

STRENGTH PROPERTIES AND SHRINKAGE BEHAVIOUR OF ROLLER COMPACTED CONCRETE

A thesis submitted in fulfillment of the requirement for the award of the degree of

DOCTOR OF PHILOSOPHY
in
Civil Engineering

Submitted by

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DECLARATION

I, Sorabh Saluja, hereby declare that the thesis titled, "Strength Properties and Shrinkage Behaviour of Roller Compacted Concrete" submitted to Thapar Institute of Engineering and Technology (Deemed University), Patiala, in fulfillment of the requirements for the award of degree of "Doctor of Philosophy" in Civil Engineering is a record of an original and independent research work done by me during the period 2012-2018, under the supervision and guidance of Dr. Shweta Goyal, Associate Professor, Department of Civil Engineering, Thapar Institute of Engineering and Technology, Patiala and Dr. Bishwajit Bhattacharjee, Professor, Department of Civil Engineering, Indian Institute of Technology Delhi, New Delhi. The work contained in this thesis has not been previously submitted to meet the requirements for a degree or diploma at this or any other higher education institution.



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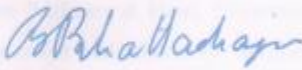
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CERTIFICATE

Certified that the work presented in this thesis titled "**Strength Properties and Shrinkage Behaviour of Roller Compacted Concrete**" which is being submitted by **Mr. Sorabh Saluja (Regd. No. 951002006)** in fulfillment of the requirements for the award of degree of "**Doctor of Philosophy**" in the **Department of Civil Engineering, Thapar Institute of Engineering and Technology, Patiala, India**, is an authentic record of the research work carried out by him under our supervision and guidance. The contents of this thesis, in full or in part, have not been submitted to any other Institute or University for the award of any degree or diploma.



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(SORABH SALUJA)

ABSTRACT

Roller compacted concrete (RCC) is a dry concrete that exhibits zero slump. It gained popularity because of economy, strength and fast placement. Initially, the adoptability of RCC was limited to dam construction because of lower heat of hydration. RCC pavements have gained momentum in the last decade because of sustainability and initial cost benefits. Other benefits of RCC include higher structural capacity per unit thickness, higher sustainability rating and earlier opening to traffic.

In order to attain sustainable development with RCC, some industrial waste products like fly-ash, ground granulated blast furnace slag (GBBS) can be used in high volumes as supplementary cementitious materials. In the present study, the effect of GGBS as partial replacement of Ordinary Portland cement (OPC) is explored in terms of strength properties and in service performance of RCC. The RCC mixes were prepared by using two types of coarse aggregates viz. crushed gravel (CG) and limestone (LS). The mix proportion of RCC was finalized based upon soil compaction approach.

The RCC mixes were evaluated on the basis of compressive strength, split tensile strength, flexural strength, abrasion resistance and drying shrinkage strain. The effect of maximum ambient temperature i.e. $70 \pm 2^\circ\text{C}$ on the shrinkage behaviour was also studied under two categories. Under the first category, specimens were subjected to a constant temperature of $70 \pm 2^\circ\text{C}$ up to a period of 270 days; while in second category, temperature cycles of 12 hour heating at $70 \pm 2^\circ\text{C}$ followed by a cooling at $10 \pm 2^\circ\text{C}$ for a period of 12 hour were applied on the specimens for a period of one month. Further, all the strength properties, abrasion resistance and drying shrinkage of RCC were correlated to the compressive strength of RCC by performing different regression analysis.

The strength properties results show that incorporation of GGBS into the RCC mix, leads to lower strength at early curing age, followed by strength higher than the control mix. The improvement in later age strength of mixes containing GGBS indicates that the glossy components in GGBS react slowly with water. RCC mix at 40% replacement level of cement

with GGBS presents the highest strength values as compared to other replacement levels. The strength results were further supported by Scanning Electron Microscopy (SEM) and stoichiometric analysis. Depth of wear results also show that replacement of cement with GGBS leads to improvement in abrasion resistance after 28 days of curing period.

The variation in aggregate type also had significant influence on the properties of resultant mix. RCC mix with limestone aggregates presents higher strength during the initial curing age, while mixes incorporating crushed gravel aggregates registered higher later age strength. Observed higher strength at early age with limestone aggregates can be attributed to the accelerated hydration occurring in mixes with limestone aggregates; while in mixes incorporating crushed gravel aggregates, hydration occurred at a steady rate. RCC mixes with limestone aggregates presents marginally better abrasion resistance as compared to crushed gravel aggregates, at all curing ages.

Drying shrinkage strain results demonstrate that GGBS incorporation into the RCC mixes exhibited higher shrinkage strain than the control mix because of higher volume of paste and denser matrix produced by the GGBS particles. Mixes incorporating crushed gravel experienced 23% higher shrinkage strain than the mixes having limestone aggregates at end of drying duration which can be attributed to lower modulus of elasticity of crushed gravel as compared to limestone. The least shrinkage strain was observed in the mix having limestone aggregates and 20% GGBS incorporation level.

Thermal shrinkage strain results show that heating the specimens at $70 \pm 2^\circ\text{C}$ accelerated the shrinkage at early duration of heating because of acceleration in hydration reaction of cement. The variation in aggregate type and incorporation of GGBS effects the thermal shrinkage strain similar with the drying shrinkage strain. Heating and cooling cycles led to large initial variation in thermal strain, which reduced with the progressive increase in heating and cooling cycles irrespective of aggregate type and GGBS content. Further, the failure cycles predicted from the S-N curve shows that all prepared RCC mixes will sustain atleast one millions heating and cooling cycles.

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ABBREVIATIONS

Abbreviation	Word(s)
RCC	Roller compacted concrete
OPC	Ordinary Portland cement
GGBS	Ground granulated blast furnace slag
RH	Relative humidity
C-S-H	Calcium silicate hydrate
Ca(OH) ₂	Calcium hydroxide
ITZ	Interfacial transition zone
SEM	Scanning electron microscopy
EDS	Energy dispersive spectroscopy
OMC	Optimum moisture content
ANOVA	Analysis of variance

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CHAPTER 1

INTRODUCTION

1.1 GENERAL

In different parts of the world, concrete has been used in pavement construction for the last several decades. Construction of pavement using roller compacted concrete (RCC) is relatively recent in India, and studies need to be carried out for the use of supplementary materials as an alternate to conventional materials for sustainable development and to make RCC pavements construction more relevant to the local conditions.

ACI 207.5R (1988) define RCC as special concrete trampled by roller compaction. It is a zero-slump concrete consisting of dense-graded aggregates, cementations materials and water. Due to zero slump consistency; it is placed in successive layers, usually not more than 10 inch (254 mm), using earthmoving construction methods (*ACI 325.10R, 2001; Harrington et al., 2010*). The layers are compacted with steel wheel vibratory rollers, with final compaction sometimes provided by rubber tire rollers. When vibratory compaction is added to these individual layers, the resulting material exhibits comparatively higher compressive strengths than the conventional concrete using similar quantities of Portland cement.

RCC has gained popularity because of economy, strength and fast placement (*ACI 325.10R, 2001; ACI 207.5R, 1988*). *Hesami et al. (2016) and Vahedifard et al. (2010)* reported that RCC require 20-28% lesser cementitious material content to attain the same range of compressive strength as that of conventional concrete. *Pittman D.W. (1994)* suggested that 60% cost saving can be achieved with the use of RCC pavements as compared to conventional concrete pavements. Additionally, low water-cement ratio used in RCC leads to lesser cement paste in the concrete matrix, thus leading to lower shrinkage which reduces the possibility of cracks in RCC pavements.

Initially, the adoptability of RCC was limited to dam construction because of lower heat of hydration. As per the statistics presented in Fig. 1.1, RCC pavements have gained momentum in the last decade because of sustainability and initial cost benefits. As a pavement material, it is

used for a wide variety of heavy vehicle applications such as lumber storage, log sorting yard, military vehicle roads, parking areas, bulk storage areas, forestry haul roads, airfield, secondary highways, truck and automobile parking etc (ACI 325.10R-95, 2001; Yazici *et al.*, 2015). For dam construction, RCC mix requires lower binder proportion, larger nominal maximum aggregate size, higher water-cement ratio and higher consistency than RCC mix used for pavements.

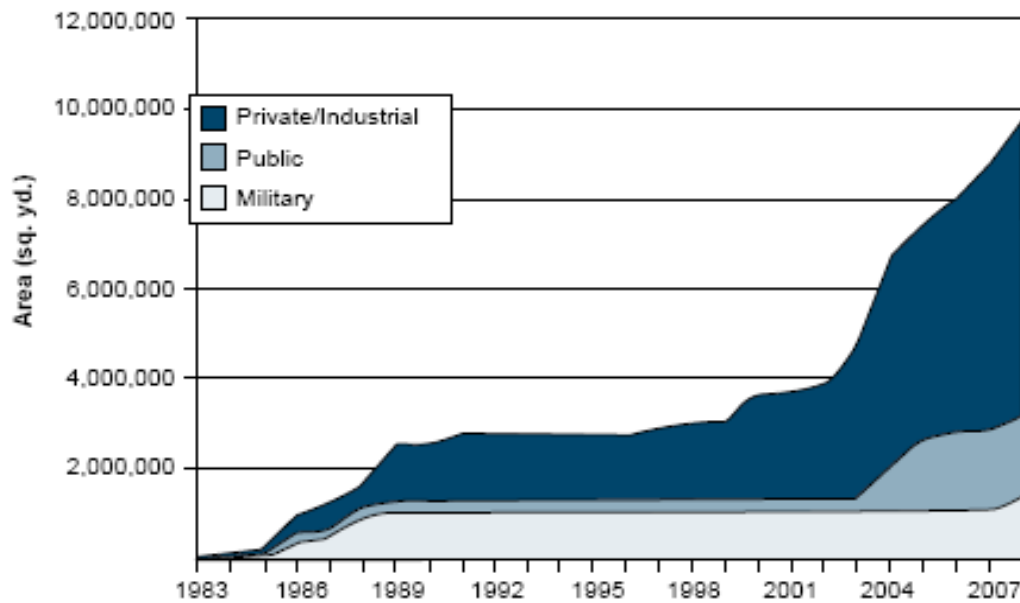


Fig. 1.1: RCC pavement construction statistics (Pittman and Anderton, 2009)

Pavement construction with RCC is economical as compared to the conventional concrete because of the cost saving achieved due to absence of formworks, reinforcing bars and dowel bars. The transverse cracking in RCC pavements is prevented through the denser particle structure leading to good interlock between aggregates and the subbase interface (Zollinger, 2016). RCC pavements are usually designed to carry traffic directly on the finished surface. A wearing course is not normally used, although a hot mix asphalt overlay is sometimes added for smoothness or rehabilitation. Various features and benefits of RCC pavements are summarized in Table 1.1.

Table 1.1: Various features and benefits of RCC pavements (*Harrington et al., 2010*)

S.No.	Features	Benefits
1.	High flexural strength (3.5 MPa to 7.5 MPa)	Supports heavy, repetitive loads without failure which reduces maintenance cost
2.	High compressive strength (28 MPa to 69 MPa)	Withstands high concentrated loads and impacts from heavy industrial, military and mining applications
3.	High shear strength (2.8 MPa to 5.1 MPa)	Eliminates rutting and subsequent repairs
4.	High density, low absorption	Provides excellent durability, even under freeze- thaw conditions; eliminates seepage through pavement
5.	Low water content, low water-cement ratio	Increases strength, reduces permeability, and enhances durability and resistance to chemical attack
6.	Aggregate interlock	Provides high shear resistance at joints and uncontrolled cracks to prevent vertical displacement or faulting
7.	No dowels bars and formwork	Speeds and simplifies construction, reduces cost
8.	No formed or sawed joints	Speeds construction, reduces cost

1.2 RCC V/S CONVENTIONAL CONCRETE

The basic differences between the RCC and conventional concrete are summarized as follows:

- (1) The constituents of RCC are similar to those of conventional concrete while the construction and placement procedure is similar to asphalt pavement (*Rao et al., 2016 a,b*). The mix design procedure of RCC is also different from the conventional concrete design. The relative percentage of basic ingredients used in conventional concrete and RCC are shown in Fig. 1.2. RCC uses higher amount of fine aggregates as compared to the conventional concrete. It consists of lesser volume of water, coarse aggregates and cementitious materials as compared to conventional concrete. The use of higher amount of fine aggregates fills the air voids in the

RCC pavements. Also, the fine aggregates are closely packed to each other which provide aggregate interlock and contribute towards initial load carrying capacity of the pavement.

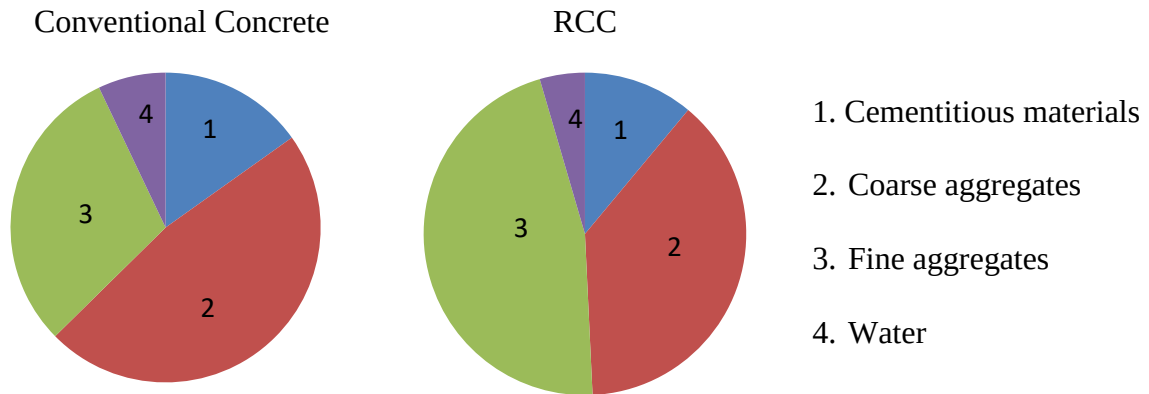


Fig. 1.2: Material proportion comparison between conventional concrete and RCC

- (2) RCC mix is relatively dry enough to be compacted by concrete paving machines; while in conventional concrete sufficient amount of free water is required to attain desired workability.
- (3) Surface of RCC pavement can be finished through diamond grinding while conventional concrete is mechanically textured to improve friction.
- (4) In RCC pavements, load transfer mechanism occurs through aggregate interlock; while in conventional concrete pavement, transverse dowel bars are provided for this.
- (5) In RCC pavements, high-density paving screeds followed by steel drum and rubber tired rollers causes the consolidation; while in conventional concrete pavements, it occur through internal paving machine vibrators.
- (6) The surface of RCC pavements is light coloured as compared to conventional concrete pavement (Fig. 1.3). The light colour surface texture reduces lighting requirements for parking and storage areas (*Harrington et al., 2010*).

1.3 CONSTRUCTION PROCESS OF RCC

RCC is generally mixed using high capacity continuous mixing equipment to provide uniform mixing of cementitious materials, aggregates and water. RCC is placed and compacted using the similar equipment utilized in the soil compaction. Consequently, RCC construction does not require any pre and post placement treatment; which is required for conventional

concrete construction. Typical construction process such as framework, use of dowel bars and surface finishing are not required in RCC pavement construction (*Vahedifard et al., 2010*).



Fig. 1.3: Surface texture of conventional concrete pavement and RCC pavement
(*Harrington et al., 2010*)

The equipment used for RCC production includes transit mixers, tilt drum mixers, mobile truck mixers and horizontal shaft mixers. The production equipment should be able to produce a uniform mixture at a rate compatible with the rate of placement. The mixed RCC material is delivered by trucks or conveyors and must be placed and compacted within an hour from the time it is mixed. Prior to the placement of RCC, the sub-grade should be graded and compacted so that a good and stable platform is available for paving. The placement of RCC can be done either with the use of asphalt-paver, high density paver or conventional paver in layers (Fig. 1.4a). After the placement, compaction of RCC is carried out so that the RCC pavements meet density requirements. Smooth wheel vibrating rollers or pneumatic tire rollers are generally used to achieve compaction of RCC surface (Fig. 1.4b). Thus, construction process results in a dense concrete layer that can be opened to traffic earlier than the conventional concrete pavement (*Lahucik J.R., 2016; Halsted G.E., 2009; Ozcan S., 2008*).



(a)



(b)

Fig. 1.4: (a) Placing of RCC with high density paver (Lahucik, 2016)

(b) Compaction of RCC with vibrating roller (Ozcan, 2008)

1.4 HISTORY OF RCC CONSTRUCTION

The first RCC pavement was introduced in North America by the U.S. Army Corps of Engineers (USACE) for runway project at Yakima, Washington in 1942 (ACI 325.10R, 2001). Another early application of RCC was the construction of two freight yards in Hokkaido, Japan in 1956 (Hiller J.E., 2000). The application of RCC pavement for the log sorting and storage yard was reported in 1976 in Canada. In 1978, the area of the same yard was expended and a thin layer of asphalt was applied to improve pavement smoothness. During the tenure of 1976-1978, three log sorting yards of RCC was constructed in the harsh climatic environment of British Columbia (ACI 325.10R, 2001).

With the ongoing success of RCC construction in British Columbia, Canada, various organizations in the United States also started showing their interest in RCC construction. The first large scale RCC pavement construction was done by the U.S. Army Corps of Engineers at Ft. Hood, Texas, in 1984. A parking area of 16.7 m^2 , for armored vehicles and tanks, was built in Texas in 11 days period, which resulted in 15% cost saving over a conventional concrete pavement (Hiller J.E., 2000). During the period of 1986-1988, large area of RCC pavement was constructed at Conley and Moran marine Terminal, Boston. The largest RCC pavement projects

undertaken to date include; more than 55 hectares of RCC pavement constructed in Tennessee and 36 hectares of RCC pavement constructed in Ft. Drum, New York (*ACI 325.10R, 2001*).

In India, the first RCC dam was constructed in 2006 in Maharashtra called Ghatghar dam, which was the 10th fastest constructed dam in the world (en.wikipedia.org/wiki/Ghatghar_Dam). Since this technology was new to Indian scenario, help of experts from U.S and Japan was taken for the mix design. Cost benefits for this project were achieved due to speed of construction, efficient use of machinery and use of fly ash.

From the referred RCC history, it is observed that a major application of RCC has been in various states of U.S and Canada having cold climate conditions. In cold regions, problems due to de-icing salt, freeze-thaw resistance have great influence on properties of RCC. Some studies have investigated the performance of RCC pavements subjected to continuous hot climatic conditions (*Deghfel et al., 2019*). In India, climatic conditions are different, with the temperature variations ranging from very hot (45-50°C) during summer to cold (5-10°C) during winters. Under this temperature variation conditions, pavements behave differently and the performance needs to be thoroughly investigated.

1.5 SCOPE AND OBJECTIVES OF PRESENT RESEARCH WORK

RCC was initially developed to produce a material exhibiting the structural properties of concrete with the placing characteristics of embankment materials. The design mix proportion of RCC differs from the conventional mix design and follows two design philosophies. One philosophy is based upon concrete approach while the other is based upon soil (geotechnical) approach (*ACI 211.3R, 2002; Madhkhan et al., 2012*). It was established that concrete approach is recommended for relatively massive sections such as dams where nominal maximum size of aggregates is 37.5 mm or larger. For relatively thin sections such as pavements or slabs generally containing aggregates of maximum nominal size of 20 mm, soil compaction approach is recommended as per ACI 211.3R (*ACI, 2002*). The basic principle involved in concrete approach is consolidation i.e. lowering of water-cement ratio; while, soil approach is based on compaction i.e the moisture-density relationship. The resultant mixes achieved by both of these approaches should have zero slump to be considered as RCC. It is said that, if other factors are kept constant,

RCC mixes obtained by using soil approach have higher compressive strength and better abrasion resistance as compared to mixes designed by using concrete approach (*Hansen and Reinhardt, 1991*). Also, soil approach is comparable to the field application of RCC in pavements since the compaction applied by surcharge in laboratory can correspond to the effort applied by the rollers in the field (*U.S. reclamation, 2017*).

The growing concern in concrete technology is to attain sustainable development, by minimizing energy consumption, reducing carbon foot prints and by using industrial by-products into concrete. To achieve this objective, many recycled and marginal aggregates, industrial by-products such as fly ash, silica fume, pumice etc. and locally available building materials were attempted to be utilized for manufacturing RCC. Till date, number of researchers investigated the effect of supplementary cementitious materials such as fly ash, silica fume, coal bottom ash and pumice on the strength and durability properties of RCC (*Cao et al., 2000; Naik et al., 1995; Atis et al., 2004; Atis, 2005; Ute, 2008; Vahedifard et al., 2010; Aghabaglou and Ramyar, 2013; Dillon et al., 2015; Rao et al., 2016a, Lam et al., 2018*).

Another industrial by-product that can be utilized as replacement of cement in RCC is ground granulated blast furnace slag (GGBS), which is generated during the manufacture of iron from its ore. Due the manufacturing process, amount of iron and slag produced are of same order (*Oner and Akyuz, 2007*). It is reported that production of 1 tone of GGBS emits approximately 0.07 tone of CO₂ equivalent which is very small amount as compared to CO₂ emitted (0.95 tone) during production of equivalent amount of OPC (*Arivalagan, 2014*). In order to reduce the green house gas emission, replacement of cement with GGBS has gained momentum because of cement and energy saving, environmental and socio-economic benefits. In India, 36 Mt/year of GGBS is produced from the steel and iron industry (www.abbujacement.com). It is reported that merely 20% of the steel slag so produced is utilized, while the unutilized slag is indiscriminately dumped in nearby landfills (*Pattanaik et al., 2019*). It is also pointed out here that, in many parts of the world thermal power point may be absent but the steel plant is always there, hence potential utilization of GGBS in RCC needs investigation, both from sustainability as well as reduction in heat of hydration point of view.

Some of the researchers investigated the effect of GGBS as replacement of cement in conventional concrete. Literature suggested that GGBS can be effectively used as supplementary cementitious material upto 50% replacement level of cement (*Elahi et al., 2010; Oner and Akyuz, 2007; Berndt M.L., 2009; Prithika and Sekar, 2016*). Although GGBS tends to decrease the early age strength of concrete but it improves the workability, later age strength and durability of the conventional concrete (*Swamy and Bouikni, 1990; Babu and Kumar, 2000; Li and Zhao, 2003; Oner and Akyuz, 2007; Liu et al., 2014; Chidiac and Panesar, 2008*). While an extensive research has been carried out on the utilization of GGBS in conventional concrete, limited research work has been reported on its effect when used for making RCC (*Karimpour A., 2010; Rao et al., 2016b*).

Apart from the effect of cementitious materials, the strength and durability properties of concrete are also influenced by many factors related to the properties of aggregates. It was reported that coarse aggregates characteristics; such as quality, type, surface texture, moisture content etc. play an important role in defining the behaviour of conventional concrete (*Mokarem et al., 2005; Yuan et al., 2015; Al-Attar, 2008; Zhang et al., 2013; Fujiwara, 2008; Seo et al., 2010; Lee et al., 2006*). The effect of replacement of natural aggregates with the recycled aggregates on mechanical performance of RCC was studied by *Courard et al. (2010)* and *Debieb et al. (2009)*. The effect of locally available aggregates on the basic properties of RCC needs to be investigated.

Based on the extensively studied literature on RCC, followings objectives were finalized for the present study:

3. To achieve roller compacted concrete with OPC and GGBS using soil compaction approach.
4. To study the effect of aggregate type and GGBS content on strength properties of roller compacted concrete; namely, compressive strength, split tensile strength and flexural strength.
5. To study the effect of aggregate type and GGBS content on abrasion resistance, drying shrinkage and thermal shrinkage of roller compacted concrete.
6. To develop regression models correlating various properties like; split tensile strength, flexural strength, abrasion resistance and drying shrinkage with compressive strength.

1.6 ORIENTATION OF THESIS

The thesis has been organized in six chapters. In the first chapter, introduction about the topic including background of RCC construction has been presented. The objectives of the present study have also been laid out in this chapter.

In the second chapter, literature review regarding the use of supplementary cementitious materials, variation in aggregate type on strength and durability properties of conventional concrete as well as RCC are presented.

The third chapter focuses on the achievement of RCC by using soil compaction approach. Properties of various materials used throughout in this study are discussed in this chapter. The details regarding finalization of RCC mix proportions are presented in detail along with the effect of GGBS and coarse aggregates variation on optimum moisture content and corresponding dry density.

In the fourth chapter, the effect of GGBS content and coarse aggregate type on various strength properties like compressive strength, split tensile strength and flexural strength are discussed. The SEM and stoichiometric analysis support the compressive strength results. On the basis of statistical analysis, the relationships between various strength properties are developed and are presented in the chapter.

In the fifth Chapter, various in-service properties like abrasion resistance, drying shrinkage and temperature effect on shrinkage behaviour of RCC are presented. On the basis of experimental data, relationship between compressive strength and abrasion resistance as well as multi variable regression model for prediction of shrinkage strain are developed and are presented in this chapter.

Sixth chapter deals with the relevant conclusions drawn from the present study and suggestion for the future work. At the end, references cited in the entire work are presented.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

Roller Compacted concrete (RCC) is a special type of concrete that has zero slump and generally has mix constituents similar to conventional concrete, but with different mix proportions. RCC has now been commonly used all around the world for the construction of dams and pavements. RCC pavements gained popularity over conventional concrete pavements because of reduced material and construction cost, earlier opening to traffic, structural capacity per unit thickness and sustainability rating (*Lahucik J.R., 2016*). RCC pavements are also beneficial for areas that endure high point load from machinery, such as loading docks and shipping yards, since it can withstand the loads that would normally deform the asphalt pavement (*Dillon et al., 2015*).

American Concrete Institute (ACI) defines RCC as “concrete compacted by roller compaction and will support roller load in plastic stage while being compacted” (*Gedeon G., 2000; Aghaeipour and Madhkhan, 2017*). The properties of RCC strongly depend upon various parameters like; cementitious material content, aggregate type, mixture proportioning, degree of compaction and quality control measures. The basic ingredients of RCC are similar to the conventional concrete but the mix design and construction procedures are different from the conventional concrete. RCC mix should be dry enough to support the load of vibratory rollers and sufficiently wet for the uniform distribution of paste binder throughout the mass.

In order to make RCC cost effective and environmentally sustainable, some industrial by-products like fly-ash, ground granulated blast furnace slag (GGBS) can be used as supplementary cementitious materials for the production of RCC. Further, in RCC production, aggregate comprises approximately 75-80% volume of the mix and therefore, sustainable alternatives for aggregates can significantly affect the overall economy of RCC. Thus, a judicious combination of cementitious materials and aggregate type can lead to sustainable development of RCC. A detailed review on the effect of utilization of various sustainable materials in the production of RCC has been presented in this chapter.

2.2 CONSTITUENTS OF RCC

The basic materials used to produce RCC include cementitious materials, fine aggregates, coarse aggregates and water. The detailed review on these materials is discussed in the following sections.

2.2.1 Cementitious Materials

The rate of hydration and strength development significantly depends upon the type of cementitious material used. For RCC mass applications, cement with lower heat of hydration is generally considered more beneficial. To further economize the overall construction cost of RCC pavements, use of blended cements, consisting of Portland cement and a pozzolanic material is considered (Ozcan S., 2008; ACI 325.10R, 2001). The use of pozzolanic materials is quite common for RCC projects, which results in reduced cost and lower heat generation. The detailed guidelines for the selection of cementitious materials for mass application and for pavement is provided in ACI 207.5R (ACI, 1988) and ACI 325.10R-95 (ACI, 2001) respectively. Based upon the strength and durability requirements, ACI 211.3R-02 (ACI, 2002) prescribed the range of cementitious material as 10 -17% of total dry mass of aggregates for RCC pavements.

2.2.2 Aggregates

The aggregates used in RCC projects must be dense graded in order to provide stability during and after construction and to minimize voids in the mix. Since, the volume of paste is lesser in RCC as compared to conventional concrete; therefore, gap graded aggregates are avoided in RCC construction (U.S. Reclamation, 2017). In RCC pavements, aggregates constitute approximately 75-80% volume of the mix and therefore significantly affect the fresh and hardened properties of RCC. Aggregates used in RCC pavement applications include both fine and coarse aggregates, although the fractions are pre-blended as a single aggregate blend (U.S. Reclamation, 2017). Since, RCC mixture is less cohesive than the conventional concrete; therefore, aggregate segregation is a major concern in RCC production. In order to minimize segregation problem and to provide a relatively smooth pavement surface, ACI 211.3R-02 (ACI, 2002) recommends the combined aggregate grading limits which are summarized in Table 2.1. Generally, nominal maximum size of the aggregates should not exceed 19 mm, and the allowable percentage passing through 75 μ m sieve should range between 2-8% (Halsted G.E., 2009).

Table 2.1: Recommended combined aggregate grading limit for RCC pavements (ACI, 2002)

Sieve Size (mm)	Cumulative percent passing (%)
25	100
19	82-100
12.5	72-93
9.5	66-85
4.75	51-69
2.36	38-56
1.18	28-46
0.600	18-36
0.300	11-27
0.150	6-18
0.075	2-8

2.2.3 Water

The water content of RCC mixes typically range between 4.6 to 5.6% of the weight of dry aggregates and cementitious materials (*Halsted G.E., 2009*). Quality of water for RCC pavements is governed by the requirement similar to the conventional concrete.

2.2.4 Admixtures

Chemical admixtures, such as water reducing admixture and retarding admixtures have limited use in RCC (*ACI, 2001*). It was reported that whenever, any admixture is used in RCC mix, extensive investigation should be conducted to determine the effectiveness and dosage rate (*Halsted G.E., 2009*).

2.3 SUPPLEMENTARY CEMENTITIOUS MATERIALS

Across the world, issue of global warming has put intensive pressure over all industries to reduce the CO₂ emission levels, which is considered as a major green-house gas. Production of cement generates 5-8% of global CO₂ emissions; directly through calcinations of lime stone and

indirectly by burning of fossil fuels in the kiln (*Andrew R.M., 2018*). It has been reported that 1.5 tones of mineral extractions along with 5000MJ of energy is consumed for the production of 1 tone of ordinary Portland cement (OPC) and this process emits 0.95 tone of CO₂ equivalent (*Elchalakani et al., 2014; Arivalagan S., 2014*). India stands as the second largest producer of cement across the world with a current consumption rate of 1m³/person/year (*Manjunatha et al., 2014*). Increased growth in cement consumption not only leads to abnormal rise in the construction cost but also has an adverse effect on the environmental health.

Another environmental concern for today's world is management of industrial waste generated due to exponential growth in industrialization and urbanization. Some of these waste products have the potential to be used as partial replacement of cement. The utilization of these industrial by-products as cementitious materials in RCC will serve the dual purpose of greenhouse gas reduction and financially viable construction. In order to reduce the green house gas emission, replacement of cement with supplementary cementitious materials has gained momentum. The various waste products such as fly ash, GGBS, rice husk, electric arc furnace slag, silica fume and pumice have the potential to be used as cementitious materials (*Dillon et al., 2015; Lam et al., 2018; Ute A.A., 2008; Aghabaglou and Ramyar, 2013; Bhat et al., 2018; Anupam et al., 2014; Akhtar and Sarmah, 2018; Vahedifard et al., 2010*). Out of all supplementary cementitious materials, only fly ash and GGBS are used at high volume replacement levels of cement, while other materials are used at low replacement levels. Thus, the manufacturing process including the production rate of these two industrial byproducts viz. GGBS and fly ash are presented in the following sections.

2.3.1 Ground Granulated Blast Furnace Slag (GGBS)

GGBS is a by-product from iron and steel manufacturing industry. During the manufacturing of steel, equal amount of coke, iron ore and limestone are fed in the blast furnace and operated at a temperature of about 1500°C. During the process of heating and further reactions, equal amount of molten iron and molten slag are produced inside the furnace (*Oner and Akyuz, 2007*). The molten slag, being lighter, floats above the molten iron inside the furnace and is rapidly water quenched after tapping off the molten iron. Fig. 2.1 shows the manufacturing process of GGBS inside the blast furnace. The water quenched molten slag results in the formation of glassy granulate which is dried and ground to the required size. The final product is

called GGBS. It was reported that production of 1 tone of GGBS emits approximately 0.07 tone of CO₂ equivalent which is a very small amount as compared to the corresponding CO₂ emissions during the manufacture of 1 tone of OPC (0.9 tone) (Arivalagan S., 2014; Akhtar and Sarmah, 2018).

GGBS so obtained constitutes of a mixture of alumina, silica, magnesium oxide and lime; which are also the major constituents of Portland cement but in different proportions, thus indicating the potential of GGBS to be used as replacement of cement. The utilization of GGBS as a cementitious material in the construction industry will lead to sufficient reduction in CO₂ emission.

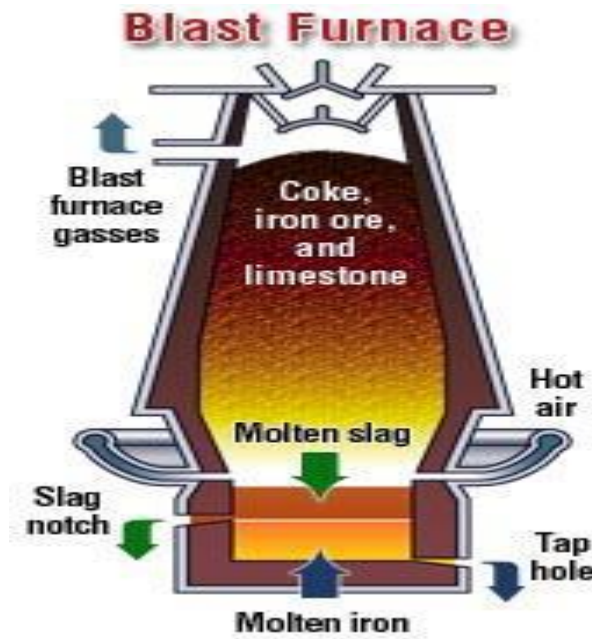


Fig. 2.1: Sources of Ground Granulated Blast Furnace Slag (Suresh and Nagaraju, 2015)

The world steel association statistics indicate that steel production increased by approximately 100% in the last 20 years as shown in Fig. 2.2. The increase in steel production ultimately leads to increased growth of GGBS production. According to the report published by World Steel Association in 2017, the five top most steel producing countries are China followed by Japan, India, U.S.A and Russia (www.worldsteel.org), with India and Japan producing almost similar amount of steel. As per Global Slag Conference (2018) report, only 75% of globally produced blast furnace slag is granulated, creating around 250 Mt/year of GGBS worldwide

(www.globalslag.com). Global CCS institute reports that China approximately produces 150 Mt/year of GGBS annually from the 49 modern slag grinding plants. In Japan, nearly 30 Mt/year blast furnace slag was produced, out of which about 75% is utilized in cement industry and remaining is used in other applications such as soil improvement etc (hub.globalccsinstitute.com).

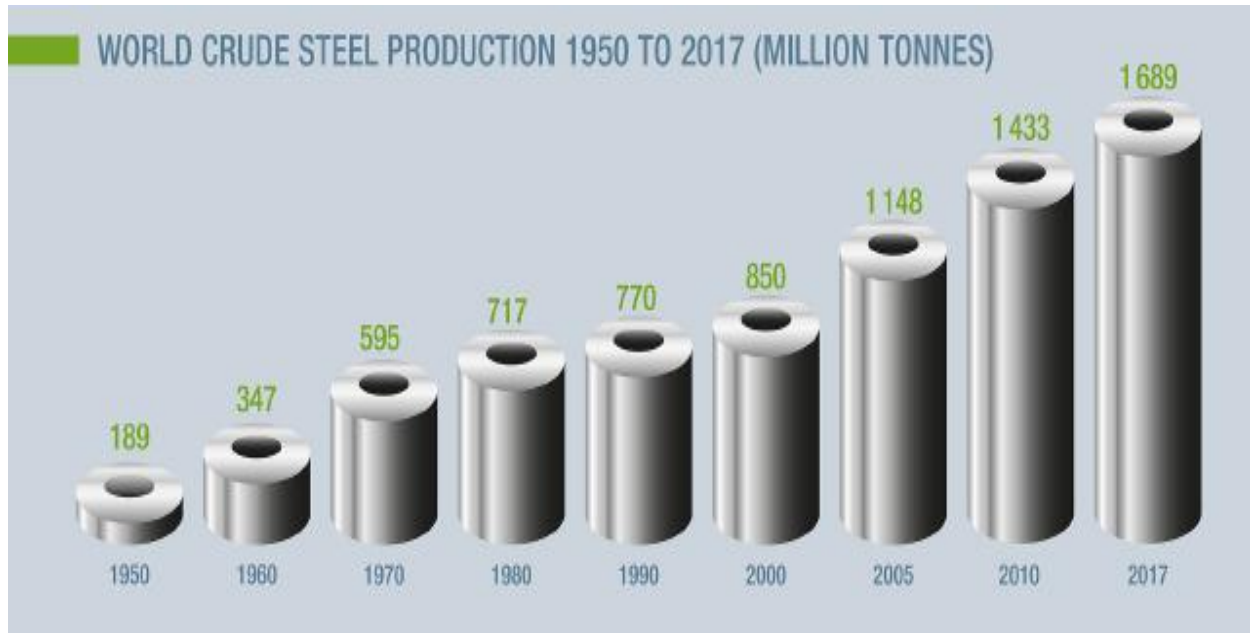


Fig. 2.2: Statistics of steel production (www.worldsteel.org)

In India, 12 Mt/year GGBS is produced from the steel industry alone, and if the slag produced from the iron industry is also converted to GGBS, than the total amount of GGBS produced will be almost 36 Mt/year (www.abbujacement.com). It has been reported that merely 20% of steel slag so produced is utilized and the rest is indiscriminately dumped in nearby landfills (*Pattanaik et al., 2019*).

2.3.2 Fly ash

Fly ash is the most widely used supplementary cementitious material and is presently being produced in millions of tones worldwide. The worldwide production rate of fly ash is represented in Fig. 2.3. Fly ash is the by-product of coal-fired electric and steam generating plants. Typically, coal is pulverized and blown with air into the boiler combustion chamber

where it immediately ignites, generating heat and producing a molten mineral residue. Boiler tubes extract heat from the boiler, cooling the flue gas and causing the molten mineral residue to harden and form ash (*American coal ash association, 2003*). Fig. 2.4 depicts the complete manufacturing process of wet and dry fly ash.

The fly ash so obtained consists primarily of oxides of calcium, silicon, iron and aluminum with lesser content of potassium, sulfur, magnesium, titanium and sodium (*Sadique et al., 2012*). Fly ash particles are generally heterogeneous and spherical in shape with size ranging between 0.5 μm to 100 μm (*Dwivedi and Jain, 2014*). Fly ash is classified into two groups viz. Class C and Class F fly ash based upon the percentage content of calcium, silicon, iron and aluminum as per ASTM C 618 guidelines (*ASTM, 1995*). Class C fly ash is produced from the burning of younger lignite or sub bituminous coal and gain strength over time. On the other hand, Class F fly ash is produced by burning of harder, older anthracite and bituminous coal. This type of fly ash contains less than 10% of lime and generally requires a cementing agent like quicklime or hydrated lime to produce cementitious compounds (*Dwivedi and Jain, 2014*).

