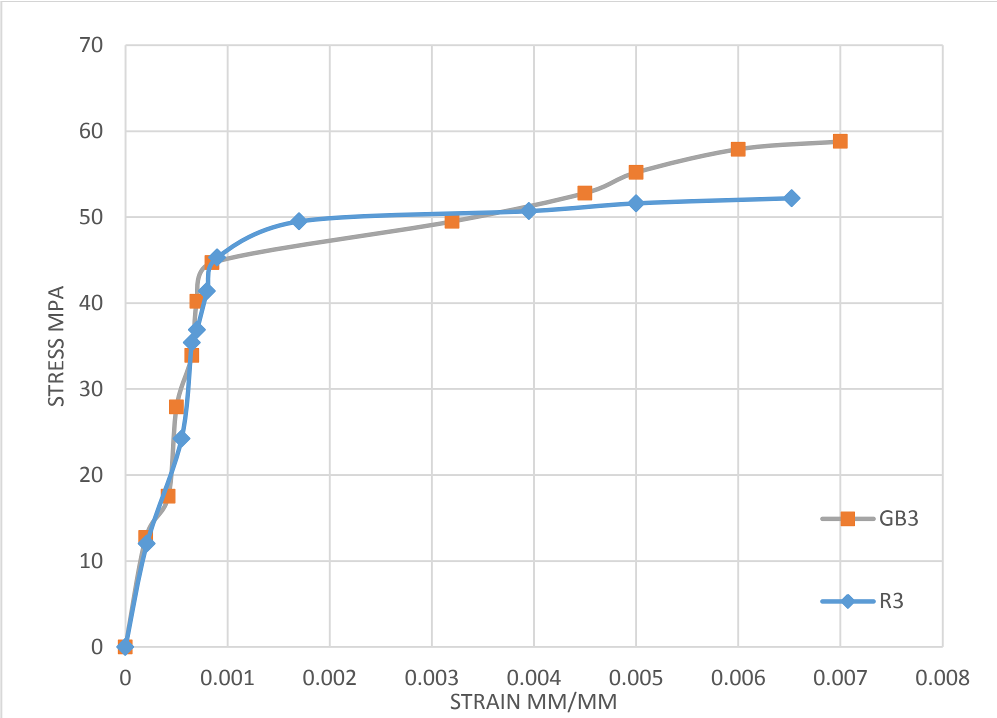


Figure (4-33): Stress-Strain of 10% limestone mix beam at 150 days

The toughness of all beams ranges between (0.287 and 0.351) MPa. The 10% limestone had the highest toughness at the late ages despite it showed low toughness value at 28 day. The calculated toughness of beams cured internally with 10% limestone was (0.278 , 0.351 and 0.338) MPa for the ages (28,90 and 150) days, respectively. While the 5% limestone beams showed (0.321,0.271 and 0.320) MPa for the test ages of (28,90 and 150) days, respectively.

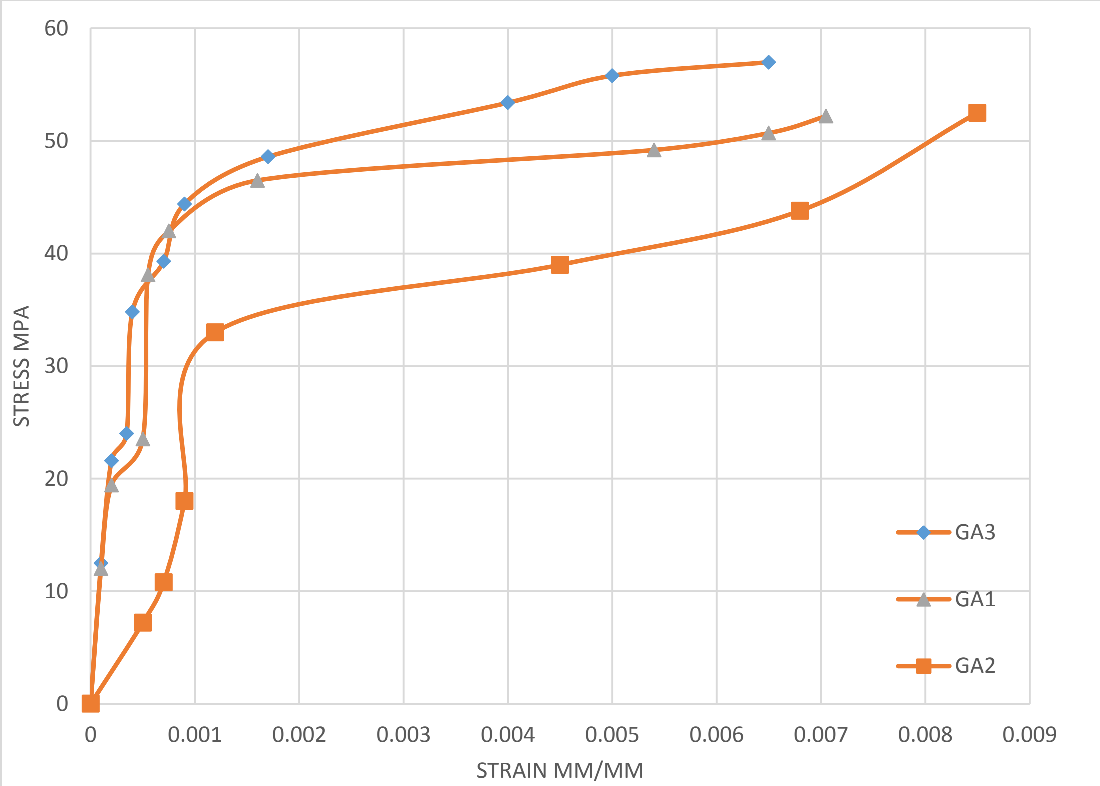


Figure (4-34): Stress-Strain of 5% limestone mix beams comparison

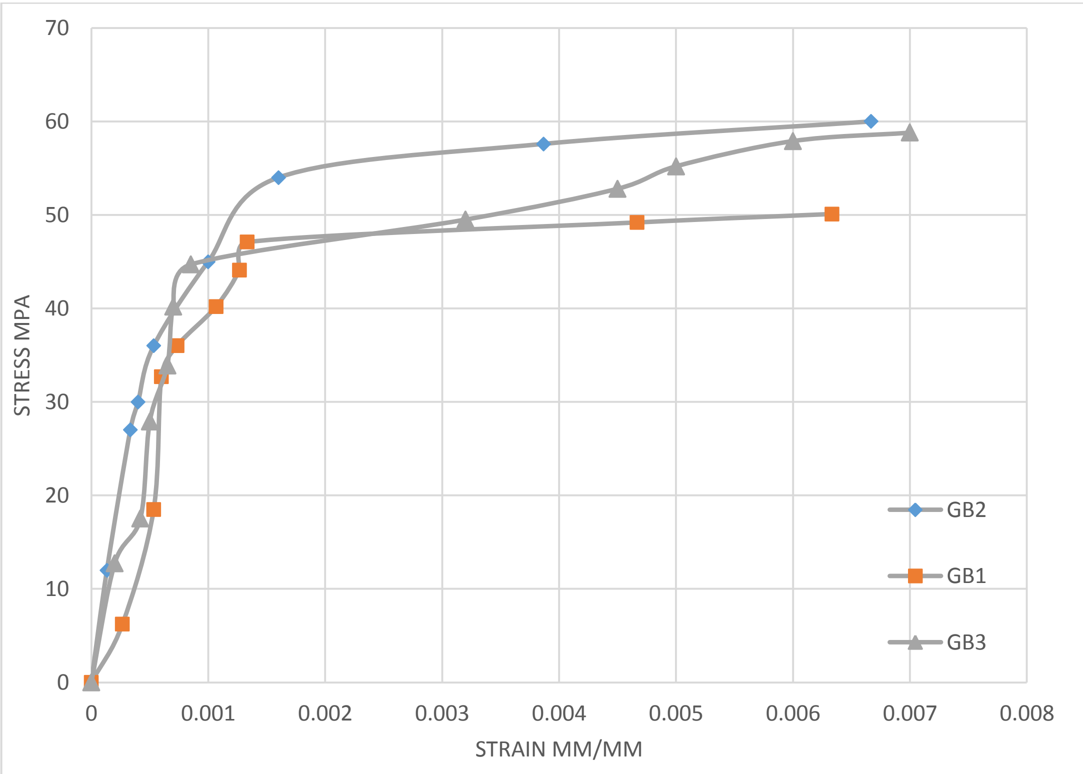


Figure (4-35): Stress-Strain of 10% limestone mix beams comparison

4.4. Effect of Replacement Percentage of The Internal Curing Materials

In this part, the results are presented to study the effect of the different percentage of replacement on structural behavior of the beams, the same parameters were taken under consideration (load capacity, ductility index and toughness).

Figure (4-32) to (4-37) shows the load deflection curve for each internal curing material with comparison with the reference mix at each age of test. The stress-strain curves are shown in Figures (4-38) to (4-43). Throughout the experimental work the following points were presented:

- For the 5% brick mix at 28 and 90 days, the ultimate load capacity was less than the reference mix, but it became higher than the reference at 150 days age. All specimens had lower ductility index than the reference mix. Figures (4-26) to (4-31) show that the area under the curve for the brick and limestone mixes were less than the reference mix which means less toughness, see Table (4-1).
- For 10% brick mix the ultimate load capacity was higher than both the reference and brick mixes (BA). Also, it had high ductility index which was nearly equall to the reference mix. As for the toughness, (BA) was higher toughness than (BB) mixes.
- The 5% limestone mix (GA) recorded higher ultimate load capacity compared to reference mix. Although, it had less ductility and toughness.
- The 10% limestone mix(GB) showed a higher load capacity, ductility and toughness compared to (GA) at late ages of a test .