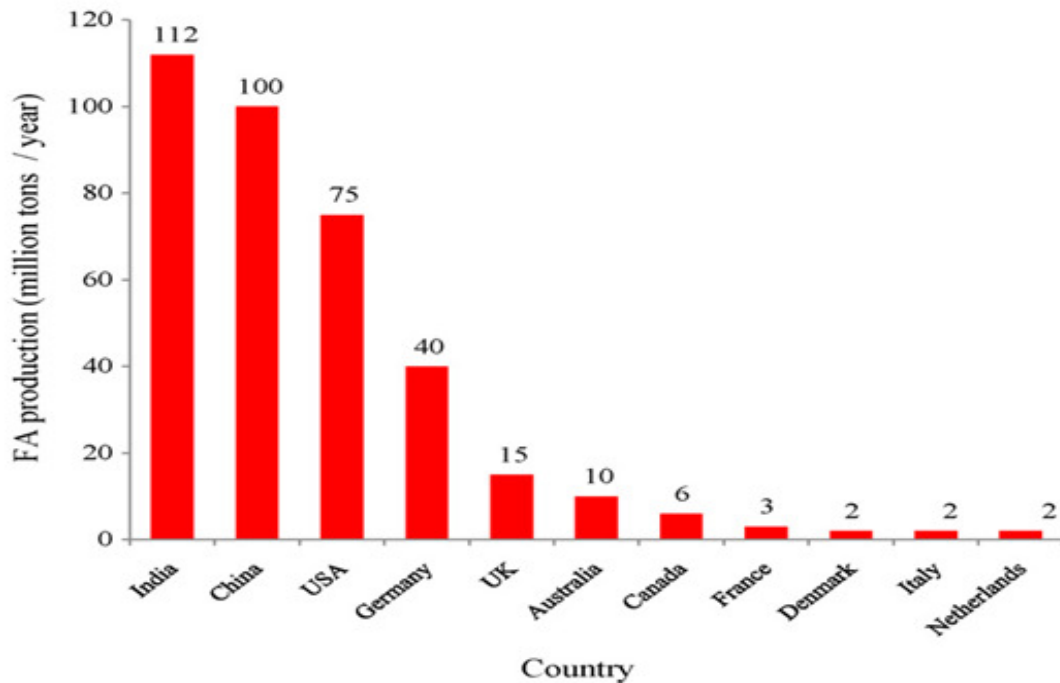


Fig. 2.3: Fly ash production rate worldwide (www.tifac.org.in)

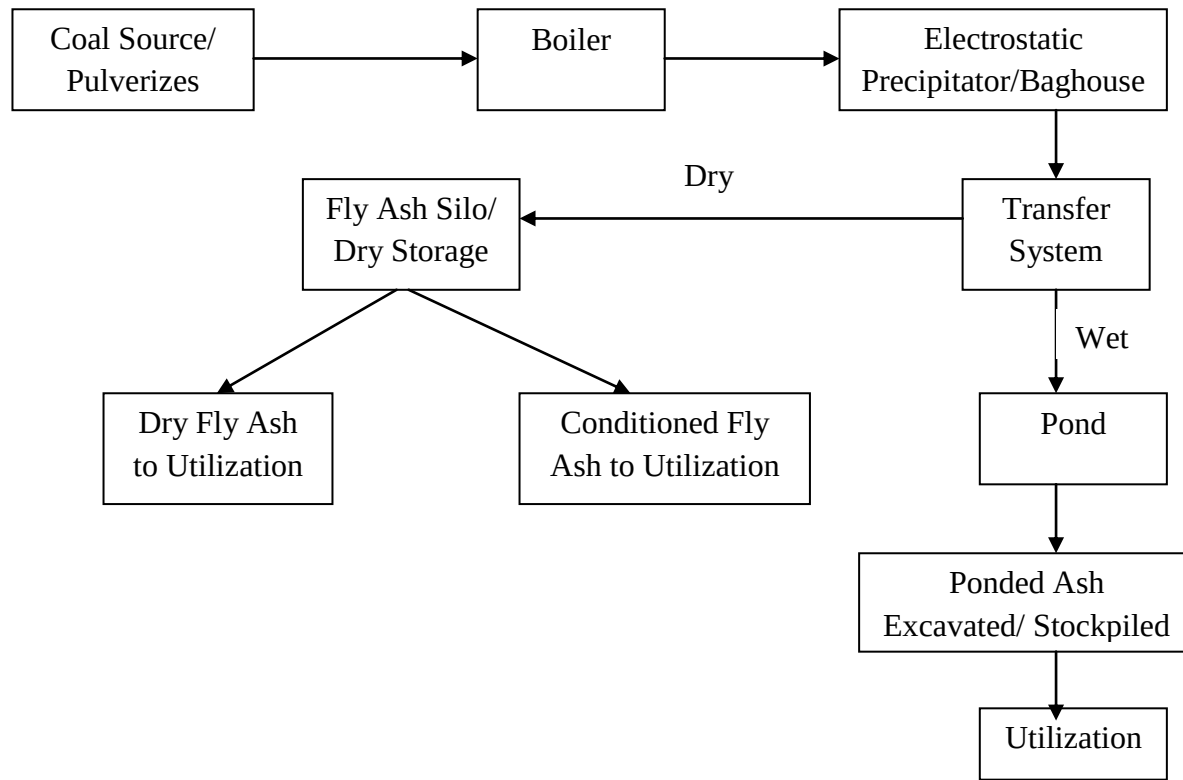


Fig. 2.4: Methods of fly ash production

2.4 EFFECT OF SUPPLEMENTARY CEMENTITIOUS MATERIALS ON WORKABILITY OF RCC

Workability is the most important property of RCC mix; which indicates the amount of energy required to achieve the maximum density of the aggregate in the mix (*Gagne R., 1999*). The workability of a conventional concrete is normally indexed by slump test; which has no significance for the RCC mixes because of their basic property of zero slump. Thus, the workability of RCC is generally measured by Vebe time, measured as per ASTM C 1170 (*ASTM, 2008*) guidelines.

According to the guidelines laid down in ACI 207.5R (*ACI, 1988*) for RCC mixes, Vebe time of 10 to 45 seconds is recommended to satisfy the degree of workability necessary for ease of compaction. *Schrader E.K. (2001)* found that dry consistency RCC mix has Vebe time ranged between 30-60 seconds, whereas wet consistency mix has Vebe time as 10-20 seconds. However, the literature suggests wider range of workability reported by researchers for achieving

RCC mixes with the various supplementary cementitious materials. *Vahedifard et al. (2010)* made a study on the workability of eight RCC mixes containing silica fume and pumice as a supplementary cementitious materials. They found that the addition of pumice significantly enhanced workability of the RCC mix; while use of silica fume had an opposite effect on the consistency properties because of their drier nature. The measured Vebe time ranged between 58-88 seconds; with the highest time registered by the mix having 10% silica fume and lowest time was corresponding to the mix having 30% pumice. *Yerramala and Babu (2011)* found that replacement of cement with high volume of fly ash helps to produce sufficient paste in the RCC mix. With 40-85% fly ash incorporation, the reported Vebe time was between 15 to 20 seconds, indicating that the RCC mixes containing fly ash were sufficiently workable as per ACI 207.5R (*ACI, 1988*). *Benouadah et al. (2017)* investigated the workability of RCC mix with the addition of polypropylene fiber at various dosages of 0, 0.5, 1, 15, 2 and 2.5 Kg/m³. It was reported that Vebe time was increased with the progressive increase in fiber content. The reduction in workability was because of the fibre network created by polypropylene fibers which interferes with water circulation and makes the RCC mix drier.

2.5 EFFECT OF SUPPLEMENTARY CEMENTITIOUS MATERIALS ON STRENGTH PROPERTIES

Around the world, huge amount of waste materials are generated as by-product from various industries. Utilization of these by-products as partial replacement of cement is an economical and environmental solution to preserve natural resources. Some of the researchers investigated the effect of various supplementary cementitious materials on resultant properties of conventional concrete and RCC. The following sections discuss the observation made by previous researchers on the effect of supplementary cementitious materials for use in both conventional concrete as well as in RCC.

2.5.1 Strength Properties of Conventional Concrete with Supplementary Cementitious Materials

Some of the researchers investigated the effect of GGBS as supplementary cementitious material for conventional concrete. Although GGBS addition led to decrease in early strength of concrete, but it improved workability, later age strength and durability of the conventional

concrete (Cheng et al. 2005; Babu and Kumar, 2000; Li and Zhao, 2003; Oner and Akyuz, 2007; Chidiac and Panesar, 2008; Liu et al., 2014). For instance, Cheng et al. (2005) reported 6.8% and 14.6% higher later age strength as compared to the control mix with 40% and 60% GGBS incorporation as a supplementary cementitious material, respectively. Oner and Akyuz (2007) studied the optimum dosage of GGBS as supplementary cementitious materials in terms of compressive strength of concrete. They observed that 55 -59% replacement level of total binder with GGBS gives the maximum strength. Prithika and Sekar (2016) reported that the replacement of cement with 25% and 50% GGBS decreased the 28 days compressive strength by 8% and 26% respectively on comparison with the control mix. Similarly, in case of flexural strength, the decrease was reported to be 9% and 18%, respectively.

The strength properties of conventional and high strength concrete with the incorporation of ternary blend consisting of GGBS and fly ash as supplementary cementitious materials were explored by various investigators. Li and Zhao (2003) studied the combined effect of fly ash and GGBS on compressive strength behaviour of high strength concrete. They observed that the combination of fly ash and GGBS improves both fresh and hardened properties of concrete, while larger replacement of cement with fly ash alone requires relatively longer time to get beneficial results. The improvement in strength shown by the ternary combination is attributed to the higher chemical modulus i.e. $(\text{CaO} + \text{MgO})/\text{SiO}_2$, which lead to acceleration in hydration rate. Berndt (2009) studied the strength properties of conventional concrete with GGBS and fly ash as supplementary cementitious materials. In the binary blend, cement was replaced with GGBS at a replacement level of 50% and 70%; while in the ternary blend, cement was replaced with fly ash and GGBS at 25% each replacement level. Test results represent that 50% GGBS incorporation significantly improve the 84 days compressive strength, while binary blend consisting of 70% GGBS and ternary blend gives the strength comparable with the control mix. Under the 28 days splitting tensile behaviour, strength higher than the control mix was achieved at all replacement levels with the highest value achieved with 50% GGBS incorporation level. It is hypothesized that incorporation of GGBS improved the interfacial bonding between the paste and aggregate thus resulting in higher strength.

Elahi et al. (2010) investigated the strength properties of high performance concrete by replacing cement with GGBS and silica fume in the binary and ternary blends consisting of 50% GGBS, 70% GGBS and 7.5% silica fume +50% GGBS respectively. It was observed that binary blends i.e. mix containing 50% GGBS shows the strength comparable with the control mix; while significantly lower strength was observed with 70% GGBS incorporation at all testing ages. On comparison with control mix, in case of ternary blend markedly lower strength was observed at early testing age, while the later age strength was marginally higher. It was concluded that the combination of silica fume and GGBS performed efficiently as compared to GGBS alone. The effect of steel slag, GGBS and composite mineral admixtures with four different GGBS to steel slag ratios on the long term (upto 1200 days) mechanical properties of concrete was studied by *Liu et al. (2014)*. It was observed that steel slag and composite mineral admixture having higher amount of steel slag have adverse effects on the compressive and splitting tensile strength of concrete. In the mix containing GGBS, markedly higher compressive strength and splitting tensile strength was registered as compared to the control mix after 7 days of testing. In composite mineral mixes, progressive increase in ratio of GGBS to steel slag leads to increase in strength with the enhancement in testing age.

2.5.2 Strength Properties of RCC with Supplementary Cementitious Materials

The properties of hardened RCC differ from the conventional concrete in terms of performance, required strength and void content present in RCC. Due to stiff no-slump consistency, RCC has unique properties other than the conventional concrete. The inclusion of various supplementary cementitious materials like fly ash, silica fume and coal waste ash in RCC has been investigated by various researchers (*Atis et al., 2004; Atis, 2005; Ute, 2008; Vahedifard et al., 2010; Aghabaglou and Ramyar, 2013; Dillon et al., 2015; Hesami et al., 2016; Rao et al., 2016a; Lam et al., 2018*).

The effect of fly ash as a supplementary cementitious material on the strength properties of RCC has been reported by various investigators. *Atis et al. (2004)* studied the efficiency of using fly ash in RCC in terms of improvement in strength properties of the resultant mix. Mix containing 15% fly ash as replacement to cement registered higher compressive strength than the control mix, while strength comparable to the control mix was observed at 30% replacement

levels of cement. *Atis (2005)* further investigated the strength characteristics of RCC mixes made with two different low lime Class F fly ashes, namely, Drax and Aberthaw, as cement replacements under moist and dry curing conditions. The investigator also made a study on the workable and zero slump concrete with the same mix proportions of cement, sand and gravel; while, the water content was kept small in zero slump concrete. It was observed that workability and type of fly ash has a great influence on properties of the mix. The experimental results show that zero slump concrete (RCC) presents 5-10% higher compressive strength as compared to workable concrete at all curing ages. Further, it was reported that good quality fly ash (Drax) at 50% replacement level gives 8.4% higher compressive strength as compared to the mix with normal Portland cement under dry curing conditions. On the other hand, low quality fly ash (Aberthaw) presents lower compressive strength at both 50% and 70% replacement levels of cement with fly ash. The inferior performance of Aberthaw fly ash as compared to Drax fly ash was attributed to the high loss of ignition (LOI) content of Aberthaw fly ash that resulted in higher water demand for the mix. Flexural tensile strength and splitting tensile strength also followed the behaviour similar with the compressive strength behaviour; with highest strength shown by 50% good quality fly ash in zero slump concrete.

Cao et al. (2000) investigated the influence of high volume fly-ash on compressive strength and flexural strength of RCC. They replaced cement with fly ash at different replacement levels of 45-95% by weight. It was reported that with the enhancement in curing age, larger amount of fly ash got activated and contributed to strength development. The test result also suggested that at early age, fly ash effect was more beneficial for compressive strength; while at later age, it was more useful to increase flexural strength. It was explained on the basis of refinement in inner structure of concrete with the curing age. The authors suggested that flexural strength is more sensitive to inner structure characteristics as compared to compressive strength. Thus, with the enhancement in curing age, larger benefit was achieved in flexural strength of concrete. The optimum level of fly ash addition was reported as 45%, beyond which strength loss was observed at both early and later ages.

Ute (2008) replaced cement with fly-ash at replacement levels of 20, 40 and 60% in RCC. It was reported that with the increase in cement replacement level from 20 to 60%,

decrease in compressive strength was observed at 7, 28 and 90 days of curing. *Aghabaglou and Ramyar (2013)* studied the mechanical properties of RCC made with fly ash, used as a replacement to both cement and fine aggregates. They observed that the increased fly ash content as cement replacement causes reduction in strength, while the use of fly ash as fine aggregates replacement gives the higher strength over the control mix. Reduction in strength in the former case was attributed to the increase in water-binder ratio of the mix due to fly ash addition and in part to the lower contribution of fly ash than cement to strength development. While in later case, improvement in strength was related with the higher compactable mix obtained by replacing the fine aggregates with the fly ash. *Dillon et al. (2015)* recommended that in RCC mix the replacement of cement with substandard fly ash having high loss on ignition, met the 28 days compressive strength requirement (27.6 MPa) for RCC to be used as a surface course as specified in ACI 325.10R (ACI, 2001) till the replacement level of 50%.

Rao et al. (2016a) investigated the strength properties of RCC till 90 days of curing, with the incorporation of 0, 20, 40 and 60% fly ash content. Compressive strength and flexural strength results show that fly ash addition had a negative effect on the both of these strength properties. As compared to the control mix, a decrease in compressive strength was registered. The reduction in compressive strength was 17%, 34% and 49% at 20%, 40% and 60% replacement level of cement with fly ash respectively. The reduction in compressive strength with the addition of fly ash is due to the fact that fly ash has slow pozzolanic reaction and hence not gained strength till the testing age of 90 days. On the other hand, *Lam et al. (2018)* reported that replacement of cement with fly ash decreased the early age strength. While at 90 days of testing, strength higher than control RCC mix was observed with 20% fly ash addition because of the pozzolanic properties of fly ash. Further, higher replacement level of cement with fly ash (40%) caused reduction in strength because at higher replacement level fly ash is not as good as cement in term of strength contribution.

Apart from fly ash, some researchers investigated the effect of other mineral admixtures such as silica fume, pumice, coal waste ash on the strength properties of RCC. *Vahedifard et al. (2010)* investigated the compressive strength of RCC mix containing silica fume and pumice as a supplementary cementitious material. The results present that 10% silica fume enhanced the

compressive strength; while pumice had a negative effect on compressive strength behaviour. Further, the increased content of pumice led to decrease in compressive strength. *Hesami et al. (2016)* investigated the effect of coal waste powder, coal waste ash and lime stone powder at 5%, 10% and 20% replacement level of cement on the strength properties of RCC pavements. It was reported that mix containing 5% coal waste powder and ash had higher compressive, splitting tensile and flexural strength as compared to other mixes. The combination of 10% coal waste ash and 7% limestone powder used as ternary blend enhanced the mechanical properties with time as compared to the binary mix having coal waste ash alone. It was suggested the collective use of coal waste powder/ash and limestone powder as supplementary cementitious materials led to an efficient utilization of waste product.

Till date, limited number of researchers investigated the effect of GGBS on the strength properties of RCC (*Rao et al., 2016b; Karimpour A., 2010*). *Rao et al. (2016b)* investigated the strength properties of RCC by replacing cement with GGBS at 10, 20, 30, 40, 50 and 60% replacement levels. Mix proportions for all RCC mixes adopted were based upon flexural strength of 5 MPa according to *ACI 211.3R (ACI, 2002)* guidelines. They reported that the maximum strength of the RCC mix was achieved at 40% replacement level of cement with GGBS; however, the strength of GGBS mixes was reported to be higher than that of the control mix, even at 60% replacement level. Further, it was observed that the addition of GGBS into the RCC mix had a greater influence on the split tensile strength as compared to compressive and flexural strength. Large increase (72%) in split tensile strength was reported at 40% replacement level of cement with GGBS as compared to control mix; while in terms of compressive and flexural strength this increase was limited to 17% and 13% respectively.

The time span between the mixing the material until the time of compacting is one of the important parameter effecting the hardened properties of RCC. The effect of changing the time span between mixing and compaction on the mechanical properties of conventional concrete and RCC containing GGBS was investigated by *Karimpour A. (2010)*. A Total of 144 specimens were cast with replacement of cement with slag, at the replacement levels of 25%, 50% and 75%. Time span between mixing and compacting of RCC was varied from 30 minutes to 180 min. It was observed that delay in compaction leads to decrease in strength properties of RCC

containing OPC only. However, by using GGBS as a part of cementitious material, delay in compaction did not cause any loss of properties, because GGBS does not show considerable reactions in short duration. So by replacing the cement with GGBS led in cement content reduction and increase in water-cement ratio in the mix. Increase in water-cement ratio postpones the formation of crystallized net in compacting activities.

2.6 EFFECT OF SUPPLEMENTARY CEMENTITIOUS MATERIALS ON DURABILITY PROPERTIES

The various durability or in-service properties of RCC and conventional concrete with the incorporation of various supplementary materials were studied by various investigators and are presented in the succeeding sections.

2.6.1 Durability Properties of Conventional Concrete with Supplementary Cementitious Materials

The effect of alternative cementitious materials on various durability or in-service properties of conventional concrete, viz. abrasion resistance and drying shrinkage was studied by various researchers. The abrasion resistance of conventional concrete with the use of fly ash, silica fume as cementitious materials was studied by numerous researchers (*Naik et al., 1995, Siddique et al., 2013; Cai et al., 2016*). For instance, *Naik et al. (1995)* found that concrete containing fly ash presented abrasion resistance comparable to ordinary concrete up to 30% replacement level of cement with fly ash. *Siddique et al. (2012)* also reported abrasion resistance comparable with the control specimens at 30% fly ash incorporation level, but beyond this level decrease in abrasion resistance as compared to ordinary concrete was observed. *Cai et al. (2016)* further reported that proper combination of fly ash and silica fume improved abrasion erosion resistance of concrete. The influence of fly ash and silica fume as a cementitious material on abrasion resistance of conventional concrete was studied by *Rashad et al. (2014)*. The cement was replaced by 70% fly ash content in binary blend, while ternary blends consist of 60% fly ash +10% silica fume and 50% fly ash +20% silica fume. It was reported that the incorporation of all supplementary materials had an adverse effect on abrasion resistance behaviour. From the refereed literature; it was observed that higher incorporation level of fly ash adversely affects the abrasion resistance of concrete.

Drying Shrinkage of conventional concrete with the incorporation of steel slag, GGBS was investigated by various researchers. For instance, *Liu et al. (2014)* studied the effect of steel slag, GGBS and composite mineral admixture with four different GGBS to steel slag ratios on drying shrinkage behaviour of concrete up to 1200 days. The test results shows that mix containing GGBS and composite mineral admixture having higher GGBS content was able to decrease the long term drying shrinkage of concrete more significantly. *Yuan et al. (2015)* reported reduction in shrinkage strain of concrete made with the slag cement as compared to the OPC concrete. The reduction in shrinkage strain improved with the increase in replacement level of cement containing slag. *Jianyong and Yan (2001)* reported that in case of high performance concrete, replacement of cement with 30% ultrafine GGBS reduced the shrinkage strain as compared to the control concrete. The reduction in shrinkage strain with the incorporation of GGBS can be correlated to the increase in C-S-H gel hydrates and density of hardened cement paste, which make concrete more resistant to deformation. On the other hand, *Lee et al. (2006)* reported that the concrete incorporating GGBS exhibited greater autogenous shrinkage than the control concrete without GGBS. The greater autogenous shrinkage observed in GGBS mixes can be attributed to greater chemical shrinkage and finer pore structure of mix incorporating GGBS. Similarly, *Shariq et al. (2016)* found that shrinkage strain increases with the increase in replacement level of cement with GGBS. The increase in shrinkage strain may be due to greater volume of paste and finer capillary pores in the GGBS concrete. Thus, from the referred literature, the effect of GGBS on the shrinkage behaviour of conventional concrete was observed to be contradictory.

2.6.2 Durability properties of RCC with Supplementary Cementitious Materials

The studies on the effect of various supplementary materials on the durability properties of RCC are limited. *Rao et al. (2016a)* investigated the combined effect of fly ash and manufactured sand (M-sand) on abrasion resistance of RCC. In the study, cement was replacement with fly ash at replacement levels of 0%, 20%, 40% and 60%; while conventional fine aggregates were replacement with M-sand. It was observed that the combined use of M-sand and fly ash leads to better performance of RCC in terms of abrasion resistance of concrete. Further *Rao et al. (2016b)* studied the effect of GGBS as partially replacement of cement on

abrasion resistance of RCC in terms of Cantabro and surface abrasion resistance. The Cantabro abrasion resistance was measured on a Los Angeles abrasion testing machine operated at 100, 200, 300, 400 and 500 revolutions. It was reported that during the initial testing ages, Cantabro and surface abrasion loss increases with the progressive increase in GGBS content. While at later age of 90 days, decrease in abrasion loss i.e. improvement in abrasion resistance was observed at all replacement levels of cement with GGBS with the least value was shown at 50% GGBS level. The strongly correlated linear regression model was developed between the Cantabro and surface abrasion resistance indicating the effectiveness of Cantabro test as an alternative of surface abrasion test.

Significant reduction in shrinkage strain with the addition of polypropylene fiber as compared to control RCC mix was reported by *Benouadah et al. (2017)*. In the control mix stabilization in shrinkage strain was observed at 14 days, whereas, the RCC mix containing polypropylene fiber (0.5, 1, 1.5, 2 and 2.5 Kg/m³) got stabilization after 7 days of testing. The incorporation of fiber creates a fiber networks, which produce a seaming effect and allows the RCC to have a low shrinkage rate. *Gholami and Modarres (2016)* studied the short and long term shrinkage behaviour of RCC with the addition of 1 to 4% superplasticiser by weight of cement. The experimental results show that short term shrinkage strain of superplasticised mixes were significantly lower than the control mix. It was observed that as compared to control mix, only 30%, 21%, 14% and 12% of shrinkage strain was registered in mixes containing 1%, 2%, 3% and 4% of superplasticiser respectively at 24 hour of testing. On the other hand, long term shrinkage strain shows the behaviour opposite to the short term shrinkage behaviour. Mix containing 1% superplasticiser showed the maximum shrinkage strain, while control mix showed the minimum value of shrinkage strain. Moreover, 3% addition of superplasticiser lowered the shrinkage strain on comparison with 1% and 2% addition, while beyond 3% addition led to reverse effect on the shrinkage strain.

2.7 EFFECT OF AGGREGATE CHARACTERISTICS ON STRENGTH PROPERTIES

Apart from the addition of cementitious materials, type of fine or coarse aggregates, bond between mortar and aggregate and shape and size of aggregate also has great influence on strength behaviour of both RCC and conventional concrete. The effects of aggregate characteristics on various properties of concrete are discussed in the succeeding sections.

2.7.1 Strength Properties of Conventional Concrete

The effect of variation in coarse and fine aggregate type on strength properties of conventional and high strength concrete was investigated by numerous researchers. *Ozturan and Cecen (1997)* investigated the mechanical properties of normal and high strength concrete made with three types of coarse aggregates viz. crushed gravel, limestone and basalt. It was observed that in case of high strength concrete, maximum compressive strength was shown by basalt followed by limestone and rounded gravel. Gravel presents 10-20% lower 28 days compressive strength as compared to basalt and limestone aggregates. Lower strength of concrete incorporating gravel aggregates is attributed with the round and smooth surface of gravel particles which result in lower bonding with the matrix. On the other hand, in case of normal strength concrete limestone aggregates presented the maximum value of compressive strength followed by basalt and crushed gravel. The relatively higher strength of concrete containing limestone aggregates may be due to some interfacial chemical reaction which improved the bond strength. Behaviour similar with the compressive strength was also reported in split tensile and flexural strength in both types of concretes.

Wu et al. (2001) investigated the effect of coarse aggregate type viz. crushed quartzite, granite, limestone and marble on the mechanical properties of normal and high strength concrete along with the properties of the parent rock. It was reported that the aggregate type does not have any significant role on strength behavior of concrete in normal strength concrete. It was because in the normal strength concrete the failure occurs in the aggregate-cement interface and the aggregate full potential strength was not utilized. On the other hand, strength of high performance concrete was strongly depended upon the type of aggregate because on loading, the cracks extent through the aggregate thus, making the use of full potential strength of coarse aggregates. The highest strength was shown by crushed quartzite aggregate as compared to other aggregates in case of high strength concrete. The influence of variation in aggregate type namely limestone, sandstone, gabbro, quartzite and basalt on the compressive and flexural strength was evaluated by *Kilic et al. (2008)*. It was reported that highest strength was shown by gabbro, while the sand stone aggregates presents the lowest strength value. It was reported that the compressive strength of concrete are strongly depends upon the strength of the parent rock i.e. the

compressive strength of the gabbro aggregates is highest (247 N/mm²) while sandstone aggregates has the least strength as 52 N/mm².

Beixing et al. (2011) investigated the influence of manufactured sand characteristics such as micro fine content and type on the 28-day compressive and flexural strength of concrete. The fineness content (<75 µm) was fixed at various levels of 4.3, 7, 10, 15 and 20% and manufactured sand possessing a characteristics of limestone, quartzite, granite and basalt was investigated. It was reported that manufactured sand with limestone micro fine content upto 10% enhanced the strength of concrete and beyond this content level, reduction in strength was observed. On the basis of variation in aggregate type, maximum strength was reported in manufactured sand having a characteristic of basalt followed by limestone, quartzite, granite and natural sand. It was reported that sand particles having rough surfaces enhance the bond between aggregate particles and cement matrix thus increasing the strength.

The mechanical properties of steel slag aggregate concrete containing 45% to 65% slag aggregate to total aggregate proportions in comparison with crushed limestone aggregate concrete was investigated by *Maslehuddin et al. (2002)*. It was reported that steel slag aggregate presents 4.7% higher compressive strength than the crushed limestone aggregate, while split tensile, flexural strength was marginally lowered by steel slag aggregates. Because of heavy weight of steel slag aggregates, it was recommended that 50% of slag aggregate can be effectively utilized in concrete construction. *Chaliasou et al. (2018)* found that mix containing 25% geopolymer recycled aggregates presents the strength higher than the mix containing natural aggregates. *Abdullahi M. (2012)* reported crushed quartzite aggregates to be the strongest aggregates based upon the highest compressive strength shown by crushed quartzite. Investigator suggested the use of aggregates made from quartzite in concrete construction.

2.7.2 Strength Properties of RCC

Aggregates from recycling may be contaminated if the original concrete has been subjected to chlorides, sulphates, siliceous gel, oil or other harmful materials. These contaminants could affect the properties of recycled concrete (fresh and/or hardened) and accelerate its degradation process. Some researchers studied the influence of this type of

aggression on the mechanical performances of RCC containing recycled aggregates. *Debieb et al. (2009)* studied the effect of non-contaminated and contaminated recycled aggregates obtained by crushing the natural concrete slab cured by water, seawater, chloride solution and sulphate solution on RCC pavement. The mechanical performance of RCC was analysed by replacing the total natural aggregates with the recycled contaminated and non-contaminated aggregates. Four concrete mixtures namely, (1) NAC: conventional concrete containing natural aggregate (2) NRCC: RCC containing natural aggregate (3) RRCC -V: RCC containing non-contaminated recycled aggregate (4) RRCC-P: RCC containing contaminated recycled aggregate were made. The mechanical test results show that NRCC presents 15% and 83% higher compressive and split tensile strength respectively; as compared to NAC. Replacement of natural aggregates with recycled aggregates presents loss in strength; while contamination of aggregates does not have a significant effect on the strength properties. It was reported that recycled aggregates resulting from crushing of polluted concrete are richer in sulphates. Recycled aggregates enriched with chlorides are leached if they are soaked into water. Therefore, after washing or immersion into water during a minimum of 2 weeks, these aggregates can be reused in RCC and even in reinforced concrete without the risk of corrosion.

Courard et al. (2010) studies the effect of replacement of natural aggregates by concrete road recycled aggregates in their research. They performed different test on recycled aggregates in order to characterize their physical, chemical and mechanical properties. Sieve analysis shows that majority of the aggregates lies between 10-20 mm. It was concluded that recycled aggregates present very good performances for use in RCC when used with very small content of fine particles, specific gravity of 2634 kg/m³, absorption coefficient of 4.58% and Los angles abrasion value equal to 25. Solid compactness of RCC with both natural and recycled aggregates remains similar; while compressive strength was observed to be higher with the natural aggregates.

Some researcher studied the utilization of by-product generated from the various plants as fine and coarse aggregate replacement in RCC (*Ghafoori and Cai, 1998; Chi and Huang, 2014; Rao et al., 2016a*). *Ghafoori and Cai (1998)* fully replaced fine aggregates with pulverized coal combustion high-calcium dry bottom ash. Along with this, to study the effect of coarse aggregate

content; it was varied in three percentages i.e. 50%, 55% and 60% of total dry solids. It was reported that development in compressive strength in RCC containing dry bottom ash is similar with the conventional concrete i.e. approximately 75% of the 28 day strength was attained at 7 days of curing period. With the increase in coarse aggregate content, improvement in various strength properties was registered. However, the incorporation of dry bottom ash in RCC had a more pronounced effect on the splitting tensile strength than on the compressive strength. It was because of the fact that the tensile strength depends upon the paste quality rather than the aggregate type and pozzolanic properties of dry bottom ash contributes more to tension than to compression. *Chi and Huang (2014)* replaced fine aggregates with circulating fluidized bed combustion (CFBC) ash at a replacement level of 5%, 10% and 15% by weight. CFBC ash which is a by-product from petroleum coke combustion power station was used to investigate its effect on the strength properties of RCC. Experimental results show that with the addition of CFBC ash, water absorption was increased with the reduction in initial surface absorption. Based upon the compressive and splitting tensile strength results, CFBC ash at 5% replacement level of fine aggregate was recommended in RCC.

Lam et al. (2018) studied the effect of electric arc furnace (EAF) slag on strength properties of RCC with partially and fully replacements of coarse aggregates with EAF slag. Along with the coarse aggregate replacement, cement was also replaced by fly ash at 20% and 40% replacement levels. It was reported that replacement of coarse aggregates with EAF slag led to decrease in strength irrespective of replacement level of coarse aggregates. Based upon the test results it was recommended that EAF slag with 20% fly ash incorporation produced RCC with sufficient strength to be used for pavement application. While EAF slag with 40% fly ash addition can be used in subbase course only. *Rao et al. (2016a)* investigated the combined effect of manufactured sand (M-sand) and fly ash on strength behaviour of RCC. The M-sand was used as full and partial replacement level of river sand in the investigation. The test results indicate that partial replacement of river sand with M-sand led to 16% increases in compressive strength and 4% decrease in flexural strength as compared to fully replacement of river sand. The fly ash addition decreased the compressive as well as flexural strength at both replacement levels of river sand. Further, *Rao et al. (2016c)* replaced the river sand partially and fully with the M-sand in RCC containing GGBS as a supplementary cementitious material. The strength results show

that fully replacement of river sand with M-sand decreased the compressive strength by 9% while flexural strength was increased by 19%. However, the highest strength value was shown by partially replacement of river sand with M-sand along with 40% replacement level of cement with GGBS.

Meddah et al. (2014) replaced the coarse aggregates of RCC pavement with shredded rubber tire at 5-30 % replacement levels. The effect of shredded rubber tire was studied on the various properties of RCC like; compressive strength, split tensile strength, bending strength, compaction energy and roughness of rubber surface. The experimental results show that incorporation of rubber tire improved the consistency of RCC. The ductility and cracking resistance was also enhanced with the rubber content, but mechanical properties decreased with the increase in rubber content. It was recommended that compressive and tensile strength of RCC pavement can be improved by 10-20%, if the roughness of rubber aggregate was modified by gluing sand. Despite from the reduction in strength properties, the use of rubber tire in RCC pavements was recommended because of high ductility, low porosity and unit weight. The rubberized RCC pavements were recommended for use in some projects such as: rural roads, low traffic pavements and pedestrian roads. *Jingfu et al. (2009)* reported that the incorporation of rubber particles as fine aggregates in RCC is beneficial in term of tensile strength and ultimate tensile elongation, when the compressive strength was kept constant at the level of 40 MPa.

2.8 EFFECT OF AGGREGATE CHARACTERISTICS ON DURABILITY PROPERTIES

The literature review emphasis on various durability or in-service properties of conventional concrete as well as of RCC with the variation in aggregate type is presented in the subsequent sections.

2.8.1 Durability Properties of Conventional Concrete

It was reported that the type of fine and coarse aggregates had a great influence on the abrasion resistance of conventional concrete (*Beixing et al., 2011; Kumar and Sharma 2014; Kilic et al., 2008*). *Kilic et al. (2008)* investigated the effect of different types of aggregates (limestone, sandstone, gabbro, quartzite and basalt) on abrasion resistance of high strength

concrete. They reported that gabbro has highest abrasion resistance, while sandstone shows the lowest values. The observed highest abrasion resistance of gabbro aggregates was related to the highest compressive strength of gabbro aggregate rock. *Kumar and Sharma (2014)* investigated the abrasion resistance of concrete made with three different types of aggregates, having variable Los Angeles abrasion value. They observed that abrasion resistance of the concrete degrades when the Los Angeles value of the aggregates exceed 30%. *Beixing et al. (2011)* studied the influence of manufactured sand as fine aggregates on abrasion resistance of conventional concrete pavement. They observed that the abrasion resistance of concrete containing manufactured sand improved because of decrease in crushing and Los Angeles abrasion value of sand particles. *Rashad A.M. (2013)* investigated the abrasion resistance of concrete by replacing fine aggregates with the metakaolin at replacement levels of 10%, 20%, 30%, 40% and 50%. The test results show that wear depth was reduced with increase in metakaolin content with the least value being observed at 40% replacement level. At 50% replacement level, wear depth higher than the control mix was observed at all testing durations.

Extensive research have been conducted to study the shrinkage of conventional concrete by using different types of coarse and fine aggregates. *Al-Attar T.S. (2008)* reported that limestone aggregates present lesser shrinkage strain than the uncrushed gravels. The drying shrinkage of concrete containing different percentages of fine limestone as a substitute of mountain sand was studied by *Carlos et al. (2010)*. It was reported that specimens with 30% fine limestone shows the shrinkage identical with specimens containing mountain sand. On the other hand, considerable reduction in shrinkage was observed in specimens containing more than 50% of fine limestone content. The effect of coarse aggregate type (limestone, granite and quartzite) in conjunction with the slag cement on drying shrinkage was investigated by *Yuan et al. (2015)*. It was reported that porous limestone aggregates in combination with slag cement greatly reduced the shrinkage strain as compared to mixture containing low absorption type coarse aggregate. *Maslehuddin et al. (2003)* investigated the shrinkage and expansion behaviour of steel slag mortar and sand mortar subjected to moist curing for a period of 7 days and 4 months respectively. After 4 months of air curing, the shrinkage observed in steel slag mortar was 0.097% while in sand mortar was 0.11% indicating the comparable shrinkage in both types of mortar. The continuous moist curing of steel slag mortar led to increase in length of the specimen

by 0.034 % while change in sand mortar this was 0 %. The expansion occur in steel slag mortar can be attributed with the presence of calcium carbonate that expands on absorbing water.

Zhang et al. (2013) found that drying shrinkage of concrete is influenced by the material characteristics of the aggregates, which includes the specific surface area, water absorption ratio, aggregate shrinkage strain and the pore structure of aggregates. It was reported that fine aggregates have insignificant influence on the drying shrinkage while, coarse aggregate play a significant role. It is because of the relative proportions of both the aggregates in concrete matrix; the volume of fine aggregates is smaller in total aggregate volume as compared to coarse aggregates. *Fujiwara (2008)* observed that the shrinkage strain of concrete reduced by using light weight aggregates. For comparison purposes, the length change of the normal aggregates was also investigated and found that it was several time higher than the light weight aggregates. Thus, concrete containing light weight aggregates presents lower drying shrinkage because of lower length change value of aggregate itself. In order to reduce the drying shrinkage of conventional concrete, *Kwan et al. (2010)* recommended the aggregate treatment by immersing it into polymer latex or suitable water repellent.

2.8.2 Durability Properties of RCC

The effect of aggregate type on durability properties of RCC is very important because of the fact that the major application of RCC is in pavements. Despite this, the influence of aggregate characteristics on durability properties of RCC is not extensively investigated. *Lam et al. (2018)* reported that full replacement of natural coarse aggregates with electric arc furnace (EAF) slag produced better abrasion resistance of RCC because of rough texture of slag particles. Further, 20% fly ash incorporation as cementitious material improved the abrasion resistance behaviour of RCC incorporating EAF slag as coarse aggregates.

Since RCC mixes are widely used in construction of pavements, so drying shrinkage becomes an essential parameter for its acceptability. The effect of fine aggregate replacement with various supplementary materials on drying shrinkage behaviour of RCC was investigated by *Ghafoori and Cai (1998)*; *Pittman and Ragan (1998)*; *Adamu et al. (2018)*; *Benouadah et al. (2017)*. For instance, *Pittman and Ragan (1998)* reported that the grading of aggregates and the

moisture content of the mix has great influence on drying shrinkage of RCC. *Ghafoori and Cai (1998)* investigated drying shrinkage of RCC containing dry bottom ash as fully fine aggregate replacement. The cement content of the mixes was kept at three levels of 9, 12, and 15%. It was reported that stabilization in drying shrinkage was observed after five months of air exposure. The important observation made in RCC containing dry bottom ash was that ultimate drying shrinkage strain reduced to approximately half the value obtained in the corresponding conventional concrete. On the basis of cement content levels, it was reported that shrinkage strain increased with the increase in cement content because of availability of more paste to undergo volumetric contraction.

Jingfu et al. (2009) found that shrinkage comparable with the control mix was observed in RCC mix containing 50 kg/m³ rubber content as replacement of fine aggregate. Whereas, in RCC mixes containing 100 kg/m³ and 120 kg/m³ rubber content shrinkage higher than the control mix was registered. The increase in shrinkage strain can be attributed to the decrease in aggregate content with the increased content of rubber; because aggregates play an important role in restraining shrinkage strain. *Adamu et al. (2018)* investigated the effect of crumb rubber as fine aggregate replacement, fly ash as cement replacement and nano silica as an additive on the creep and drying shrinkage behaviour of RCC pavement. The replacement level of fine aggregate and cement with crumb rubber and fly ash was 10% and 50% respectively. The experimental results demonstrated that incorporation of crumb rubber slightly increased the drying shrinkage strain, while; drastical increase in shrinkage strain was observed with fly ash addition in to the mix containing 10% crumb rubber as fine aggregates. On the other hand, 1% addition of nano silica in the mix decreased the shrinkage strain as compared to the corresponding mix without silica fume addition.

2.9 CLOSING REMARKS

In this chapter, a brief review on the various benefits of RCC construction is presented first. In order to promote the utilization of GGBS as a sustainable material, the detailed study regarding the manufacturing process and production rate of GGBS globally are discussed initially. Further, a detailed literature review focusing on the effect of various percentages of

GGBS addition as supplementary cementitious material on various properties of conventional concrete and RCC mixes is presented.

Apart from the addition of cementitious materials, type of fine or coarse aggregates, bond between mortar and aggregate and shape and size of aggregates also has great influence on strength and durability behaviour of conventional concrete. Thus, a detailed literature covering the effect of variation in aggregate type, utilization of industrial byproducts such as steel slag, manufactured sand etc. as fine aggregates on the properties of conventional concrete and RCC mixes are also presented.

Through the extensive review of literature carried out related to the proposed research, it is inferred that most of the previous studies investigating the effect of GGBS as supplementary cementitious material were focused on the properties of conventional concrete. The studies investigating the effect of GGBS on various strength properties and in-service performance of RCC are limited. Also, it is well established that the properties of RCC differ from the conventional concrete because of difference in mix proportioning and placement procedure. Therefore, in the present research work, various strength and in-service properties of RCC are explored with the incorporation of GGBS as supplementary cementitious material.

Further, the referred literature suggested that most of the researchers have studied the properties of RCC by replacing either cement or fine aggregates with the industrial by-products. However, the replacement of aggregates, which constitute about 70-80% of total concrete, has not been studied. The present investigation is undertaken to explore the possibility of using the variation in coarse aggregates viz. crushed gravel aggregates and limestone aggregates, along with GGBS incorporation in RCC. The strength and in-service performance of RCC made with limestone aggregates will help to promote the practical use of this aggregate in pavements. Based upon the extensive literature review, objectives purposed for the present research work have already been presented in Chapter 1.

CHAPTER 3

ROLLER COMPACTED CONCRETE: MATERIAL DESIGN

3.1 GENERAL

The basic ingredients of roller compacted concrete (RCC) are similar to the conventional concrete, with some changes in the proportioning of the ingredients (*Rao et al., 2016a*). RCC requires larger proportion of sand, lesser water and cement content as compared to the conventional concrete. Since a relatively smaller amount of water is used in RCC, the resultant mix is super dry and cannot be placed by the general conventional concrete compaction techniques.

The mix design procedure of RCC is also different from the conventional concrete design. ACI 211.3R (*ACI, 2002*) suggest two philosophies for the design of RCC mixes; one is based upon concrete approach while the other is based upon soil approach. The basic principle involved in concrete approach is consolidation, while soil approach is based on compaction. The resultant mixes achieved by both the approaches should have zero slump. The mix design procedure in concrete approach is based upon the water-cement relationship. The mix designed by this approach has wetter consistency and higher paste content to fill all the voids in the aggregate for making a fully compacted mix. On the other hand, in soil approach, RCC is considered as a cement-enriched processed soil with a mix design based on moisture-density relationship (*U.S. reclamation, 2017; Hansen and Reinhardt, 1991*). Under this approach, optimum moisture content is determined for an individual cementitious material and aggregate type.

The RCC designed by concrete philosophy has more fluid consistency or is more workable than the mixes designed by soil approach (*Aslan O., 2006; U.S. reclamation, 2017*). It is said that if other factors are kept constant, RCC mixes obtained by using soil approach have higher compressive strength and better abrasion resistance as compared to mixes designed by using concrete approach (*Hansen and Reinhardt, 1991*). The concept used in soil approach is comparable to the field application of RCC in pavements since the compaction applied by surcharge in laboratory can correspond to the effort applied by the rollers in the field (*U.S. reclamation, 2017*).

The concrete approach is recommended for relatively massive sections such as dams where nominal maximum size of aggregates (NMSA) is 37.5 mm or larger. For relatively thin sections such as pavements or slabs generally containing 19 mm or lesser NMSA, soil compaction approach is recommended as per ACI 211.3R (ACI, 2002). Beside these well established approaches, some researchers suggest the mix proportioning of RCC by using theoretical models that attempt to decrease the number of compaction voids by maximizing the compactness of the granular skeleton. Some of the well documented models include compressible packing model and optimal paste volume method (De Larrard, 2009; Gagne, 2000). Since the present study is on the use of RCC for pavement construction, RCC mix was achieved by using soil compaction approach, as recommended by ACI 211.3R. Since soil compaction approach is based upon the moisture-density relationship, which will get altered with the variation in aggregate type and addition of ground granulated blast furnace slag (GGBS) as supplementary cementitious material.

3.2 MATERIALS

The major materials used for the experimental programme are cement, aggregates and GGBS. The details of each material are described in the succeeding sections.

3.2.1 Cement

In this study, Ordinary Portland Cement (OPC) conforming to Indian standards IS: 8112-1989 (BIS, 2005) was used. OPC is obtained by burning of naturally occurring argillaceous containing alumina with calcareous materials to a partial fusion at high temperature of around 1450°C (Gambhir, 2004). The energy dispersive spectroscopy (EDS) along with the scanning electron microscopy (SEM) for OPC used in the present study is presented in Fig. 3.1. SEM images give the information regarding the topographic and compositional behaviour of the material. From EDS, it is clearly observed that the major constituents of OPC are calcium oxide (CaO), Silicon dioxide (SiO₂) with the other constituents present in minor proportions. The exact chemical composition and physical properties of OPC used in the study are presented in Table 3.1 and Table 3.2 respectively.

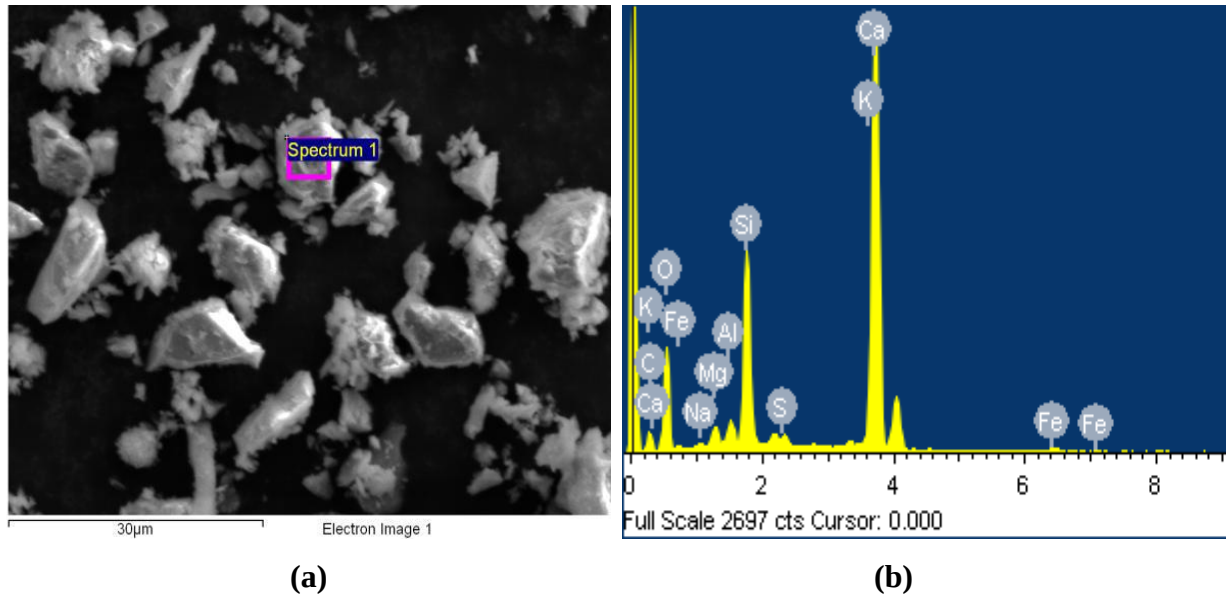


Fig. 3.1: (a) Scanning electron microscopy (SEM) image and (b) Energy dispersive spectroscopy (EDS) of OPC cement

Table 3.1: Chemical composition of cement

Composition (%)	OPC result	BIS Limit
Silicon dioxide, SiO_2	21.68	-
Aluminum oxide, Al_2O_3	4.87	-
Ferric oxide, Fe_2O_3	3.35	-
Calcium oxide, CaO	62.79	-
Magnesium Oxide, MgO	1.73	<6.00
Sulfur trioxide, SO_3	2.43	<2.50
Sodium Oxide, Na_2O	0.21	-
Potassium oxide, K_2O	0.69	-
Loss on ignition, LOI	1.94	<5.00
Aluminum/Iron ratio	1.45	>0.66
Lime saturation factor (lsf)	0.88	0.66-1.02

Table 3.2: Physical properties of cement

Properties	OPC result	BIS Limit
Fineness (m^2/kg)	305	>225
Specific gravity	3.12	-
Initial setting time (min)	130	>30
Final setting time (min)	170	<600
Compressive strength (N/mm^2)		
3 days	33.0	>23
7 days	41.3	>33
28 days	50.5	>43

3.2.2 Ground Granulated Blast Furnace Slag (GGBS)

GGBS is a by-product during the manufacturing of iron and steel. Molten iron and molten slag are produced in the blast furnace operated at 1500°C . Molten slag is lighter and floats on the top of molten iron. After removal of the molten iron from the furnace, the remaining molten slag is then water quenched rapidly below 800°C in order to prevent the crystallization of merwinite and melilite, resulting in the formation of a glassy granulate. This glassy granulate is then dried and grounded into fine powder, which is known as GGBS. GGBS so produced is a mixture of alumina, silica, magnesium oxide and lime; which are also the basic constituents of Portland cement.

In the present study, GGBS conforming to IS: 12089- 1987 (*BIS, 1987*) was used as supplementary cementitious material as a partial replacement of cement. The replacement level of cement with GGBS are 0%, 20%, 40% and 60% by weight basis. The SEM morphology and EDS of the GGBS used in the present study are presented in Fig. 3.2. From Fig. 3.2(a) it was observed that most of the slag particles have anomalous shape with sharp edges and clear angles. The surface textures of the GGBS are also smooth as compared to the OPC. In GGBS samples the intensity of peak observed in EDS (Fig. 3.2 b) are also different than OPC (Fig. 3.1 b). GGBS is rich with silica content with relatively low lime content as compared to OPC. The chemical composition and physical properties of the GGBS are reported in Table 3.3.

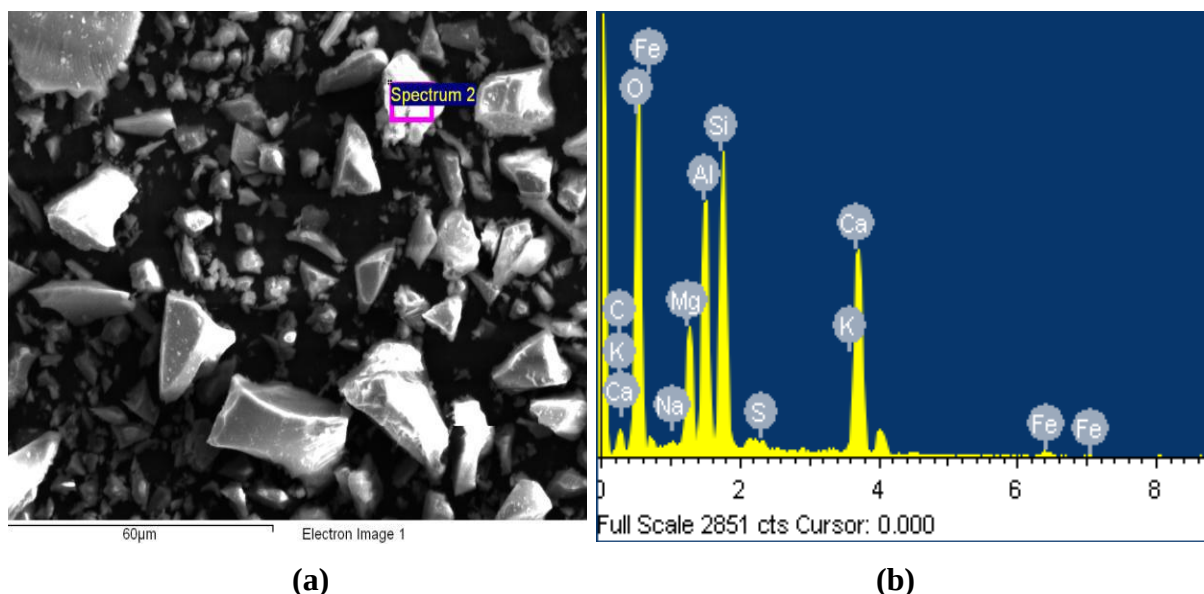


Fig. 3.2: (a) Scanning electron microscopy (SEM) image and (b) Energy dispersive spectroscopy (EDS) of GGBS

Table 3.3: Chemical compositions and physical properties of GGBS

Properties		GGBS result	BIS Limit
Chemical Compounds (%)	Silicon dioxide, SiO_2	31.61	
	Aluminum oxide, Al_2O_3	22.60	
	Ferric oxide, Fe_2O_3	2.50	
	Calcium oxide, CaO	33.21	
	Magnesium Oxide, MgO	8.00	<17
	Sulfur trioxide, SO_3	0.18	<2
	Sodium Oxide, Na_2O	0.85	
	$\frac{\text{CaO} + \text{MgO} + 0.33 \text{Al}_2\text{O}_3}{\text{SiO}_2 + 0.66 \text{Al}_2\text{O}_3}$	1.02	≥ 1
	Loss on ignition, LOI	0.98	
Physical properties	Fineness (m^2/kg)	385	
	Specific gravity	2.83	
	Slag activity index	1.73	

3.2.3 Aggregates

Selection of aggregates plays an important role in composition of RCC mixes. It constitutes approximately 75-85% of total volume of RCC and hence has a significant role in fresh and hardened concrete properties. The maximum size of coarse aggregates used in RCC varies depending upon its application in dams or in pavements, with the smaller size aggregates used in later application. According to ACI 325.10R-95 (ACI, 2001) the nominal maximum size of aggregates should be limited to $\frac{3}{4}$ inches (19 mm) in order to avoid segregation in RCC pavements. In the present study, two types of coarse aggregates were used; viz crushed gravel (CG) and limestone (LS) aggregates. The basic properties of both the aggregates are presented in the following sections.

(a) Coarse aggregates: Two types of coarse aggregates viz. crushed gravel and limestone aggregates were used in the present investigation. Both types of aggregates had a nominal maximum particle size of 20 mm and 10 mm. EDS spectra along with SEM images for crushed gravel aggregates and limestone aggregates are presented in Figs. 3.3 and 3.4 respectively. From the SEM images, it is found that both of aggregates show different microscopic morphology. Exfoliated characteristics are observed for the crushed gravel aggregate while; limestone aggregate shows dense irregular protrusions. Through EDS analysis, it was observed that crushed gravel aggregates have silica as its major composition, while limestone aggregates contain lime (Ca) as major element. The exact chemical compositions of both the aggregates are given in Table 3.4; while physical properties of both the aggregates are presented in Table 3.5. Sieve analysis results for 20 mm and 10 mm crushed gravel aggregates are presented in Table 3.6 and 3.7 respectively. For limestone aggregates, the corresponding sieve analysis is shown in Tables 3.8 and 3.9 respectively.

Table 3.4: Chemical composition of coarse aggregates

Material	Chemical Composition (%)							
	SiO ₂	Al ₂ O ₃	FeO	CaO	MgO	CaCO ₃	Na ₂ O	K ₂ O
Crushed Gravel (CG)	85.16	9.24	1.86	-	0.75	-	0.23	2.77
Limestone (LS)	-	-	-	11.72	16.38	71.90	-	-

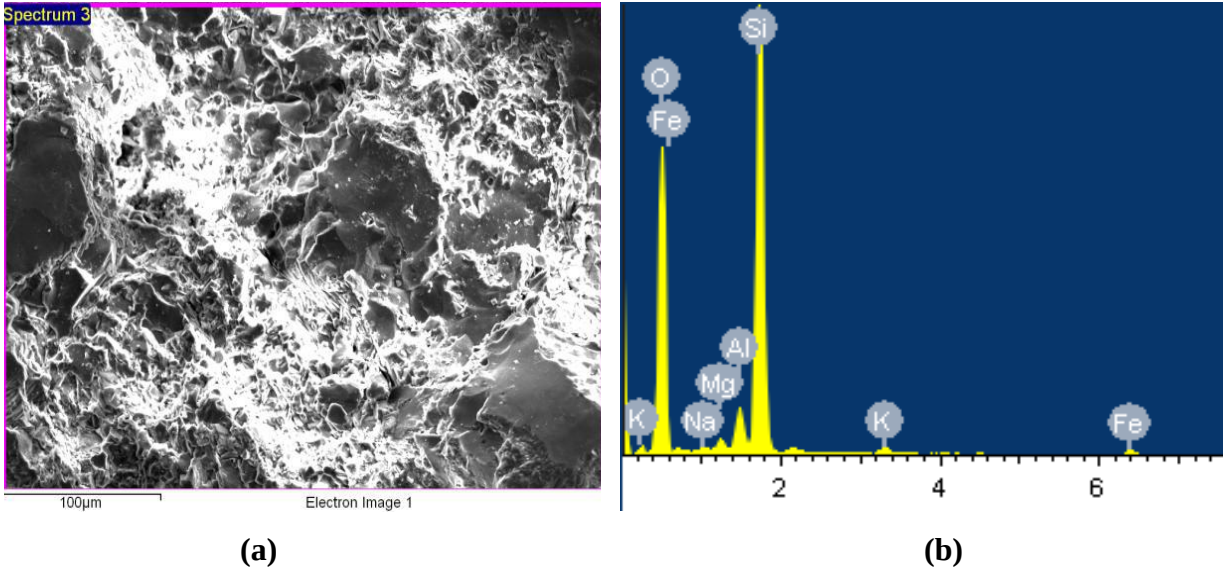


Fig. 3.3: (a) Scanning electron microscopy (SEM) image and (b) Energy dispersive spectroscopy (EDS) for crushed gravel aggregates

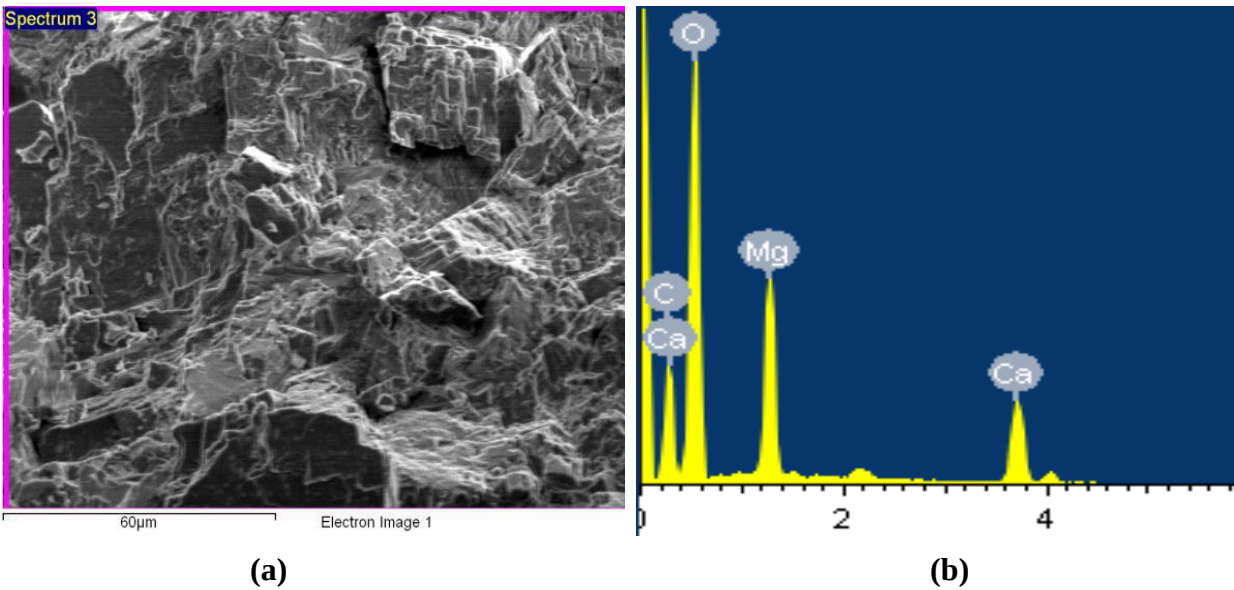


Fig. 3.4: (a) Scanning electron microscopy (SEM) image and (b) Energy dispersive spectroscopy (EDS) for limestone aggregates

Table 3.5: Physical properties of coarse aggregates

Physical Properties	Coarse Aggregates			
	Crushed Gravel (CG)		Limestone (LS)	
	20 mm	10 mm	20 mm	10 mm
Fineness modulus	6.80	6.04	6.69	5.97
Specific gravity	2.63	2.65	2.51	2.58
Water absorption (%)	1.3	1.4	1.6	1.7
Los angles abrasion value (%)	24.00*		28.20*	
Impact Value (%)	14.31*		18.28*	
Crushing value (%)	16.19*		24.16*	

*values were measured after combining 20mm and 10 mm aggregates in equal proportions

Table 3.6: Sieve analysis of 20 mm crushed gravel coarse aggregates

S. No.	IS-Sieve size (mm)	Weight retained (%)	Cumulative weight retained (%)	Weight passing (%)
1.	40	0.00	0.00	100.00
2.	20	3.73	3.73	96.27
3.	12.5	79.01	82.74	17.26
4.	10	10.64	93.38	6.62
5.	4.75	6.58	99.96	0.04
6.	Pan	0.04	100.00	0.00

Table 3.7: Sieve analysis of 10 mm crushed gravel coarse aggregates

S. No.	IS-Sieve size (mm)	Weight retained (%)	Cumulative weight retained (%)	Weight passing (%)
1.	20	0.0	0.0	100.0
2.	12.5	3.63	3.63	96.37
3.	10	8.41	12.04	87.96
4.	4.75	75.63	87.67	12.33
5.	2.36	12.31	99.98	0.02
6.	Pan	0.02	100	0.00

Table 3.8: Sieve analysis of 20 mm limestone coarse aggregates

S. No.	IS-Sieve size (mm)	Weight retained (%)	Cumulative weight retained (%)	Weight passing (%)
1.	40	0.00	0.00	100.00
2.	20	1.37	1.37	98.63
3.	12.5	71.70	73.07	26.93
4.	10	21.69	94.76	5.24
5.	4.75	5.20	99.96	0.04
6.	Pan	0.04	100	0.00

Table 3.9: Sieve analysis of 10 mm limestone coarse aggregates

S. No.	IS-Sieve size (mm)	Weight retained (%)	Cumulative weight retained (%)	Weight passing (%)
1.	20	0.0	0.0	100.0
2.	12.5	2.84	2.84	97.16
3.	10	9.76	12.60	87.40
4.	4.75	71.50	84.10	15.90
5.	2.36	15.81	99.91	0.09
6.	Pan	0.09	100	0.00

(b) Fine Aggregates: The fine aggregates used in the present study conformed to Zone-II as per IS: 383-1970 (BIS, 2002). The values of specific gravity, water absorption and fineness modulus were 2.70, 1.8% and 2.85 respectively. The sieve analysis results of fine aggregates used in the present study are presented in Table 3.10. From sieve analysis results it can be seen that the percentage passing of fine aggregates satisfied the Zone –II recommendations of IS 383-1970 (BIS, 2002). The SEM image for the fine aggregates is presented in Fig. 3.5.

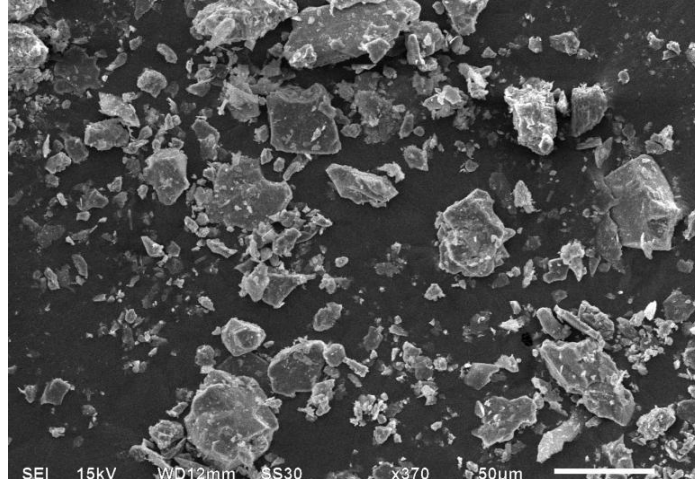


Fig. 3.5: Scanning electron microscopy (SEM) image for fine aggregates

Table 3.10: Sieve analysis results for fine aggregates

S. No.	IS-Sieve size (mm)	Weight retained (%)	Cumulative weight retained (%)	Weight passing (%)	IS limits for Zone II
1	10 mm	0.0	0.0	100.0	100
2	4.75 mm	2.9	2.9	97.1	90-100
3	2.36 mm	7.6	10.5	89.5	75-100
4	1.18 mm	27.0	37.5	62.7	55-90
5	600 micron	18.3	55.8	44.2	35-59
6	300 micron	27.7	83.5	16.5	8-30
7	150 micron	12.1	95.6	4.4	0-10
Total			285.8		
F.M. 2.85					

(c) Particle Size Distribution: Typical particle size distribution of all aggregates; viz. fine aggregates, crushed gravel coarse aggregates and limestone coarse aggregates is presented in Fig. 3.6. From the figure, it was clearly revealed that fine aggregates (sand) show a continuous gradation in size and falls under the category of well graded or continuously graded aggregate; while both the coarse aggregate (crushed gravel and limestone) show a size friction in a limited range and comes under the category of uniformly graded aggregates. The uniformly graded aggregates have a grain to grain contact in concrete matrix and hence produce a large number of

voids, leading to difficulty in compaction. Therefore, to ascertain the best packing of aggregates, the proportions of coarse and fine aggregates were selected in such a way so as to produce continuous gradation of size. The proportioning of aggregates used is addressed in the succeeding section.

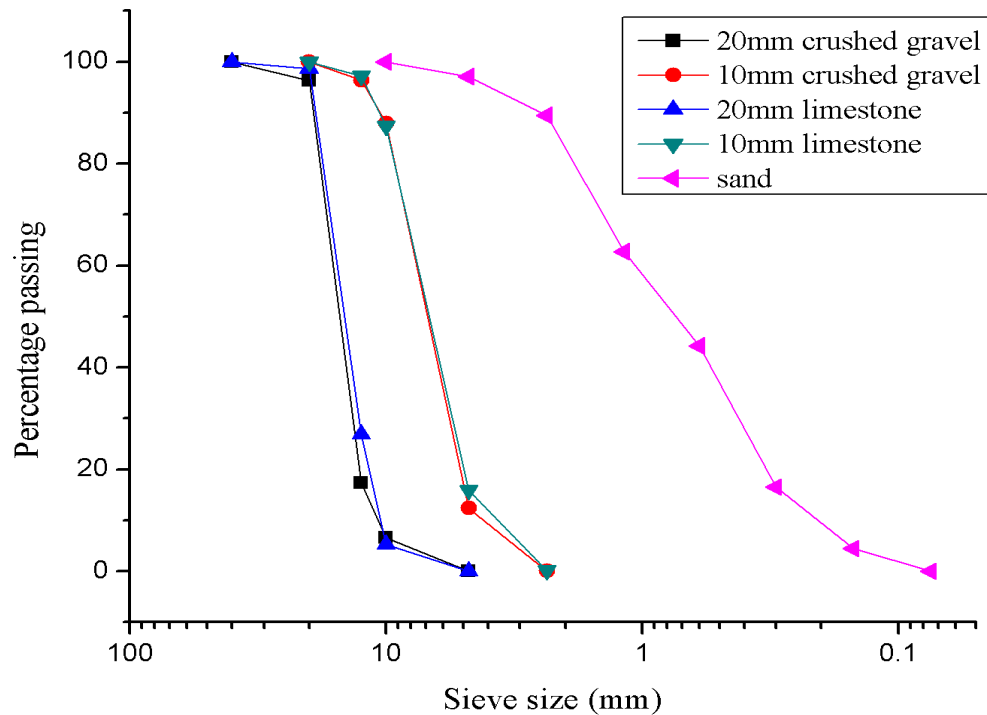


Fig. 3.6: Particle size distribution of fine and coarse aggregates

3.2.4 Water

In the present study, tap water was used for casting and curing of the specimens.

3.3 EXPERIMENTAL PROGRAMME AND RESULTS

In this study, soil compaction approach was used for achieving RCC mixes. According to this approach, optimum moisture content for specified solid content was obtained by laboratory compaction effort. The compaction applied by surcharge in laboratory corresponds to the effort applied by rollers in the field. The detailed procedure for finalizing composition of RCC mixes is discussed in the subsequent sections.

3.3.1 Selection of Combined Aggregates Gradation

The first step for achieving RCC mix by using soil compaction approach involves finalization of aggregate proportions. The aggregate proportions are decided in such a way that a dense graded combined aggregate curve as per ACI 211.3R (ACI, 2002) is attained. To achieve this target, various blends of fine aggregates, 10 mm coarse aggregates and 20 mm coarse aggregates were tried, which is summarized in Table 3.11. The cumulative percentage passing for all the tried blends is shown in Table 3.12, along with the ACI specified limits for dense graded combined aggregate curve. As can be seen from the table, cumulative percentage passing of all the trial blends fall under the ACI specified guidelines. The proportions of aggregates corresponding to the blend having highest regression coefficient was chosen in the present study for obtaining the reference RCC mix. From Table 3.11, it can be observed that blend number 4 (B.N4) gives the maximum regression coefficient of 0.982. This blend consisted of a combination of 55% of fine aggregates, 30% of 10 mm coarse aggregates and 15% of 20 mm coarse aggregates. The proportioning of 10 mm and 20 mm coarse aggregates were kept constant for both type of coarse aggregates i.e. crushed gravel and limestone.

Further, for all the trial blends, maximum density gradation curve was drawn as represented in Fig. 3.7. The maximum density gradation curve corresponds to a typical aggregate arrangement in which the smaller aggregates pack within the voids between the larger aggregates as shown in Fig. 3.8. It will result in low void content and high stability. The most acceptable maximum density gradation curve referred by the Federal Highway Administration (FHWA) is in the form of power gradation curve in which the sieve size was raised to n^{th} power (usually 0.45) as the x-axis units (www.kth.se). The combined 0.45 power gradation curves of all the tried mixes are shown in Fig. 3.7. From the figure, it was observed that the linear fit of all blends represent the maximum density gradation i.e. the aggregate particles fit together in their densest possible arrangement along this gradation line. It can be seen from the curves that B.N4 represents the maximum gradation density and hence was finalized for RCC aggregate mix proportions for both types of coarse aggregates. The combined aggregate grading curve corresponding to B.N4 for both types of aggregates along with the ACI specified lower and upper bounds are presented in Fig. 3.9.

Table 3.11: Summary of blends tried for combined aggregates gradation

Blend No.	Gradation proportion of aggregates (%)		
	20 mm	10 mm	sand
B.N1	17%	20%	63%
B.N2	20%	20%	60%
B.N3	17%	26%	57%
B.N4	15%	30%	55%

Table 3.12: Cumulative percentage passing of combined aggregates

Sieve size (mm)	Cumulative percentage passing				
	ACI 211.3R limits	B.N1	B.N2	B.N3	B.N4
20	82-100	99.3	99.2	99.3	99.4
12.5	72-93	82.9	80.1	82.5	84.1
10	66-85	79.8	76.9	78.7	79.9
4.75	51-69	65.1	62.2	60.4	59.3
2.36	38-56	52.1	49.6	47.1	45.4
1.18	28-46	36.3	34.5	32.8	31.6
0.6	18-36	25.5	24.3	23.1	22.3
0.3	11-27	8.8	8.4	8.0	7.7
0.15	6-18	7.3	6.9	6.6	6.3
0.075	2-8	4.9	4.6	4.4	4.3
Regression Coefficient (R^2)		0.961	0.969	0.978	0.982

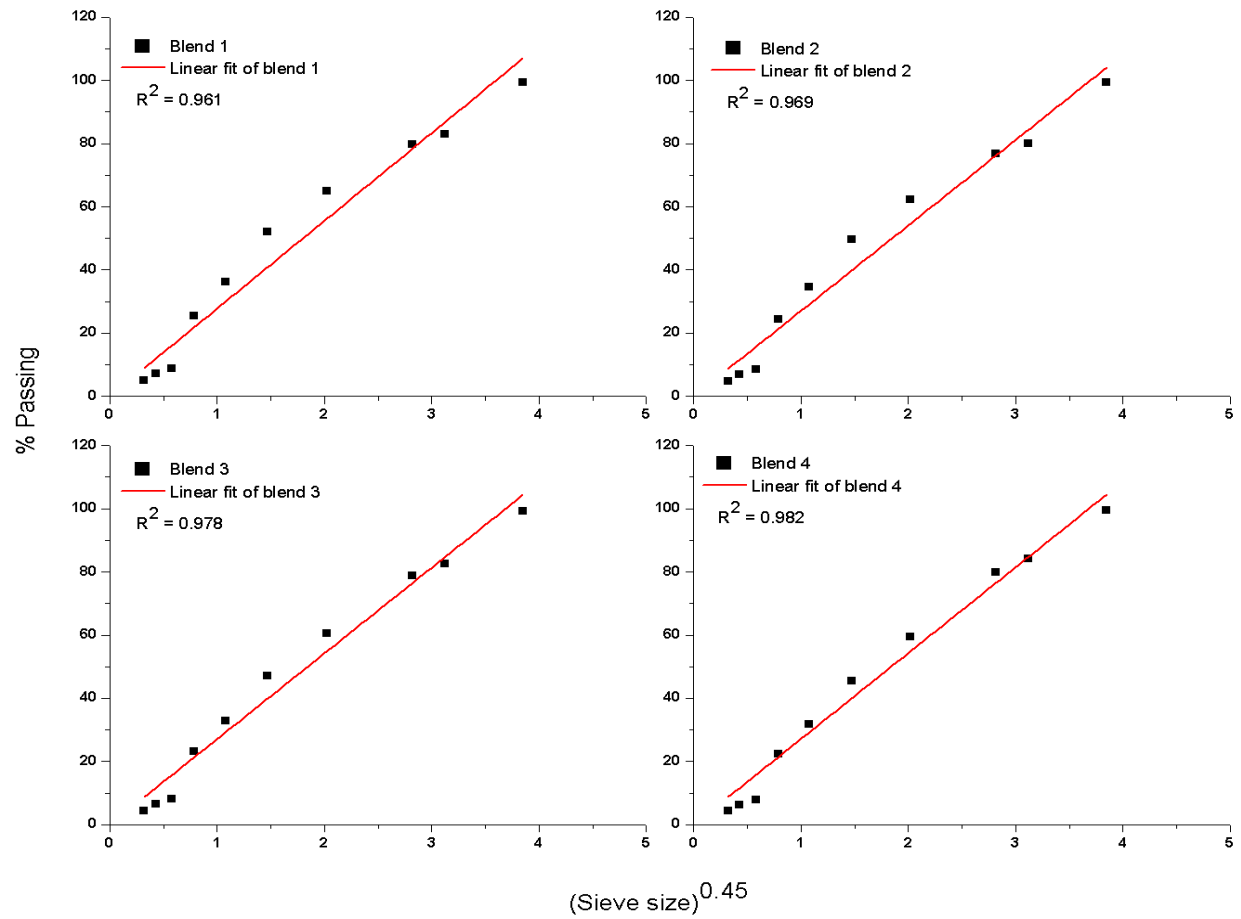


Fig. 3.7: Combined aggregates 0.45 power gradation curves for all trial blends

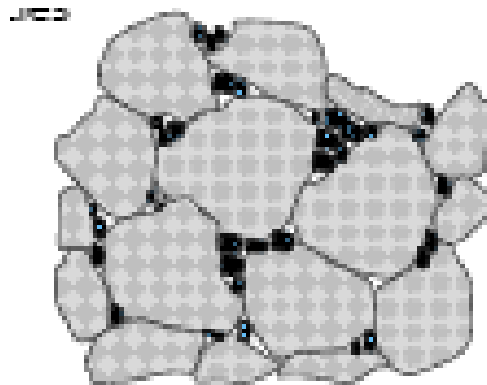


Fig. 3.8: Aggregate packing achieved in maximum density gradation (www.kth.se)

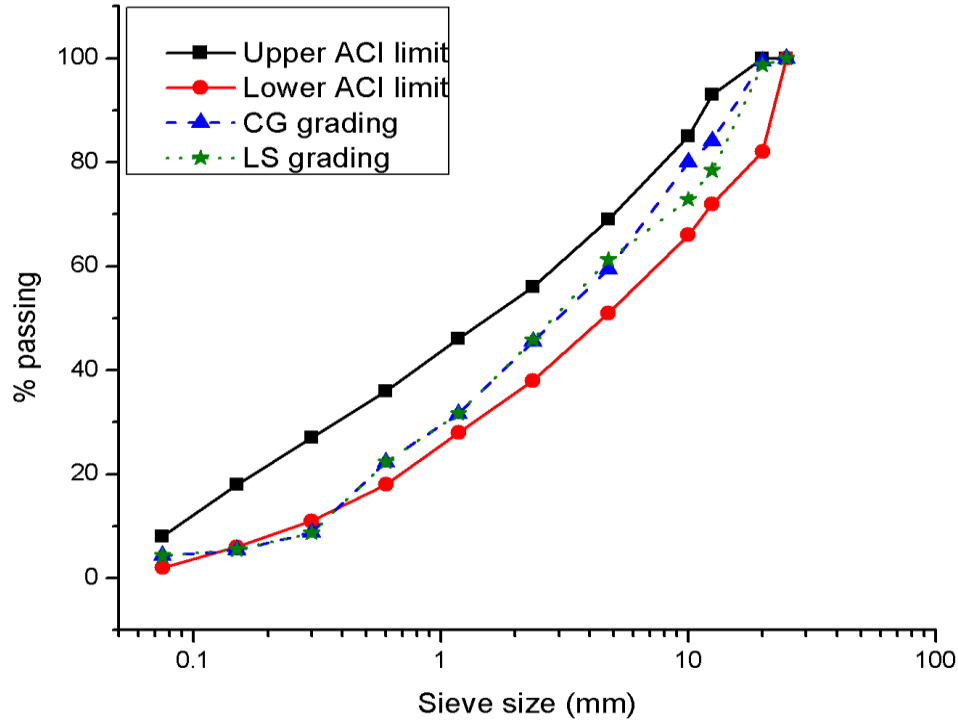


Fig. 3.9: Gradation curve corresponding to blend B.N4 with Upper and Lower ACI limits

3.3.2 Optimum Cement and Water Content

The optimum cement and water content should be selected very carefully, because they govern the compressive strength and void distribution in RCC mixes. Water content below the optimum value will produce insufficient paste and contribute to the presence of voids between aggregate particles; while higher water content results in higher water to cementitious material ratio (ACI, 2002).

In the soil approach, optimum water content is obtained for a specified solid content of material by laboratory compaction effort. The compaction applied by surcharge in laboratory can be correlated to the effort applied by rollers in the field (*U.S. reclamation, 2017*). The proportion of the aggregates in RCC mixture is first finalized in such a way so as to create dense-graded combined aggregate curve lying in the limits specified by ACI 211.3R (ACI, 2002). After finalizing the aggregate proportions, the cementitious material content and water content is determined by following combined guidelines laid down in ACI 211.3R (ACI, 2002) and ASTM D 1557 (ASTM, 2009). According to ACI 211.3R (ACI, 2002), the cementitious material content for RCC pavements range from 10 to 17% of the total dry mass of coarse and fine aggregates,

depending upon strength and durability requirements. For each mix, moisture content of the mix was determined by moisture-density test as described in method B of ASTM D 1557 (ASTM, 2009).

In the present study, aggregate proportions of B.N4 were finalized for obtaining RCC mix, cementitious material content was varied from 11% to 17% in various trial mixes and the optimum moisture content of each mix was determined from moisture-density curve. For moisture-density test, 7 numbers of trial mixes were prepared with cement content varying from 11% to 17% of total dry mass of aggregates, with an increment of 1% in each successive mix. Further, the water content of each mix was varied in five ranges, viz. 5%, 5.5%, 6%, 6.5%, 7% of total dry mass of RCC mixture. Thus, a total of 35 samples were prepared with seven levels of cement content and five water levels.

The prepared sample was filled in 100 mm diameter mould having volume of 920.89cm³ and was then compacted using an automatic compaction machine consisting of 4.54 kg circular rammer. The whole arrangement of automatic compaction machine is shown in Fig. 3.10. The compaction was done in five layers with 25 blows per layer. Thereafter, the wet mass of the compacted sample was noted which was further used to obtain wet density (γ_{wet}) of the mix. From the prepared compacted sample, a reference sample was obtained by collecting small fractions of the mix from three different locations i.e. top, centre and bottom. This reference sample was used to obtain optimum water content. For this, the wet mass of reference sample was noted. The sample was then kept in an oven for 24 hours to obtain dry mass of the sample. Optimum water content (w) and dry density (γ_{dry}) was then measured from the obtained data, according to equations (3.1) and (3.2) respectively (Aghabaglou and Ramyar, 2013).

$$w(\%) = \frac{m_{wet} - m_{dry}}{m_{dry}} \times 100 \quad \dots\dots(3.1)$$

$$\gamma_{dry} = \frac{\gamma_{wet}}{1 + w} \quad \dots\dots(3.2)$$

where, m_{wet} = wet mass of the reference sample

m_{dry} = oven dry mass of the reference sample

w = the water content (%)

γ_{dry} = dry density of the sample

γ_{wet} = wet density of sample which was calculated by dividing wet mass of the compacted concrete by the volume of concrete.



Fig. 3.10: Compaction arrangement for determination of moisture –density content

The detailed calculation work to obtain the optimum moisture content corresponding to crushed gravel aggregates with 16% OPC cement content is shown in Table 3.13 and corresponding moisture-density curve of the prepared mix is presented in Fig. 3.11. It can be seen from the figure that dry density of the mix increases with the increase in water content, thereafter the value decreases with further increase in water content. In this case, the optimum value of dry density and optimum water content was reported to be 2.39 g/cc and 5.83% respectively. Similarly, the values of optimum dry density and optimum water content were obtained for other mixes as well and the results are presented in Fig. 3.12. It can be seen from the figure that with the increase in cement content, optimum dry density of the mix increases, with the corresponding decrease in optimum water content. The increase in dry density value can be attributed to the increase in particle packing density with higher cement content. Previous study

by Choobbasti *et al.* (2015) shows a decrease in moisture content and marginally increases in dry density value with the increase in cement content. Shooshpasha and Shirvani (2015) also reported a similar behaviour under the cement-soil stabilized system.