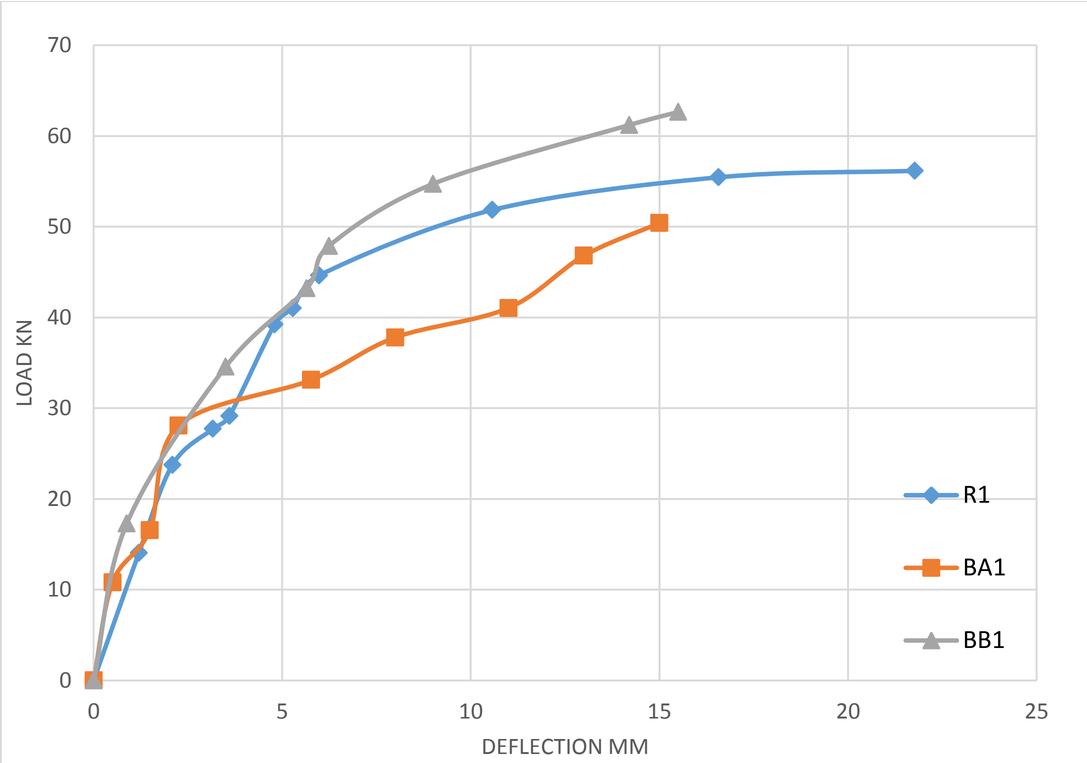


Figure (4-36): Deflection of brick mix beams at 28 days

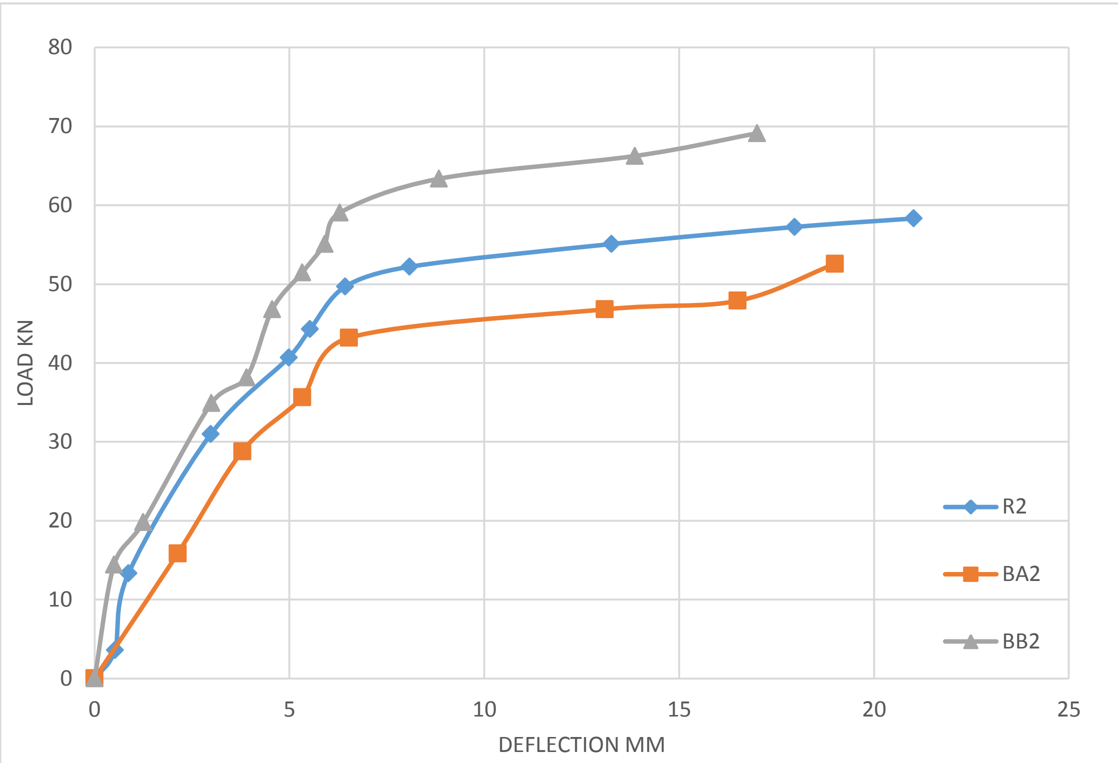


Figure (4-37): Deflection of brick mix beams at 90 days

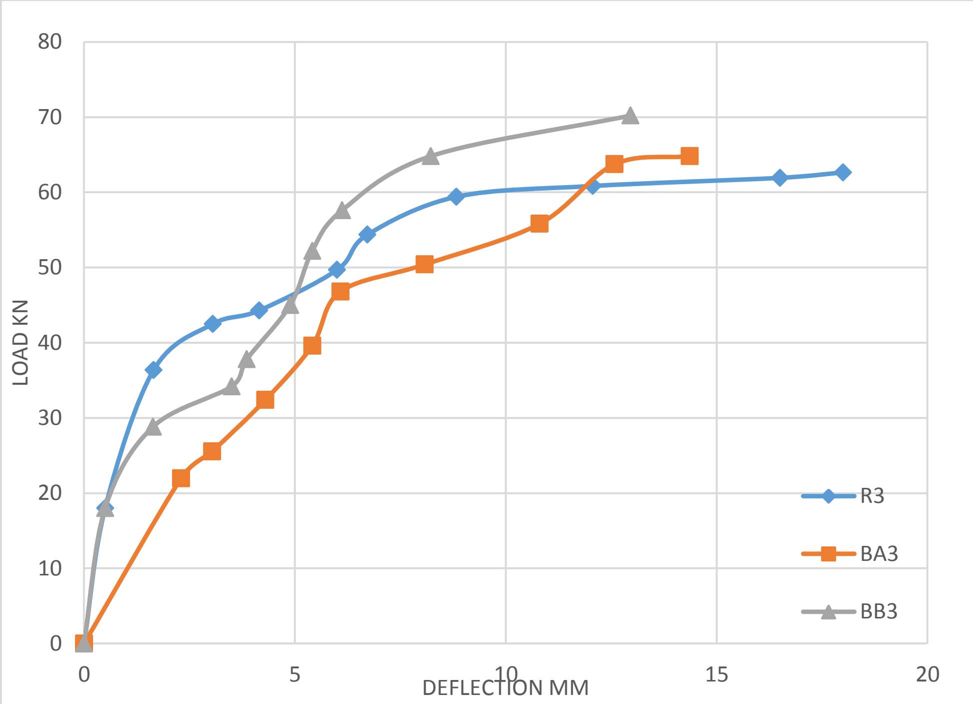


Figure (4-38): Deflection of brick mix Beams at 150 days

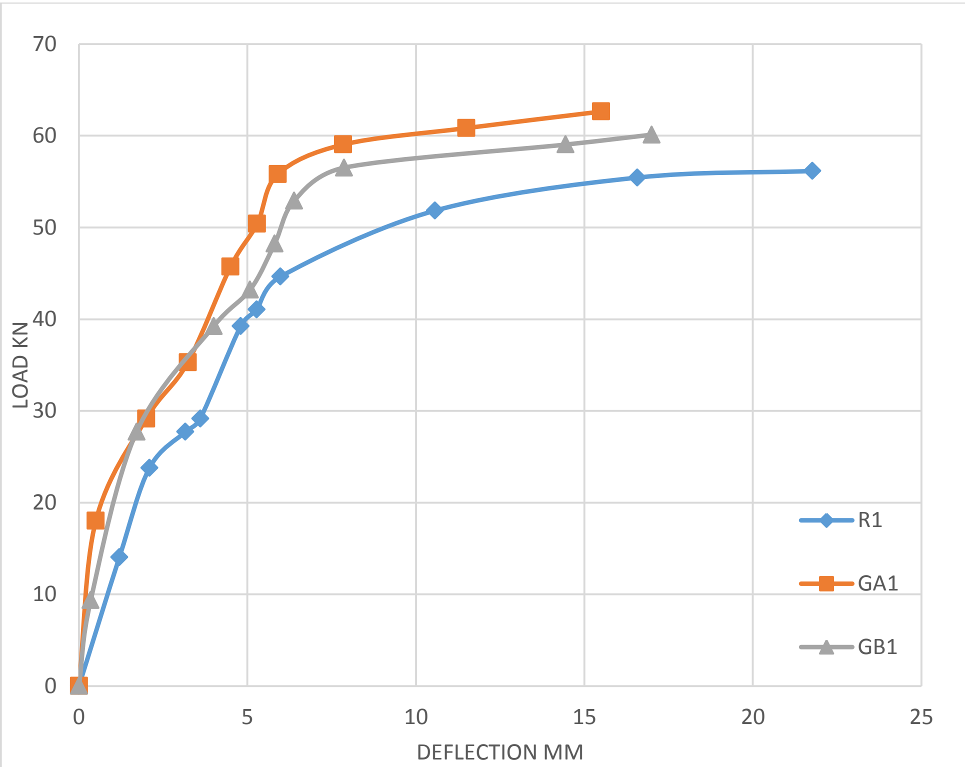


Figure (4-39): Deflection of limestone mix beams at 28 days

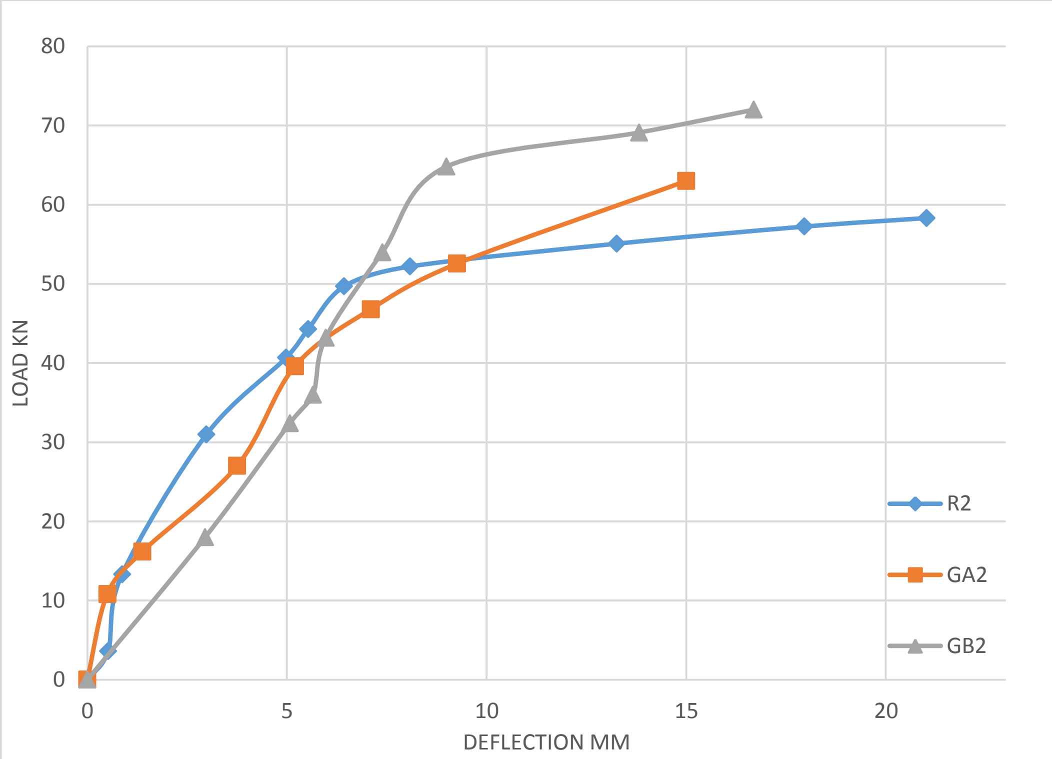


Figure (4-40): Deflection of limestone mix beams at 90 days

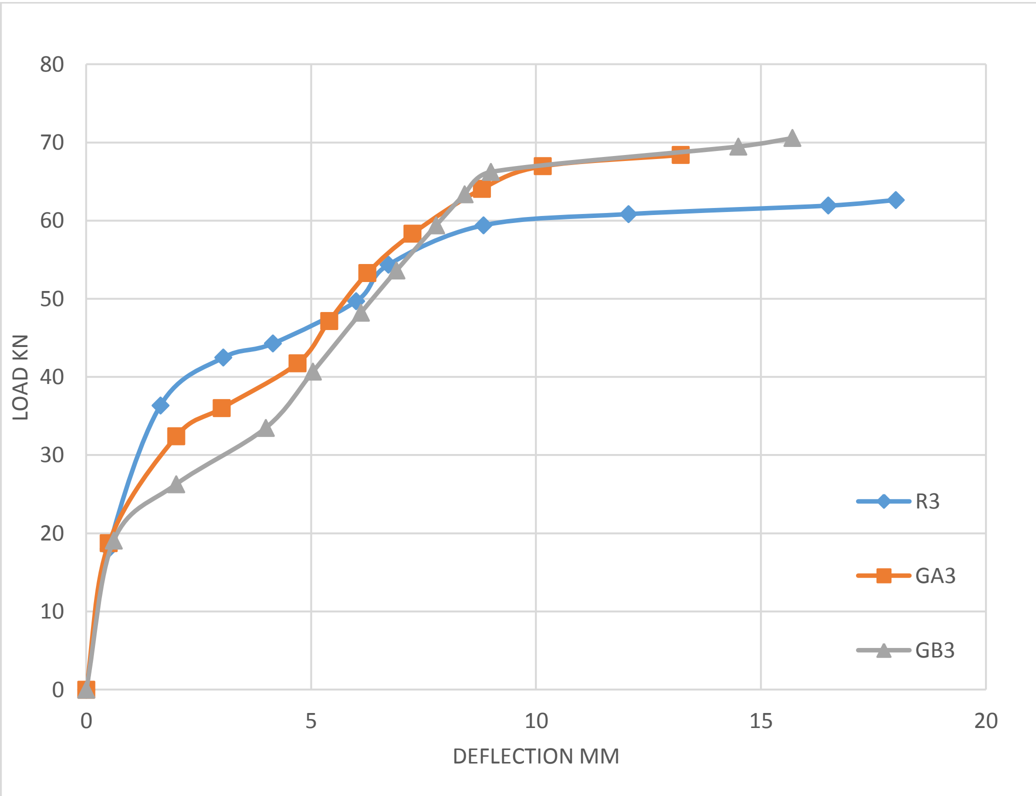


Figure (4-41): Deflection of limestone mix beams at 150 days

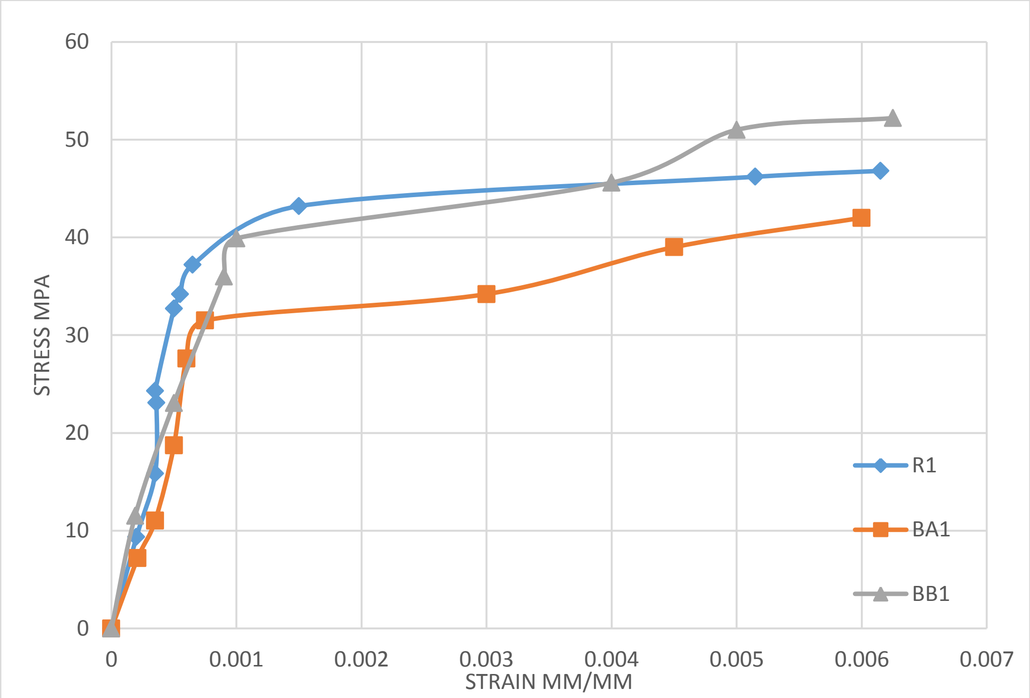


Figure (4-42): Stress-Strain of brick mix beams at 28 days

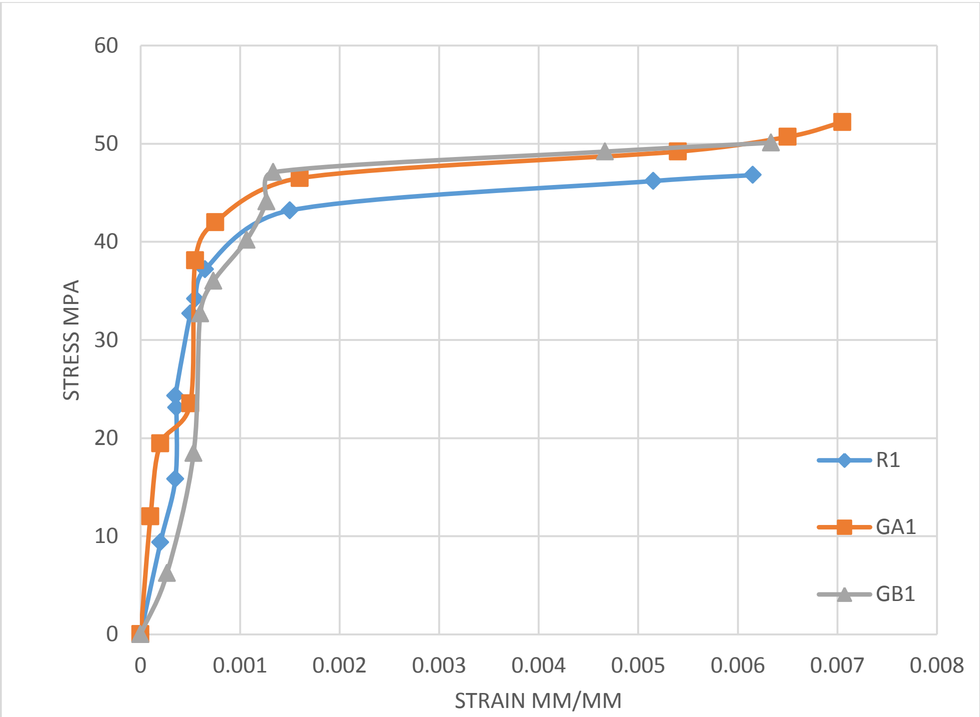


Figure (4-43): Stress-Strain of limestone mix beams at 28 days

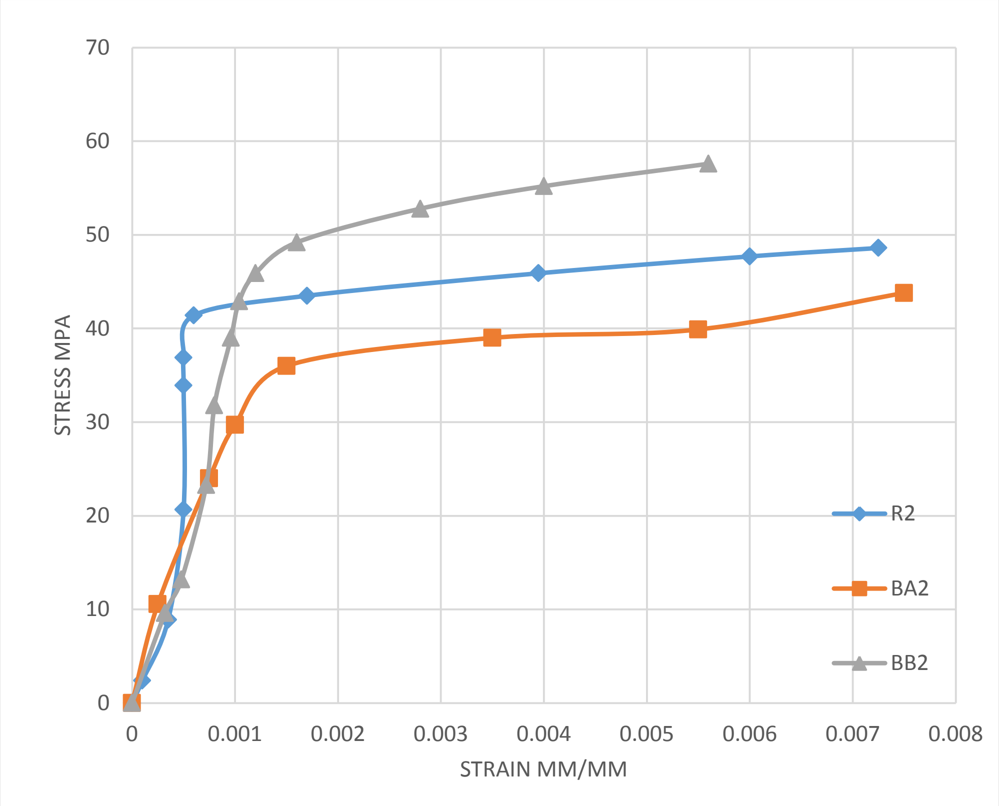


Figure (4-44): Stress-Strain of brick mix beams at 90 days

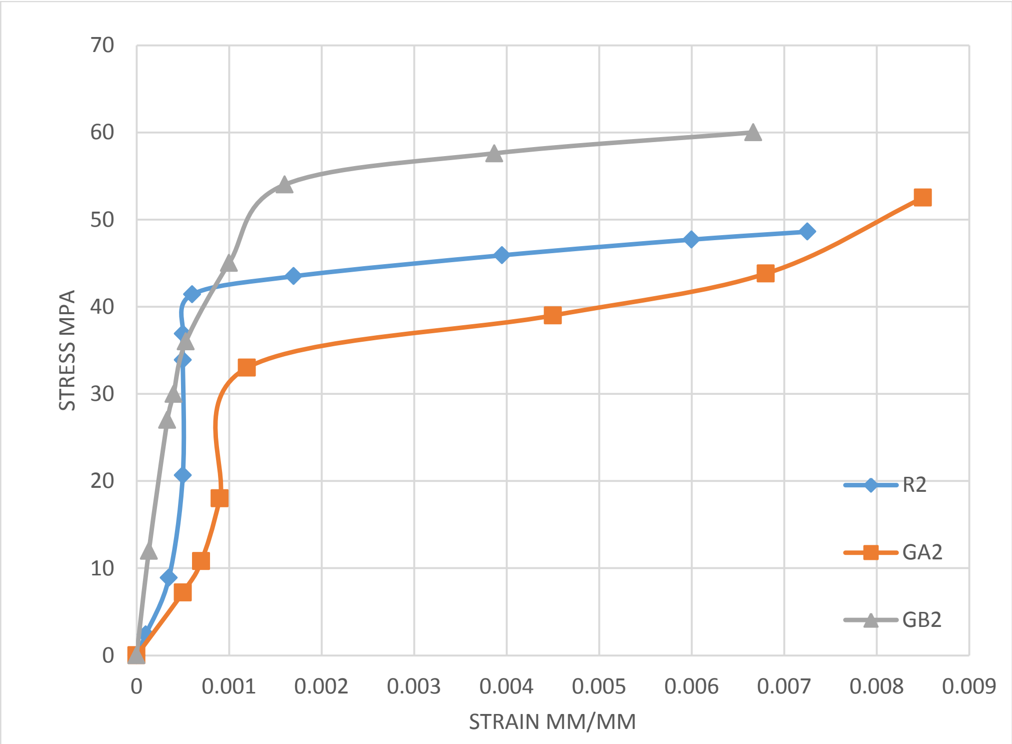


Figure (4-45): Stress-Strain of limestone mix beams at 90 days

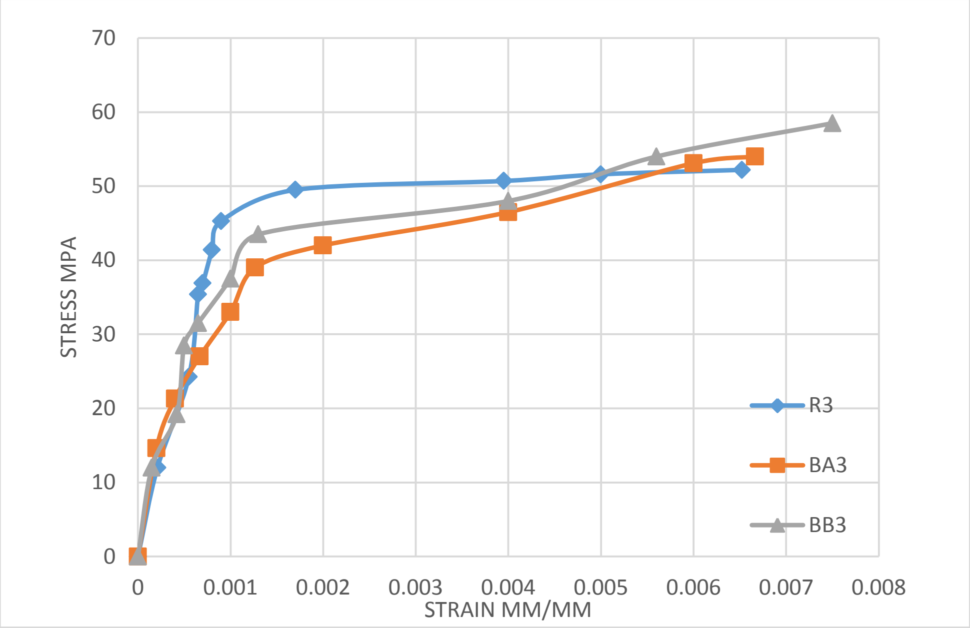


Figure (4-46): Stress-Strain of brick mix beams at150 days

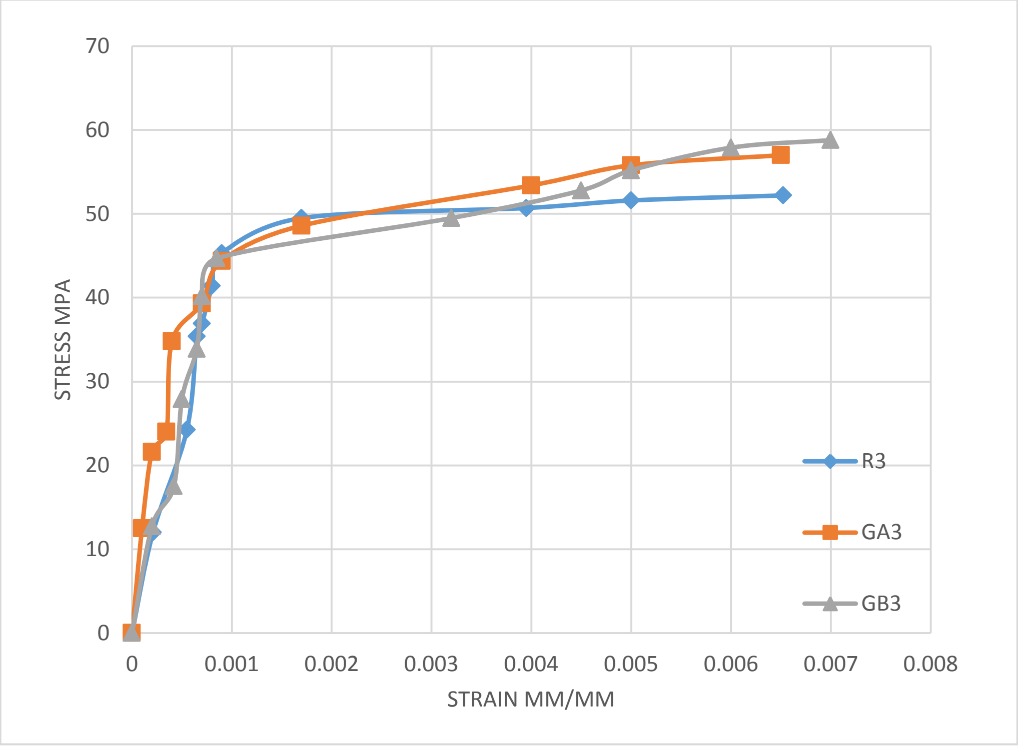


Figure (4-47): Stress-Strain of limestone mix beams at 150 days

4.5. Effect of Testing Age on Structural Behavior

The experimental program throughout the present work aims to study the effect of testing age on the concrete properties to examine the changes with time on the concrete and the structural behavior. Thus, there were three different ages (28, 90 and 150) days to determine the most durable mix of the different mixes. The following articles discuss the properties of all mixes and beams at each age of test.

Table (4-3) : Increase of ultimate load ratio with age

Mix	28 days	90 days	150 days
5% brick	0	4.2	28
10% brick	0	10.3	12
5% limestone	0	.5	9.1
10% limestone	0	19.7	17.3

4.5.1. Behavior of Beams at 28 Days

Figures (4-44) and (4-45) demonstrate the load-deflection and stress-strain curves of all the beams at the test age of 28 days it can be seen that:

The beam specimens of internally cured materials had a higher maximum loading capacity than the reference mix except for (BA1) which means that the internal curing works as a good factor to enhance loading capacity.

Although, the ductility of all the mixes was less than the reference mix. While the highest recorded durability was closely equal to the reference beam ductility value .