Table 3.13: Calculation of optimum-moisture content and dry density value for 16% cement content

Description	Water content (%)				
	5	5.5	6	6.5	7
Compacted sample mass (gm)	2240.52	2283.10	2334.10	2319.79	2305.98
γ_{wet} (g/cc)	2.43	2.48	2.54	2.52	2.50
Crucible wt. (g)	46.5	43.0	43.5	39.0	42.5
Crucible + m_{wet} (g)	299.5	269.0	286.4	302.8	284.1
Crucible + m_{dry} (g)	287.5	257.5	273.0	287.5	269.5
w (%)	4.970	5.324	5.833	6.156	6.421
γ_{dry} (g/cc)	2.318	2.354	2.395	2.373	2.353
Optimum values	γ_{dry} optimum 2.395 g/cc and w optimum 5.833 %				

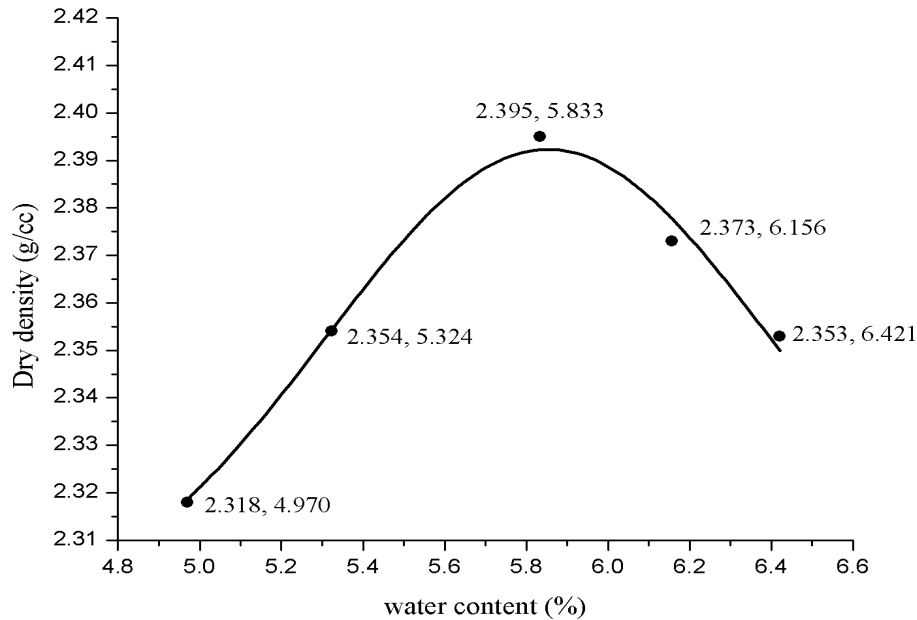


Fig. 3.11: Dry density variation with water content for crushed gravel aggregates with 16% cement content

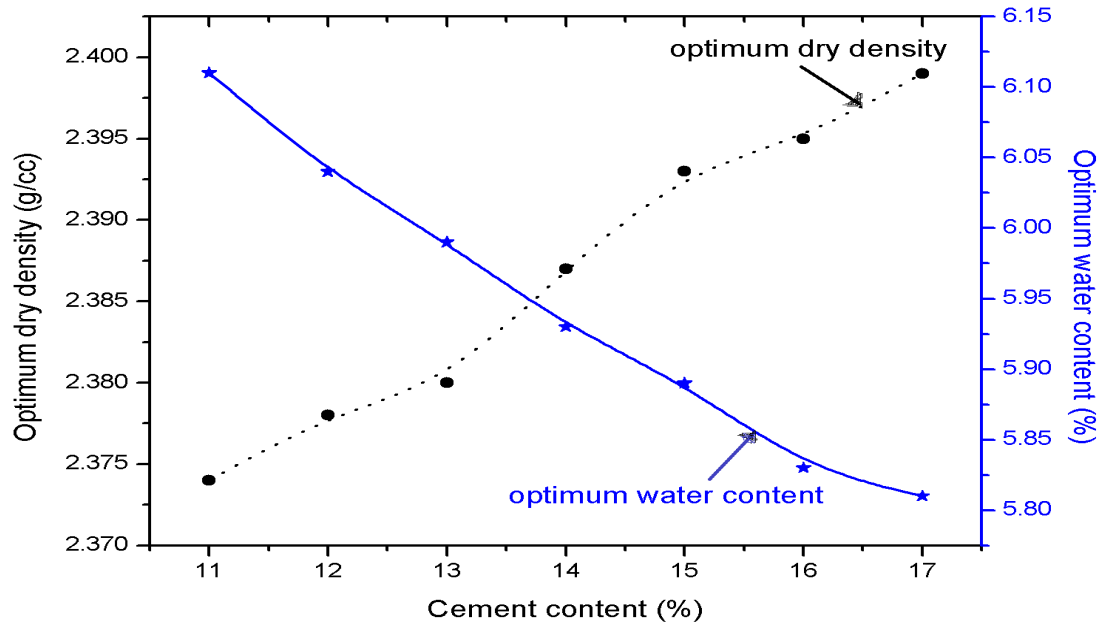


Fig. 3.12: Effect of cement content variation on optimum values of dry density and water content

3.3.3 Finalization of Cement Content Based on 28-day Compressive Strength

After getting optimum water content for all mixes having cement content in the range of 11 – 17%, one mix was to be finalized as a control mix having optimum cement content and maximum compressive strength. For this, cylindrical specimens of size 150 x 300 mm were prepared for seven concrete mixes with mix proportions listed in Table 3.14.

Table 3.14: RCC mix proportion with variation in cement content

S.No.	Cement Content (%)	Water Content (%)	Mix Proportions (kg/m ³)				
			Cement	Water	Fine Aggregates	Coarse Aggregates	
						10 mm	20 mm
1.	11	6.11	232.3	128.8	1161.2	633.4	316.7
2.	12	6.04	251.6	126.5	1153.3	629.1	314.5
3.	13	5.99	271.0	124.8	1146.6	625.4	312.7
4.	14	5.93	290.2	122.9	1140.2	621.9	310.9
5.	15	5.89	309.2	121.4	1133.6	618.3	309.2
6.	16	5.83	327.8	119.3	1127.0	614.7	307.3
7.	17	5.81	346.3	118.4	1120.6	611.2	305.6

For preparing the RCC mix, all dry ingredients were taken in required proportions and mixed thoroughly for about two minutes to get a uniform mix. After this, water was added to the dry mix and mixed for another three minutes. The RCC mix so obtained was filled into cylindrical moulds in three layers and compacted by putting surcharge on the top of each layer. According to ASTM C 1176 (ASTM, 2008), for cylindrical samples 4.9 kPa (9kg) surcharge was used to facilitate consolidation. Compaction was done by using a vibrating table having vibration frequency of 60Hz, as specified in ASTM C 1170 (ASTM, 2008). All the specimens were consolidated till the formation of ring between the surcharge and inside face of the mould. The formation of this ring shows that the prepared RCC mix has reached its maximum density. The specimens were removed from the moulds after 24h of casting and were kept in a curing tank for 28 days, maintained at a temperature of $25 \pm 2^\circ\text{C}$ and 100% relative humidity. After 28 days of water curing, the specimens were tested as per IS: 516- 1959 (BIS, 2004). The 28 days cylinder compressive strength results are presented in Fig. 3.13. The compressive strength value shown in Fig. 3.13 corresponds to the average value of three specimens, while the error bar represents the variations between the test results. From the figure, it was observed that the compressive strength of the mix increases with increase in cement content till 16% cement content level, thereafter the compressive strength value got stabilized.

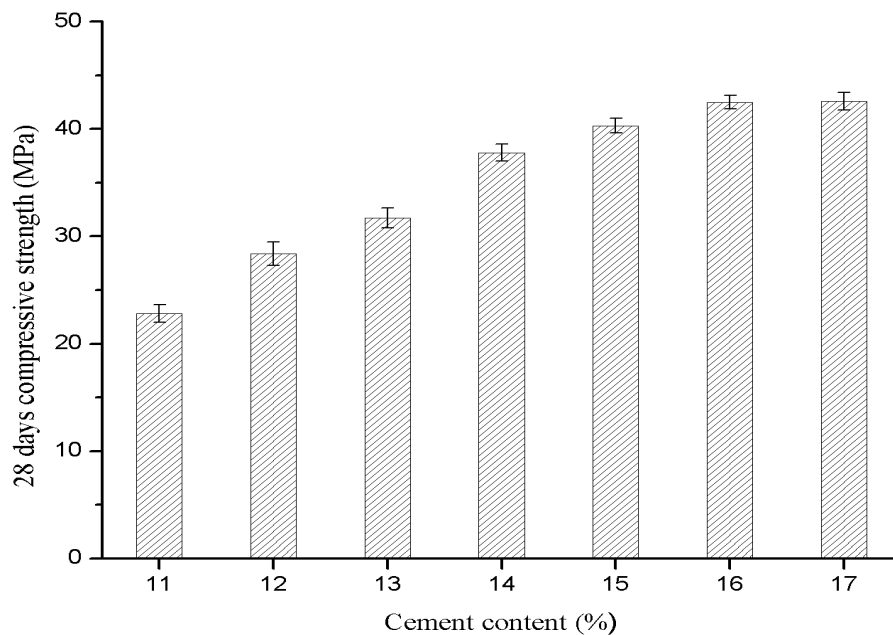


Fig. 3.13: Effect of variation in cement content on 28-day compressive strength

Based upon the 28 days compressive strength results, the mix containing 16% of cement content as total dry mass of coarse and fine aggregates was finalized as the control mix for crushed gravel aggregate. In order to maintain uniformity in all mixes, the same cement content (16%) was used for mixes having limestone aggregates.

3.3.4 Influence of GGBS Content and Aggregate Type on Optimum Values

Industrial wastes like fly ash, bottom ash, GGBS, metakaolin etc are released from the various industries in huge amount. These industrial wastes pose various difficulties in their disposal. To overcome these waste management issues and for sustainable development, the best solution is to utilize these industrial by-products in construction industry. For evaluating the influence of GGBS and aggregate type, subsequent mixes were prepared by using GGBS as partial replacement of cement and by taking two types of aggregates, viz. crushed gravel and limestone aggregates. For all mixes, the cementitious content was fixed at 16%, while the GGBS content was varied from 20% to 60%. The nomenclature adopted for all mixes is presented in Table 3.15.

Table 3.15: Nomenclature used for various RCC mixes

Cement type	Aggregate type	GGBS %	Nomenclature
OPC	Crushed gravel	0	CG-0
	Crushed gravel	20	CG-20
	Crushed gravel	40	CG-40
	Crushed gravel	60	CG-60
	Limestone	0	LS-0
	Limestone	20	LS-20
	Limestone	40	LS-40
	Limestone	60	LS-60

Further, optimum water content of each mix was finalized by the moisture-density curves. Since GGBS is finer than cement, thus, water content variation was taken as 6%, 6.5%, 7%, 7.5% and 8% of total dry mass of RCC mixture. Mixes containing GGBS were compacted

by using the arrangement similar to the control mix and optimum water content (w) and dry density (γ_{dry}) was measured as discussed earlier in section 3.3.2.

The moisture-density curves for mix containing crushed gravel and limestone aggregate are presented in Fig. 3.14 and Fig. 3.15 respectively. Peak point in each moisture density curve gives the corresponding maximum dry density and optimum water content. The optimum values are summarized in Table 3.16. It was observed from the table that optimum water content increases with the increase in GGBS content, irrespective of the type of aggregate used. This increase in water content may be associated with the higher fineness of GGBS as compared to cement.

Table 3.16: Optimum water content and maximum dry density of RCC mixes achieved by compaction test

Nomenclature	CG-0	CG-20	CG-40	CG-60	LS-0	LS-20	LS-40	LS-60
Optimum water content (%)	5.83	6.31	6.39	6.64	6.15	6.38	6.71	6.87
Maximum dry density (g/cc)	2.395	2.373	2.368	2.347	2.368	2.345	2.330	2.318

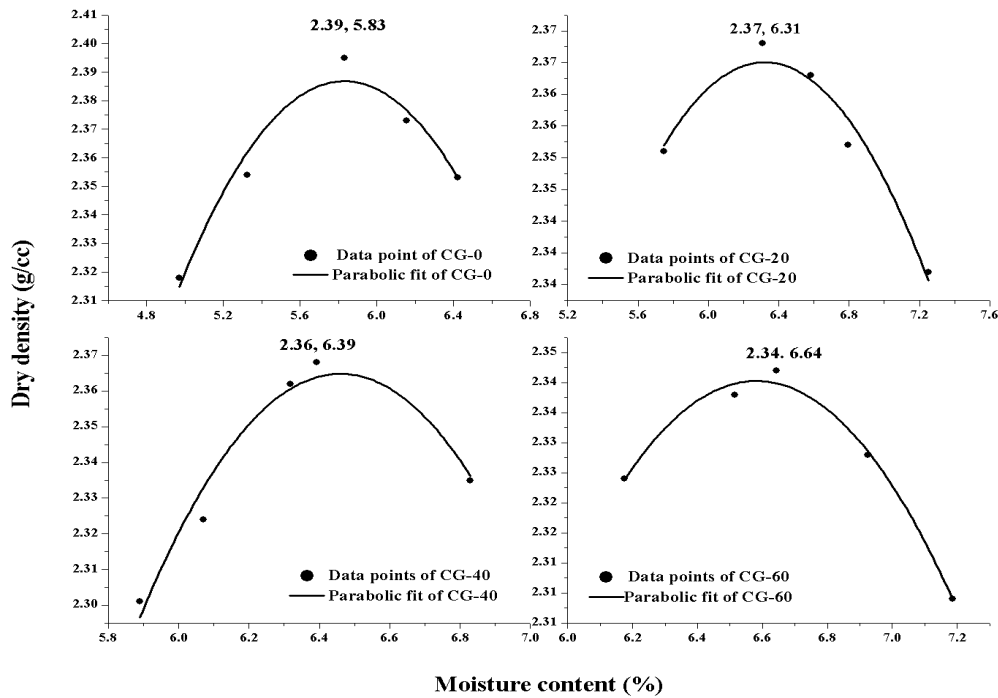


Fig. 3.14: Dry density variation with water content for mixes having crushed gravel aggregates and variable percentage of GGBS

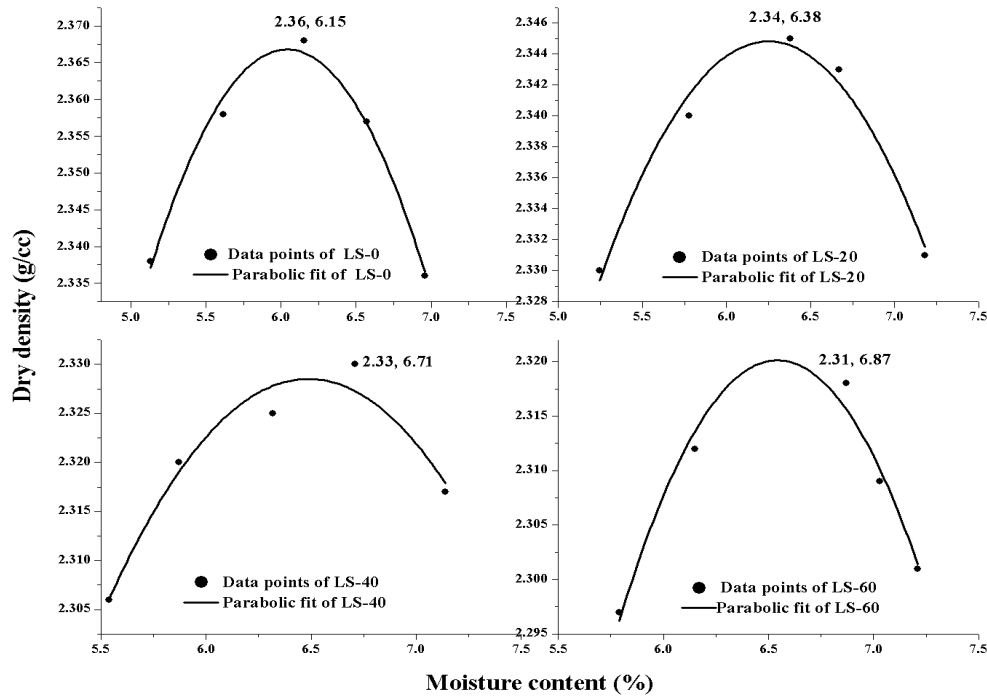


Fig. 3.15: Dry density variation with water content for mixes having limestone aggregates and variable percentage of GGBS

It can be seen from Figs. 3.14 and 3.15 that the type of aggregate influence optimum moisture content and hence, dry density of the resultant RCC mixes. RCC mix made with limestone aggregates is found to have higher optimum water content as compared to the corresponding mix made with crushed gravel aggregates. It can be due to the higher water absorption of limestone aggregates as compared to crushed gravel (Table 3.5). Higher water absorption of limestone aggregates increased the water demand of the RCC mix containing limestone. *Videla and Aguilar (2006)* also reported that limestone aggregates concrete has a higher water demand than the concrete made by using siliceous aggregates.

Further, for each aggregate type, increase of GGBS content led to increase in moisture content and decrease in dry density. This behaviour can be due to lower specific gravity of GGBS as compared to cement. During the early age, GGBS remains inert and does not contribute in the hydration reaction. Previous investigator, *Aghaeipour and Madhkhani (2017)* also reported an increase in moisture content with the progressive increase in slag content to achieve maximum compaction. From Fig. 3.16 it was observed that maximum dry density decreases with increase in optimum water content of the RCC mix, irrespective of the type of

aggregates. Similar observation regarding the maximum dry density behaviour with optimum water content was made by Yazici *et al.* (2015); Hesami *et al.* (2016); Aghabaglou and Ramyar (2013).

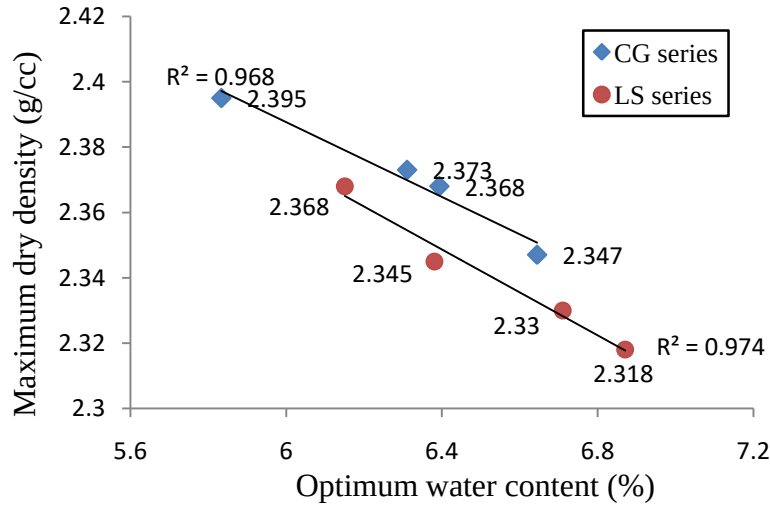


Fig.3.16: Maximum dry density variation with optimum water content for all mixes

3.3.5 Finalized RCC Mix Proportions

After getting the optimum water content of each mix, proportions of RCC were finalized for each mix based upon the ACI 211.3R-02 (ACI, 2002) guidelines. By assuming 1% air voids, and by using finalized proportions of aggregates, cementitious material and corresponding optimum water content, the mix proportions for each RCC mix was finalized. Briefly, aggregate proportions corresponding to B.N4, 16% cementitious material content and optimum water content presented in Table 3.16 was used for deciding the final proportions. The finalized RCC mix proportions are presented in Table 3.17.

3.3.6 Effect of GGBS Content and Aggregate Variation on 28-day Compressive Strength

The finalized mix proportion as shown in Table 3.17 is used to determine the influence of GGBS content and aggregates variation on 28 days compressive strength. Cylindrical specimens of size 150 x 300 mm as discussed earlier in section 3.3.3 were used to determine the 28 days compressive strength. The 28 days compressive strength was measured on three specimens and average of the three specimens was taken as the reference value. Compressive strength results obtained after 28 days of curing are presented in Fig.3.17. From the figure, it was observed that

in case of limestone aggregates, compressive strength had a marginal decrease with the addition of GGBS. While for crushed gravel aggregates, a reduction of 3.8% (CG-20), 4.8% (CG-40) and 13.9% (CG-60) was registered as compared to the corresponding control mix (CG-0). However, the observed strength of all mixes satisfied the limit prescribed by ACI 325.10R-95 (ACI, 2001), that requires a minimum value of 28-day compressive strength of 27.6 MPa for RCC to be used as surface course. It indicates that all the prepared RCC mixes can be used as surface course in pavement construction.

Table 3.17: RCC mix proportions of various mixes used in the present study

Nomenclature	Mix proportions (kg/m ³)					
	Cementitious materials		Water	Fine Aggregate	Coarse Aggregate	
	Cement	GGBS			10 mm	20 mm
CG-0	327.87	-	119.30	1127.06	614.76	307.38
CG-20	259.38	64.85	127.86	1114.51	607.92	303.96
CG-40	193.65	129.10	129.10	1109.46	605.16	302.58
CG-60	128.14	192.22	133.04	1101.23	600.67	300.34
LS -0	321.77	-	123.52	1105.68	603.10	301.77
LS-20	255.48	63.87	127.34	1097.76	598.78	299.61
LS-40	189.87	126.58	132.74	1087.81	593.35	296.90
LS-60	125.96	188.95	135.15	1082.52	590.46	295.45

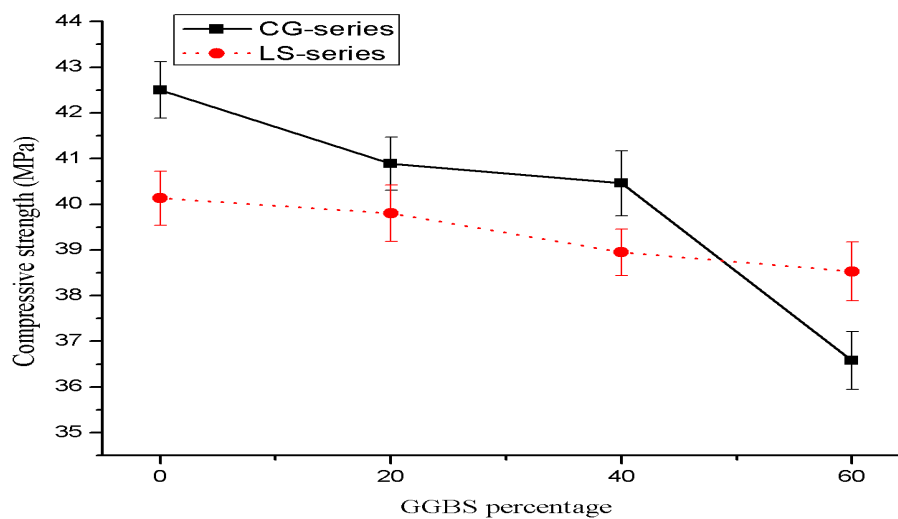


Fig.3.17: Influence of GGBS content and aggregate type on 28-days compressive strength

3.4 CLOSING REMARKS

The present chapter focuses on the finalization of mix proportion for zero slump concrete with the soil compaction approach. RCC achieved by soil approach has less fluid consistency as compared to the concrete approach because in soil approach, concept of optimum water content is used. The finalized aggregate proportions i.e. 55% of fine aggregate, 30% of 10 mm coarse aggregates and 15% of 20 mm coarse aggregates fitted well with R^2 as 0.982 in the maximum density gradation curve. Obtained higher correlation coefficient represents that the aggregate particles fit together in their densest possible arrangement along the gradation line. The cementitious material and water content were decided based upon the moisture-density test after finalization of aggregate mix proportion. From the moisture-density test, optimum water content and dry density value was obtained with the cement content variation from 11% to 17%. At obtained optimum value of water content with each cement content level the cylindrical compressive strength specimens were cast. Based upon the highest value of 28 days compressive strength, mix containing 16% of cement content as total dry mass of coarse and fine aggregates was finalized for both aggregate types.

Further, the influence of aggregate type and GGBS content on optimum value of water content and dry density was studied by replacing the cement with GGBS at replacement level of 20%, 40% and 60% by weight for both aggregates series. Moisture-density results show that limestone aggregates increased the optimum water content requirement with the corresponding decrease in maximum dry density value as compared to the crushed gravel aggregates. Addition of GGBS into the control mix also increased the optimum water content for both types of aggregate series. Optimum water content of all RCC mixes decreased with the increase of maximum dry density.

CHAPTER 4

STRENGTH PROPERTIES OF ROLLER COMPACTED CONCRETE

4.1 GENERAL

The focus of the present study is to determine the effect of Ground Granulated Blast Furnace slag (GGBS) and coarse aggregates variation on the properties of roller compacted concrete (RCC). Various researchers have evaluated the effect of GGBS as cementitious material on strength properties of conventional concrete mixes (*Swamy and Bouikni, 1990; Liu et al., 2014; Chidiac and Panesar, 2008; Oner and Akyuz, 2007*). It has also been observed that apart from the characteristics of cementitious materials, aggregate type has a great influence on the properties of concrete (*Ozturan and Çeçen, 1997; Tuncay et al., 2016; Maslehuddin et al., 2002; Adbullani, 2012; Kilic et al., 2008; Topcu and Uygunoglu, 2010; Beshr et al., 2003; Carlos et al., 2010*). In this chapter the effect of various percentages of GGBS and variation in coarse aggregate type on strength properties of RCC such as compressive strength, split tensile strength and flexural strength are described. The various materials used are briefly discussed, followed by the experimental details and test results of the strength properties.

4.2 TEST MATRIX

The test matrix adopted in the present study, to evaluate the effect of coarse aggregate variation and replacement level of cement with GGBS on strength properties of RCC, is presented in Fig. 4.1. The illustration provides description about the major test variables, viz. cement replacement levels with GGBS and coarse aggregate type. Briefly, GGBS was used as a cementitious material, at 20%, 40% and 60% replacement levels of cement. Further, two types of coarse aggregates, viz. crushed gravel (CG) and limestone (LS) were used in the study.

4.3 MATERIAL SYSTEM

The major materials used are cement, GGBS, aggregates and water. The details of each material are described in the following sections.

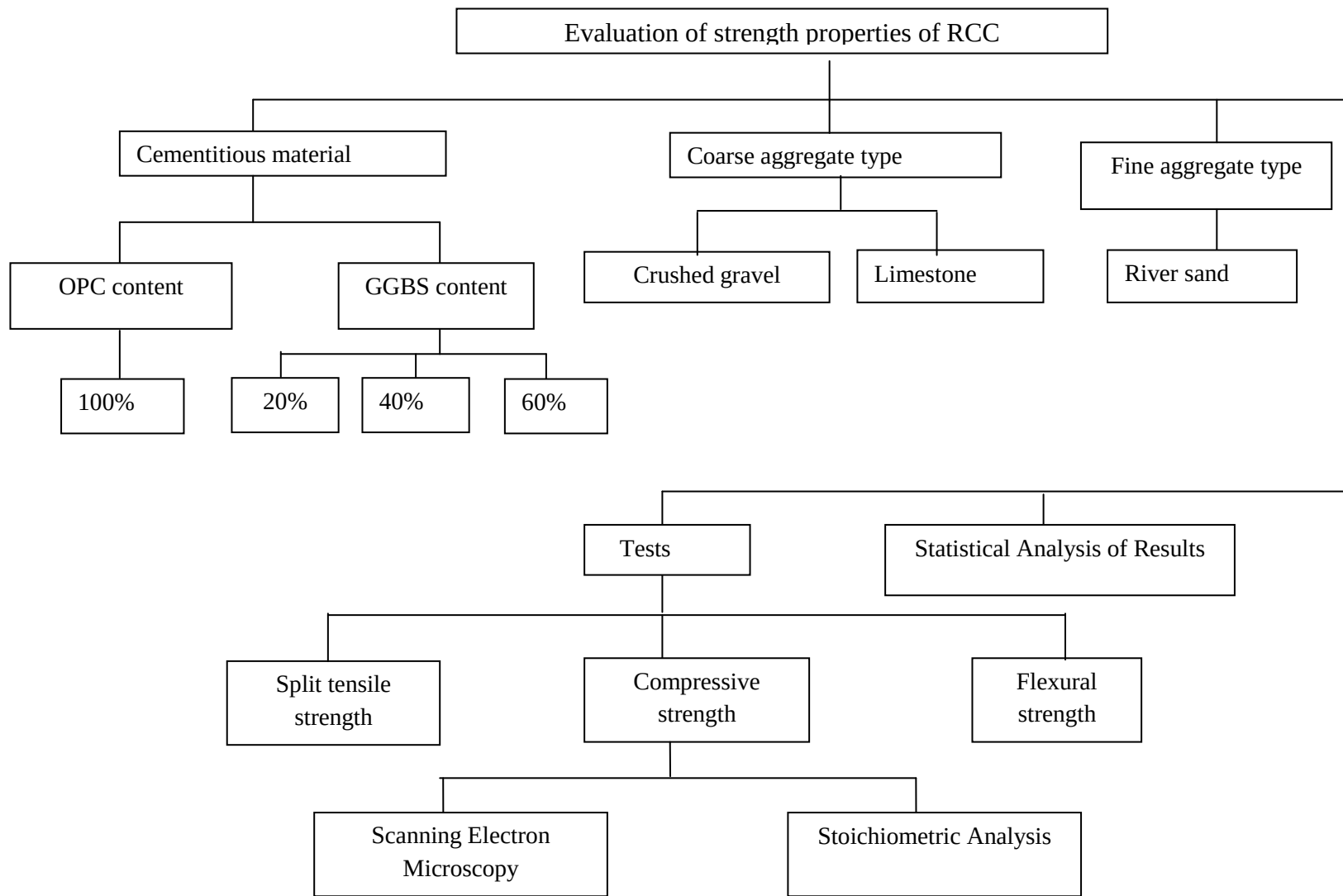


Fig.4.1: Test matrix to evaluate strength properties of RCC

4.3.1 Cement

In this study, Ordinary Portland Cement (OPC) conforming to IS: 8112-1989 (*BIS, 2005*) was used. The detailed information about the OPC has already been discussed in Chapter 3 under Section 3.2.1.

4.3.2 Ground Granulated Blast Furnace Slag

In the present study, GGBS conforming to IS: 12089- 1987 (*BIS, 1987*) was used. The chemical composition and physical properties of GGBS has already been presented in Table 3.3 and detailed information has been given in Section 3.2.2. In this study, cement was replaced by GGBS at a replacement level of 0%, 20%, 40% and 60% by weight basis.

4.3.3 Coarse Aggregates

Two types of coarse aggregates, viz. crushed gravel (CG) and limestone (LS), were used in the present investigation. The basic properties of both the aggregates have already been discussed in Section 3.2.3. Briefly, the Los angles abrasion, impact and crushing value for crushed gravel was 24.00, 14.31 and 16.19 respectively. For limestone aggregates, the corresponding values were 28.20, 18.28 and 24.16 respectively. Both types of aggregates were used in two size frictions, i.e. nominal maximum particle size of 20 mm and 10 mm. The sieve analysis results for both types of aggregates have already been discussed in Section 3.2.3.

4.3.4 Fine Aggregates and Water

The fine aggregates used was river sand conforming to Zone-II as per IS: 383-1970 (*BIS, 2002*). The values of specific gravity, water absorption and fineness modulus of fine aggregates were 2.70, 1.8% and 2.85 respectively. The sieve analysis result for fine aggregates has already been provided in Table 3.10. Normal tap water was used for casting and curing of the specimens.

4.4 MIX PROPORTIONS OF RCC

The proportioning of constituent materials is an important phase of RCC mix design to ensure dual purpose of strength and economy. RCC mixes are designed based on the concepts of either consolidation or compaction. The design based on consolidation is called concrete approach; while the one based on compaction is called soil approach (*ACI, 2002; Hansen and*

Reinhardt, 1991). The mix proportions of RCC, in the present study, are finalized based upon the soil approach. The detailed procedure related to the achievement of RCC using soil approach has been discussed in Chapter 3. The brief methodology adopted in the present study to achieve RCC by using soil approach is as under:

1. The proportions of coarse and fine aggregates were finalized in order to create a dense graded combined aggregate curve as per ACI 211.3R (*ACI, 2002*). In the present research work, a combination of 55% of fine aggregates, 30% of 10 mm coarse aggregates and 15% of 20 mm coarse aggregates was used.
2. After finalizing the aggregates proportions, water content and cement content were finalized based upon the combined guidelines laid down in ASTM D 1557 (*ASTM, 2009*) and ACI 211.3R (*ACI, 2002*). According to ASTM D 1557 (*ASTM, 2009*), water content of each mix was finalized by performing moisture-density tests. ACI 211.3R (*ACI, 2002*) suggests that for RCC pavements, the cementitious material content should range from 10 to 17% of the total dry mass of coarse and fine aggregates. For the present study, cementitious material content was varied from 11% to 17% in various trial mixes and based upon the 28 days cylinder compressive strength, 16% cement content was finalized for achieving RCC mix.
3. After obtaining the proportioning of all ingredients, the absolute volume and mass of the materials were calculated for a unit volume of concrete by assuming 1% air voids as per ACI 211.3R (*ACI, 2002*) guidelines.
4. Further, for evaluating the influence of GGBS, the finalized percentage of cement (16%) was systematically replaced by GGBS, with the replacement levels varying from 20% to 60% by weight. Along with this, the effect of using different types of coarse aggregates (viz. crushed gravel and limestone) was investigated by changing the aggregate type in the finalized mixes.

In this way, eight RCC mixes were created by using different proportions of GGBS and two types of coarse aggregates. The finalized mix proportion of all mixes has already been presented in Table 3.17.

4.5 CASTING AND CURING OF SPECIMENS

The test specimens were cast with the mix proportions given in Table 3.17. The RCC mix was prepared in a rotating mixer keeping a mix time of five minutes. To obtain the RCC mix,

coarse aggregates, fine aggregates and cementitious materials were taken in required proportions and mixed thoroughly for about two minutes to get a uniform mix in the dry state. After this, water was added to the dry mix and was mixed for another three minutes. The prepared mix was poured in moulds of variable dimensions based on the tests to be performed. All the moulds were greased with oil before pouring of concrete in the moulds. The specimens of different sizes were cast corresponding to the evaluation of different strength properties. Cylindrical specimens of size 150 x 300 mm were cast for determination of compressive strength and split tensile strength of the prepared mix; while beam specimens of size 150 x 150 x 700 mm were cast to evaluate flexural strength of the mix.

To achieve complete compaction, a vibrating table with a vibration frequency of 60 Hz, as specified in ASTM C 1170 (ASTM, 2008), was used. The RCC mix was filled into the moulds in three layers and compacted by putting surcharge on the top of each layer. According to ASTM C 1176 (ASTM, 2008), for cylindrical samples, surcharge of 4.9 kPa (9 kg) was used to facilitate compaction as shown in Fig. 4.2 (a). For beam specimens no standard was available, therefore the surcharge was kept as 4.9 kPa (54 kg) for proper consolidation as shown in Fig. 4.2 (b). The dimension of the surcharge was kept smaller than the inside dimensions of corresponding moulds. All the specimens were consolidated till the formation of ring between the inside face of the mould and the surcharge. The formation of this ring indicates that the RCC has reached its maximum density. After 24h of casting, all the specimens were de-moulded and kept in curing tank maintained at a temperature of $25 \pm 2^{\circ}\text{C}$ and 100% relative humidity.

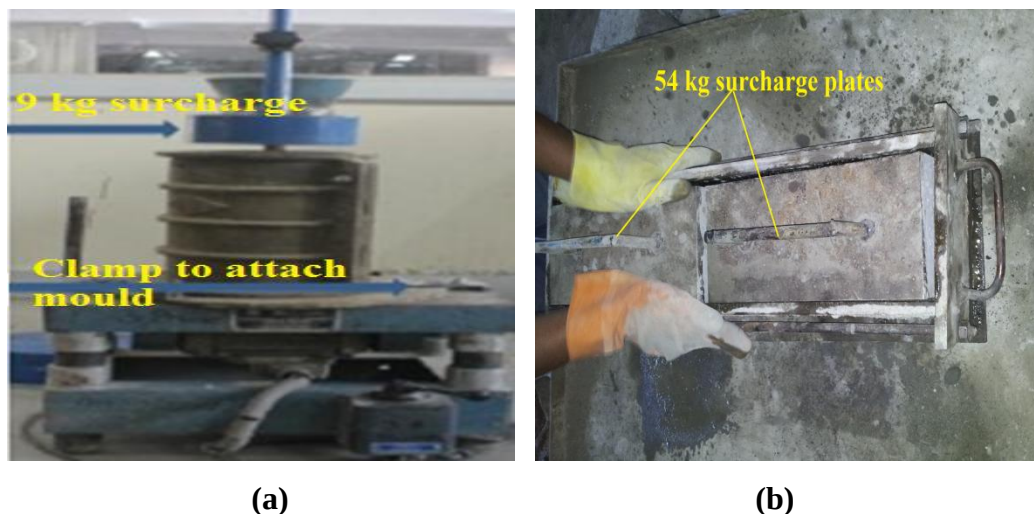


Fig. 4.2: Preparation of (a) Cylindrical and (b) Beam shaped specimens

4.6 TESTING PROCEDURE

All the strength properties were measured on three specimens and average of the three specimens was taken as the reference value. The detail of each property is presented in the following sections.

4.6.1 Compressive Strength

Compressive strength of the specimens was measured at the age of 7, 28, 60, 90 and 365 days of casting. A total of 120 cylindrical specimens were cast for compressive strength measurements, with two types of aggregates and three replacement levels of cement with GGBS. On the testing date, the specimens were taken out of the curing tank and were kept in the laboratory environment for 2 hours. The testing was performed in accordance with IS: 516-1959 (BIS, 2004), on compression testing machine of 5000 kN capacity, by applying load at a rate of 5 kN/s. The test set-up is shown in Fig. 4.3. At each testing age, three specimens were tested and the average of three specimens was taken as the reference compressive strength of the mix. The compressive strength was calculated according to the equation 4.1.

$$f_c = P / A \quad \text{.....(4.1)}$$

where, f_c = compressive strength (MPa or N/mm²),
 P = maximum load (N),
 A = cross section area of cylinder (mm²)



Fig. 4.3: Test set-up for measuring compressive strength of RCC

4.6.2 Split Tensile Strength

Split tensile strength was measured on the cylindrical specimens of size 150 x 300 mm according to IS: 5816-1999 (BIS, 2004). A total of 96 cylinders were tested for split tensile strength at the curing age of 7, 28, 90 and 365 days. The test specimen was placed horizontally between the plates of the compression testing machine and progressive compressive line load was applied between the opposite generators of the test specimen as shown in Fig. 4.4 (a). As obtained from the elastic analysis, a fairly uniform tensile stress is produced over 2/3 of the loaded diameter of the cylinder because of applied line loading (Singh G., 2012). The intensity of load at which the specimens fail by splitting into two halves along the loaded diameter is taken as the failure load. The test set-up used for split tensile strength test is shown in Fig. 4.4 (a), while the typical broken specimens are shown in Fig. 4.4 (b). The magnitude of split tensile strength was calculated from the failure load by using equation 4.2.

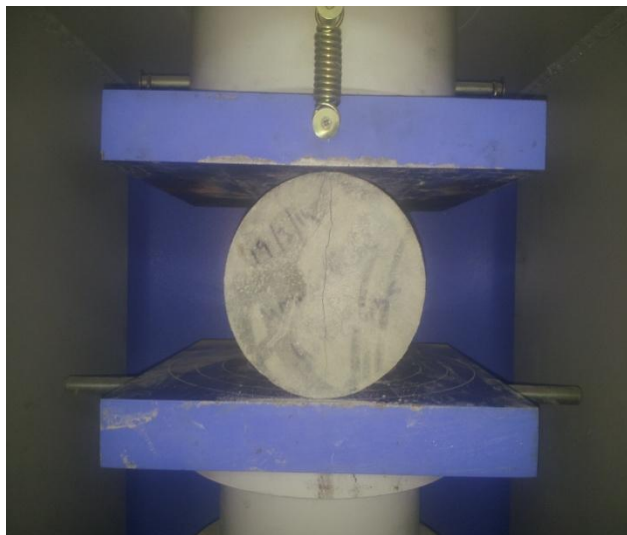
$$f_{st} = 2P / \Pi dL \quad \dots(4.2)$$

where, f_{st} = splitting tensile strength (N/mm²)

P = applied load at failure (N)

d = diameter of the cylinder (mm)

L = length of the cylinder (mm)



(a)



(b)

Fig. 4.4: (a) Test arrangement for split tensile strength (b) Tested split tensile strength test specimens

4.6.3 Flexural Strength

Concrete pavements fail because of bending stresses, therefore flexural strength becomes an important parameter for the design of concrete pavements. The deflection and cracking behaviour of concrete pavement depends upon the flexural strength or modulus of rupture of concrete (*Ahmed et al., 2016*). The flexural strength was determined on beam specimens of size 150 x 150 x 700 mm as per guidelines laid down in IS: 516--1959 (*BIS, 2004*). A total of 96 beam specimens were cast and tested for flexural strength at the curing age of 7, 28, 90 and 365 days.

The flexural strength test was performed on a universal testing machine of capacity 1000kN. At each testing age, the specimens were taken out of the curing tank and were allowed to air dry. In order to ensure that the axis of beam specimens was aligned with the axis of the loading machine, marking was done carefully on the uppermost surface of the beam specimens at center, 50 mm on both ends and 200 mm apart for the remaining span. Prior to testing, surface of the beam specimen as well as bearing surface of the supporting rollers was cleaned with dry cloth in order to ensure good contact between the rollers and the beam specimen. After that, the beam specimens were mounted on the steel roller having 38 mm diameter and having a center-to-center distance of 600 mm. The beam specimens was subjected to external load by means of three point loading on a loading frame as shown in Fig. 4.5. The load was applied at a rate of 0.065 kN/s with constant increment until failure of the specimen. In all the specimens, line of fracture was within the distances of 20 cm from the nearer support on the tensile side as shown in Fig. 4.6. Therefore, the flexural strength also known as modulus of rupture “ f_r ” was calculated according to the formula given in equation 4.3.

$$f_r = \frac{3p \times a}{b \times d^2} \quad \dots\dots(4.3)$$

where, f_r = flexural strength of concrete specimens (N/mm²)

p = maximum load applied to the specimen (N)

a = distance between the line of fracture and nearest tensile side support (mm)

b = width of the specimen (mm)

d = depth of the specimens at point of failure (mm)



Fig. 4.5: Test setup for measurement of flexural strength of the RCC



Fig. 4.6: Beam specimens after fracture showing the crack position

4.6.4 Scanning Electron Microscopy (SEM) and Energy Dispersive Spectroscopy (EDS)

To study the micro-structural characteristics of RCC, fractured small size pieces of specimens obtained after conducting compressive strength tests were collected and were used for Scanning Electron Micrographs (SEM) and Energy Dispersive Spectroscopy (EDS) analysis. SEM gives the information regarding the topographic and compositional behaviour of the material; while, EDS identifies the different phases present in the chemical composition.