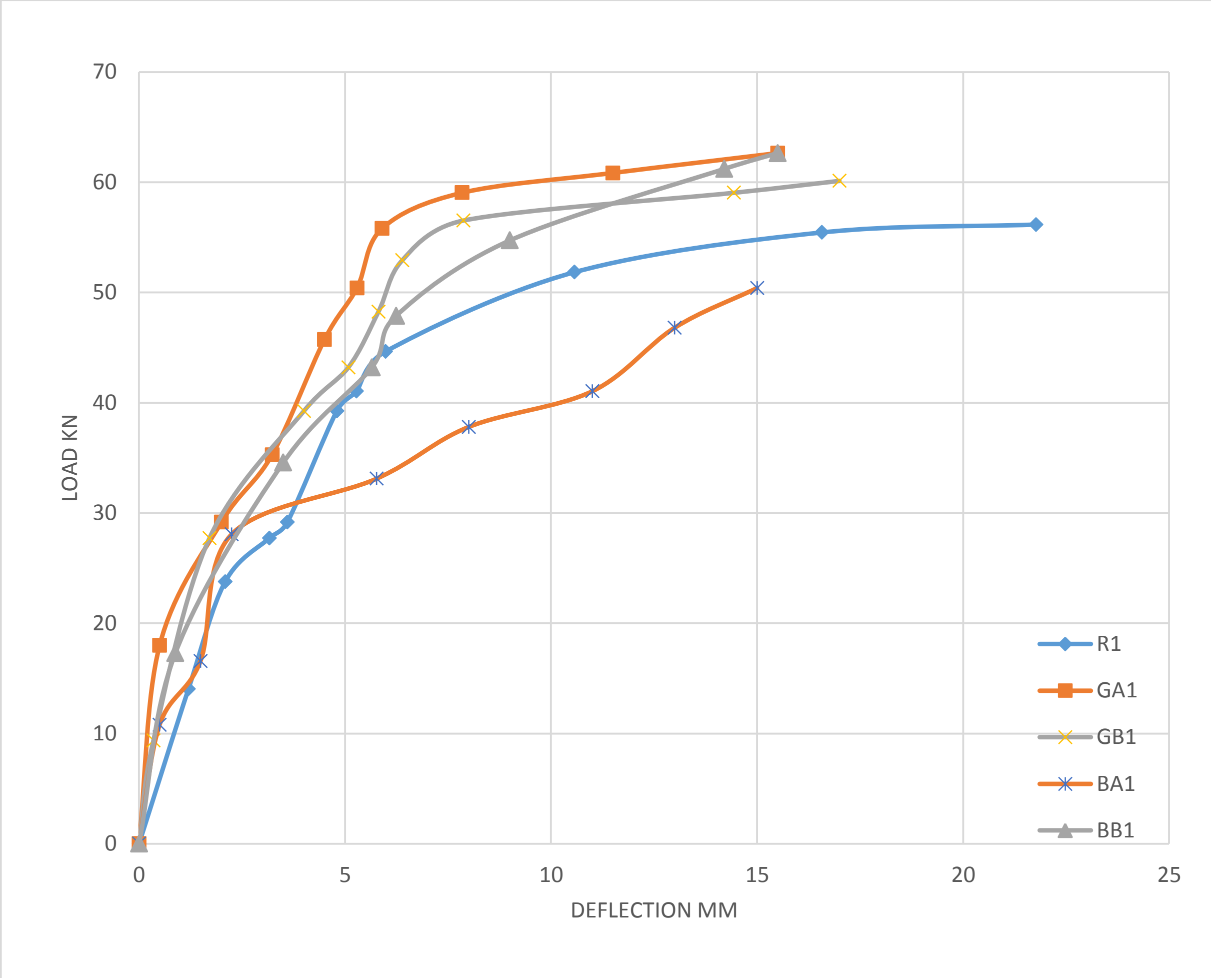


Figure (4-48): Deflection curve comparison for beams at 28 days

The internally cured beams GA1, GB1 and BB1 were higher in toughness than the reference, which means that they have a good ability to absorb energy before failure.

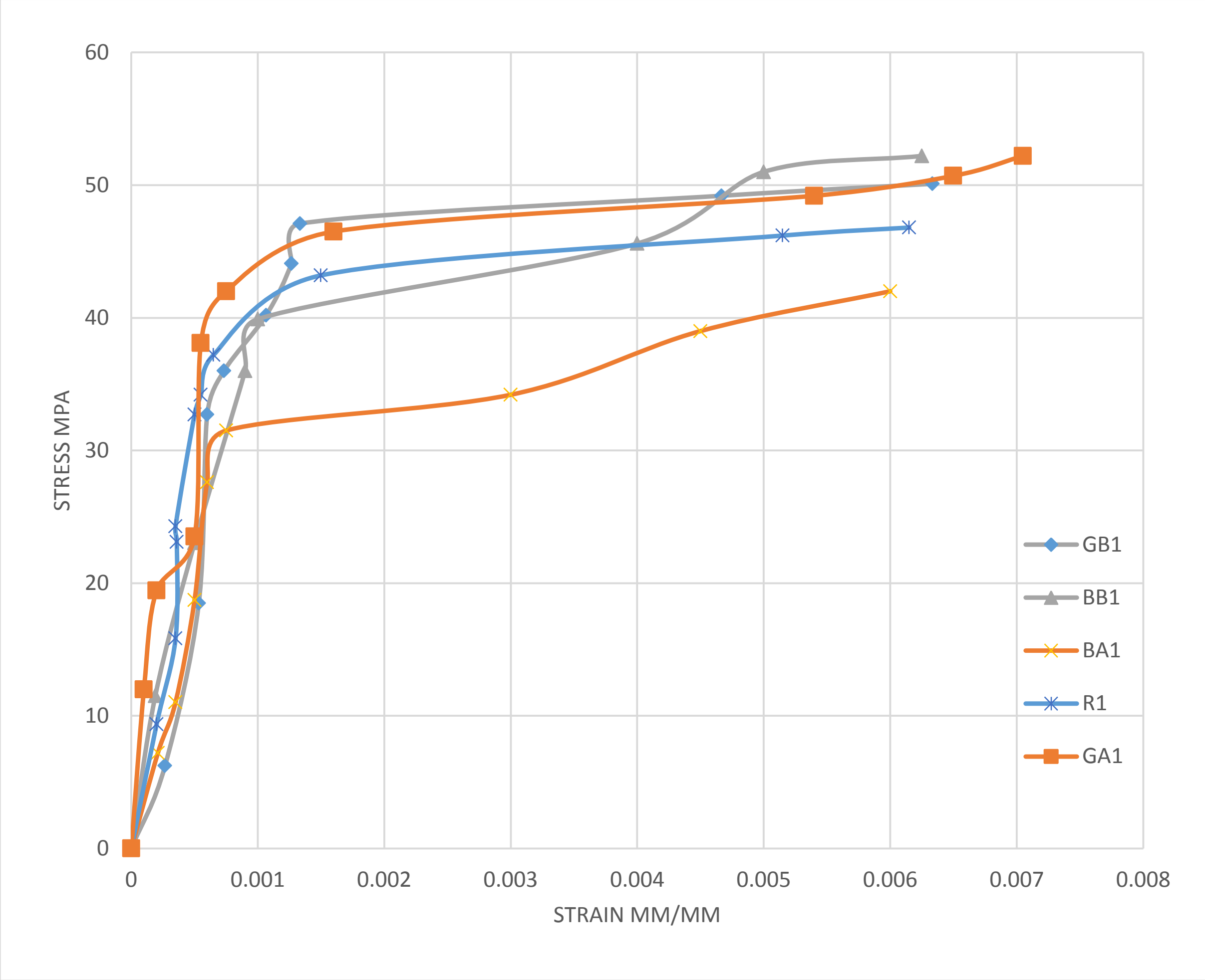


Figure (4-49) Stress-Strain curve comparison for beams at 28 days

4.5.2. Behavior of Beams at 90 Days

Figures (4-46) and (4-47) represent the load-deflection and stress-strain curves of all the beams at the test age of 90 days it can be seen that:

All the internally cured beams had ultimate loading capacity, higher than the reference mix except for (BA2). The ductility index of the internally cured beam specimens was less than the ductility index of the reference which means that the ductility did not improve significantly with age.

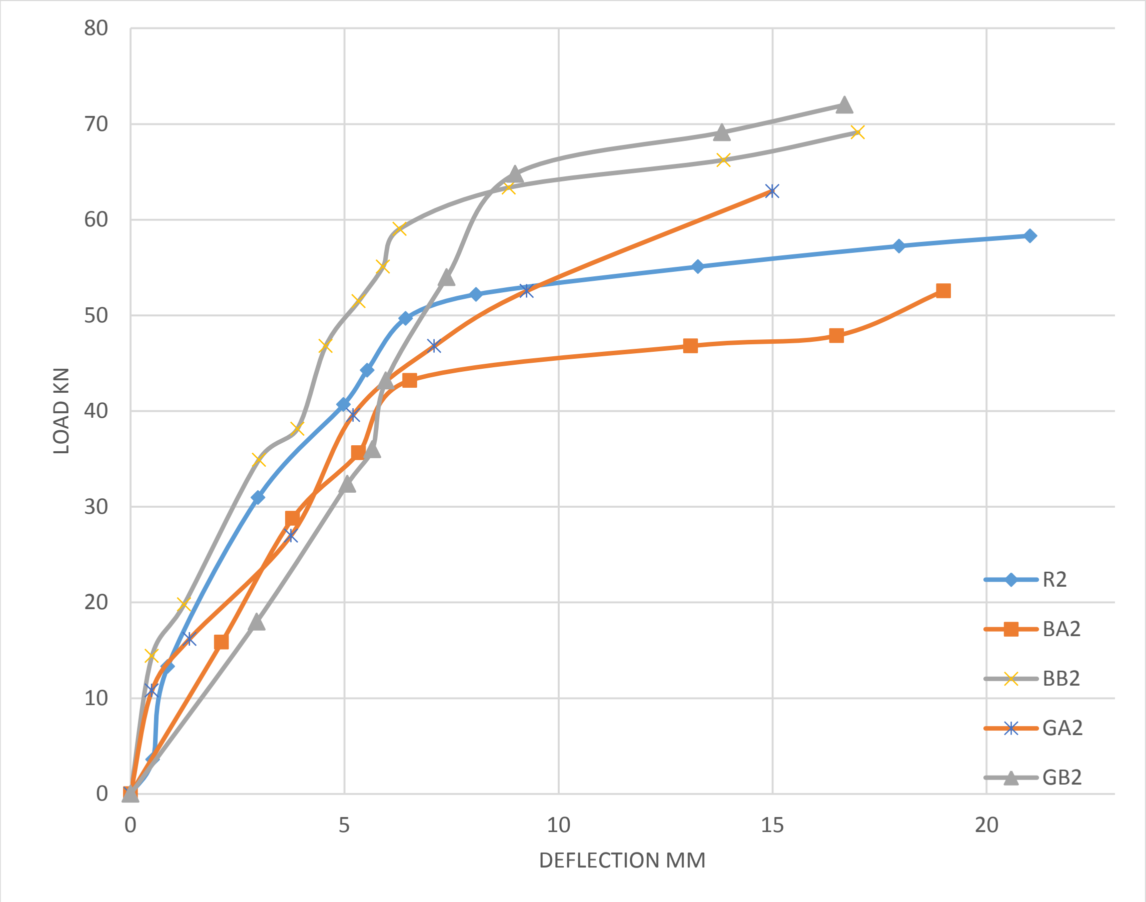


Figure (4-50): Deflection curve comparison for beams at 90 days

The toughness values and the shape of stress-strain curves showed that the internally cured beams (GA2) and (GB2) had very good results and were higher than the reference mix while the brick mixes were lower in toughness than the reference.

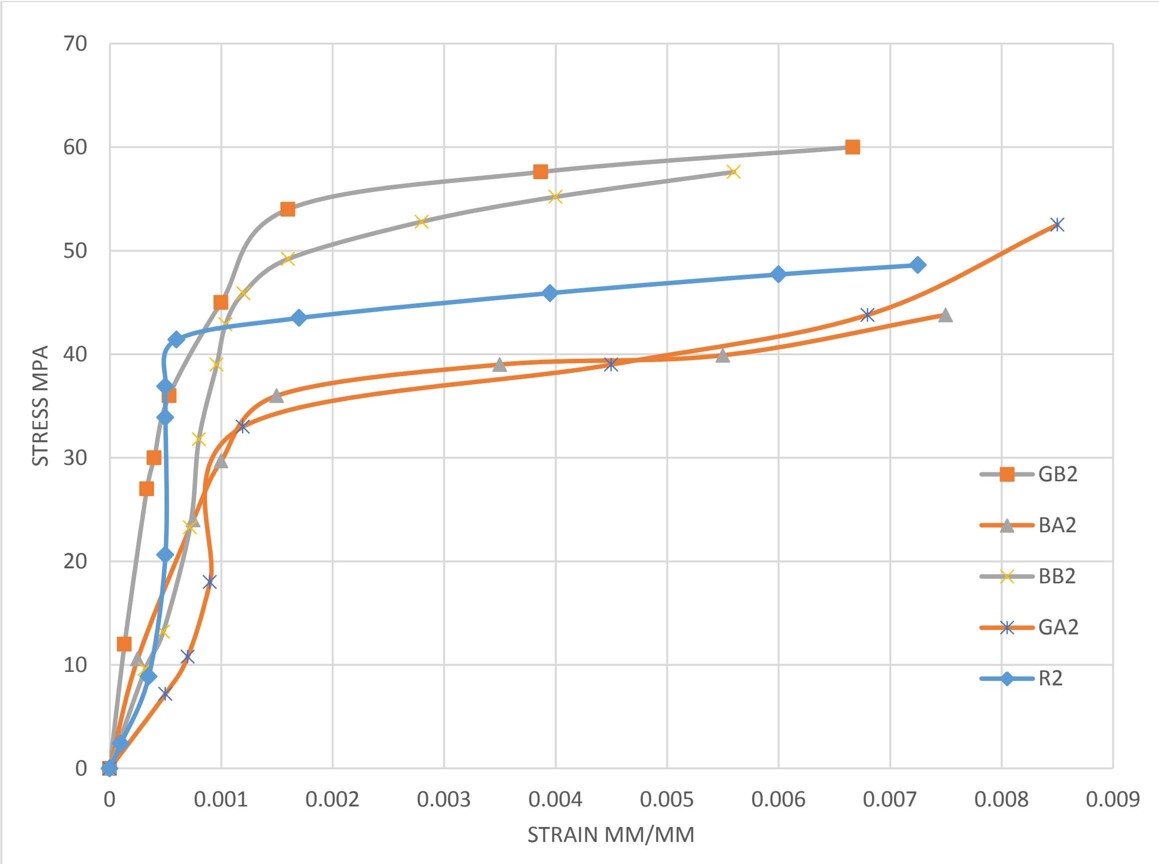


Figure (4-51): Stress -Strain curve comparison for beams at 90 days

4.5.3. Behavior of Beams at 150 Days

Figures (4-48) and (4-49) present the load-deflection and stress-strain curves of all the beams at the test age of 150 days, it can be seen that:

Noticeable improvement of the ultimate loading capacity of the internally cured mix beams which was higher than the reference beam in general. However, the ductility was lower than the reference. While, (BA3) and (GB3) were the closest values of ductility index to the reference mix.

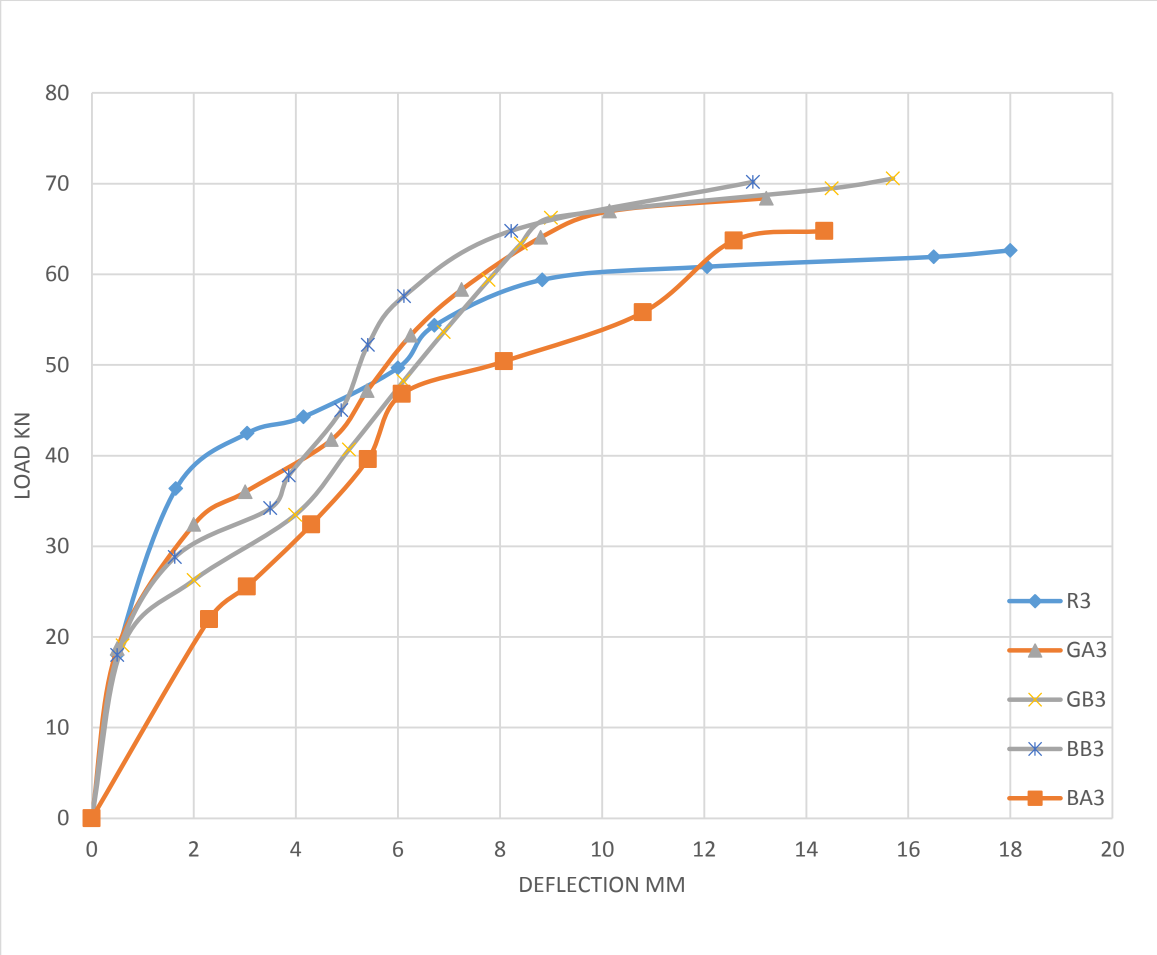


Figure (4-52): Deflection curve comparison for beams at 150 days

Toughness value of the internally cured beam (BB3) had the highest toughness (0.348) MPa, which was higher than the toughness of the reference beam. Also beams (GA3 and GB3) showed higher toughness. while (BA3) had lower toughness than the reference beam.

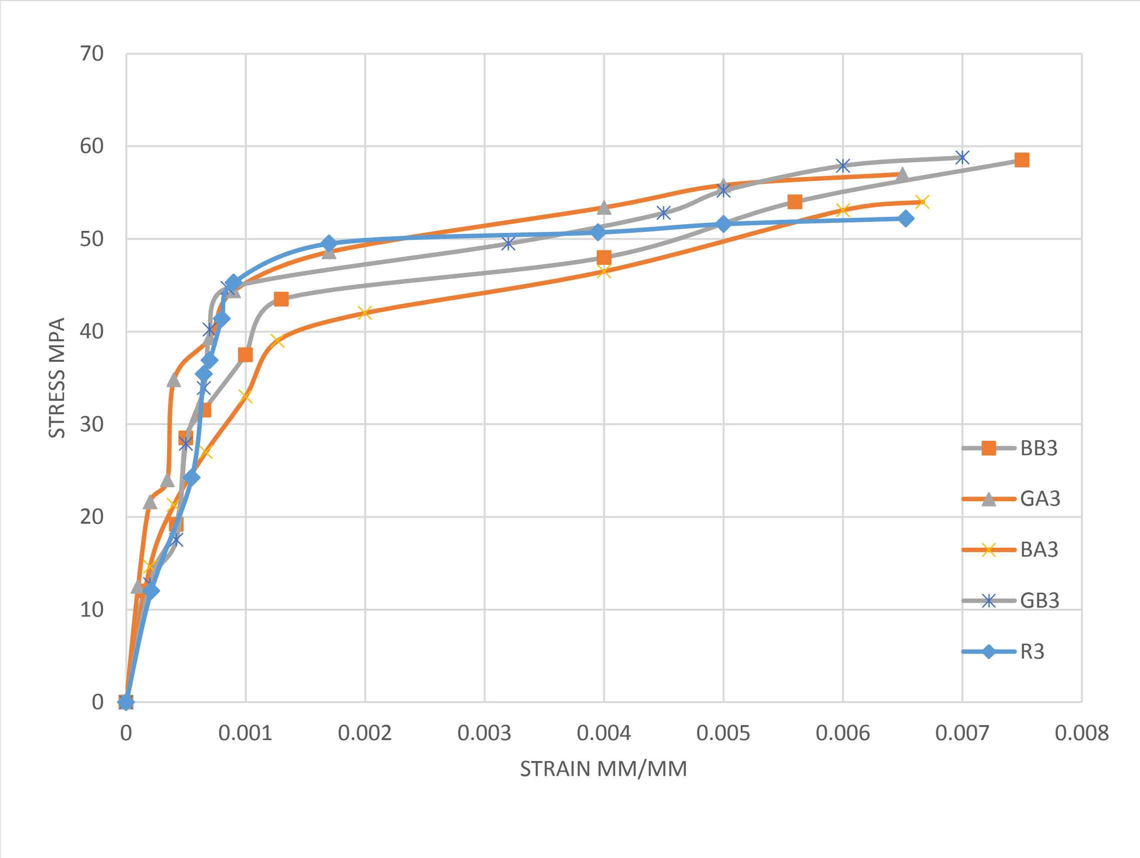


Figure (4-53): Stress-Strain curve comparison for beams at 150

4.6. Crack Patterns

Cracks initiation and propagation had been noticed and recognized in every loading step of the beams. The cracks were not within the elastic portion of the stress-strain responce (the linear loading stage) . Then micro cracks began to propagate at the bottom of the beam in the pure bending area between the loading points. The first cracks were barely noticeable and were accompanied by clear audible sound that indicates the cracking of concrete in the tension zone. As the loading became higher new cracks were propagated and the older cracks began to grow longer towards the upper face of the beam in a straight path.

Another types of diagonally orientated cracks also propagated from the sides of the beam under the loading point . At final stages some of the beams experienced crash of the concrete and spalling of the cover the compression zone while some showed high stains in steel which indicates a tension failure. Figures (4-50) to (4-64) show the tested beams after failure . The beams were painted with white emulsion to distinguish the cracks easily. Colored pens were used to mark the cracks after every loaing stage, the numbers on each crack represents the reading of a loading cell used in the laboratory to measure the load and then must be multiplied by a factor of (3.6) to obtain the load in kN. The failure of the beams (R1, BA1 and BB1) was by crushing in the concrete of the compression area after the cracks reached too close to the top of the beam. The other beams reached high loadings and experienced high tension at the steel .