SEM uses a high-energy beam of electron in a raster scan pattern to produce images of the surface topography of the specimen. Signal produced during the interaction of high energy beam with the specimen surface provides the information about the surface composition, topography and other properties. EDS works by measuring X-rays emitted from core electrons that move from higher energy levels to the ground state (*Dodds L., 2012*). In this study, fractured small pieces of the specimens were mounted on the SEM stub and images were obtained using secondary electron (SE) image mode. To ensure electrical conductance, fractured small pieces of the specimens were coated with thin layer of gold. The set up used in the present study for SEM and EDS is shown in Fig. 4.7.

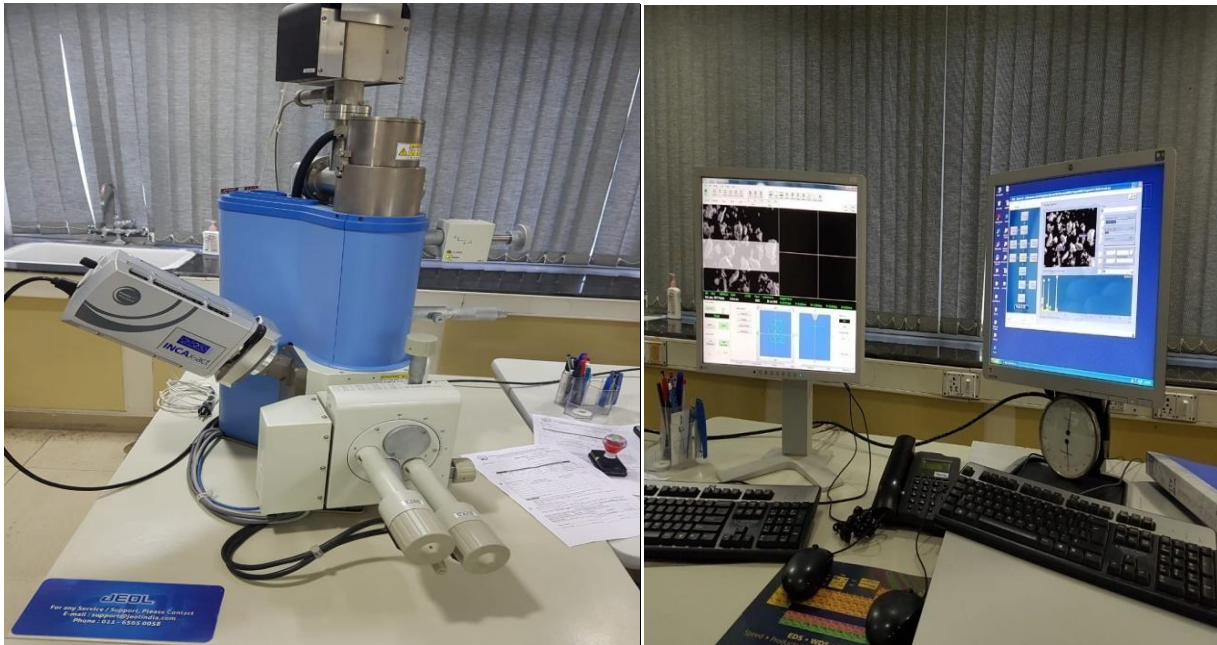


Fig. 4.7: Experimental set up for SEM and EDS spectroscopy

4.7 RESULTS AND DISCUSSIONS

The effect of variation in coarse aggregate type and incorporation of GGBS on various strength properties of RCC like compressive strength, split tensile strength and flexural strength is discussed in the succeeding sections. The average values of the strength properties obtained from three replicate specimens are plotted along with the error bar to exhibit the variability of the measured values. The results are supported by the micro-structural characteristics and stoichiometric analysis of the mixes.

4.7.1. Compressive Strength of RCC

The compressive strength results are shown in Figs. 4.8 - 4.10, it can be seen from all figures that the error bars are quite small indicating small variation in strength value from replicate to replicate. Also since there is small variability of results, all the results are discussed considering the average values.

(a) Effect of coarse aggregate type and curing age

The variation in compressive strength of RCC made with different types of aggregates viz. crushed gravel (CG) and limestone (LS) is shown in Fig. 4.8. From the figure, it can be observed that curing age and type of aggregates play an important role in compressive strength development. At 7 days of curing age, RCC containing limestone aggregates (LS-0) have 11.4% higher compressive strength in comparison with mixes containing crushed gravel aggregates (CG-0). However, when the curing time is extended from 28 days to 365 days, reversal in compressive strength trend was observed i.e. crushed gravel exhibited higher compressive strength than the corresponding strength achieved by using limestone aggregates. At 365 days of curing, CG-0 mix exhibited 9% higher strength as compared to mix LS-0. *Poitevin (1999)* also reported that in case of normal strength concrete, to get the same target strength almost 10-15% more cement was required with limestone aggregates against gravel aggregates. However, in present work, with same cement content for both aggregates mixtures, approximately 6-9% lower compressive strength was observed in LS-0 mix as compared to CG-0 mix.

Observed higher strength at early age with limestone aggregates can be attributed to the increase in hydration at early age as compared to the crushed gravel. Due to porous nature, limestone aggregates were able to absorb significant amount of water. Hence the

cement-aggregate bond in limestone aggregates becomes superior to that achieved in crushed gravel aggregates. From 7 days to 365 days of curing, mix LS-0 exhibited smaller compressive strength gain (31.7%) on comparison with mix CG-0 (61.2%). As the curing time become longer, slow down in hydration reaction occurs which results in small percentage increase in compressive strength. In case of limestone aggregates, decrease in compressive strength gain at longer curing age is also supported by other researchers. *Menendez et al. (2003)* investigated the effect of limestone filler on the strength development for conventional concrete. They observed that addition of limestone filler increases the early age hydration which results in high early strength, but it can reduce the later age strength. *Beshr et al. (2003)* also reported that mixes having limestone aggregates give higher early age strength.

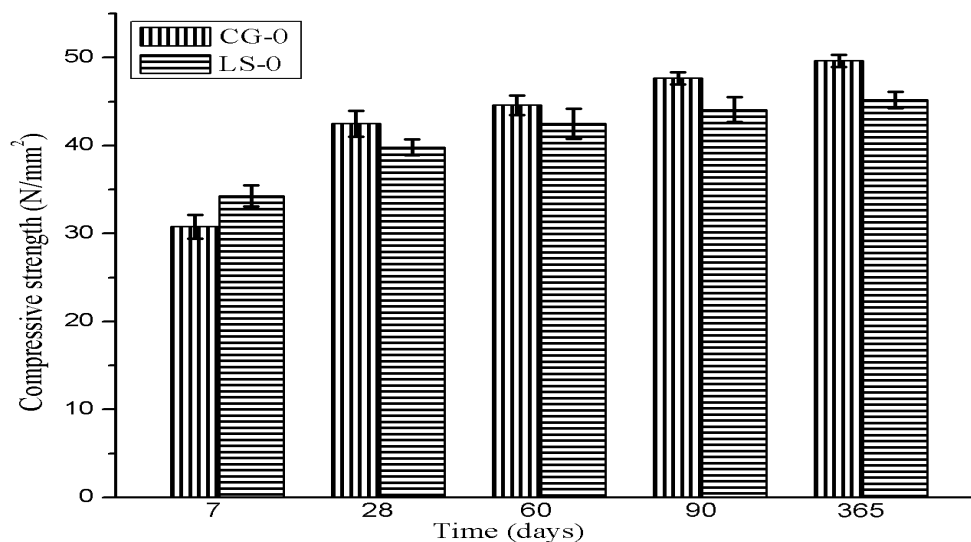


Fig.4.8: Effect of aggregate type on compressive strength development of RCC

When the 28 days compressive strength value of the mixes is compared with the prescribed guidelines of ACI 325.10R -95 (*ACI, 2001*), for the use of RCC as a surface course. It can be observed that RCC made with both crushed gravel aggregates and limestone aggregates achieve strength level higher than the prescribed values.

(b) Effect of GGBS content

The variation in compressive strength for RCC containing GGBS as supplementary cementitious material is shown in Figs. 4.9 and 4.10 for crushed gravel aggregates and for limestone aggregates, respectively. From both the figures, it can be observed that irrespective of

type of aggregate, compressive strength behaviour changes with the addition of GGBS into RCC mix. Fig. 4.9 indicates that at early curing ages, compressive strength for RCC containing GGBS was lower than the corresponding strength of control mix. At 7 days of curing age, the compressive strength of control mix (CG-0) was 30.8 N/mm^2 , whereas 6.7%, 13.2 % and 28.5% reduction in strength was observed for mixes CG-20, CG-40 and CG-60 respectively. Similarly, at 28 days, a strength reduction of 3.8%, 4.8% and 13.9% in comparison with the strength of corresponding control mix was registered. However, the 28 days strength of all mixes satisfied the limit of ACI 325.10R (ACI, 2001), that requires a minimum value of compressive strength of 27.6 N/mm^2 at 28 days for RCC to be used as a surface course.

The later age strength of mixes incorporating GGBS was found to be better than the corresponding control mix. Beyond the curing period of 28 days, RCC mixes containing 20% and 40% GGBS exhibited even higher strength than the control RCC, whereas CG-60 mix shows strength comparable to the control mix. At 365 days of curing age, maximum strength was observed for CG-40 mix followed by CG-20, CG-60 and CG-0 mix. The improvement in later age strength of mixes containing GGBS indicates that the glossy components in GGBS react slowly with water and therefore, the secondary pozzolanic reaction between GGBS and OH^- produced during hydration of cement is slow. Further, at large replacement levels of GGBS, whole of GGBS do not get consumed in the pozzolanic reaction due to lesser availability of Ca(OH)_2 from primary hydration reaction. Previous researchers, *Babu and Kumar (2000)*; *Li and Zhao (2003)*; *Chidic and Panesar (2008)*; *Patra and Mukharjee (2016)* also observed that the addition of GGBS into conventional concrete decreases the early age strength of concrete, but improves the later age strength and durability of conventional concrete mixes.

Fig. 4.10 depicts that RCC having limestone aggregates had compressive strength trend similar with the mixes incorporating crushed gravel aggregates. However, the mixes with limestone aggregates exhibited better compressive strength improvement with the replacement of cement with GGBS. With the increase in percentage of GGBS in the mix, the compressive strength increases till 40% replacement level, there after strength of the mix decreases. However, the cement replaced with 60% GGBS presents higher strength than the corresponding control mix.

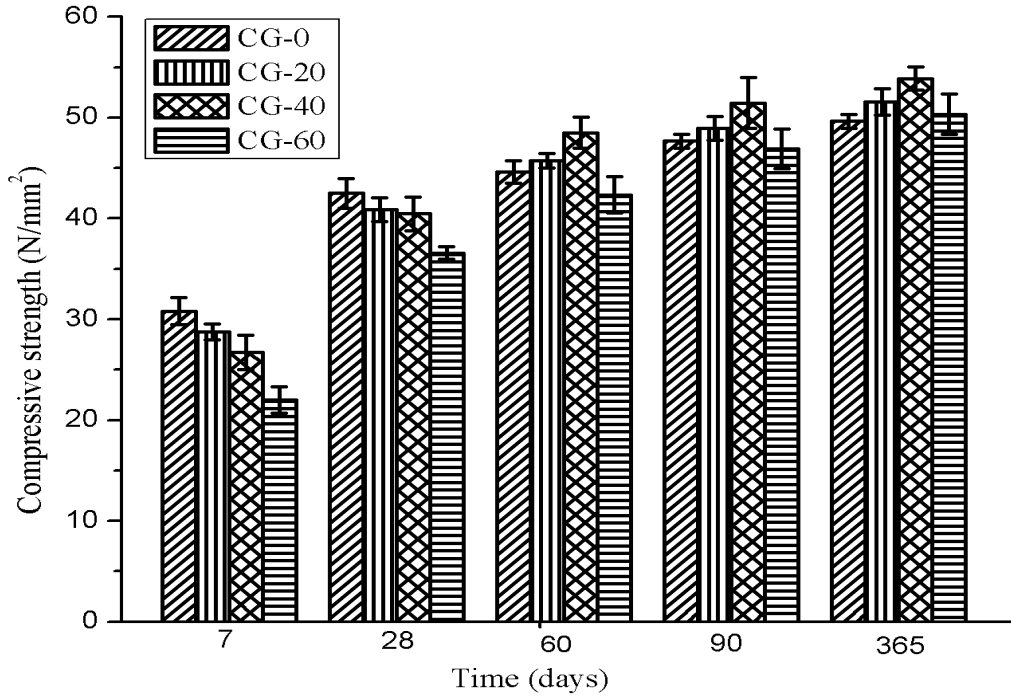


Fig. 4.9: Effect of GGBS content on compressive strength of RCC incorporating crushed gravel aggregates

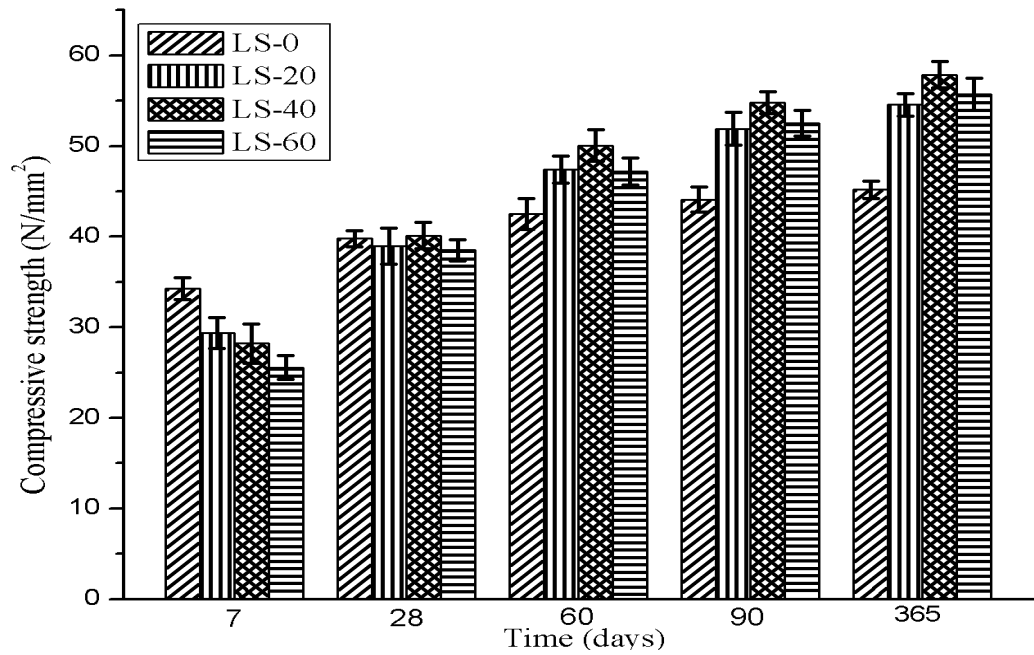


Fig. 4.10: Effect of GGBS content on compressive strength of RCC incorporating limestone aggregates

At early curing age of 7 days, compressive strength of all mixes having GGBS as supplementary cementitious material was lower than the corresponding control mix i.e. LS-20, LS-40 and LS-60 shows 14.2%, 17.7% and 25.5% lower strength as compared to LS-0. At 28 days of curing, the strength of mixes with GGBS was almost similar to the control RCC mix. After this age, all replacement levels of cement with GGBS shows compressive strength higher than the control RCC. At 365 days of curing, 20.6%, 28.0% and 23.3% increase in strength was observed in mixes LS-20, LS-40 and LS-60 respectively as compared to LS-0 mix.

Further, compressive strength development of all mixes at various ages with respect to 28 days strength of the corresponding mix was analyzed. In CG-0 mix smaller strength gain of 16.8% was observed from 28 days to 365 days; while the corresponding strength gain was highest 37.6% in CG-60 mix. In mix CG-20 and CG-40 mix, the strength gain was observed to be 26.1% and 33.1% respectively. In case of RCC containing limestone aggregates, strength gain observed was 13.5%, 40.0%, 44.20% and 44.6% in mixes LS-0, LS-20, LS-40 and LS-60 respectively. The strength gain behaviour shows that the percentage of GGBS has a great influence on compressive strength development. The strength gain of all the mixes increased with progressive increase in GGBS content irrespective of aggregate type. Previous investigator, *Patra and Mukharjee (2016)*, also reported that increasing content of GGBS led to higher strength gain in conventional concrete.

4.7.2. Scanning Electron Microscopy (SEM) and Energy Dispersive Spectroscopy (EDS)

The concrete microstructure generally consist of calcium hydroxide ($\text{Ca}(\text{OH})_2$), calcium-silicate-hydrate gel (C-S-H gel), ettringites (calcium sulfo-aluminate hydrate) and interfacial transition zone (ITZ) present between the aggregates and hardened cement paste. To study the microstructure behavior of RCC, small size pieces were taken from fractured compressive strength specimens and used for SEM and EDS analysis. It was established that higher magnification is generally used to get an insight into the cement composition, but this large interaction volume was interfering (*Dodds L., 2012*). Thus, all the SEM images were taken at low magnification of (1000 x) to get the reasonable overview of the surface.

SEM images of RCC mixes incorporating crushed gravel and limestone aggregates at 28 days of curing are shown in Figs. 4.11 and 4.12 respectively along with the EDS spectra. Both figures clearly indicate that with the variation in coarse aggregate type, microstructure of concrete gets altered. In both figures, Ca(OH)_2 in the form of plate shaped structure, C-S-H gel in form of cotton shaped structure and ettringites in the form of needle like structure with variation was observed. The particle structures of portlandite (Ca(OH)_2) observed in case of limestone aggregates are different from the ones observed in crushed gravel aggregates. In mix incorporating limestone aggregates, flat plate type structures of Ca(OH)_2 was observed (Fig. 4.12), while in case of mix with crushed gravel aggregates, hexagonal type structure of Ca(OH)_2 was observed (Fig. 4.11). In RCC mix containing crushed gravel aggregates, needle like structures observed are less as compared to the mix having limestone aggregates. Also in RCC mix containing limestone aggregates the cotton shaped C-S-H gel is less monolithic than the C-S-H gel observed in crushed gravel RCC mix. The presence of larger number of ettringites along with the less monolithic C-S-H gel in mix containing limestone aggregates can be attributed to lower 28 days compressive strength observed in limestone aggregates as compared to the crushed gravel aggregates.

The EDS spectrum of RCC containing crushed gravel and limestone aggregates shows that with the variation in aggregate type, major variation in the quantum of elements viz. Si, C and Ca was registered while all other elements differ by minor proportions. In crushed gravel aggregates the major compound was SiO_2 (64%) while in limestone aggregates its composition was limited to 23%. Since, limestone aggregates are rich in carbon content therefore, in limestone aggregates, the highest CO_2 content was 50% and CaO content was 56%. On the other hand, in crushed gravel, maximum content of CO_2 was 17% and CaO was 6% only.

The incorporation of GGBS into the control mix greatly altered the microstructure of RCC. The SEM images along with the EDS obtained at 28 days of curing for RCC containing 20%, 40% and 60% GGBS are presented in Figs. 4.13 to 4.18 for both type of aggregates. The chemical composition obtained from the EDS analysis is presented in Table 4.1 and 4.2 for crushed gravel and limestone aggregates respectively. The main constituents of GGBS are silicon (Si), aluminium (Al), magnesium (Mg) and iron (Fe) in their oxide forms. Thus, RCC

mixes containing GGBS had higher percentages of this compound viz. SiO_2 , Al_2O_3 , FeO , MgO in comparison with the control RCC mix irrespective of the aggregate type.

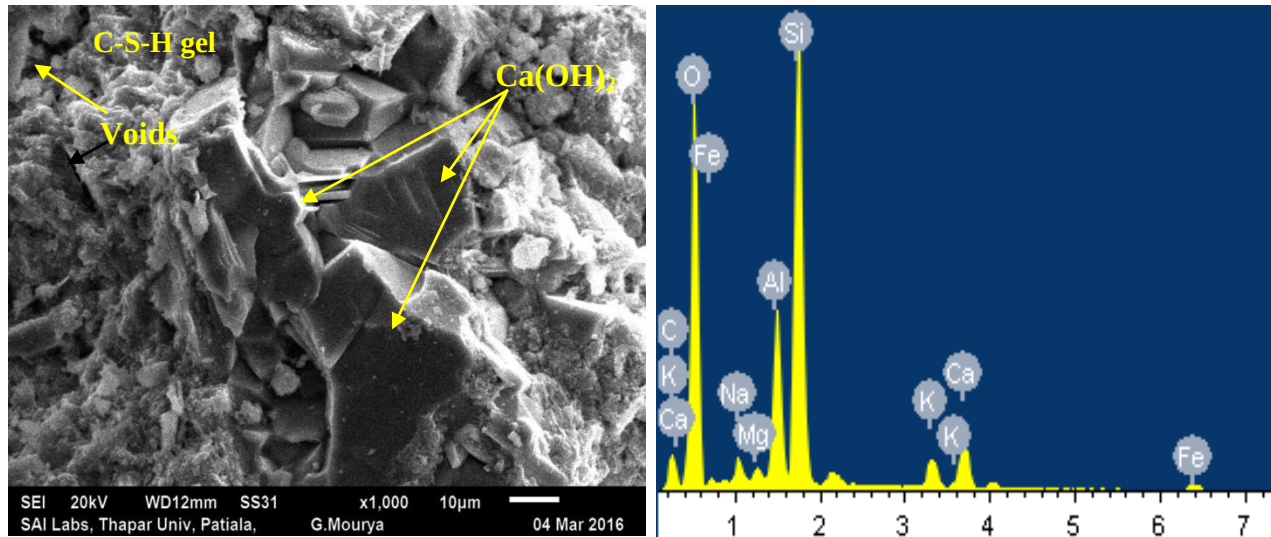


Fig. 4.11: SEM and EDS for CG-0 mix at 28 days of curing period

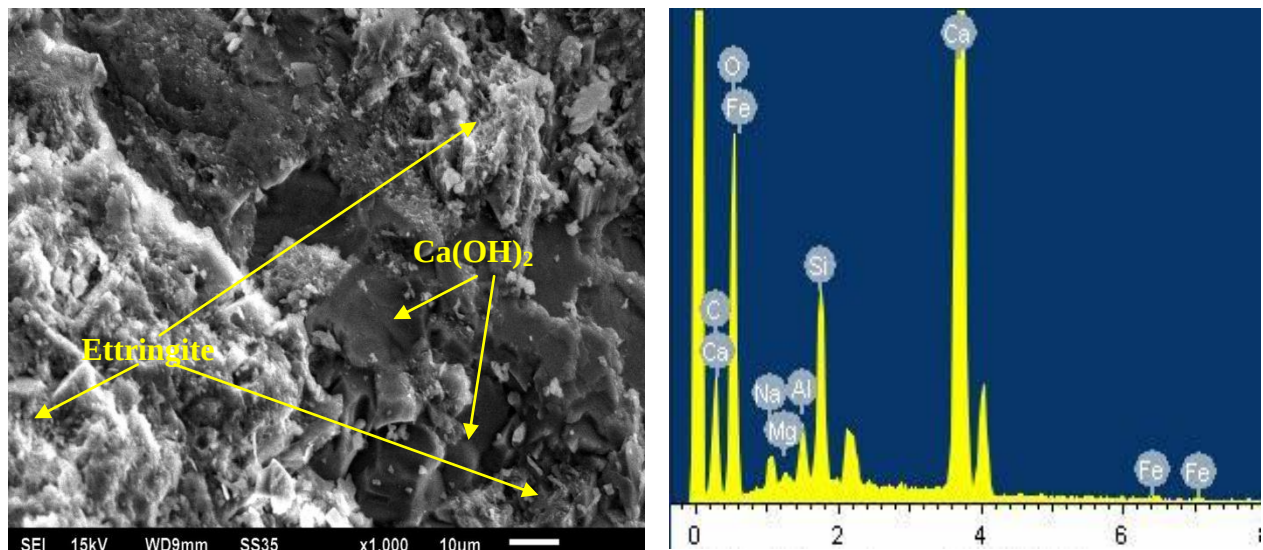


Fig. 4.12: SEM and EDS for LS-0 mix at 28 days of curing period

Table 4.1: Chemical composition obtained from EDS analysis for various RCC mixes containing crushed gravel aggregates

Chemical compound (%)	Nomenclature			
	CG-0	CG-20	CG-40	CG-60
CO ₂	17.93	16.59	7.43	3.41
Na ₂ O	1.87	4.89	3.87	4.69
MgO	0.48	1.11	1.98	2.01
Al ₂ O ₃	15.05	15.55	15.08	15.01
SiO ₂	51.44	49.06	58.82	64.46
CaO	6.63	6.11	6.63	3.78
FeO	3.12	6.46	5.83	6.42
K ₂ O	3.49	0.24	0.36	0.22

Table 4.2: Chemical composition obtained from EDS analysis for various RCC mixes containing limestone aggregates

Chemical compound (%)	Nomenclature			
	LS-0	LS-20	LS-40	LS-60
CO ₂	50.27	15.58	22.02	24.94
Na ₂ O	1.76	0.28	0.40	0.57
MgO	0.43	1.58	3.43	2.83
Al ₂ O ₃	2.24	4.13	8.08	11.35
SiO ₂	8.10	18.84	20.14	23.45
CaO	36.72	56.90	44.51	33.14
FeO	0.49	2.70	1.42	3.73

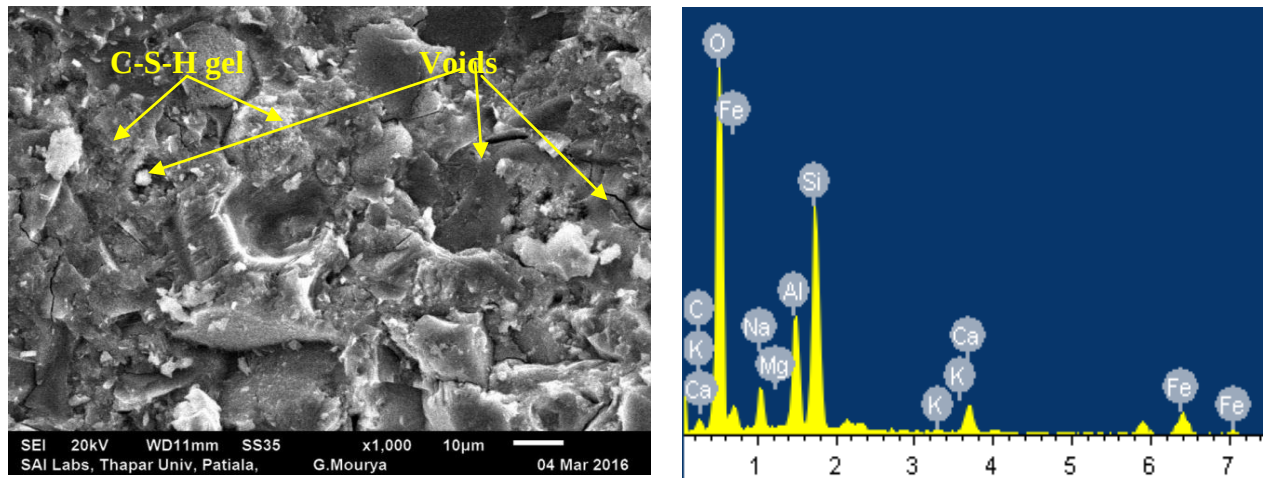


Fig. 4.13: SEM and EDS for CG-20 mix at 28 days of curing period

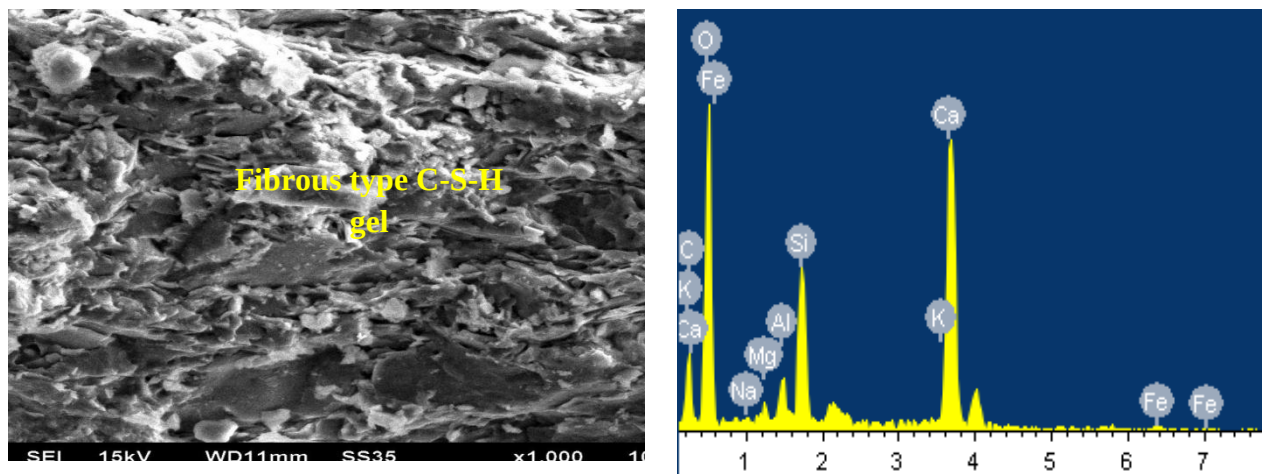


Fig. 4.14: SEM and EDS for LS-20 mix at 28 days of curing period

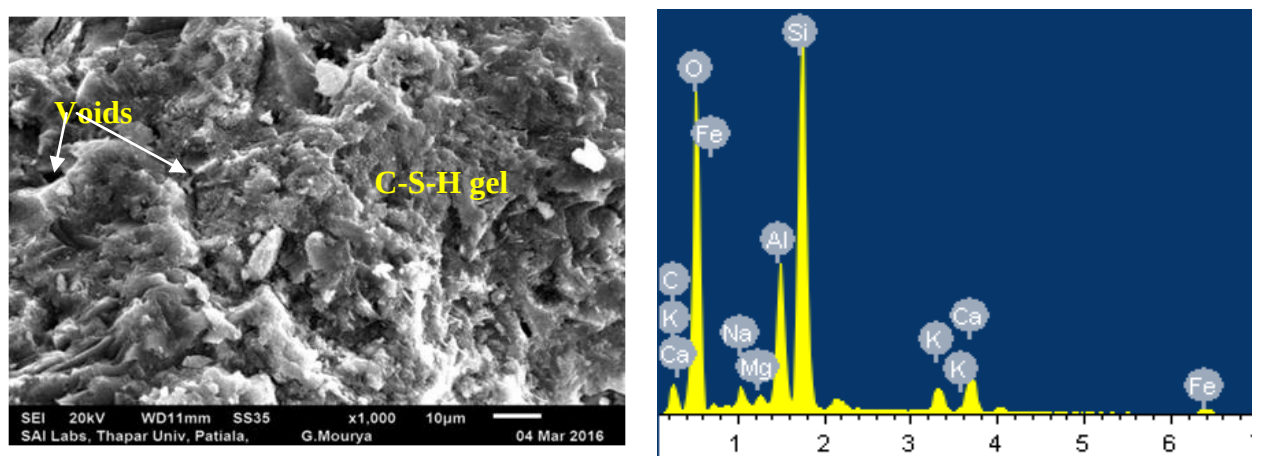


Fig. 4.15: SEM and EDS for CG-40 mix at 28 days of curing period

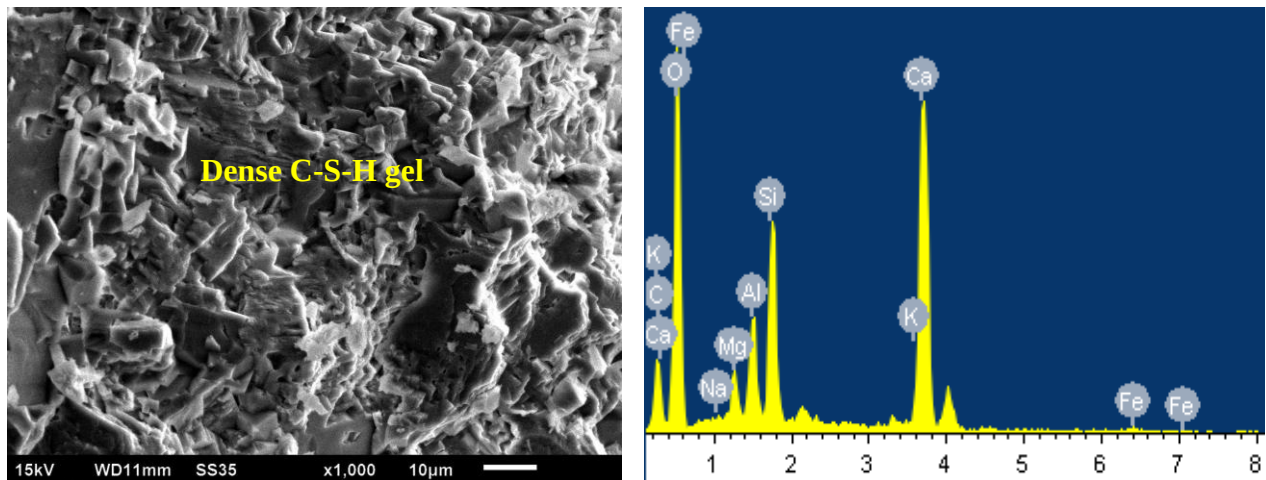


Fig. 4.16: SEM and EDS for LS-40 mix at 28 days of curing period

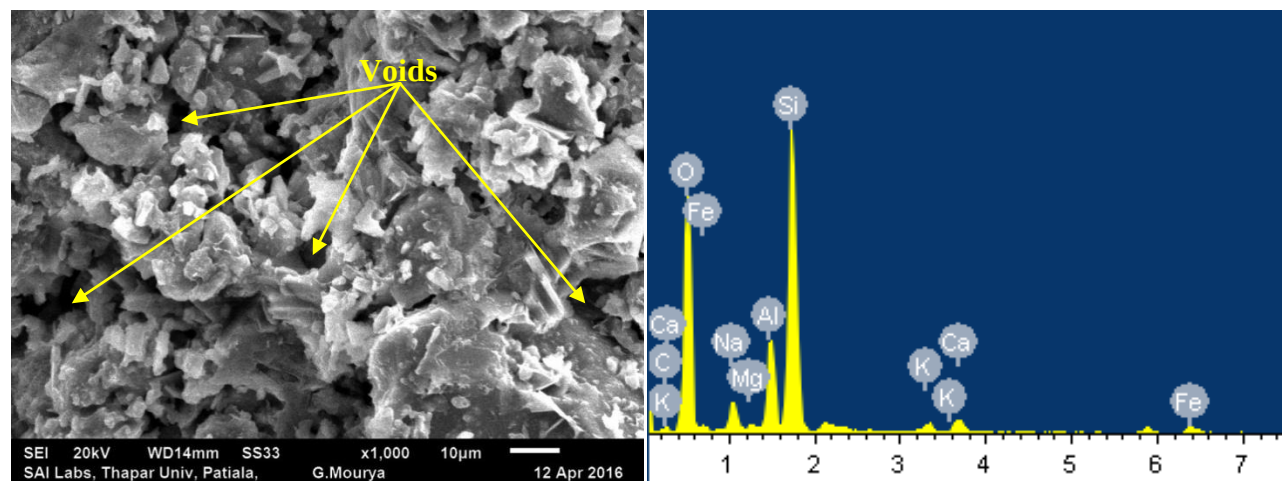


Fig. 4.17: SEM and EDS for CG-60 mix at 28 days of curing period

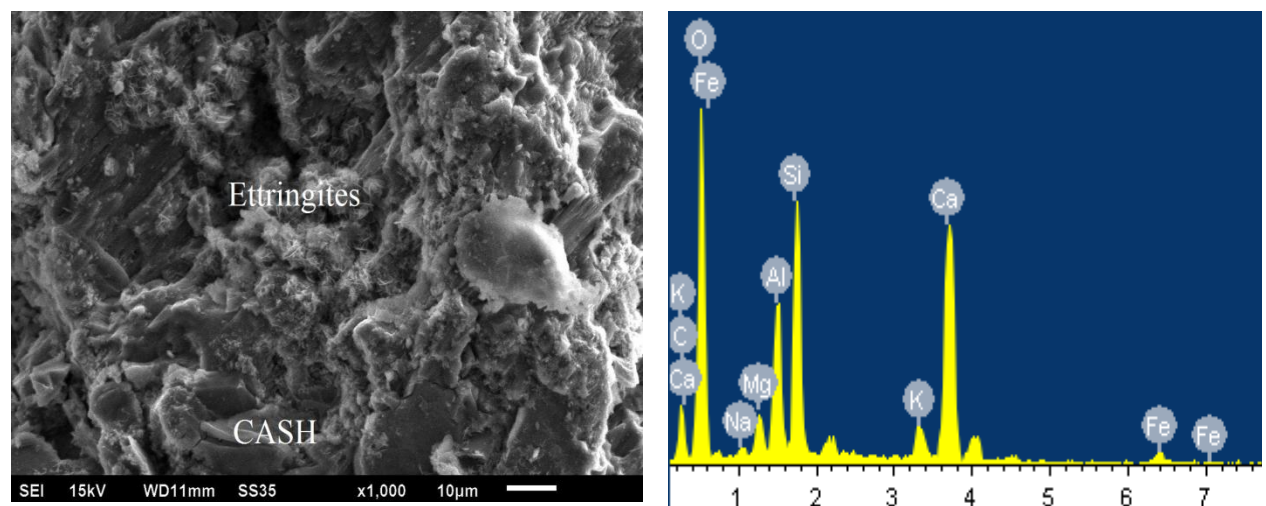


Fig. 4.18: SEM and EDS for LS-60 mix at 28 days of curing period

Figs. 4.13-4.18 clearly show that the addition of GGBS alters the surface morphology of the mix. For all RCC mixes containing GGBS, light grayish surface with white spots was observed; while in control mix dark grayish surface was seen. In RCC containing crushed gravel aggregates with 60% GGBS incorporation, large numbers of white spots with lightest grayish surface as compared to other mixes was registered. It indicates the presence of high density material such as Fe, which has a large capture cross section compared to other constituents of the specimen (Dodds L., 2012). Table 4.1 also support this agreement, highest percentage of FeO (6.42%) was reported in CG-60, while in corresponding control mix (CG-0) this value was limited to 3.12%.

Figs. 4.13-4.16 reveal that irrespective of aggregate type, the mixes having GGBS have more amount of C-S-H gel as compared to the control mix. The calcium hydroxide crystals get converted into C-S-H gel due to secondary pozzolanic reaction with the GGBS incorporation. For RCC containing 20% and 40% GGBS, C-S-H gel becomes more visible and porous in structure. Less monolithic behavior of C-S-H gel may be due to partial reaction of GGBS with water and presence of glossy slag parcels at this stage of curing. It can also be observed that GGBS addition influences the interfacial transition zone (ITZ) present between the aggregates and hardened cement paste (Gao *et al.*, 2005). With 20% and 40% incorporation level of GGBS, ITZ of the paste gets altered with very few needle shaped ettringites as compared to corresponding control mixes. Further, ITZ zone observed in the mix having 40% GGBS is denser than the corresponding zone observed with 20% GGBS incorporation level. It shows that 40% GGBS incorporation improves the transition zone on comparison with 20% GGBS replacement levels.

RCC incorporating 60% GGBS with crushed gravel aggregates shows the microstructure entirely different from the remaining structures. At 60% replacement level (CG-60), larger voids with ettringites in the form of needle like structures were observed. The slight decrease in 28 days compressive strength in CG-60 mix can be attributed to variation in the microstructure of RCC containing GGBS as compared to the control specimen. Also, in LS-60 mix, flat structures are seen, indicating formation of calcium aluminium silicate hydrate (CASH).

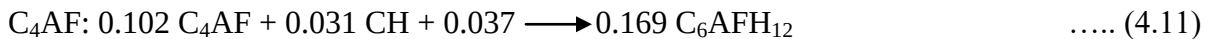
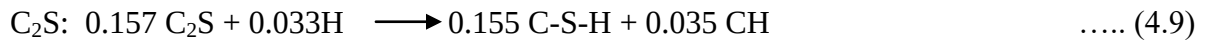
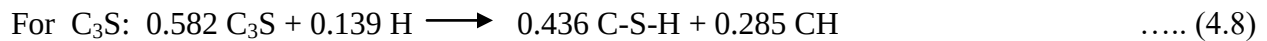
4.7.3 Stoichiometric Analysis

The variation in strength of the RCC mixes with change in replacement level of GGBS can be understood from basic stoichiometric equations of the hydration reaction. The stoichiometric analysis of cement hydration involves reaction of cement compounds in the presence of water. The major compounds of Portland cement are: alite, belite, celite and brownmillerite. The phase compositions of these compounds are mostly found by Bogue equation, which is based on the chemical composition of cement (*Prochon and Piotrowski, 2016*).

On the basis of chemical composition of cement used in the present study (Table 3.1), the phase composition of various cement compounds, viz. C_3S , C_2S , C_3A and C_4AF is obtained as 58.19%, 15.71%, 7.24% and 10.18% respectively. In addition to these major compounds, some minor compounds such as MgO , SO_3 and Na_2O are present, which are together taken as 8.67%. The basic stoichiometric equations of hydration of major cement compounds can further be summarized as (*Neville, 2012*):



On the basis of atomic masses, it can be observed that 100 grams of C_3S and C_2S produces 49 grams and 22 grams of CH respectively and C_4AF consumes approximately 30 grams of CH. Therefore, the stoichiometric equations for C grams of cement can be written as:



From the above stoichiometric equations, it was estimated that C grams of cement produced 0.862 grams of C-S-H and 0.289 grams of CH after complete hydration.

Addition of GGBS into the mix leads to alternation in the hydration process. GGBS comprises of higher quantities of SiO_2 , Al_2O_3 and MgO which promote the formation of hydrotalcite (M_5AH_{13}) and C-S-H (Stephant *et al.*, 2015; Kolani *et al.*, 2012). In mixtures containing GGBS, a secondary pozzolanic reaction occurs between CH liberated during the hydration of cement and silica component of GGBS. The stoichiometric equation for the secondary pozzolanic reaction for pure silica can be written as:



By considering the atomic masses of all the elements, it was estimated that 1 part of silica would react with 1.851 parts of CH to produce 2.851 parts of C-S-H gel. The chemical composition of GGBS shows that it has 31.6% of silica in it. Therefore, one gram of GGBS will react with 1.851×0.316 (i.e. 0.585 C) gram of CH to produce C-S-H gel. Since 0.289 C grams of CH is produced after complete hydration, it will require maximum of 0.494 grams of GGBS for its conversion into C-S-H gel during secondary pozzolanic reaction. Therefore, by stoichiometric analysis, upper limit of substitution of cement by GGBS can be taken as 49.4%. Stoichiometric upper limits indicate that GGBS does not contribute towards strength development by chemical reactions beyond 50% replacement level, after which it will simply act as filler.

The hydration reaction of a mix containing secondary cementation materials like GGBS can be termed as lime controlled or pozzolanic controlled based on the stoichiometric analysis. If the amount of CH produced by the hydration reaction is more than the amount that can be consumed by pozzolanic material present in the mix, the reaction is said to be pozzolanic controlled; and if CH produced is less than the amount required by the pozzolanic material, the reaction is said to be lime controlled. The stoichiometric analysis of RCC mixes used in the present study is presented in Table 4.3. It can be observed from the table that the reaction is pozzolanic controlled till 20% replacement of cement by GGBS, and the reaction becomes lime controlled at 40% and 60% replacement level. In CG-40 mix the imbalance between the CH liberated and CH consumed during pozzolanic reaction is minimum. Therefore, CG-40 mix is the most optimum mix in terms of theoretical CH consumption and in terms of strength development.

Table 4.3: Stoichiometry of hydration reaction for all mixes

Mix Designation	Cement content (kg)	GGBS content (kg)	Maximum CH librated (kg)	CH consumed by pozzolanic reaction (kg)	Amount of CH left/required	Limiting parameter
CG-0/LS-0	327.87	0	94.76	-	94.76/--	Pozzolana
CG-20/LS-20	259.38	64.85	74.96	37.94	37.02/--	Pozzolana
CG-40/LS-40	193.65	129.10	55.96	75.52	--/19.56	Lime
CG-60/LS-60	128.14	192.22	37.03	112.45	--/75.42	Lime

4.7.4 Split Tensile Strength of RCC

Split tensile strength was measured at the testing age of 7, 28, 90 and 365 days of curing. The results shown in following section corresponds to the average of three specimens tested for each mix.

(a) Effect of coarse aggregate type

Fig. 4.19 shows the variation in split tensile behaviour of control RCC mix, with the variation in coarse aggregate type and exposure duration. With the variation in coarse aggregate type, split tensile strength follows the behaviour identical with the compressive strength behaviour. At early duration of testing, mixes with limestone aggregates present marginally higher split tensile strength but at later durations, crushed gravel aggregates exhibited marginally higher split tensile strength values. At 365 days of curing, marginally higher (5.4%) split tensile strength was observed in mixes incorporating crushed gravel aggregates as compared to limestone aggregates. Thus, from the observed marginal variation in split tensile value, it can be concluded that the variation in aggregate type does not have any significant effect on split tensile behaviour of RCC.

The development in split tensile strength from 7 to 365 days of curing was higher in mixes having crushed gravel aggregates as compared to the corresponding values obtained in mixes having limestone aggregates. The strength gain in case of crushed gravel aggregates was 63.9%; while for limestone aggregates this value was limited to 45.3%. The lower strength gain in limestone aggregates can be attributed to the acceleration in hydration reaction at early ages followed by slow down of the hydration reaction at later ages.

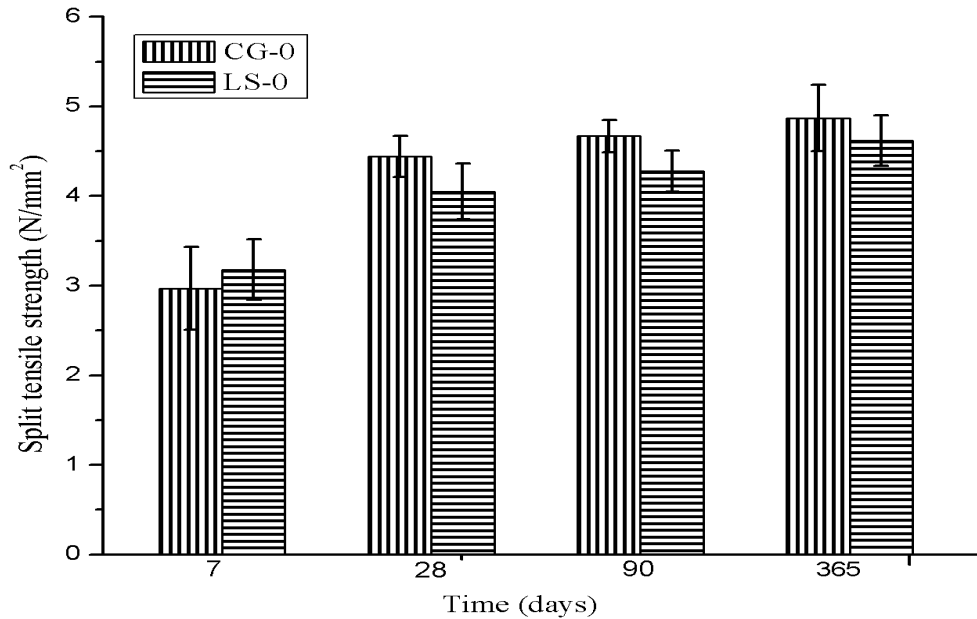


Fig.4.19: Effect of aggregate type on split tensile strength of RCC mixes

(b) Effect of GGBS content

The effect of GGBS content on split tensile behaviour of RCC is presented in Figs. 4.20 and 4.21, for the mixes incorporating crushed gravel aggregates and limestone aggregates respectively. From both the figures, it was observed that at early curing ages, GGBS incorporation adversely affect the split tensile strength of RCC, while with the enhancement in curing age, improvement in strength was registered. The highest split tensile strength was registered against 40% GGBS incorporation, irrespective of the aggregate type. To use RCC in pavements, according to *ACI 325.10R (2001)* splitting tensile strength range at 28 days is 2.8 to 4.1 MPa. From both figures, it can be clearly observed that all RCC mixes satisfied the specified limit at 28 days of age, indicating that all the prepared mixes can be used for RCC pavements.

Fig. 4.20 depicts a 33.6%, 30.9% and 29.2% lower split tensile strength for mixes incorporating 20%, 40% and 60% GGBS respectively with respect to control mix at 7 days of curing. After 28 days of curing, significant improvement in split tensile strength was observed in mixes CG-40 and CG-60. At 365 days of curing, the increase in split tensile strength for CG-40 and CG-60 over that of CG-0 mixture was 26.6% and 21.7% respectively. RCC containing

limestone aggregates presents the behavior similar with the crushed gravel aggregates at all replacement levels of cement with GGBS. At 28 days of curing, 14.1% and 10.3% increase in strength was observed in mixes LS-40, LS-60 respectively as compared to mix LS-0, while in mix LS-20, 5.4% drop in strength was noticed (Fig. 4.21). At 365 days of curing large increase in strength was observed in LS-40 as compared to control mix. At this curing age, 2.3%, 38.1% and 28.3% increase in split tensile strength was registered in LS-20, LS-40 and LS-60 mixes respectively with reference to LS-0 mix.

The observed behaviour shows that the addition of GGBS in concrete improved the splitting tensile strength of RCC after 28 days of curing age irrespective of aggregate type. It was established that split tensile strength of concrete is affected with quality of paste and interfacial transition zone in the concrete (*Beshr et al., 2003*). The fineness and pozzolanic properties of GGBS improves the quality of paste at later ages of curing, which results in increase in split tensile strength of RCC. Observed behavior for split tensile strength are in good agreement with those reported by *Atis et al. (2004)* and *Cao et al. (2000)*.

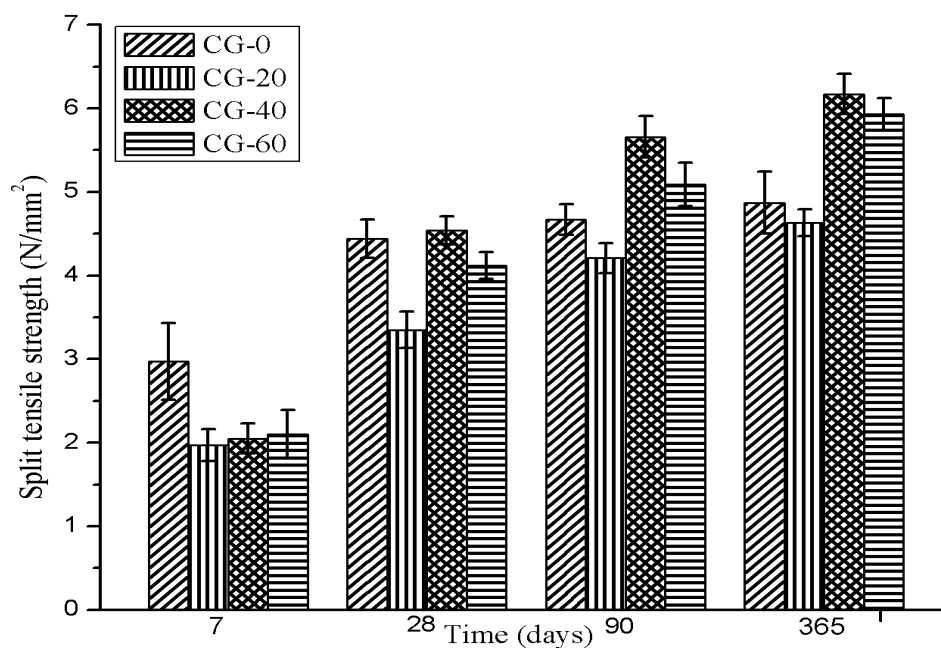


Fig. 4.20: Effect of GGBS content on split tensile strength of RCC incorporating crushed gravel aggregates

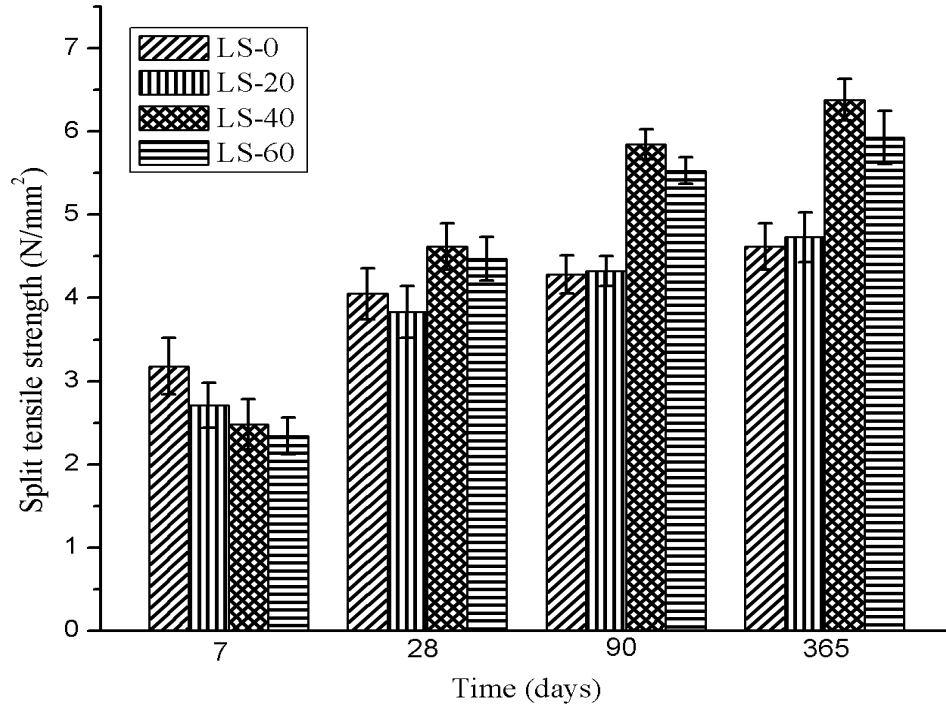


Fig. 4.21: Effect of GGBS content on split tensile strength of RCC containing limestone aggregates

Further, the strength gain from 7 to 365 days for compressive and split tensile strength were calculated and summarized in Table 4.4. From the table, it was clearly observed that for the control RCC mix comparable gain in compressive and split tensile strength was observed for both aggregates series. On the other hand, the incorporation of GGBS affects the splitting tensile behavior differently than the compressive strength behavior. For RCC mixes containing GGBS the strength gain in split tensile was higher than the compressive strength. For instant, in CG-40 mix, gain in split tensile strength from 7 days to 365 days was 200% while for compressive strength this gain was limited to 101% only. Similarly, for limestone aggregates, in LS-40 mix, the gain in split tensile strength was 157%, while 105% strength gain was registered in case of compressive strength.

Table 4.4: Compressive and split tensile strength gain for all RCC mixes

RCC mixes incorporating	Nomenclature	Strength gain (%) from 7 days to 365 days	
		Compressive	Split Tensile
Crushed gravel aggregates	CG-0	61	63
	CG-20	79	135
	CG-40	101	200
	CG-60	128	182
Limestone aggregates	LS-0	31	45
	LS-20	85	74
	LS-40	105	157
	LS-60	118	153

4.7.5 Relationship between Split Tensile and Compressive Strength for RCC

In order to establish a relationship between split tensile strength and compressive strength, graphs were plotted for both crushed gravel aggregates and limestone aggregates, and are presented in Figs. 4.22 and 4.23, respectively. Both figures revealed the split tensile strength for both RCC mixes increased with increase in compressive strength, but the increasing range is different for the control RCC mix as compared to mixes incorporating GGBS.

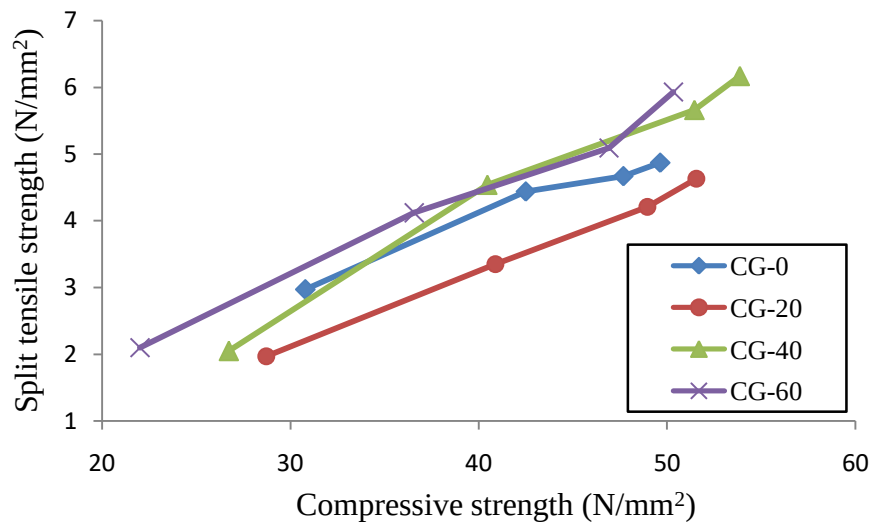


Fig. 4.22: Split tensile versus compressive strength for RCC containing crushed gravel aggregates

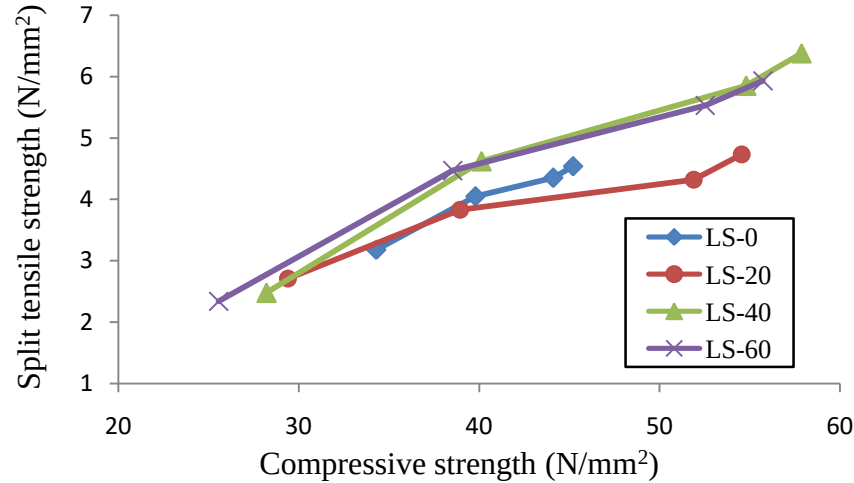


Fig. 4.23: Split tensile versus compressive strength for RCC containing limestone aggregates

Till date, a number of relationships are available for the conventional concrete to evaluate the split tensile strength based upon the compressive strength. These relationships are presented in Table 4.5 and also employed on the present compressive strength results of control RCC mixes.

Table 4.5: Details of existing relationship between compressive and splitting tensile strength for conventional concrete

S.No.	Source	Relationship	Predicted values of $*f_{st}$ for CG-0 mix by various relationships			
			7d	28d	90d	365d
1.	Gardner et al. (1988)	$f_{st} = 0.47 f_c^{0.59}$	3.55 **(19.5%)	4.29 (-3.2%)	4.59 (-1.5%)	4.70 (-3.3%)
2.	CEB-FIB model (1990)	$f_{st} = 0.3 f_c^{0.66}$	2.88 (-2.9%)	3.56 (-19.7%)	3.84 (-17.6%)	3.95 (-18.9%)
3.	Oluokun et al. (1991)	$f_{st} = 0.294 f_c^{0.69}$	3.13 (5.3%)	3.91 (-11.9%)	4.23 (-9.4%)	4.35 (-10.6%)
4.	ACI committee 318 (ACI, 1999)	$f_{st} = 0.56 f_c^{0.5}$	3.10 (4.6%)	3.65 (-17.8%)	3.87 (-17.2%)	3.95 (-18.9%)
5.	Selim (2008)	$f_{st} = 0.106 f_c^{0.948}$	2.73 (-8.0%)	3.71 (-16.5%)	4.14 (-11.5%)	4.29 (-11.8%)
6.	Ahmad and Shah (1985)	$f_{st} = 0.462 f_c^{0.55}$	3.04 (2.4%)	3.63 (-18.2%)	3.87 (-17.1%)	3.96 (-18.7%)

$*f_{st}$: split tensile strength

**variation of predicted strength from the experimental split tensile strength

Table 4.5 reveals the split tensile strength value obtained by using the various relationships based upon the measured compressive strength value for CG-0 mix. The values shown in bracket represent the variation between the predicted split tensile values from the experimental values. From the table; it was observed that at 7 days of testing duration, some relationships underestimate the strength value, while some overestimate the split tensile strength. But after this duration, all the existing relationships underestimate the split tensile strength. This underestimation in strength at later testing duration represents the better performance of RCC mix in terms of split tensile strength on comparison with compressive strength.

Figs. 4.24 and 4.25 illustrate the comparison of split tensile strength obtained by various relationships with the experimentally observed strength value for control RCC mixes incorporating crushed gravel and limestone aggregates, respectively. From both figures, it was observed that the *Gardner et al. (1988)* relationship overestimate the split tensile strength at early duration of testing, but at later testing duration this relationship closely fit the experimental value. While all others relationship shows the opposite trend i.e. closely related to the experimental value of split tensile strength at 7 days of testing. Beyond this testing duration, all relationships predict much lower value of split tensile strength than the experimental observed value for both controls RCC mixes. Thus, it can be concluded that the existing relationships which fitted well in conventional and high strength concrete cannot be used to accurately predict the split tensile strength of RCC based upon the compressive strength.

Thus, a regression analysis was performed to develop the relationship between the compressive strength and split tensile strength. The power relationship as presented in equation 4.13 yielded the high coefficient of determination (R^2) amongst the all other linear regression relationships tried for the experimental data. The power relationship presented in equation 4.13 is used commonly for both types of aggregates viz. crushed gravel and limestone at all replacement level of cement. The correlation equation is:

$$f_{st} = 0.05 f_c^{1.18} \quad R^2 = 0.86 \quad \text{.....(4.13)}$$

where, f_{st} = split tensile strength of RCC (N/mm²)

f_c = compressive strength of RCC (N/mm²)

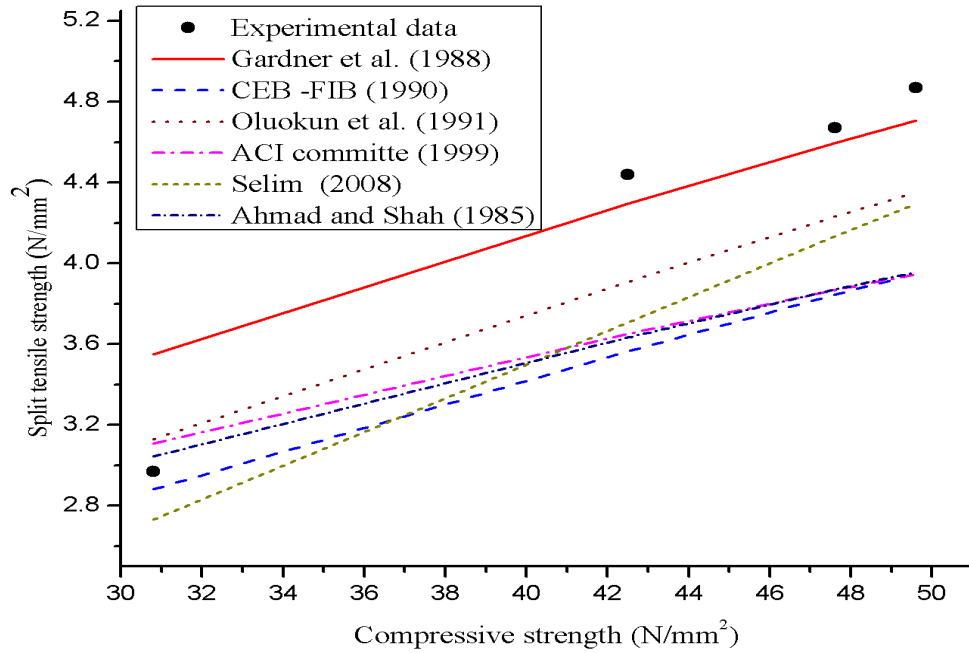


Fig. 4.24: Observed split tensile strength for control mix CG-0 and its comparison with existing relationship

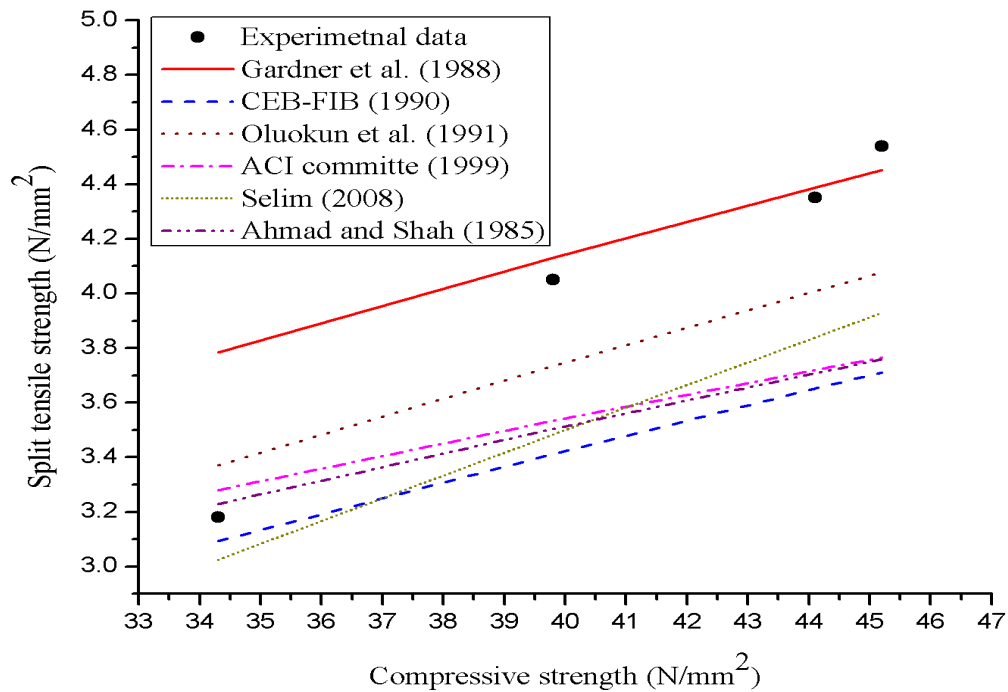


Fig. 4.25: Observed split tensile strength for control mix LS-0 and its comparison with existing relationship

Fig. 4.26 depicts the power relationship between compressive strength and split tensile strength at 95% confidence interval for RCC containing crushed gravel aggregates, while for limestone aggregates this relationship are presented in Fig. 4.27. It was observed from both figures that strong correlation exists between the split tensile strength and compressive strength of RCC in power format.

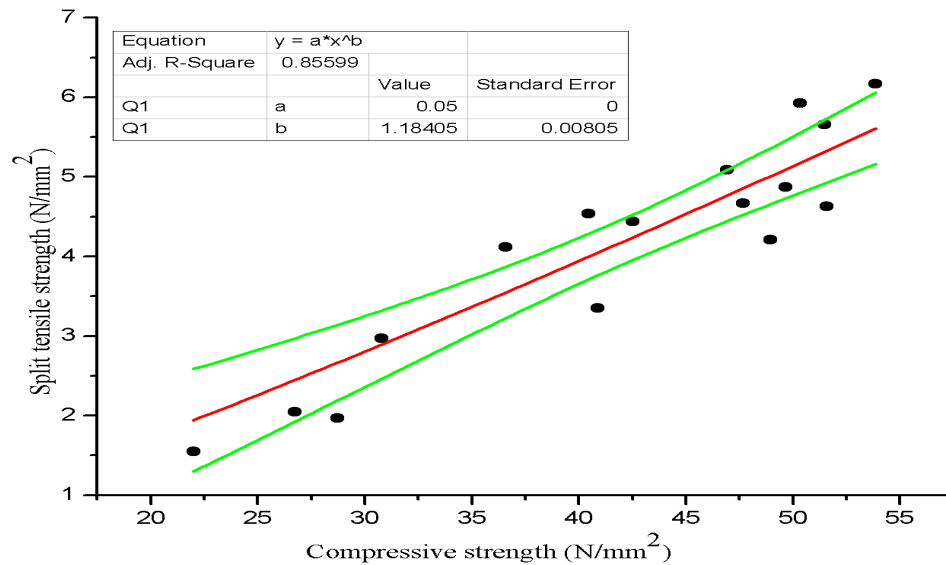


Fig. 4.26: Relationship between compressive and split tensile strength at 95% confidence interval for RCC containing crushed gravel

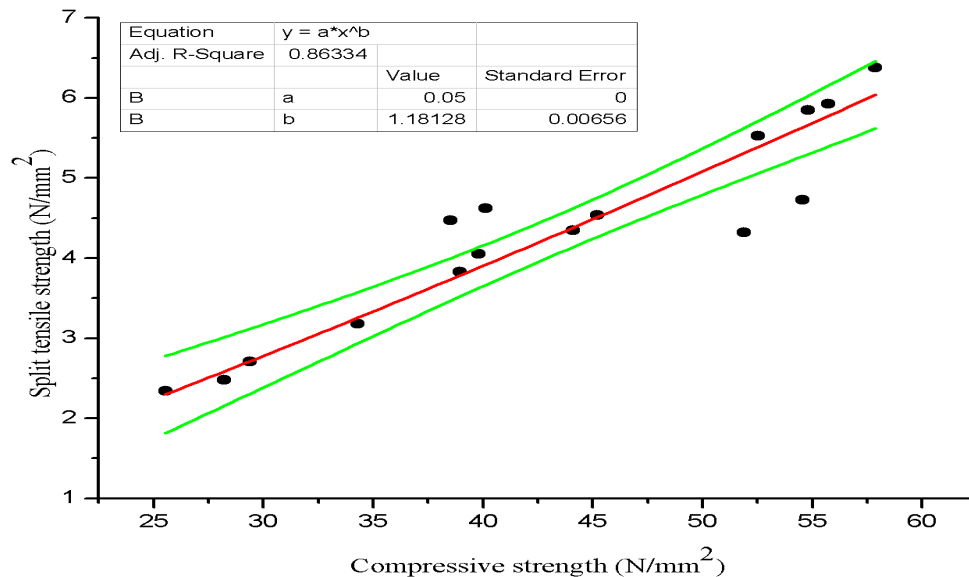


Fig. 4.27: Relationship between compressive and split tensile strength at 95% confidence interval for RCC containing limestone

4.7.6 Flexural Strength of RCC

Present study is oriented to the application of RCC as a pavement material. Since concrete pavements fail because of bending stresses, therefore flexural strength becomes an important parameter for the design of concrete pavements. According to IRC-58 (IRC, 2002), the 28 days flexural strength of concrete pavement should lie between 3.8 to 4.2 MPa. Therefore, the effect of coarse aggregate type and GGBS replacement levels was investigated on flexural strength behaviour of RCC and presented in the subsequent sections.

(a) Effect of coarse aggregate type

The effect of variation in coarse aggregate type on 7, 28, 90 and 365 days flexural strength is presented in Fig. 4.28. The figure indicates that with the variation in coarse aggregate type, marginal variation in flexural strength was registered. During the early curing duration, limestone aggregates presents marginally higher flexural strength as compared to crushed gravel aggregates; while at later durations, crushed gravel aggregates show marginally higher flexural strength. For instant, at the age of 7 days, the RCC containing limestone aggregates presents 3.1% higher flexural strength as compared to crushed gravel aggregates. On the other hand, at the testing age of 365 days, 4.4% decrease in flexural strength was observed in mixes containing limestone aggregates as compared to the mixes with crushed gravel aggregates.

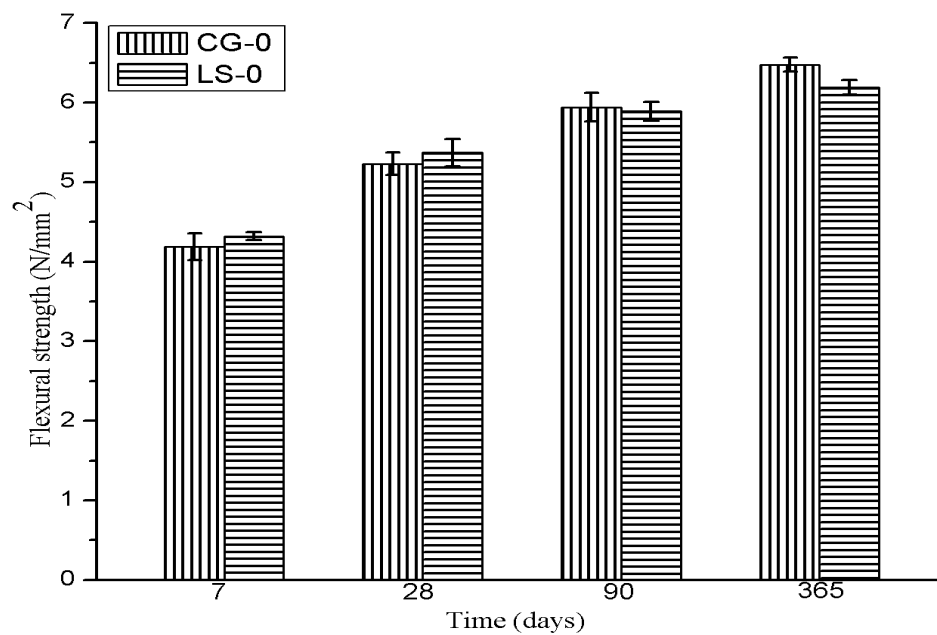


Fig.4.28: Effect of aggregate type on flexural strength of RCC

On making a comparison based upon the strength development from 7 days to 365 days, it was observed that mixes with crushed gravel aggregates have higher strength gain. For crushed gravel aggregates, 54.5% gain in flexural strength was observed, while in limestone aggregate this gain was 43.2%. Overall, it can be concluded that the variation in coarse aggregate type does not have any significant effect on the flexural strength behaviour of the RCC.

(b) Effect of GGBS content

The flexural strength of the mixes containing variable percentages of GGBS are presented in Figs. 4.29 and 4.30 corresponding to crushed gravel aggregates and limestone aggregates respectively. At early duration of testing, inclusion of GGBS into the mix decreases the flexural strength at each replacement level irrespective of the aggregate type. With the enhancement in curing period, improvement in flexural strength was registered with the maximum enhancement at 40% replacement level of cement with GGBS.

The flexural strength of RCC mixes containing crushed gravel aggregates; namely CG-20, CG-40 and CG-60 decreased by 28.8%, 31.86% and 43.43% as compared to the flexural strength of the corresponding control mix (CG-0) at 7 days of curing. At 365 days of testing, in RCC mixes containing CG-20 and CG-60 decrease in flexural strength observed was 6.9% and 13.3% respectively while in CG-40, 5.45% increase in strength was observed as compared to control mix. RCC made with limestone aggregates presents the behavior identical with the mixes incorporating crushed gravel aggregates upto 28 days of curing age; at all replacement levels of cement (Fig. 4.30). Beyond the age of 28 days, LS-20 and LS-60 mixes gives the flexural strength comparable with the control RCC mix (LS-0). However, large increase in flexural strength was registered with LS-40 mix. At 365 days of curing, 1.3%, 16.8% higher strength was shown by mix LS-20 and LS-40 respectively as compared to control mix.

Improvement in flexural strength observed in case of RCC containing limestone aggregates with GGBS incorporation can be attributed to the fineness of GGBS particles as well as physical properties of the aggregates. The porous irregular structure of limestone aggregates can make better bonding to the mortar which was further strengthen by GGBS incorporation. It was reported that the flexural strength depends upon the surface characteristics of coarse

aggregates, strength of the mortar and quality of the interfacial zone, which can be improved with the incorporation of finer supplementary materials (*Ozturen and Cecen, 1997*).

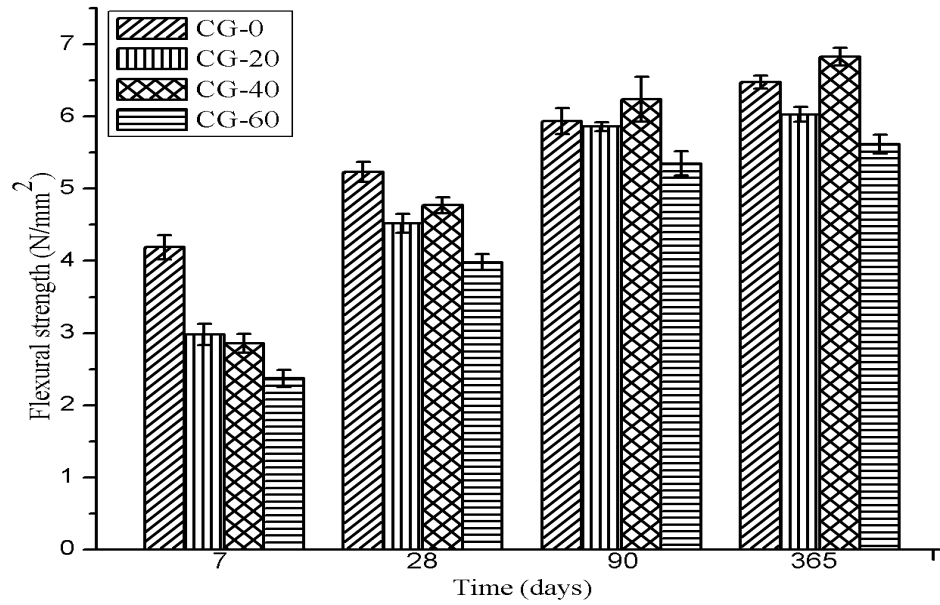


Fig. 4.29: Effect of GGBS content on flexural strength of RCC incorporating crushed gravel aggregates

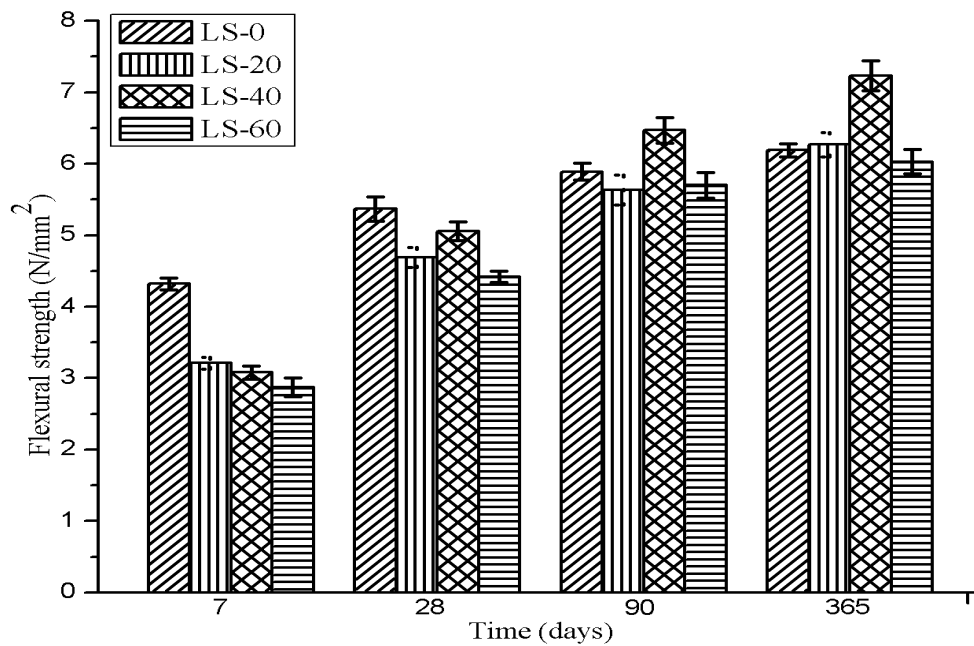


Fig. 4.30: Effect of GGBS content on flexural strength of RCC containing limestone aggregates

Further, the compressive and flexural strength gain from 7 days to 365 days was calculated, in order to get the better insight information with the effect of GGBS incorporation on the gain in flexural strength and is presented in Table 4.6. From the table it was also observed that for the control specimens the compressive and flexural strength gain differ by small variation. While for mixes containing GGBS, this difference in strength gain occur at larger variation especially for mixes CG-40 and LS-40. The observed behaviour indicates that the 40% replacement level of cement with GGBS gives the better gain in strength in terms of flexural strength.

Table 4.6: Compressive and flexural strength gain for all RCC mixes

RCC mixes incorporating	Nomenclature	Strength gain (%) from 7 days to 365 days	
		Compressive	Flexural
Crushed gravel aggregates	CG-0	61	54
	CG-20	79	102
	CG-40	101	139
	CG-60	128	137
Limestone aggregates	LS-0	31	43
	LS-20	85	95
	LS-40	105	134
	LS-60	118	110

4.7.7 Relationship between Flexural and Compressive Strength for RCC

The variation in flexural strength with the variation in compressive strength of RCC containing crushed gravel and limestone aggregates is shown in Figs. 4.31 and 4.32, respectively. As expected, both figures clearly show the increase in flexural strength with the increase in compressive strength of corresponding mix.

For design purpose, many investigators and codes have standardized the relationship between flexural strength and compressive strength (*Ahmed et al., 2016; Hesami et al., 2016; ACI 325-10R-95*). The equations recommended by various investigators are generally expressed

in the square root format as (Ahmed et al., 2008; Legeron and Paultre, 2000; ACI committee 363, 1997; Hesami et al., 2016):

$$f_r = b\sqrt{f_c} \quad \text{.....(4.14)}$$

where, f_r = flexural strength in N/mm^2

f_c = compressive strength in N/mm^2

b = constant generally varies between 0.33 to 1.2, depending upon the curing condition, testing age, strength levels and moisture content of the specimens etc.