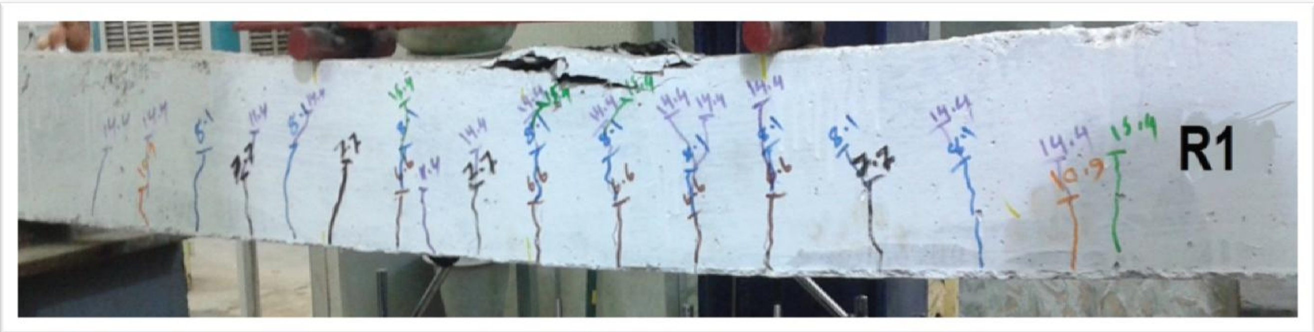


Figure (4-54): Crack patterns of beam R1

Figure (4-55): Crack patterns of beam BA1

Figure (4-56): Crack patterns of beam BB1

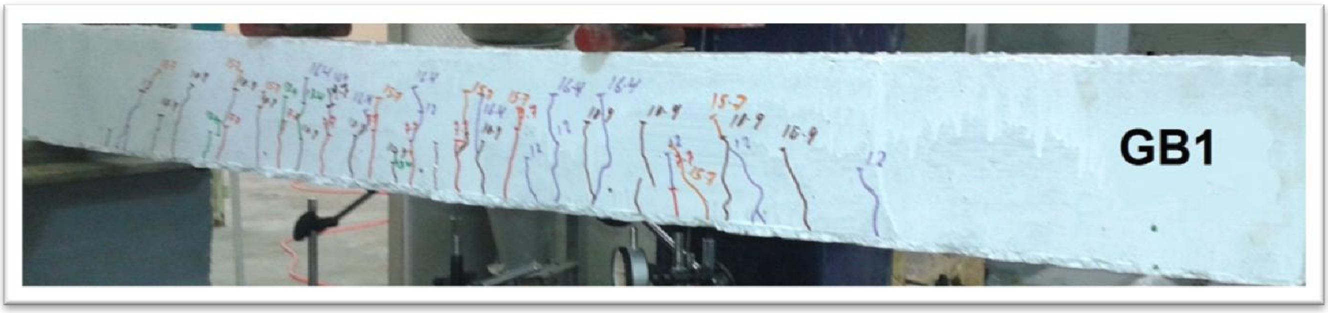


Figure (4-57): Crack patterns of beam GB1

Figure (4-58): Crack patterns of beam GA1

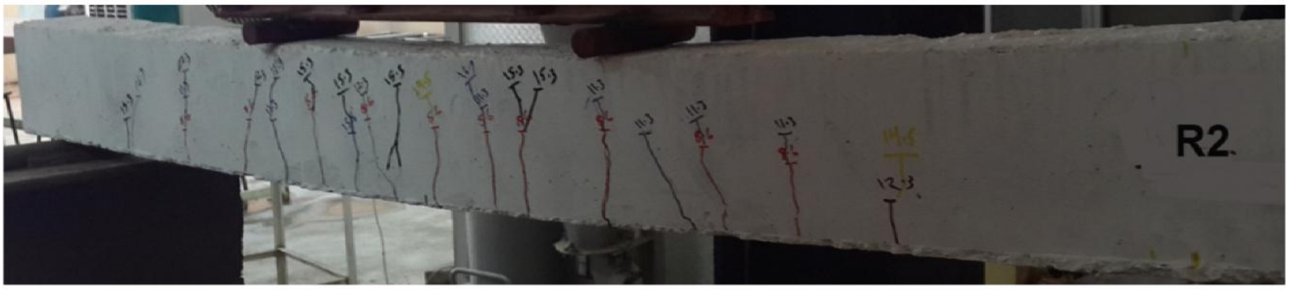


Figure (4-59): Crack patterns of beam R2

Figure (4-60): Crack patterns of beam BA2

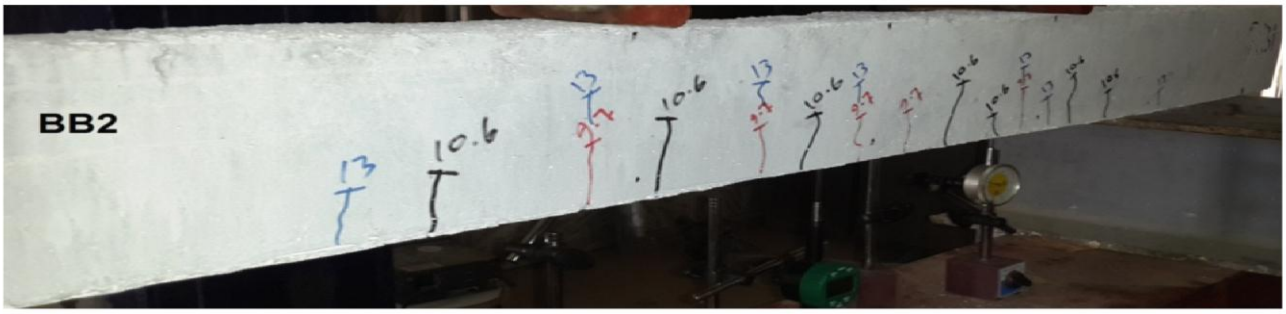


Figure (4-61): Crack patterns of beam BB2

Figure (4-62): Crack patterns of beam GA2

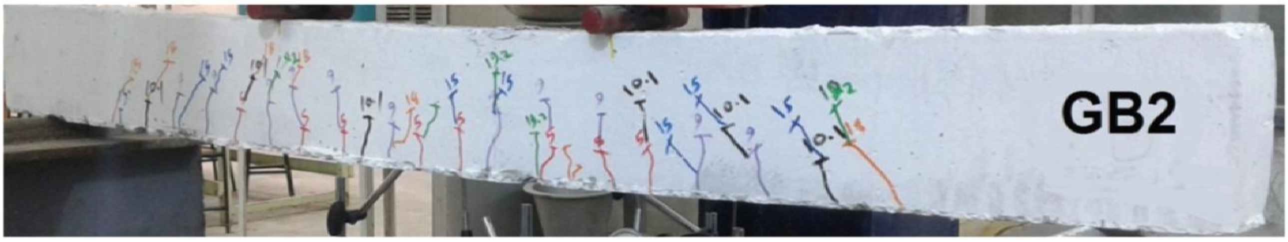


Figure (4-63): Crack patterns of beam GB2

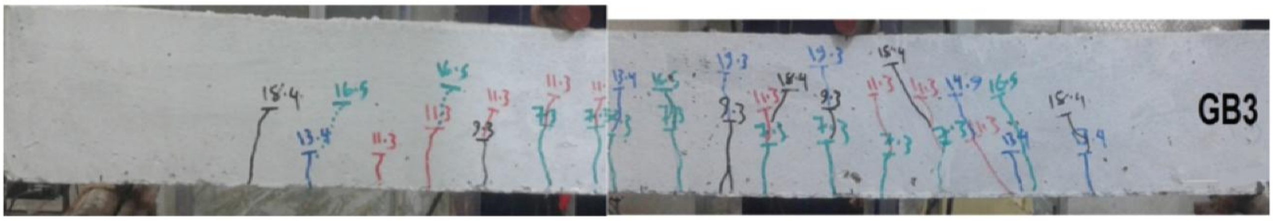


Figure (4-64): Crack patterns of beam GB3

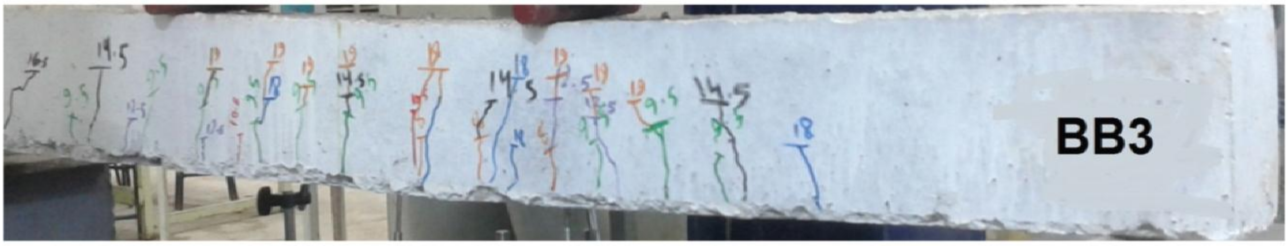


Figure (4-65): Crack patterns of beam BB3

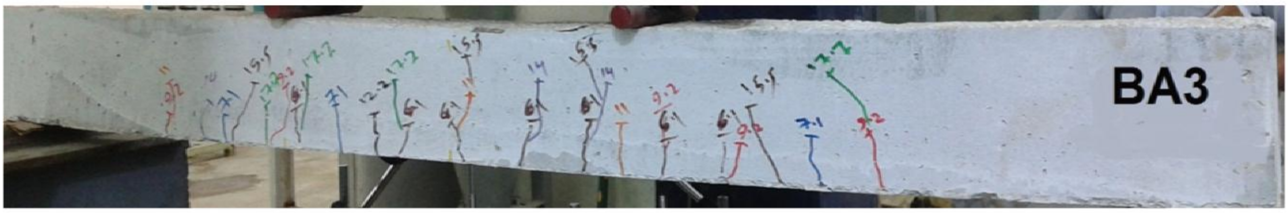


Figure (4-66): Crack patterns of beam BA3

Figure (4-67): Crack patterns of beam R3

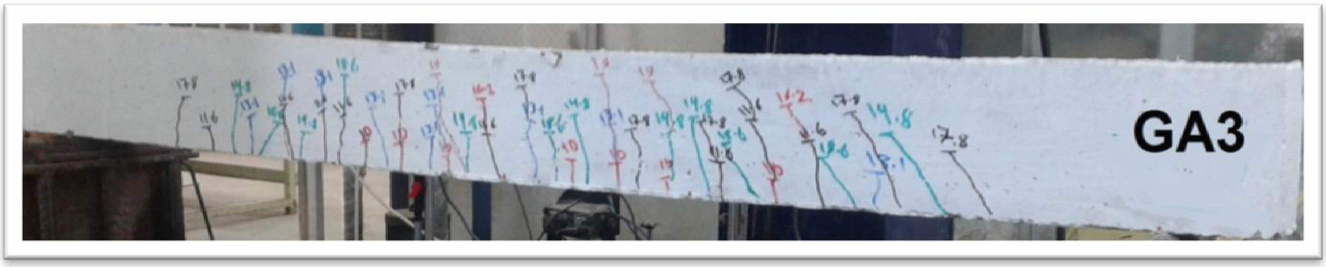


Figure (4-68): Crack patterns of beam GA3

4.7. Concrete Properties Results

All the concrete tests that are related to the fresh and hardened concrete properties, compressive strength and splitting tensile strength were detailed and discussed in the following subsuctions. A number of concrete cubes and cylinders had been casted for each type of mix to examine those properties.

4.7.1. Compressive Strength

This paragraph presents and discusses the test results of the compressive strength of the various mixes at different ages as shown in Table (4-3). The results represent the average strength value of 3 cubes (100*100*100)mm which were tested for each concrete mix at ages of 28, 90 and 150 days.

The reference concrete mix had the compressive strength of (55.8, 58 and 59) MPa for ages (28, 90 and 150) days, respectively. This trend showed that the concrete was developing the strength with time. The other internally cured mixes showed a similar advancement in gaining strength with time for each mix.

Table (4-4) : Compressive strength and percentage of increase compared to ordinary mix

Mix	28 days	increase%	90 days	increase%	150 days	increase%
Ordinary	55.8	0.0	58.0	3.9	59.0	5.7
5% brick	55.5	-0.6	59.1	5.9	59.7	7.0
10% brick	57.2	2.4	60.9	9.1	62.8	12.5
5% limestone	58.2	4.3	60.5	8.4	61.9	10.9
10% limestone	57.5	3.0	58.3	4.5	63.4	13.6

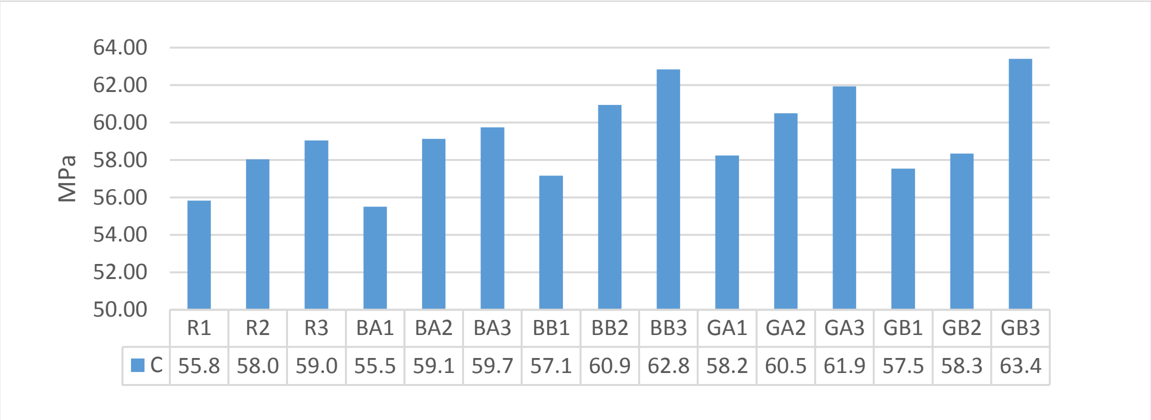


Figure (4-69): Compressive strength of all mixes

As shown from the values of compressive strength in Table (4-3) and Figure(4-65): There is a considerable improvement of the strength of internally cured mixes through the time. The highest compressive strength was gained by the mix GB3 of (63.4) MPa And the improvement percentage with age was 13.6 % . that means 10% limestone used as internal curing had the most efficient effect.

Also the mix of 10% crushed brick showed excellent performance with compressive strength 62.8 MPa . The lowest strength was BA1 which had strength of 55.5 MPa. Figures (4-66) and (4-67) show the development of the strength with the age of the mix.

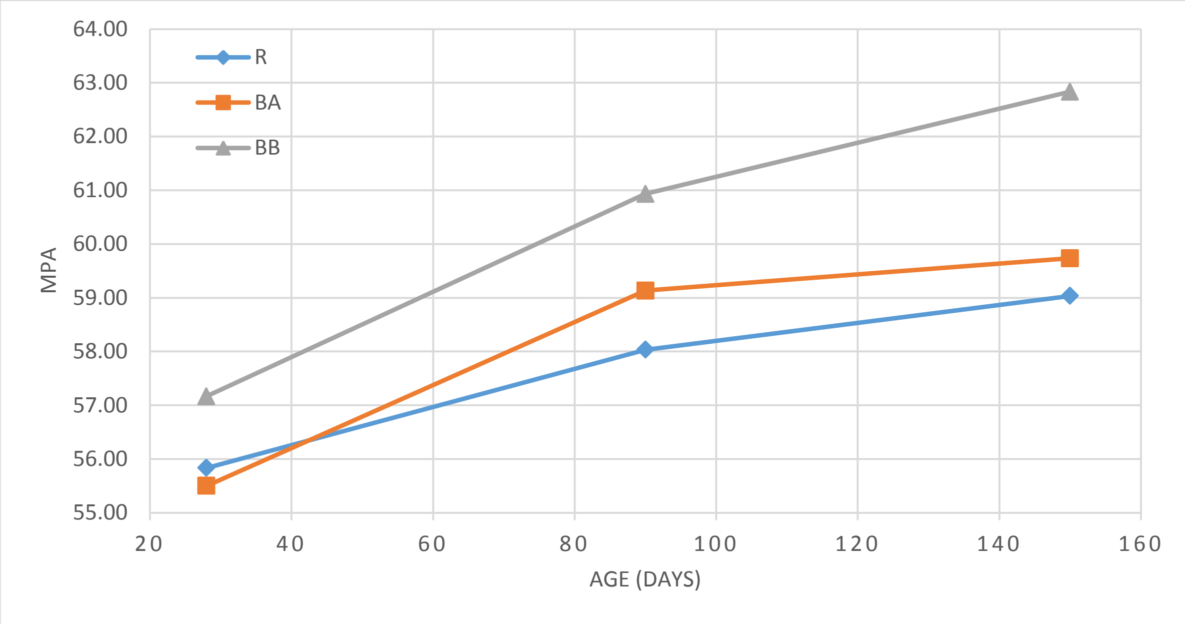


Figure (4-70): Compressive strength development with age (brick mixes)

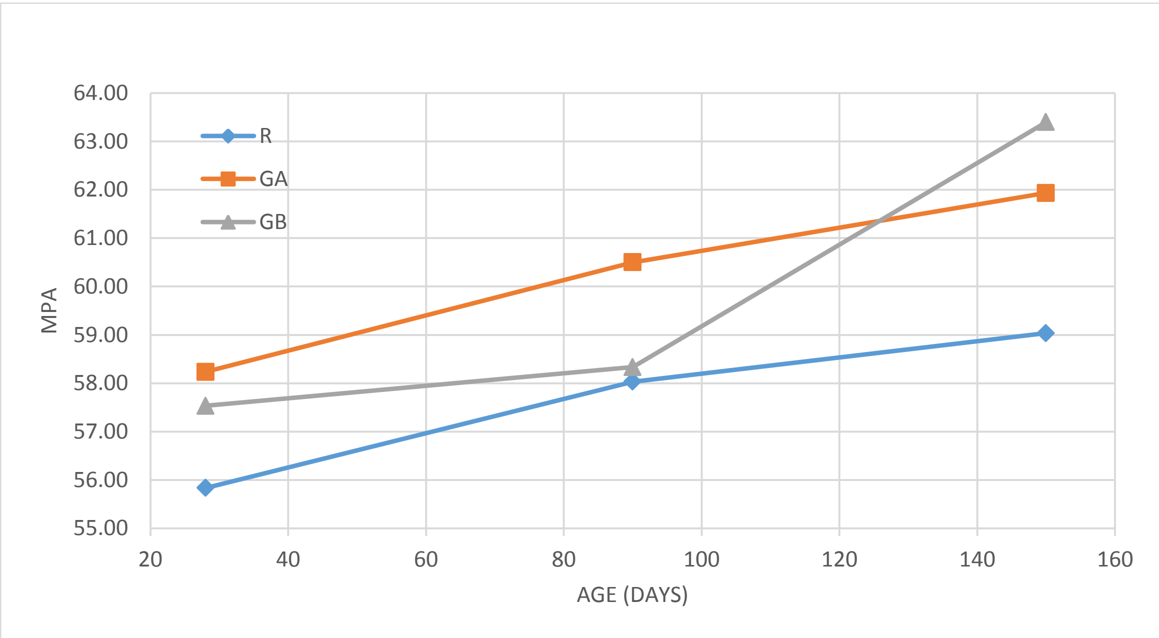


Figure (4-71): Compressive strength development with time(limestone mixes

4.7.2. Tensile Strength

One of the important factor for assessing the concrete is determining splitting tensile strength.

The test for cylinders of (100*200) mm for each mix were made by taking the mean of three specimens for each mix at each age of the test. As shown in Table (4-4), the values of splitting tensile strength ranged between 4.01 and 5.2 MPa. All mixes showed a drop in the strength at 28 days, but exhibited an improvement in the latter ages. This behaviour might be regarded to the effect of internal curing action in the LWA.

Table (4-5): Splitting Tensile Strength Results (MPa)

mix	28 day	increase %	90 day	increase %	150 day	increase %
reference	4.70	0	5	6.5	5.2	10.7
5% brick	4.40	-6.35	5.1	8.9	5.5	17.1
10% brick	4.31	-8.2	5.2	11.4	5.3	12.5
5% limestone	4.54	-3.3	5.1	9.3	5.4	14.8
10% limestone	4.49	-4.5	4.8	2.6	5.3	13.5

It is also noticed that there is a drop of strength at 28 days and regain in the latter ages. This can be noticed in Figures (4-68) through (4-70).

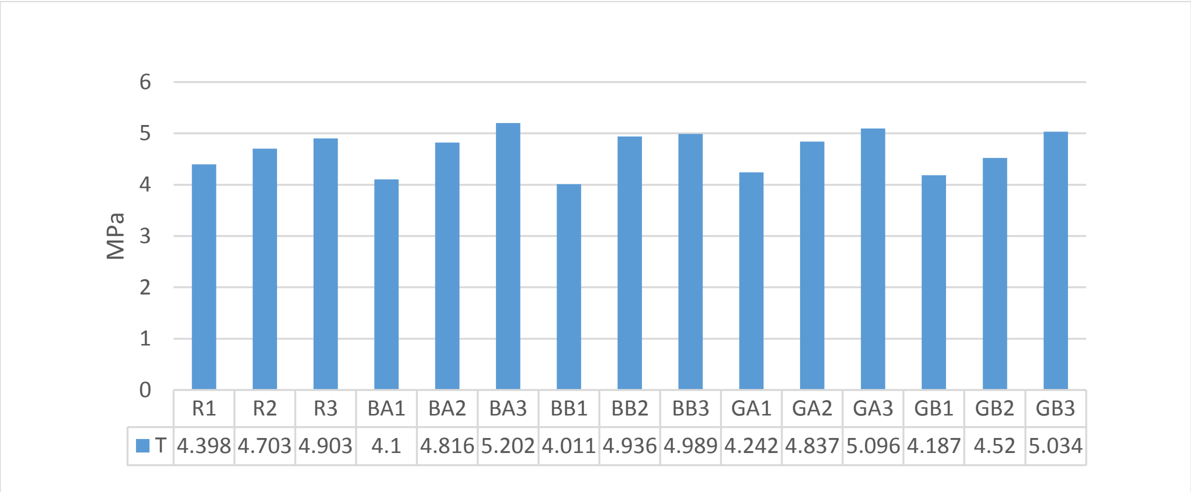


Figure (4-72) : Splitting Tensile Strength for All Mixes

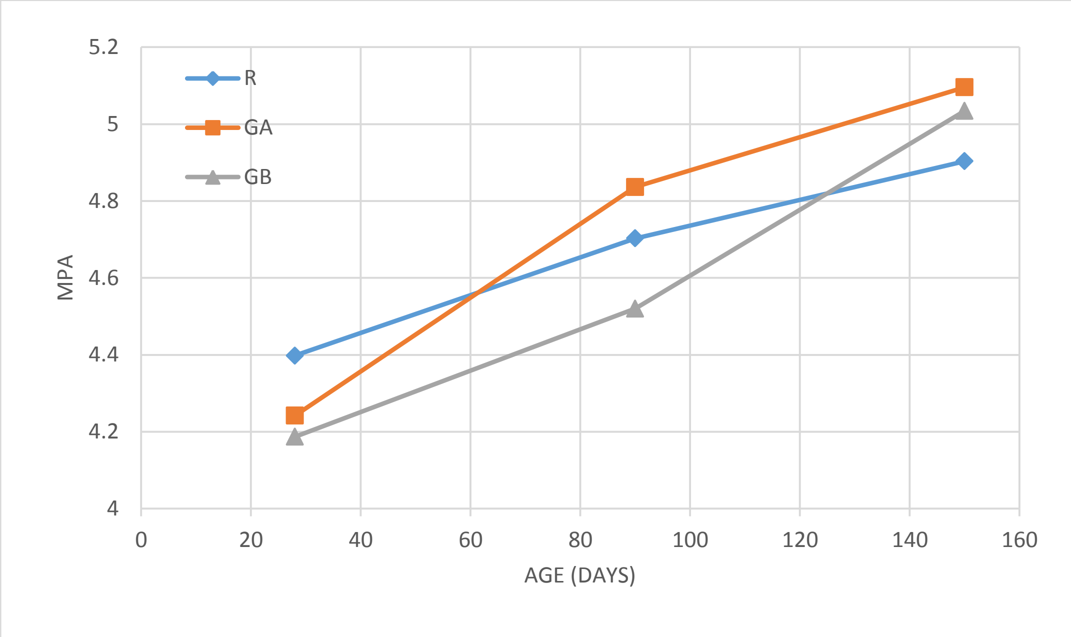


Figure (4-73) : Splitting tensile strength development with age (limestone)

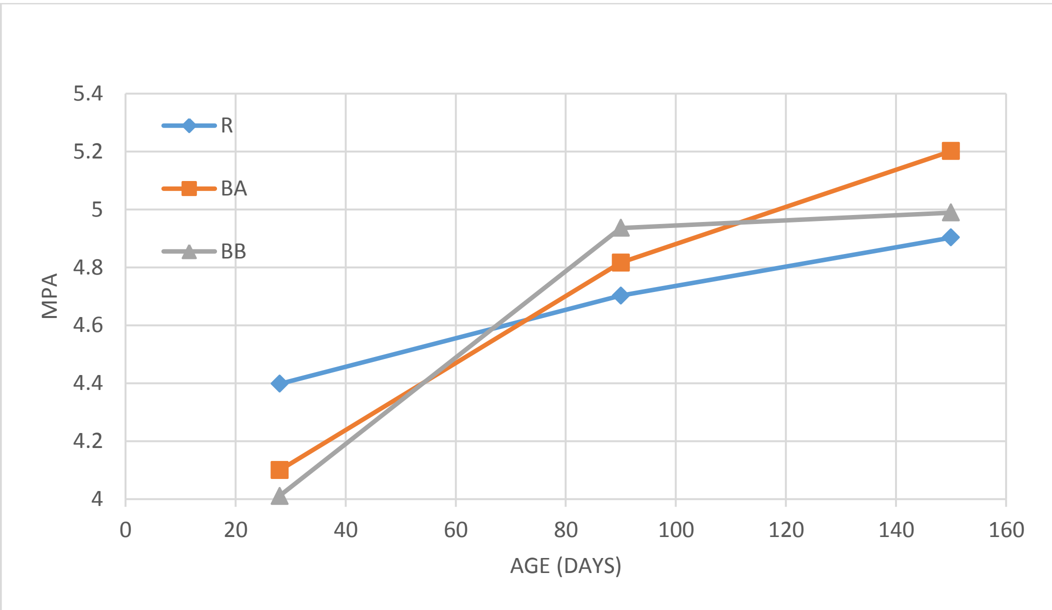


Figure (4-74): Splitting tensile strength development with age (brick)

The ratio of tensile to compression is illustrated in Table (4-5) and Figure (4-71)

Also Figure (4-72) showed the relationship between the compressive and tensile strength as a scatter diagram.

Table (4-6) : Ratio of Tensile to Compressive Strength (%)

T/C	R	BA	BB	GA	GB
28	8.41	7.93	7.54	7.80	7.80
90	8.62	8.65	8.59	8.49	8.26
150	8.81	9.21	8.42	8.71	8.41

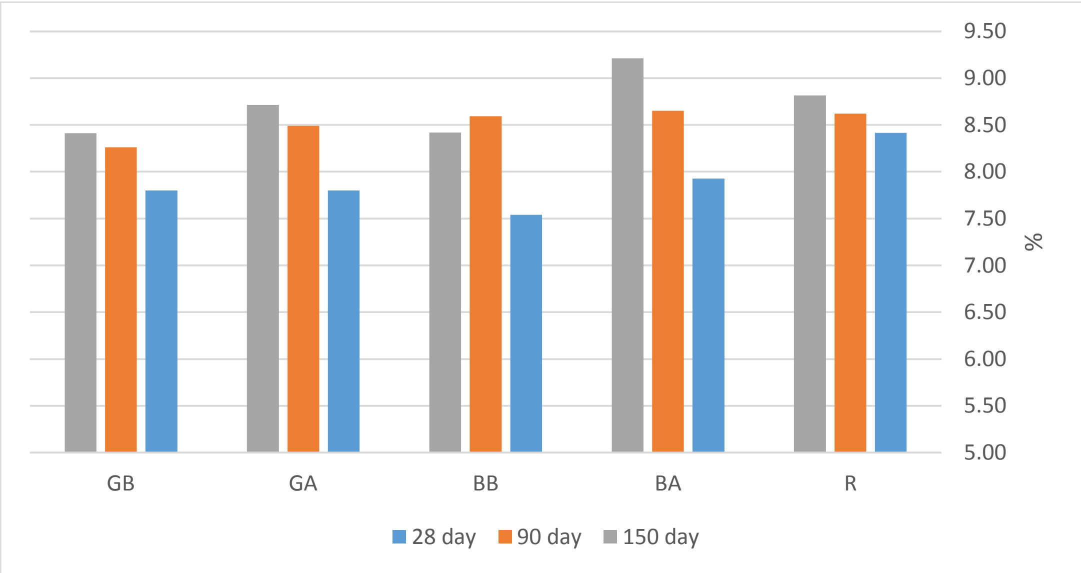


Figure (4-75): Splitting Tensile Strength to Compressive Strength Ratio(%)

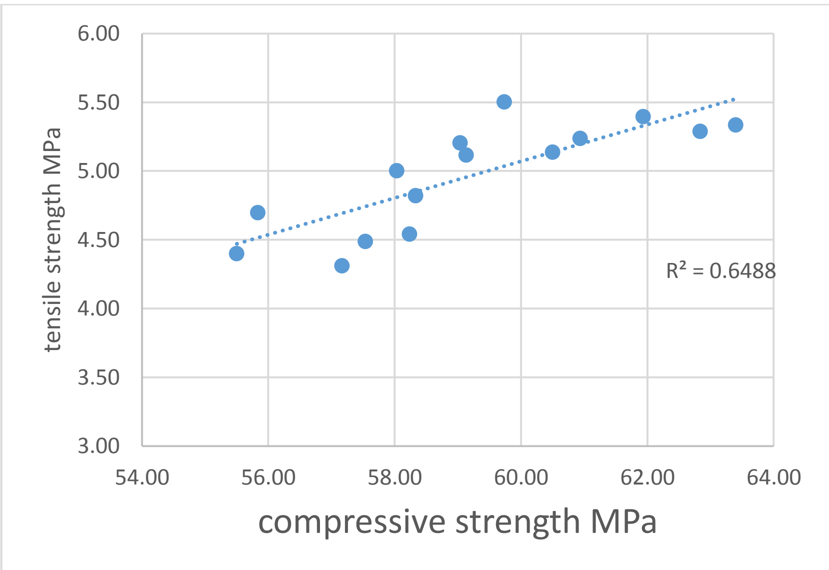


Figure (4-76) :Relationship Between Tensile Strength and Compressive Strength

4.7.3. Density and Absorption

The test results of absorption for each mix are listed in Figures (4-73) to (4-75). As presented in Table (4-6), the absorption ratio ranged between (0.6 and 0.9) % for the various mixes at each age.

The absorption of the brick mixes was noticeably higher than that of the reference mix about (0.89%) and that is mainly due to the nature of the LWA. However the mixes of limestone showed a drop of absorption compared to the reference.