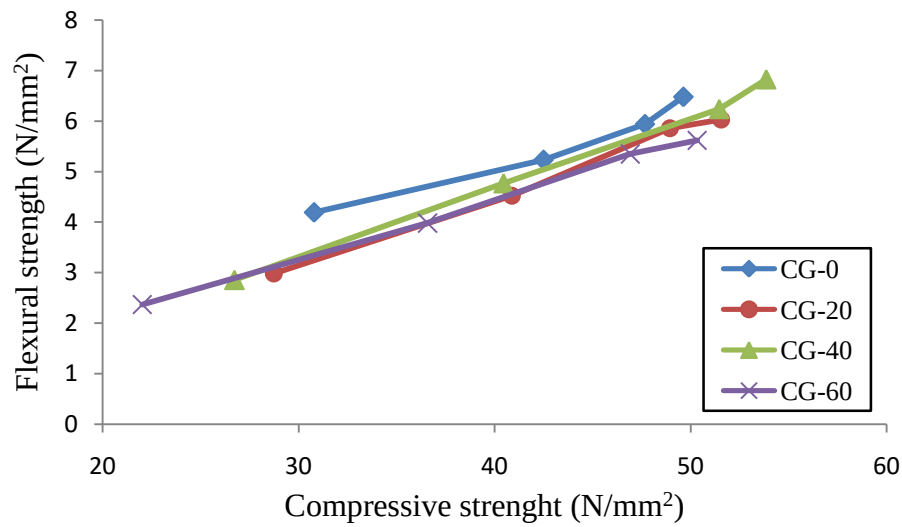


Fig.4.31: Flexural versus compressive strength of RCC containing crushed gravel aggregates

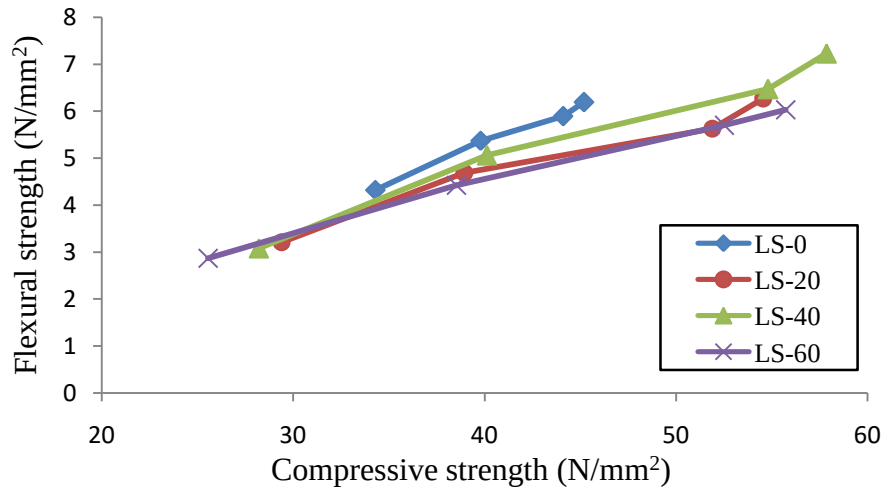


Fig.4.32: Flexural versus compressive strength of RCC containing limestone aggregates

Details of all these existing relationship between compressive and flexural strength are summarized in Table 4.7. Further, these relationships were employed on the experimental data of compressive strength and variation between predicted values of flexural strength from the experimentally observed value is also shown in Table 4.7. From Table 4.7, it was observed that lower limit of *Legeron and Paultre (2000)* and *Ahmed et al. (2008)* relationship as well as relationship proposed by *Selim (2008)* underestimate the flexural strength of RCC, while all others relationship overestimate the flexural strength value based upon the corresponding compressive strength values. The comparative analysis between the observed flexural strength values of both control RCC mixes, namely CG-0 and LS-0 with the relationship developed by various investigators in the graphical form are shown in Figs. 4.33 and 4.34 respectively.

Table 4.7: Details of existing relationship between compressive and flexural strength for conventional concrete

S.No.	Source	Relationship	Predicted values of $*f_r$ for CG-0 mix by various relationships			
			7d	28d	90d	365d
1.	ACI committee 363 (1997)	$f_r = 0.94 f_c^{0.5}$	5.22 **(24.5%)	6.12 (17.17%)	6.49 (9.2%)	6.63 (2.2%)
2.	Legeron and Paultre (2000)	$f_{r,min} = 0.68 f_c^{0.5}$	3.77 (-9.9%)	4.43 (-15.2%)	4.69 (-20.9%)	4.79 (-26.0%)
		$f_{r,max} = 1.2 f_c^{0.5}$	6.66 (58.9%)	7.82 (49.5%)	8.28 (39.4%)	8.45 (30.5%)
3.	Ahmed et al. (2008)	$f_r = 0.3 f_c^{0.5}$	1.66 (-60.2%)	1.96 (-62.6%)	2.07 (-65.1%)	2.11 (-67.3%)
		$f_r = 1.0 f_c^{0.5}$	5.55 (32.4%)	6.52 (24.6%)	6.90 (16.2%)	7.04 (8.7%)
4.	Hesami et al. (2016)	$f_r = 0.9 f_c^{0.5}$	4.99 (19.2%)	5.87 (12.1%)	6.21 (4.6%)	6.34 (-2.1%)
5.	Selim (2008)	$f_r = 0.034 f_c^{1.286}$	2.79 (-33.4%)	4.22 (-19.3%)	4.89 (-17.6%)	5.15 (-20.4%)
6.	Chhorn et al. (2018)	$f_r = 0.678 f_c^{0.605}$	5.39 (28.7%)	6.55 (25.3%)	7.02 (18.2%)	7.19 (11.1%)

$*f_r$: flexural strength

**variation of predicted strength from the experimental flexural strength

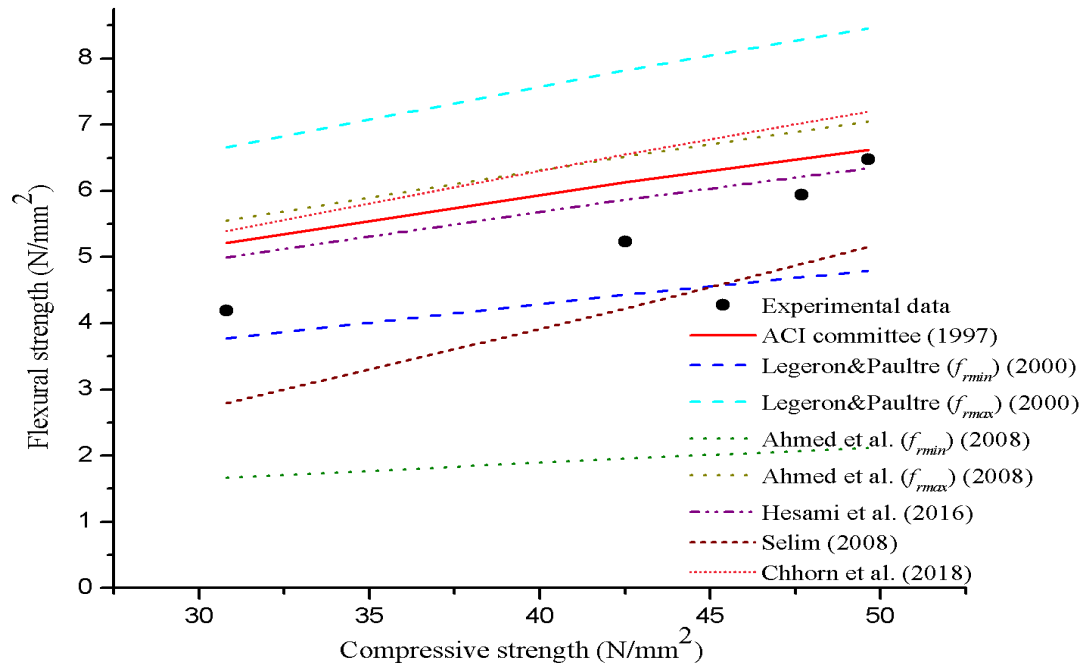


Fig. 4.33: Observed flexural strength for control mix CG-0 and its comparison with existing relationship

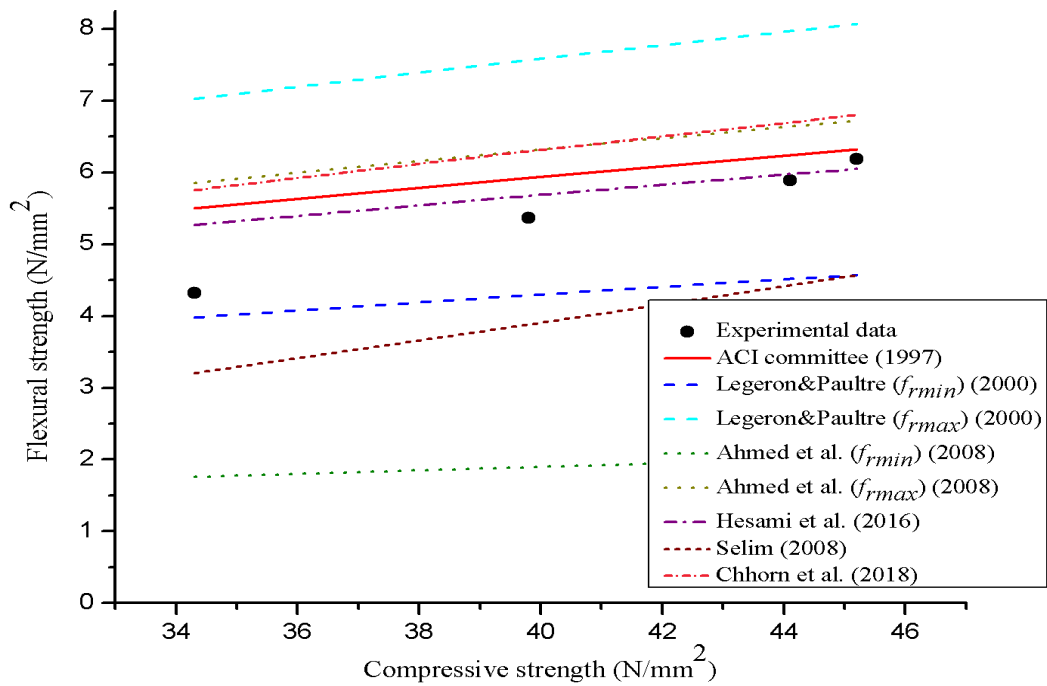


Fig. 4.34: Observed flexural strength for control mix LS-0 and its comparison with existing relationship

From both figures, it was observed that ACI committee relationship and relationship proposed by *Hesami et al. (2016)* fitted well with the experimentally observed flexural strength values at compressive strength higher than 40 N/mm² irrespective of type of coarse aggregate. At strength lower than 40 N/mm² the lower limit of relationship proposed by *Legeron and Paultre (2000)* predicted the strength comparable with the experimentally observed value. While rest of all others relationships highly under or over estimate the flexural strength of RCC irrespective of aggregate type. From the above discussion it was concluded that none of the existing relationship efficiently predicted the flexural strength of RCC. Thus, keeping in mind the various relationships proposed by the previous researchers and based upon the experimental data regression analysis was performed. The developed correlation equation is as under:

$$f_r = 0.144 f_c^{0.95} \quad \text{.....(4.15)}$$

where, f_r = flexural strength of RCC (N/mm²)

f_c = compressive strength of RCC(N/mm²)

Figs. 4.35 and 4.36 show the power relationship between the compressive strength and flexural strength at 95% confidence interval for crushed gravel and limestone aggregates respectively. Observed higher correlation coefficient (R^2) 0.93 for crushed gravel and 0.88 for limestone aggregates proves that developed relationship can be used to predict flexural strength of RCC for specified compressive strength with acceptable accuracy.

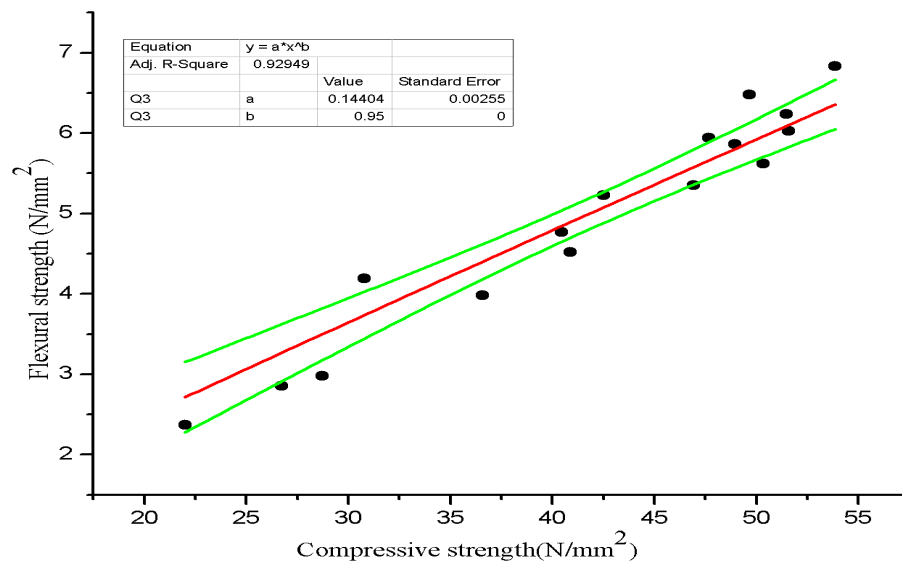


Fig. 4.35: Relationship between compressive and flexural strength at 95% confidence interval for RCC containing crushed gravel aggregates

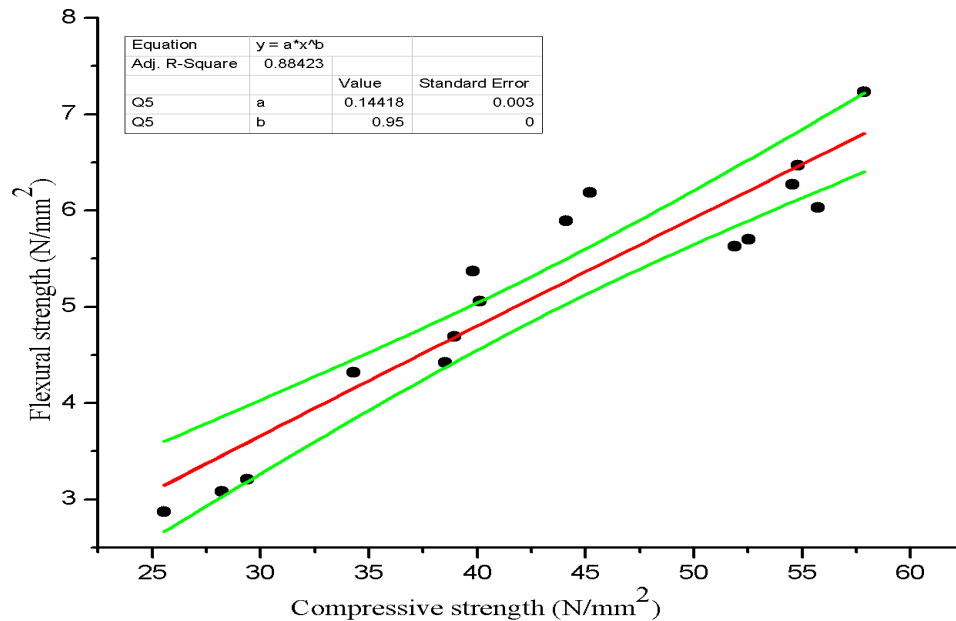


Fig. 4.36: Relationship between compressive and flexural strength at 95% confidence interval for RCC containing limestone aggregates

4.8 CLOSING REMARKS

The present chapter discusses the mechanical properties viz. compressive strength, split tensile strength and flexural strength of RCC incorporating GGBS at various replacement level of cement, containing either crushed gavel aggregates or limestone aggregates. Compressive strength result shows that limestone aggregates gives higher strength at early curing age followed by reversal in strength. Incorporation of GGBS into the mix altered the compressive strength behaviour irrespective of the type of aggregates. After the activation of GGBS and pozzolanic activity (i.e. beyond 28 days), 20% and 40% replacement level of cement with GGBS gives compressive strength higher than the control mix with the highest value was attributed to the 40% GGBS incorporation level. The stoichiometric analysis also showed that the upper limit of substitution of cement by GGBS can be taken as 49.4% i.e. GGBS doesn't contribute towards strength development by chemical reactions beyond 50% replacement level, after which it will simply act as filler. The SEM analysis also support that the addition of GGBS influence the interfacial transition zone present in the aggregate and hardened cement paste.

Split tensile strength results shows that variation in aggregate type does not have any significant effect on tensile strength. It is due to the fact that the tensile strength of concrete is influenced by the quality of paste and interfacial transition zone in the concrete rather than the type of aggregate. Incorporation of GGBS into the mix as replacement of cement led to improvement in split tensile strength of RCC after 28 days of curing. Similar to the split tensile, variation in aggregate type does not have any significant effect on the flexural strength behaviour. Similar to the compressive strength behavior, the 40% incorporation level of GGBS gives the highest flexural strength amongst all other mixes irrespective of aggregate type.

On the basis of statistical analysis, it was observed that the well existing conventional concrete model for predication of split tensile and flexural strength from compressive strength are not suitable for predicting strength of RCC mixes. Therefore, a relationship in the power format for both aggregates and GGBS incorporation are proposed for the estimation of split tensile and flexural strength from the corresponding compressive strength. The values predicted for split tensile and flexural strength, based upon the proposed relationships are in good agreement with the experimental value at 95% prediction band.

CHAPTER 5

IN-SERVICE PERFORMANCE OF ROLLER COMPACTED CONCRETE

5.1 GENERAL

The pavement surfaces are designed to meet various durability requirements throughout their service life. Since the major application of RCC corresponds to pavements; abrasion resistance, drying shrinkage and thermal shrinkage of RCC becomes an essential parameter for its acceptability. The effect of supplementary cementitious materials like; fly ash, silica fume and slag on the abrasion resistance behaviour of conventional concrete have been studied by *Naik et al. (1995)*; *Siddique et al. (2012)*; *Rashad et al. (2014)*; *Siddique and Khatib (2010)*. Beside the supplementary cementitious materials, various scholars investigated the effect of coarse aggregates characteristics on abrasion resistance of conventional concrete (*Beixing et al., 2011*; *Kumar and Sharma, 2014*; *Kilic et al., 2008*; *Al-Attar, 2008*; *Zhang et al., 2013*; *Fujiwara, 2008*; *Kwan et al., 2010*). The abrasion resistance behaviour of RCC has not been extensively investigated.

Drying shrinkage is defined as the volumetric shrinkage of concrete, because of loss of moisture during drying with time (*Mokarem et al., 2005*). This volumetric change introduces tensile stresses in concrete, ultimately leading to cracking of RCC pavements. Shrinkage cracks adversely affect the pavement surface with decrease in durability and increase in roughness of pavements. *Pittman and Ragan (1998)* reported that the grading of aggregates and moisture content of the mix has great influence on drying shrinkage of RCC. It was reported that characteristics of coarse aggregates such as quantity, type, surface texture, moisture content as well as supplementary cementitious materials play an important role in controlling shrinkage behaviour of conventional concrete (*Mokarem et al., 2005*; *Yuan et al., 2015* ; *Al-Attar, 2008*; *Zhang et al., 2013*; *Lee et al., 2006*; *Shariq et al., 2016*; *Fujiwara, 2008*; *Kwan et al., 2010*; *Jianyong and Yan, 2001*).

The variations in environmental temperature affects the temperature related properties of concrete, viz. rate of hydration, thermal stress, drying shrinkage cracking, permeability; that further decreases long-term strength and durability (*Bhal and Jain, 1999*; *Schindler and*

McCullough, 2002). Numerous researchers investigated the effect of ambient and high temperature on mechanical properties of concrete (Kim *et al.*, 2014; Chakrabarti and Jain, 1999; Bhal and Jain, 1999; Barnett *et al.*, 2006; Malhotra, 1956; Tanyildizi and Coskun, 2008; Lin *et al.*, 2015). Yang *et al.* (2014) investigated the temperature effect on drying shrinkage behaviour of conventional concrete up to 28 days of drying period. However, the effect of temperature on the long term shrinkage behaviour of RCC is not investigated.

From the referred literature, it was observed that while abrasion resistance, drying shrinkage and temperature effect on mechanical properties of conventional concrete is extensively studied, the long term performance of RCC has not been fully investigated. Also it is well established that the properties of RCC are different than the conventional concrete, because of difference mix proportioning and placement procedure. The present chapter deals with the study on abrasion resistance, drying shrinkage and thermal shrinkage behaviour of RCC. The experimental procedures with the corresponding results are described in the succeeding sections.

5.2 TEST MATRIX AND MATERIALS

The test matrix describing the effect of GGBS as supplementary cementitious material and variation in coarse aggregate type on the abrasion resistance, drying shrinkage and temperature effect on shrinkage behaviour of RCC is shown in Fig. 5.1. The detailed description of all mix constituents has already been described in Chapter 3.

5.3 CASTING AND CURING OF SPECIMENS

Two types of test specimens were prepared, for measuring abrasion resistance and shrinkage behaviour of RCC mixes. Concrete blocks of size 65 x 65 x 30 mm were prepared for measuring abrasion resistance, while concrete prisms of size 75×75×290 mm were cast for the measurement of unrestrained shrinkage strain and studying temperature effect on shrinkage behaviour of RCC. The test specimens were cast with the mix proportions provided in Table 3.17 in Chapter 3. For preparing the RCC mix, all mix constituents were taken in required proportions and mixed thoroughly for about 2 minutes to get a uniform mix in the dry state. After this, water was added to the dry mix and mixed in a rotating mixture keeping 5 minute mixing time.

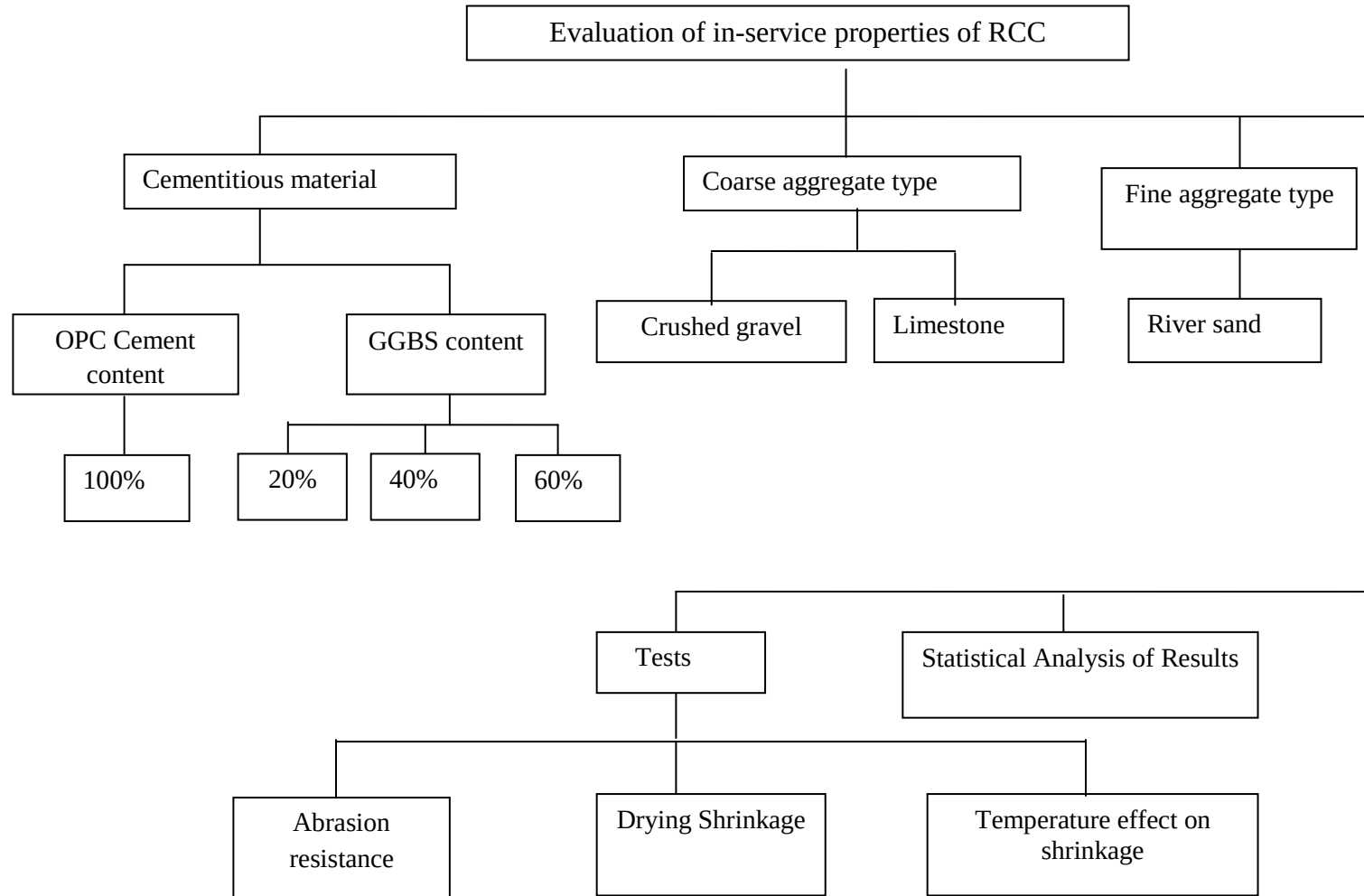


Fig.5.1: Test matrix to evaluate in-service properties of RCC

The RCC mix so obtained was filled into the moulds in three layers and compacted by putting surcharge of 4.9 kPa on the top of each layer, as specified by *ASTM C 1176 (2008)*. The required surcharge was obtained by using steel plates of 2.1 kg and 3 kg weight for abrasion resistance blocks and shrinkage prisms respectively. To achieve complete compaction, a vibrating table with a vibration frequency of 60 Hz specified in *ASTM C 1170 (2008)* was used. All the specimens were consolidated till a ring between the inside face of the mould and the surcharge was visible. The formation of this ring indicates that the RCC has reached its maximum density (*ASTM C 1176, 2008*). The prepared test specimens are shown in Fig. 5.2. After 24h of casting, all the specimens were de-moulded and kept in curing tank for the required period. The tank was maintained at temperature of $25 \pm 2^\circ\text{C}$ and 100% relative humidity.



Fig. 5.2: RCC blocks for abrasion resistance measurement and prisms for shrinkage properties

5.4 TESTING PROCEDURE

The major in-service properties investigated in the present study are abrasion resistance, drying shrinkage and temperature effect on the drying shrinkage of RCC. The detail of each property is discussed hereunder.

5.4.1 Abrasion Resistance

Abrasion resistance was measured after 3, 7, 28, 90 and 365 days of curing as per IS:1237-1980 (*BIS, 1980*). A total of 120 test specimens having size 65 x 65 x 30 mm were prepared for abrasion resistance measurements. The dimensions of the abrasion resistance

specimens were considered according to IS: 1237-1980 (BIS, 1980) with minor modification. The cross section under abrasion was 65 x 65 mm, which was same as adopted by the previous investigators (Siddique, 2013; Siddique and Khatib, 2010). The depth of specimens was kept in accordance with the requirements of holding device of the machine.

The set up used for conducting abrasion test is shown in Fig. 5.3. For conducting abrasion resistance test at a specified testing age, the specimens were taken out of the curing tank and were oven dried at $110 \pm 5^\circ\text{C}$ for 2h. The specimen was then weighed accurately on a digital balance, having least count 0.01 gram. Thickness of oven dried specimens was measured with the help of vernier caliper having least count of 0.02 mm at five locations (one at the center and at four corners). The test specimen was then fixed in the holding device of the abrasion testing machine and loaded at the centre with a load of 300N. Thereafter, the specimen was grind over the grinding machine rotating at a speed of 22 revolutions per minute. The grinding path of the disc of the abrasion testing machine was evenly distributed with 20 gram of abrasive (aluminum) powder. The abrasive powder was continuously spread on the grinding disc so that it remained uniformly distributed on the track corresponding to width of the test specimens (Siddique *et al.*, 2012; Siddique and Khatib 2010). Weight of the specimens were measured after each 22 revolutions and specimens was turned about the vertical axis through an angle of 90° in clockwise direction until the end of test (220 revolution). The extent of abrasion was determined from the difference in values of thickness measured before and after the abrasion test, by using the following formula:

$$T = \frac{(w_1 - w_2)t}{w_1} \quad \dots\dots(5.1)$$

where, T = average loss in thickness (mm)

w_1 = initial mass of specimen (gram)

w_2 = final mass of abraded specimen (gram)

t = the initial average thickness of the specimens (mm).

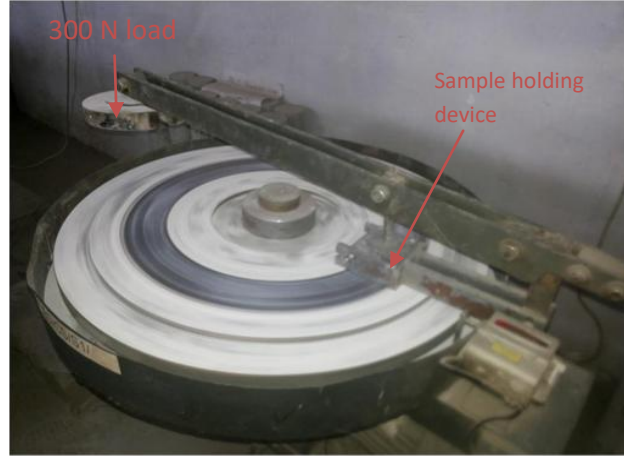


Fig.5.3: Experimental setup for measurement of abrasion resistance of concrete

5.4.2 Drying Shrinkage

Drying shrinkage strain of each mix was measured according to the *ASTM C 157* (*ASTM, 2003*) guidelines. With the three replicates for each mix, a total of 24 prism specimens of size 75×75×290 mm were cast for the measurement of drying shrinkage strain. For measuring drying shrinkage, prism specimens were demoulded after 24 hours of casting and initial comparator readings, known as “initial CRD” for shrinkage was taken with the help of length comparator mounted with the dial gauge, as shown in Fig. 5.4. The least count of the dial gauge was 0.01 mm. A steel rod having low thermal expansion with 290 mm length is used as a reference bar against which the reading of the gauge was taken during the entire process. The reference rod as well as the specimens was marked so that the same end was kept upward during measurement. After taking the initial comparator reading, the specimens were moist cured for 28 days followed by air curing in the controlled environment having 27±2°C temperature and 60-65% relative humidity. During air curing period, a clearance of 25 mm was maintained on all sides between the specimens. Subsequently, the comparator readings was taken at 4, 7, 10, 14, 21 and 28 days of air curing for initial one month and thereafter, as shrinkage slowed down, the readings were taken at 15 days interval for a drying period of 9 months. Drying shrinkage strain was calculated according to equations 5.2 and 5.3.

$$\Delta L_x = \frac{CRD - initialCRD}{G} \quad \dots 5.2$$

$$\varepsilon_{sh} = \frac{\Delta L_x}{L} \quad \dots\dots 5.3$$

where, ε_{sh} = shrinkage strain (μ mm/mm)

ΔL_x = length change of the specimen at any age

L = original length of the specimen (290 mm)

CRD = difference between the comparator reading of the specimen and the reference bar at each testing age

G = gauge length

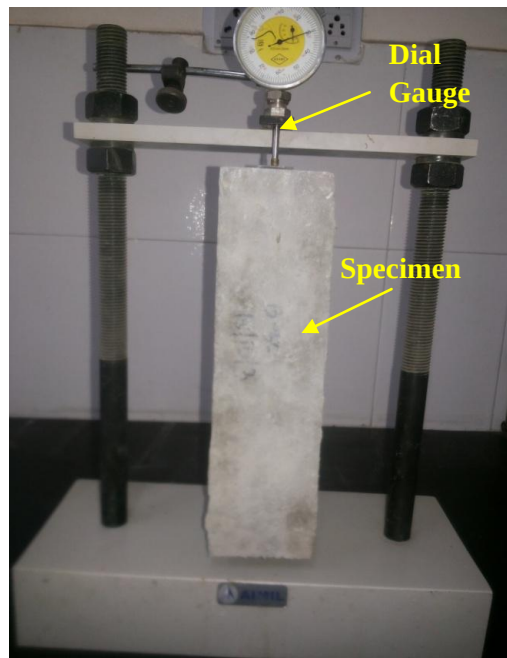


Fig. 5.4: Unrestrained drying shrinkage test setup

5.4.3 Temperature Effect on Shrinkage Strain (Thermal Shrinkage Strain)

The prism specimens of size 75×75×290 mm were used to measure the temperature effects on the shrinkage strain of RCC made with two types of aggregate and various percentage levels of GGBS. The apparatus used for thermal shrinkage strain measurement was kept similar to the ones used for drying shrinkage measurement. The effect of maximum ambient temperature i.e. $70 \pm 2^\circ\text{C}$ on the shrinkage behaviour formally known as thermal shrinkage strain was studied under two categories. Under the first category, specimens were subjected to a constant temperature of $70 \pm 2^\circ\text{C}$ upto a period of 270 days; while in second category, temperature cycles

were applied on the specimens for a period of one month. The test procedure for both the categories is explained in the following sections:

(1) First category: Maximum ambient temperature

The main objective of this category is to study the effect of maximum ambient temperature on the shrinkage behaviour of RCC pavements. The maximum ambient temperature that a concrete structure is subjected to is termed as sol - air temperature, which is composed of atmospheric temperature and the component of solar radiations (*Khandelwal A., 2009*). In Indian climatic conditions, maximum temperature recorded in last decade was approximately 48°C-51°C during the hot dry climatic conditions. Thus, by considering the effect of actual temperature of surroundings and the component due to solar radiations, the prism specimens were subjected to a constant temperature of $70 \pm 2^\circ\text{C}$ with relative humidity of 60-70% in a temperature control chamber after the completion of curing period of 28 days. The thermal shrinkage strain measurements were taken on the same intervals as for drying shrinkage. Prior to each reading, the specimens were taken out of the chamber and kept in air for a period of 2h. Total 24 prism specimens were cast and tested under this category.

(2) Second category: Heating and cooling cycles

The main objective of this category is to study the effect of thermal cycles (heating and cooling cycle) on the shrinkage behaviour of RCC pavements. Under this category, additional 24 prism specimens of similar size 75×75×290 mm were prepared and were water cured for 7 days. After the completion of 7 days of water curing, the prism specimens were subjected to heating and cooling cycles. For this, the specimens were put in the temperature control chamber and a temperature cycle of 12 hour heating at $70 \pm 2^\circ\text{C}$ followed by a cooling at $10 \pm 2^\circ\text{C}$ for a period of 12 hours was applied for a period of one month. Thus one cycle consisted of total 24 hours and readings were taken after every third cycle of heating and cooling.

The temperature control chamber used in the present study is shown in Fig. 5.5. The prism specimens were placed inside the chamber by maintaining an approximate gap of 10 mm between the specimens such that each specimen is subjected to uniform temperature.



Fig. 5.5: Temperature control chamber with prism specimens

5.5 RESULTS AND DISCUSSIONS

The variation in abrasion resistance, drying and thermal shrinkage with the variation in coarse aggregate type and GGBS percentage is described in the following sections.

5.5.1 Abrasion Resistance

The abrasion resistance of all RCC mixes made with two types of aggregates and having GGBS as partial replacement of cement was measured at 3, 7, 28, 90 and 365 days of casting. The effect of each parameter on abrasion resistance behaviour of RCC is discussed in the following sub-sections.

(a) Effect of coarse aggregate type and curing age

The abrasion resistance was measured in terms of depth of wear (*Ghafoori and Sukandar, 1995; Siddique et al., 2012; Nanni 1989; Naik et al., 1995; Siddique and Khatib, 2010*). As stated earlier also, depth of wear was measured in 10 sets of 22 revolutions, until the end of total 220 revolutions. From the measured depth of wear generally two parameters are noticed; firstly, the progressive variation in depth of wear with each set and secondly, the variation in depth of wear at end of 10 sets i.e. 220 revolution with curing age. The final depth of wear after 220 revolutions for the prepared RCC mixes containing crushed gravel and limestone aggregates is presented in Fig. 5.6. The progressive variation in depth of wear with each set of 22 revolutions at 7, 28, 90 and 365 days of curing is presented in Fig. 5.7 for both the mixes.

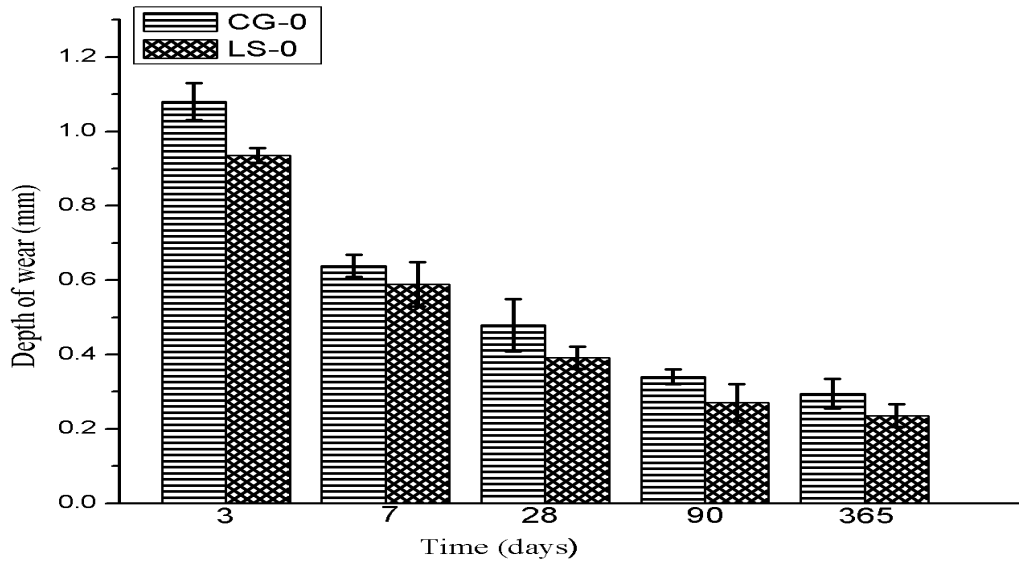


Fig. 5.6: Effect of aggregate type on final depth of wear

From Fig. 5.6, it was observed that curing age plays an important role in abrasion resistance behaviour of RCC, irrespective of aggregate type. It is evident from the figure that depth of wear decreases with the enhancement in curing period. At 7 days of curing, 41% and 37% lower depth in wear was observed on comparison with 3 days wear, for CG-0 and LS-0 mix respectively. Enhancement in curing period to 365 days reduced the depth of wear to approximately 73-75% as compared to 3 days curing for both types of aggregates. Improvement in abrasion resistance with age can be primarily attributed to the increased hydration of concrete, thereby improving the properties of concrete. The decrease in depth of wear with the increase in curing age is also reported by various researchers (*Siddique and Khatib, 2010; Siddique, 2013; Kumar and Sharma, 2014*).

On making a comparison based upon the aggregate type, it can be observed that RCC mix made with limestone aggregates presents marginally better abrasion resistance as compared to crushed gravel aggregates, at all curing ages. At 365 days of testing, approximately 20% lower depth of wear was registered in RCC mix containing limestone aggregates as compared to the mixes incorporating crushed gravel aggregates. Enhancement in abrasion resistance of RCC made with limestone aggregates can be correlated with its surface characteristics. From Table 3.16, it is evident that optimum moisture content and maximum dry density of both CG-0 and LS-0 mixes are similar, however the water absorption of limestone aggregates is about 15—20%

higher than the crushed gravel aggregates (Table 3.5). Thus, limestone aggregates are more porous than crushed gravel aggregates, which lead to better mechanical interlocking of fine particles with the aggregates. It was also established that softer, porous and rougher texture aggregates result in a greater adhesion between the particles and the cement matrix (*Neville and Brooks, 2010; Beixing et al., 2011*). Therefore, it can be said that the porous nature of limestone aggregates is leading to better abrasion resistance of RCC incorporating these aggregates.

Fig. 5.7 depicts that depth of wear increases with increase in number of revolutions. Also, the slope of progressive depth of wear decreases with the increase in curing time. In general, during the initial curing ages, the depth of wear increased at a constant rate with the increase in number of revolutions. However, with the enhancement in curing period, the depth of wear occurred at higher rate during the initial number of revolutions. For instance, at 365 days of curing, with 22 revolutions 23% and 21% depth of wear was observed in CG-0 and LS-0 mix, respectively, and at 154 revolutions this percentage goes to 73% and 80%. After this, increase in number of revolution increased the depth of wear at decreased rate. Reason behind this behaviour is that, the softer layer of concrete surface erodes at faster rate; once the hardened layer is reached the wear occurred at a slower rate (*Kumar and Sharma, 2014*).