Table (4-7) : Density and Absorption Data

	reference			5% brick			10% brick			5% limestone			10% limestone		
mix	R1	R2	R3	BA1	BA2	BA3	BB1	BB2	BB3	GA1	GA2	GA3	GB1	GB2	GB3
dry wt. gm	2490	2514	2530	2490	2493	2475	2460	2470	2480	2515	2486	2510	2505	2495	2490
saturated wt. gm	2510	2531.5	2550	2510	2515	2500	2480	2490	2500	2530	2502	2525	2525	2511.5	2505
Density gm/cm^3	2495	2518	2528	2485	2490	2470	2465	2470	2478	2505	2480	2500	2500	2490	2485
Mean density gm/cm^3	2513			2481			2471			2495			2492		
Mean aborption %	0.76			0.89			0.8			0.6			0.68		
Absorption %	0.80	0.7	0.79	0.80	0.88	1.0	0.81	0.81	0.8	0.59	0.64	0.59	0.79	0.66	0.60

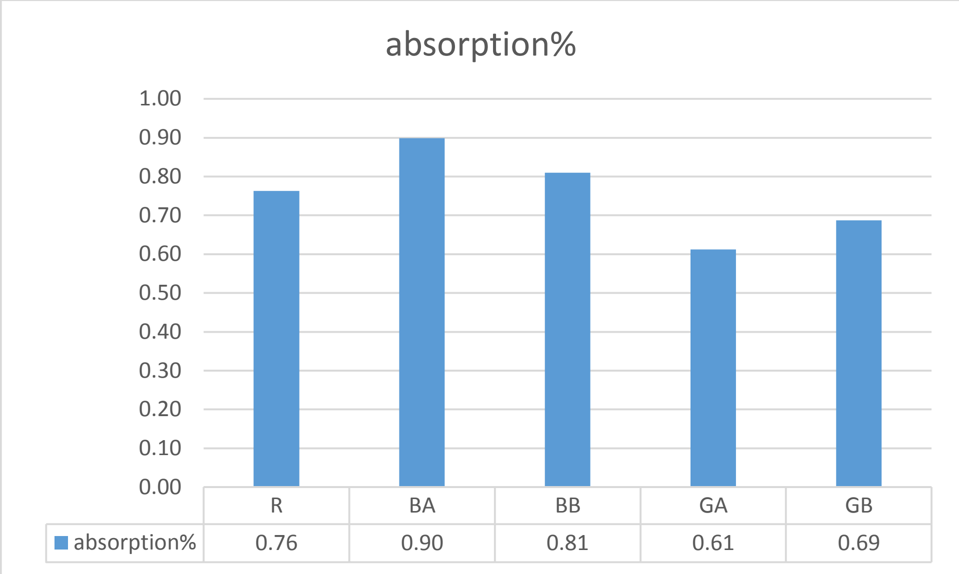


Figure (4-77): Absorption of The Hardened Concrete (%) for All Concrete Mixes

Also, the density of the mixes was affected by the added materials while the reference mix mean density was 2513 Kg/m3 the two brick mixes were (2481and 2471) Kg/m3 for the (5% and 10 %) replacement, respectively and the limestone mixes were (2495 and 2492) Kg/m3 for the (5% and 10%) replacement, respectively.

This decrease in density is probably due to the addition of LWA in these mixes.

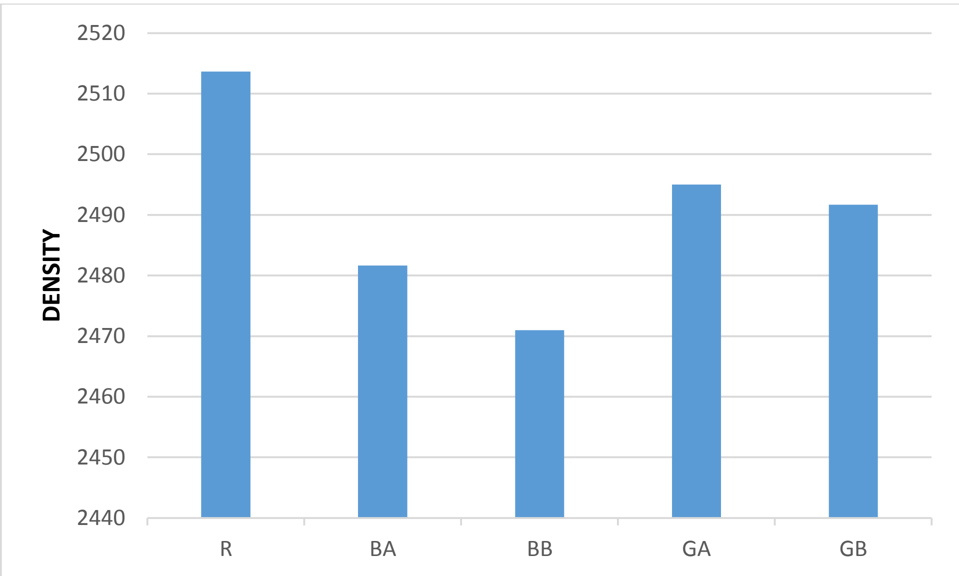


Figure (4-78): Mean Density (gm/cm^3) for All Concrete Mixes

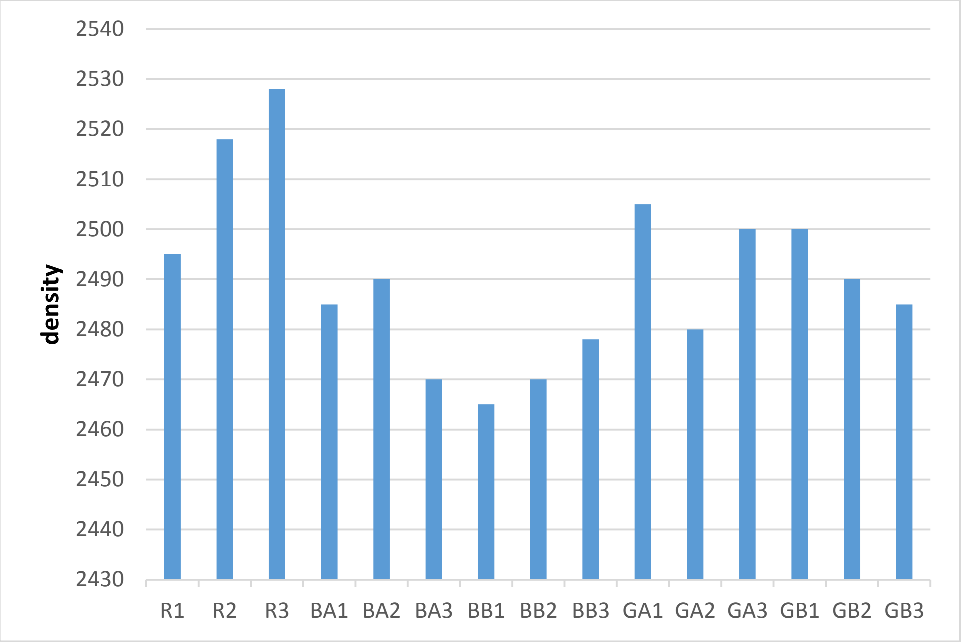


Figure (4-79): Density for All Mixes (gm/cm^3)

Conclusions and Recommendations

5.1. General

This chapter presents the conclusions that derived from the results of the experimental testing program. This study results was based on an experimental program regarded on the results of structural tests of a reinforced concrete beam tested on four points loading till failure. Also the mechanical properties were examined by testing a set of cubes and cylinders.

5.2. Conclusions

The main findings obtained from the results of this work are summarized as follows:

1. The use of crushed brick and limestone as internal curing materials was useful in improving the concrete properties in general and especially at the later ages.
2. The internal curing effect on the ductility was negative effect, where the ductility index of the reference mixes ranges between (3.1 and 3.3). While, the ductility index of the internally cured beams was less than 3 in all mixes.
3. The ultimate load capacity of the internally cured beams showed a noticeable improvement with a maximum increase of 23.5% for the 10% limestone beam at 90 days .
4. Internal curing had a remarkable effect on the HPC by improving the compressive strength with the time by percentage up to 13.6% for the mix of 10% limestone replacement at 150 days.
5. The splitting tensile strength was increased by using the internal curing by 14.8 % for the 5% limestone mix at 150 days.
6. The internal curing improved the toughness of the beams significantly, where the 10% limestone beam at 90 days had the highest toughness of 0.351 MPa which was higher than the toughness of the reference beams 0.311 MPa .

7. The density of the internally cured mixes was lower than the reference concrete mix which was (2513 gm/ cm^3). The highest density of the internally cured mixes was (2495 gm/ $cm^{3)}$ for the mix of 5% limestone .

8. The use of limestone showed better enhancement in all tested properties than the crushed brick mixes.

9. The use of 10% percentage replacement for both the two materials showed better improvement than 5% replacement.

10. The late ages of the internally cured beams had the highest load capacity, which indicates its remarkable improvement to the durability of concrete.

5.3. Recommendations For Future Works

Here are listed some of the suggestions for the future work that would examine more variables :

- Study the effect of fibers on the ductility of internally cured HPC beams.
- Study the effect of internal curing with ultra-high strength concrete.
- Study the structural behavior of internally cured pre-stressed high performance concrete.
- Study the effect of using three or more replacement percentages to obtain the optimum ratio.

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Appendix

Flexural Analysis for HPC :

$f_y = 530MPa,\ f_c^- = 55MPa,$

f_c^- (cylinder equivelnt) = 44 MPa

$\rho = 0.016,\ \rho_b = \beta_1 \dfrac{0.85 fc^-}{fy} \dfrac{600}{600 + fy}$

$\beta_1 = 0.65$

$\rho_b = o.65 \dfrac{0.85 * 44}{530} \dfrac{600}{600 + 530}$

$\rho_b = 0.0243,\ \rho_{\max} = 0.0182$

$\rho_{\min} = \dfrac{1.4}{fy} = \dfrac{1.4}{530} = 0.0026$

$\rho_{\min} < \rho < \rho_{\max}$

$\therefore$ **Section is flexural adequate.**

$Mu = 1.4 * 530 * 0.016 * (1 - 0.59 * 0.016 * \dfrac{530}{44})$

$Mu = 10.52kN.m$

$0.25P = 10.9 \Rightarrow P = 42.1kN(Totalload)$

(Check Shear)

$Vc = 0.17\sqrt{fc^-} * b_w d$

$Vc = 0.17\sqrt{44} * 100 * 125$

$Vc = 14.1kN$

$Vs = \dfrac{51}{65} * 125 * 520$

$Vs = 51kN$

$Vn = Vc + Vs = 14.1 + 51 = 65.1kN$

$Vu = 0.75 * 65.1 = 48.8kN$

$Totalload = 97.65kN$

Shear failure load > bending failure load

$\therefore$ **Bending failure controls**

الخلاصة

عوارض الجسور الخرسانية تعتبر واحدة من العناصر الانشائية الهامة في مشاريع البنية التحتية ، والتي تتطلب المتانة لضمان عمر خدمي طويل. أثبت استخدام مواد المعالجة الداخلية أنه مفيد في تعزيز خواص الخرسانة عالية الاداء. يهدف هذا البحث إلى دراسة تأثير المعالجة الداخلية باستخدام مواد فائضة عن الحاجة متوفرة محليًا عن طريق استبدال جزئي من الركام الناعم على السلوك الانشائي للجسور الخرسانية عالية الأداء. لذلك ، تم استخدام مادتين كعناصر معالجة داخلية في هذه الرسالة: الطابوق المطحون والحجر الجيري ، بنسبتي استبدال (٥٪ و ١٠٪) لكل مادة. تمت دراسة السلوك الإنشائي لخمسة عشر جسرا خرسانيا مسلحا بأبعاد (١٧٠٠ * ١٥٠ * ١٠٠) مم تمثل ثلاث اعمار للفحص (٢٨،٩٠ و ١٥٠) يوماً لكل مادة ونسبة استبدال بالإضافة إلى فحص الخواص الميكانيكية للخرسانة. أظهر استخدام المعالجة الداخلية زيادة في قدرة التحميل النهائية للجسور ، مع تحسن بنسبة ١١.٥٪ في ٢٨ يومًا لخلطات الطابوق بنسبة ٥٪ و ٢٣.٥٪ لخلطات الحجر الجيري بنسبة ١٠٪ في ٩٠ يومًا و ١٢.٦٪ لحجر الجير عند ١٥٠ يوما. أيضا ، وكذلك تحسين قوة الانضغاط بنسبة ١٣.٦ ٪ لخلطات ١٠ ٪ من الحجر الجيري. علاوة على ذلك ، لوحظ تحسن في مقاومة الشد للخلطات ذات المعالجة الداخلية تصل إلى ١٤.٨٪ لخلطة الحجر الجيري بنسبة استبدال ٥٪. كذلك، زادت متانة الجسور ، حيث كانت خلطة الحجر الجيري بنسبة استبدال ١٠ ٪ في ٩٠ يوما أعلى متانة مسجلة بمقدار (٠.٣٥١) ميغا باسكال و التي كانت أعلى من الخلطات المرجعية. على الرغم من ذلك ، كان لمواد المعالجة الداخلية تأثير سلبي على المرونة للخلطات ، حيث كانت معاملات المرونة تتراوح بين للخلطات المرجعية (٣.٣ إلى ٣.٦) ، ولكن جميع الخلطات المُعالجة داخليًا كان لديها معامل مرونة أقل من (٣). أظهر استخدام الخرسانة عالية الاداء المعالجة داخليًا تحسنًا جيدًا في كل من السلوك الانشائي والخواص الميكانيكية للخرسانة وأظهرت الاعمار المتأخرة قدرة تحمل نهائية عالية مما يعني أن المعالجة الداخلية عززت ديمومة العناصر الانشائية.

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

السلوك الانشائي للجسور المسلحة من الخرسانة عالية الاداء والمعالجة داخلياً باستخدام مواد فائضة متوفرة محلياً

مقدمة
إلى كلية الهندسة في جامعة كربلاء
قسم الهندسة المدنية
كجزء من متطلبات نيل درجة الماجستير
في علوم الهندسة المدنية
(البنى التحتية)

من قبل
احمد عباس شريف
(بكالوريوس هندسة مدنية – جامعة بغداد - ٢٠٠٩)

إشراف

أ.م.د. ليث شاكر رشيد
أ.م.د. علي طالب جاسم

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