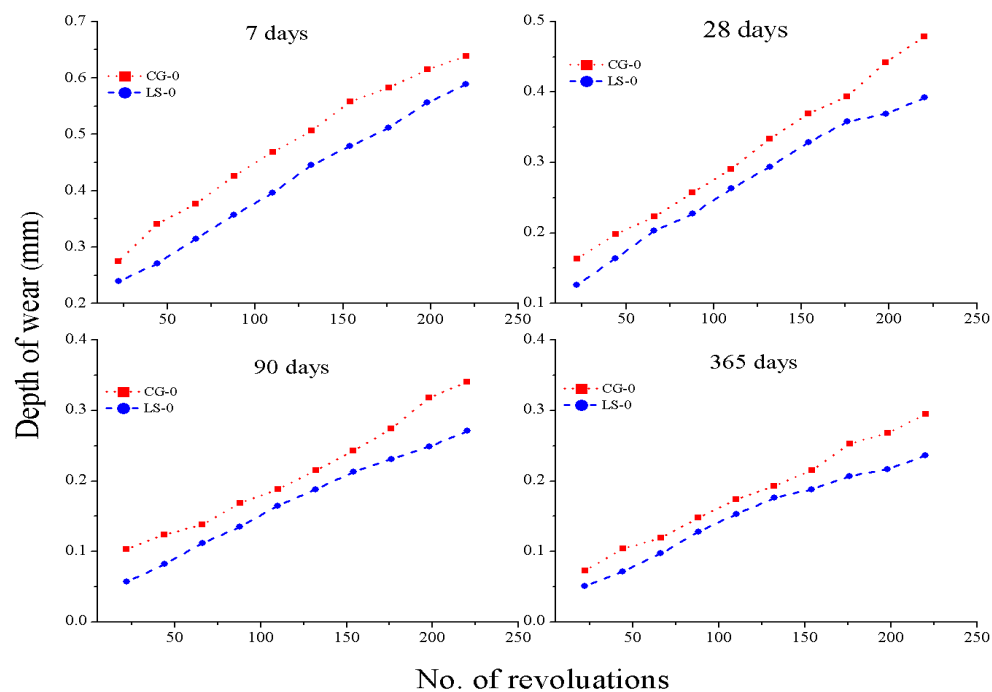


Fig. 5.7: Depth of wear versus number of revolutions at 7, 28, 90 and 365 days of curing for both aggregate type

(b) Effect of GGBS content

The effect of GGBS content on the final depth of wear obtained after 220 revolutions is presented in Figs. 5.8 and 5.9, for RCC mixes incorporating crushed gravel and limestone aggregates, respectively. Also, the progressive variation in depth of wear with number of revolutions with variation in GGBS content at 7, 28, 90 and 365 days of casting are presented in Figs. 5.10 and 5.11. As expected, there is a linear increase in the depth of wear with the increase in number of revolutions for all RCC mixes (Figs. 5.10 – 5.11). This increase in depth of wear shows that surface layer of RCC gets eroded by increasing the number of revolutions.

Figs. 5.8 and 5.9 depicts that at the initial curing period of 3 days, addition of GGBS into the control RCC mix had a negative impact on the abrasion resistance behaviour of RCC, irrespective of aggregate type. At early curing age of 3 days, in mixes CG-20, CG-40 and CG-60 increase in depth of wear was observed as 17.6%, 37.0% and 51.8% respectively as compared to control mix CG-0. Similarly, in mixes containing limestone aggregates abbreviated as LS-20, LS-40 and LS-60, increase in depth of wear was observed as 10.6%, 40.4% and 61.7% respectively as compared to control mix LS-0. The increase in depth of wear with the increase in GGBS content indicates poor abrasion resistance of mixes containing GGBS, since at early ages most of the GGBS remains inert and has not participated in C-S-H gel formation.

However, when the curing period is increased from 3 to 28 days, reversal in depth of wear was registered. After the curing period of 28 days, improvement in abrasion resistance (indicated by decrease in depth of wear) was observed at all replacement levels of cement with GGBS. Both Figs. 5.8 and 5.9 represents that at 365 days of curing, RCC containing 40% GGBS content shows the lowest value of depth of wear against all other corresponding mixes. Highest increase in abrasion resistance in CG-40 and LS-40 mix is analogous to the observed highest compressive strength in the respective mixes. At the age of 365 days, CG-40 mix shows maximum enhancement in abrasion resistance (33.9%) while in mixes CG-20 and CG-60 this improvement was 15.9% and 11.5% respectively as compared to the control mix. For RCC containing limestone aggregates, the maximum decrease in depth of wear was observed for mix LS-40 (33.1%) followed by LS-60 (21.6%) and LS-20 (13.1%), as compared to the corresponding control mix.

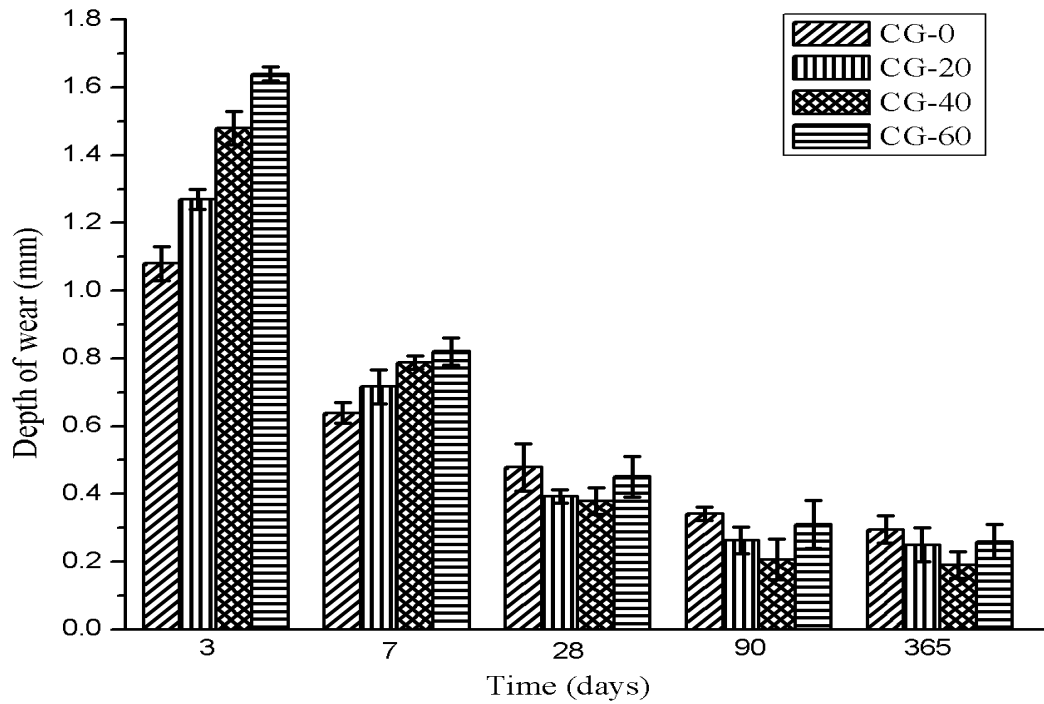


Fig. 5.8: Effect of GGBS content on final depth of wear of RCC incorporating crushed gravel aggregates

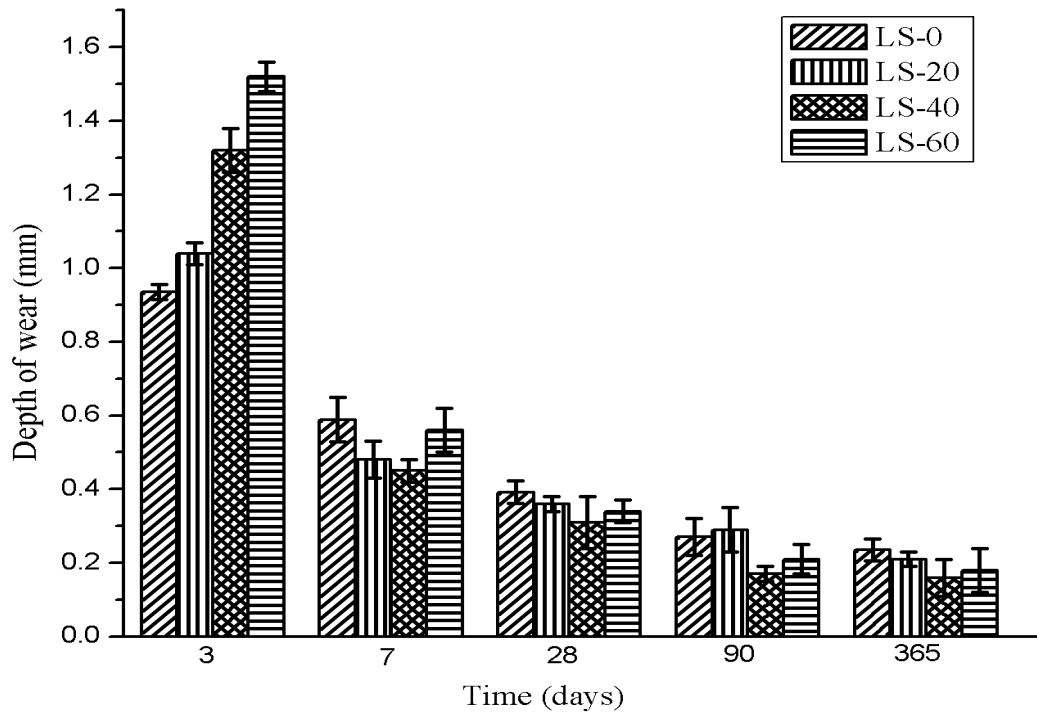


Fig. 5.9: Effect of GGBS content on final depth of wear of RCC incorporating limestone aggregates

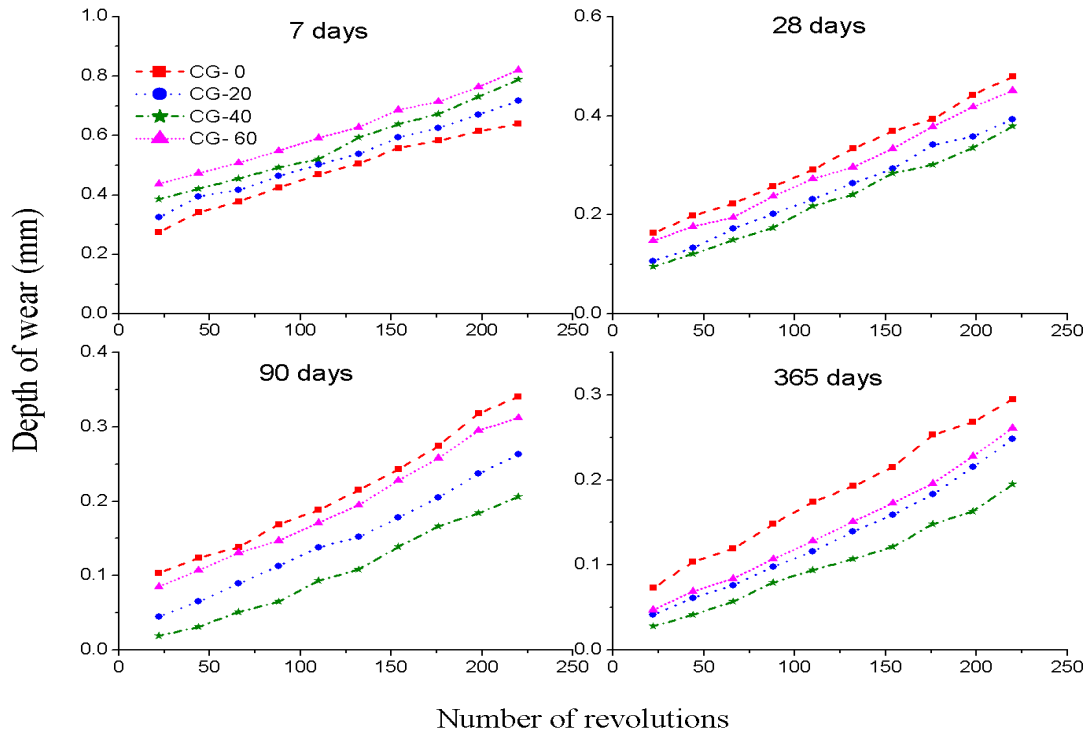


Fig. 5.10: Effect of GGBS content on depth of wear versus number of revolutions at 7, 28, 90 and 365 days for crushed gravel aggregates

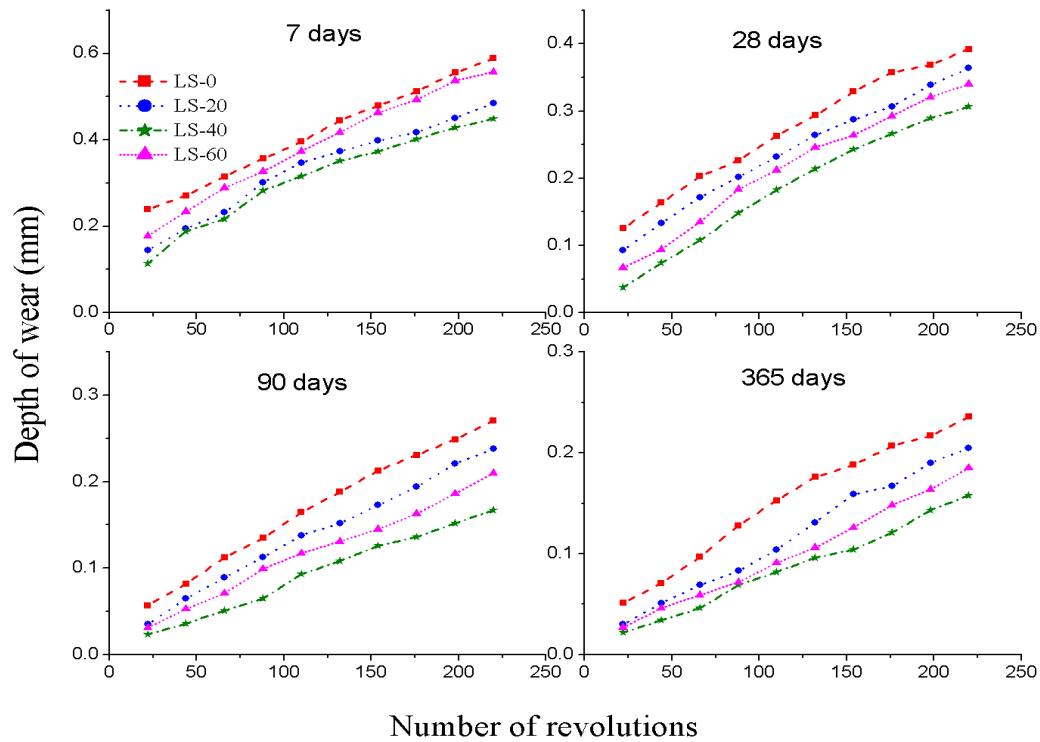


Fig. 5.11: Effect of GGBS content on depth of wear versus number of revolutions at 7, 28, 90 and 365 days for limestone aggregates

Observed better performance of mixes containing GGBS shows that the addition of GGBS in RCC mix improves the properties of paste and the transition zone between the aggregates and paste, which led to an improvement in abrasion resistance of RCC. Similar trend were also reported by *Wu et al. (1994)*, *Kumar and Sharma (2014)* and *Rao et al. (2016)*. *Wu et al. (1994)* reported that concrete containing 45% slag showed the best abrasion erosion resistance. *Kumar and Sharma (2014)* observed that GGBS based mixes performed better in abrasion resistance as compared to non-pozzolanic mixes. *Rao et al. (2016)* reported the improvement in abrasion resistance upto 50% replacement level of cement with GGBS, and thereafter with farther increase in replacement level, decrease in abrasion resistance was registered.

(c) Comparative effect of coarse aggregate type and GGBS content on compressive strength and abrasion resistance

Figs. 5.12-5.15 were prepared to compare the combined effect of aggregate type and GGBS content on compressive strength and abrasion resistance of RCC. From the figures it can be seen that GGBS content and type of aggregates play an important role in abrasion resistance and compressive strength development of RCC mixes.

Fig. 5.12 depicts that with the enhancement in curing age improvement in compressive strength and abrasion resistance was observed. In term of compressive strength behaviour, crushed gravel aggregates presents higher strength over limestone aggregates at all curing ages except at 7 days of curing. While, in terms of abrasion resistance, better performance was shown by limestone aggregates because of porous nature of limestone aggregates, which leads to better mechanical interlocking of fine particles with the aggregates. At 365 days of curing, mixes with crushed gravel (CG-0) exhibited 9% higher compressive strength as compared to mix LS-0. While abrasion resistance of limestone mix remained higher by 18-20% than the crushed gravel mix at all curing ages. *Beixing et al. (2011)* also reported that the abrasion resistance of concrete is not totally dependent upon the corresponding compressive strength. They also observed higher abrasion resistance for concrete containing quartzite aggregate while the compressive strength of this concrete was lower.

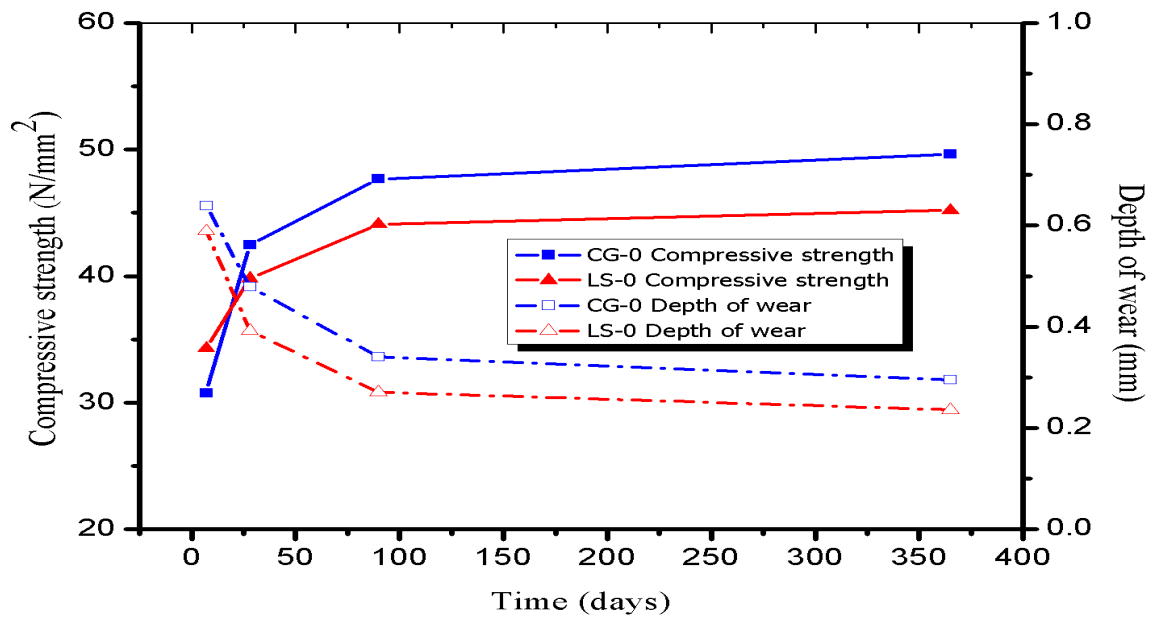


Fig. 5.12: Compressive strength and final depth of wear variation with time for control RCC mix

Figs. 5.13-5.15 show the combined action of aggregate type and GGBS content on compressive strength and abrasion resistance behaviour of RCC. As can be seen from all these figures, the addition of GGBS leads to better performance of mixes containing limestone aggregates as compared to the corresponding crushed gravel mixes. In general, RCC mix containing limestone aggregates shows higher compressive strength and abrasion resistance at all replacement levels of cement with GGBS as compared to the corresponding mixes incorporating crushed gravel aggregates. At curing age of 365 days, limestone RCC mixes with addition of 20%, 40% and 60% GGBS shows 5.7%, 7.5% and 10.7% higher compressive strength as compared to crushed gravel RCC mixes with similar content of GGBS. The increase in abrasion resistance for limestone RCC mixes was 17.3%, 18.8% and 29.1% corresponding to 20%, 40% and 60% GGBS addition respectively; as compared to counterpart RCC mix containing crushed gravel aggregates. The better performance of RCC mixes made with limestone aggregates and GGBS is due to combined action of porous structure of limestone aggregates which lead to better mechanical interlocking of fine particles with the aggregates and the fineness of GGBS which help in getting the refined concrete matrix. Both these effects lead to an improved mix and the performance improves further with the curing age. Improvement in compressive strength and abrasion resistance in RCC containing GGBS and limestone aggregates leads to two folded

benefit. On the one hand, utilization of industrial by-product is achieved by incorporating GGBS in higher replacement levels. On the other hand, an alternate to traditional coarse aggregates is explored.

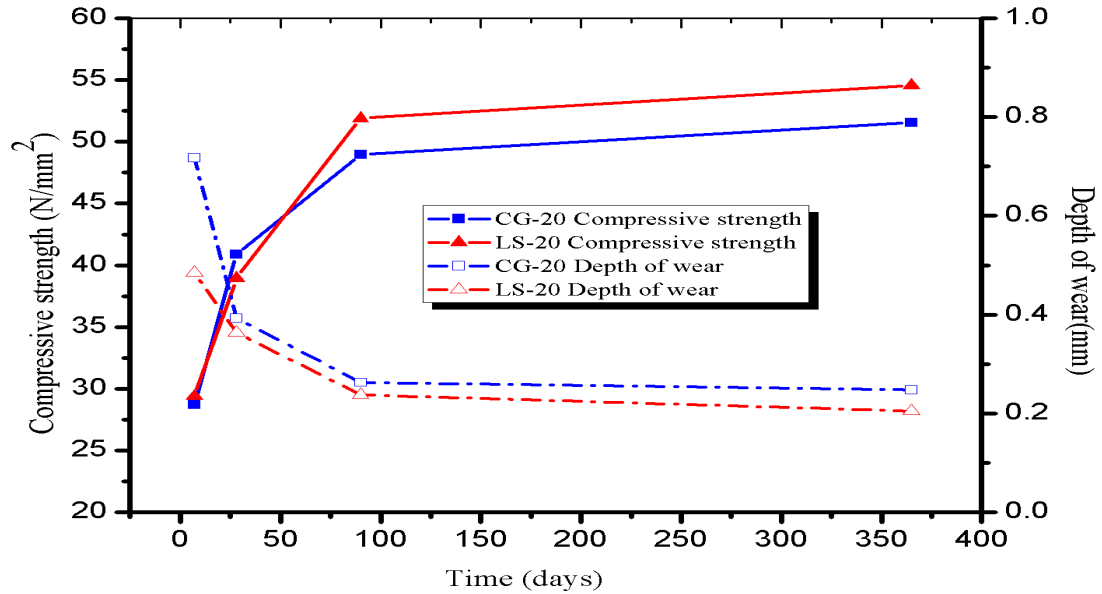


Fig. 5.13: Compressive strength and final depth of wear variation with time for RCC containing 20% GGBS

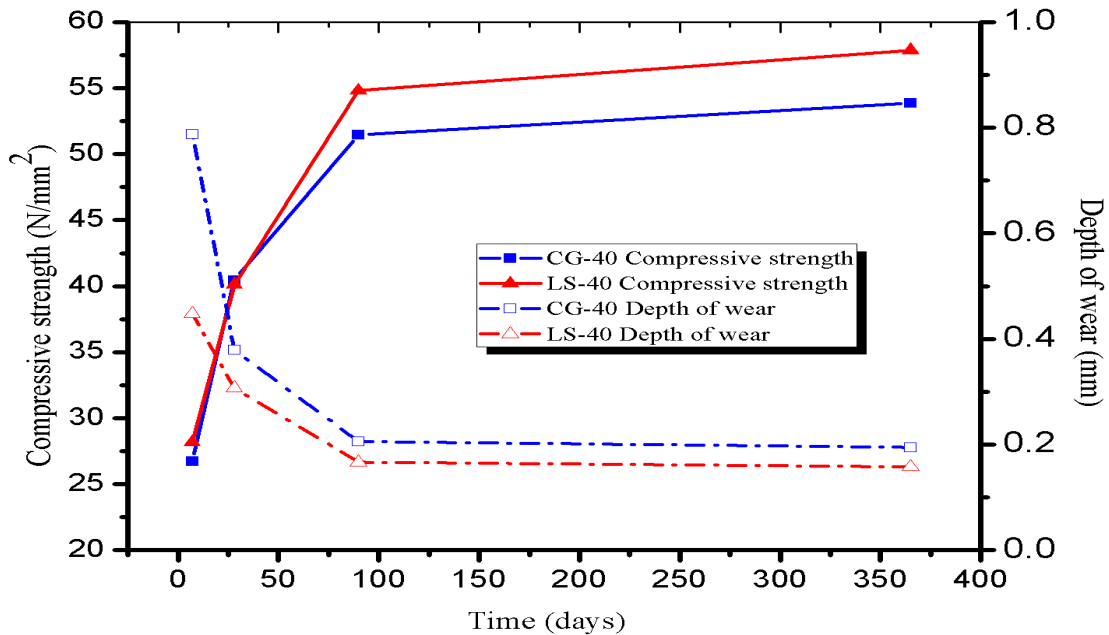


Fig. 5.14: Compressive strength and final depth of wear variation with time for RCC containing 40% GGBS

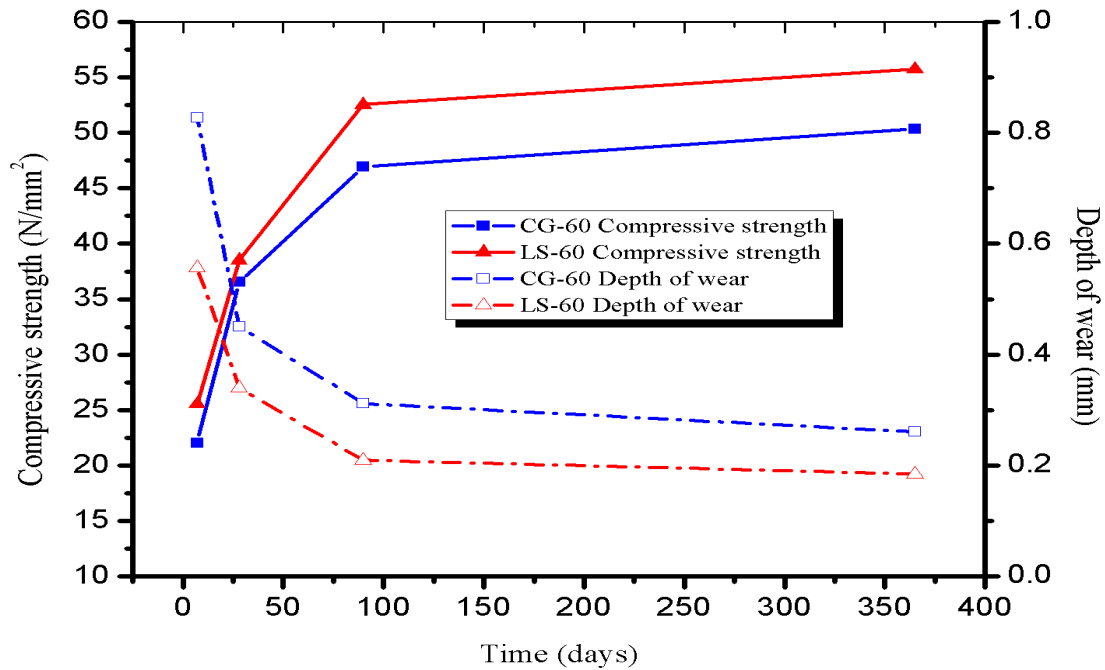


Fig. 5.15: Compressive strength and depth of wear variation with time for RCC containing 60% GGBS

5.5.2 Statistical Analysis of Abrasion Resistance Results

It is well established that compressive strength is the most important factor governing other properties, including abrasion resistance of concrete (*Rashad et al., 2014*). Various regression models correlating the compressive strength with abrasion resistance have been proposed by different researchers. Some of the researchers suggested polynomial relationship between these two properties (*Ghafoori and Sukandar, 1995; Horszczaruk E, 2005; Rashad et al., 2014*). Some authors suggested a relationship between compressive strength and abrasion resistance in power format (*Naik et al., 1994; Konin and Kouadio, 2012; Siddique et al., 2012; Siddique and Khatib, 2010*), while *Topcu and Uygunoglu (2010)* suggested this relation in exponential format. The various relationships suggested by previous researchers for prediction of abrasion resistance based upon compressive strength results are summarized in Table 5.1.

Table 5.1: Details of existing relationship between compressive strength and abrasion resistance for conventional concrete

S.No.	Source	Relationship	Applicability
1.	Ghafoori and Sukandar (1995)	$d_w = 699 - 11670949f_c + 671703159f_c^2$	Conventional concrete with cement content ranged between 200-600 kg/m ³
2.	Rashad et al. (2014)	$d_w = 0.0002f_c^2 - 0.0223f_c + 1.5159$	High volume fly ash concrete
3.	Horszczaruk E. (2005)	$d_w = af_c^2 - bf_c + c$ <i>a, b, c</i> are constant depends upon the testing duration with general range of <i>a</i> : 0.004-0.007 <i>b</i> : 0.843 -1.51 <i>c</i> : 45.126-81.649	High strength concrete
4.	Topcu and Uygunoglu (2010)	$d_w = 60.47e^{-0.0311f_c}$	Self-consolidating light weight concrete
5.	Siddique and Khatib (2010)	$d_w = 40.202f_c^{-0.8201}$	High volume fly ash concrete
6.	Siddique et al. (2012)	$d_w = 9.0256f_c^{-0.5545}$	High volume fly ash concrete
7.	Konin and Kouadio (2012)	$d_w = 27.167f_c^{-0.817}$	Conventional concrete containing natural aggregates
8.	Naik et al. (1994)	$d_w = Cf_c^{-17}$ <i>C</i> ranged between 71-114 corresponds to compressive strength of 30 MPa to 70MPa	Conventional concrete with fly ash inclusion
9.	Atis et al. (2009)	$d_w = -1.08f_c + 2.75$	Conventional concrete containing fly ash as cementitious material

d_w : depth of wear, f_c : compressive strength

From the table, it can be observed that most of the previous relationships were developed for conventional concrete containing fly ash as supplementary cementitious material. Also, the effect of type of aggregates was not included in the existing relationships. Apart from

compressive strength other parameters like type of aggregates play a significant role in defining abrasion resistance of concrete (Beixing et al., 2011). Therefore, an attempt has been made to develop a regression equation correlating compressive strength and depth of wear for both type of aggregates with GGBS as supplementary cementitious material as shown in Fig.5.16. The correlation equation is as follows:

$$d_w = Ae^{-0.04(f_c)} \quad \text{.....5.4}$$

where, d_w = depth of wear of RCC (mm)

f_c = compressive strength of RCC (N/mm²)

A = constant depending on the aggregate type

In the present study, A is having a value of 2.518, 1.567 for crushed gravel and limestone aggregates respectively. The correlation coefficient (R^2) is higher than 0.9 for both type of aggregates.

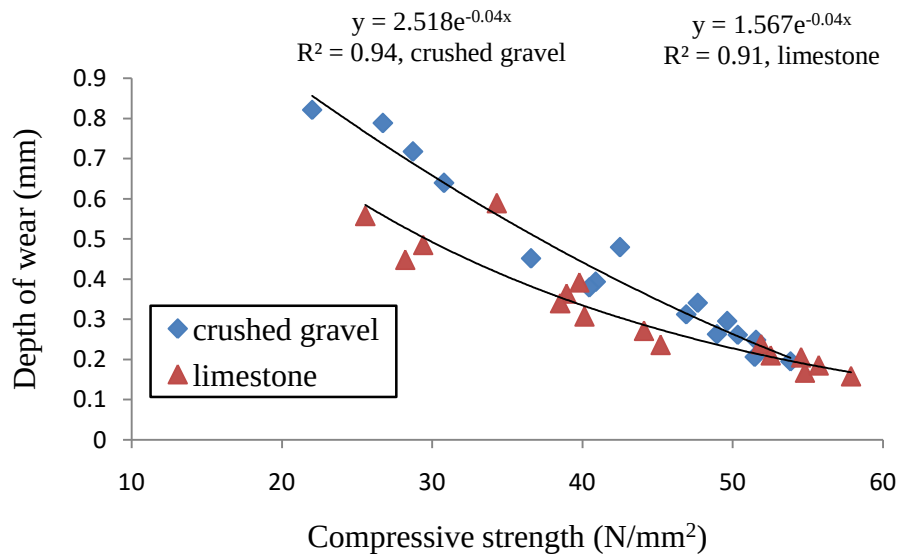


Fig. 5.16: Relationship between depth of wear and compressive strength for RCC containing crushed gravel and limestone aggregates

From Fig. 5.16, it can be observed that RCC mixes incorporating limestone aggregates exhibit lower depth of wear for lower compressive strength RCC; while for higher compressive strengths, both the aggregates exhibit similar depth of wear. The lower strengths, up to about 40 N/mm², mostly correspond to 7 days and 28 days curing age, that is the period in which the activation of GGBS has not occurred yet. It implies that prior to activation of GGBS and

pozzolanic activity; the limestone aggregates incorporated RCC exhibit better abrasion resistance. At later ages and at higher compressive strength RCC with both aggregates show similar abrasion resistance in terms of depth of wear.

Further, the predicted values of depth of wear based upon the above relationships was calculated and plotted in Fig. 5.17 against the experimental values. From Fig. 5.17, it can be observed that the predicted values of depth of wear are in good agreement with the experimental value at 95% prediction band.

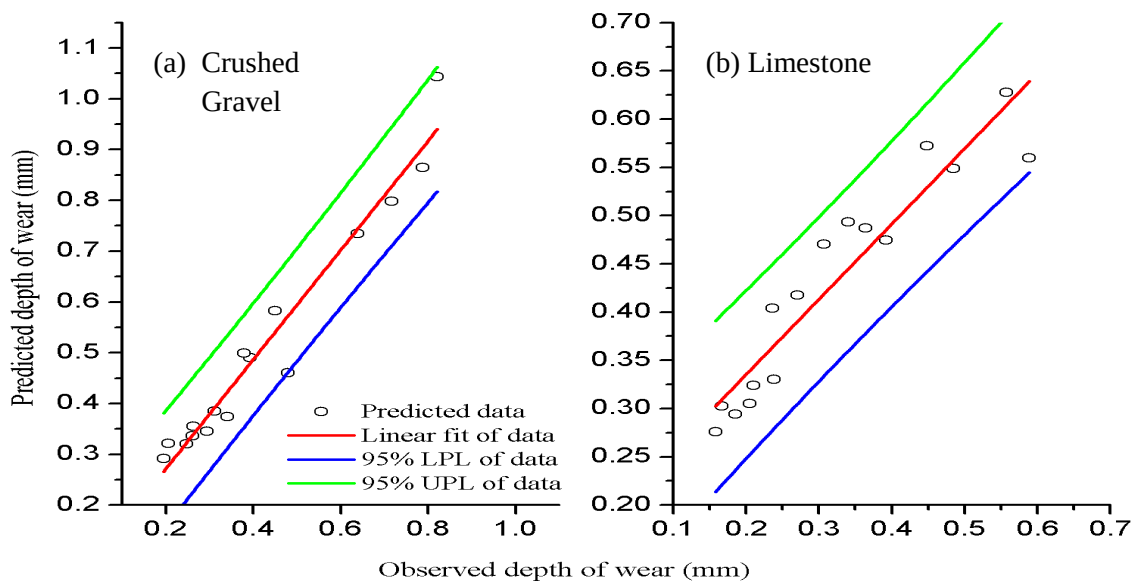


Fig. 5.17: Comparison between observed and predicted depth of wear using developed relationship at 95% prediction band

5.5.3 Drying Shrinkage

The unrestrained shrinkage strain of RCC was measured at 4, 7, 10, 14, 21 and 28 days of air curing for initial one month and thereafter at each 15 days interval for a total drying period of 9 months. The effect of drying time, aggregate type and GGBS content on drying shrinkage strain are discussed in the succeeding sections.

(a) Effect of aggregate type and drying duration on drying shrinkage

Fig. 5.18 depicts the effect of aggregate type viz. crushed gravel and limestone aggregates on development of drying shrinkage strain of RCC upto 270 days of drying period.

From the figure, it was observed that RCC mixes with crushed gravel aggregates presents higher shrinkage strain than the corresponding mixes incorporating limestone aggregates, at all drying durations. At 270 days of drying, the drying shrinkage strain observed for crushed gravel and limestone aggregates was 372 and 302 $\mu\text{m}/\text{mm}$ respectively. RCC made with crushed gravel aggregates presents 23% higher shrinkage as compared to limestone aggregates at the end of drying duration. *Carlos et al. (2010)* also observed 15% lesser shrinkage strain in conventional concrete prepared with fine limestone aggregates as compared to the mixes incorporating natural sand. *Videla and Aguilar (2006)* reported that lesser shrinkage observed in limestone aggregates can be attributed to the higher modulus of elasticity of these aggregates, which resist a greater degree of restrained.

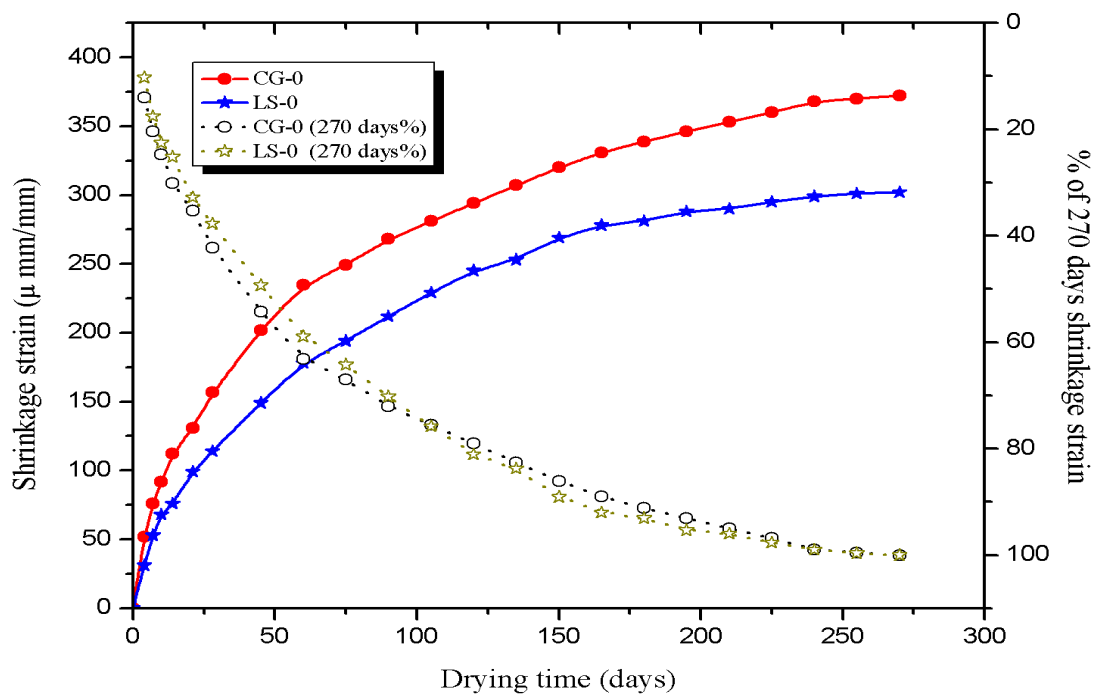


Fig. 5.18: Effect of aggregate type on development of drying shrinkage strain of RCC

Secondly, it can be observed from the figure that shrinkage strain development curve consists of two phases; in first phase, shrinkage occurred at faster rate (initial 90 days) followed by a second phase where, it increased with a decreasing rate for both types of aggregates. If the final shrinkage strain at 270 days of drying is taken as the base, the relative shrinkage strain at all ages can be obtained and is represented in Fig. 5.18. From second ordinate of the figure, it is

observed that approximately 70-75% of total shrinkage developed during the initial 90 days of drying duration.

When the obtained shrinkage strain of RCC is compared with the strain values available in literature for conventional concrete, it is observed that RCC experiences lesser shrinkage than the conventional concrete irrespective of the type of aggregates. For instance, *Kwan et al. (2010)* reported a shrinkage strain as high as 700 $\mu\epsilon$ at 300 days of drying. Similarly, *Seo et al. (2010)* observed shrinkage strain values in the range of 800 $\mu\epsilon$ in conventional concrete using ordinary Portland cement. Experimental shrinkage results show that RCC experiences approximately 50% lesser shrinkage than the conventional concrete. It can be due to lesser amount of cement and water content used for producing RCC, which are considered to be major factors affecting drying shrinkage (*Khay et al., 2010*).

(b) Effect of GGBS content on drying shrinkage

Figs. 5.19 and 5.20 illustrate the progressive variation in shrinkage strain with drying duration for RCC incorporating GGBS as supplementary cementitious material made by using respectively crushed gravel and limestone aggregates. In the figures, the symbolic representation was used for the experimentally obtained shrinkage strain values while the solid lines represent the predicted values obtained from the model generated by performing multi-regression analysis. The model is discussed in the next section.

Both of these figures show that RCC mixes incorporating GGBS exhibit higher shrinkage strain than the control RCC mix, irrespective of the aggregate type. The final shrinkage strain at 270 days of drying observed in crushed gravel RCC mixes at 20%, 40% and 60% GGBS incorporations levels was 395, 413 and 471 $\mu\text{ mm/mm}$ respectively; while for the control mix the value was limited to 372 $\mu\text{ mm/mm}$ (Fig. 5.19). The percentage increase in shrinkage strain observed was 6.2% (CG-20), 11.0% (CG-40) and 26.6% (CG-60) as compared to the corresponding control mix (CG-0). RCC mixes made with limestone aggregates also presents shrinkage behaviour identical with the mixes containing crushed gravel aggregates i.e. 2.3%, 9.6% and 17.8% higher shrinkage strain was observed in mix LS-20, LS-40 and LS-60 as compared to control LS-0 mix (Fig. 5.20).

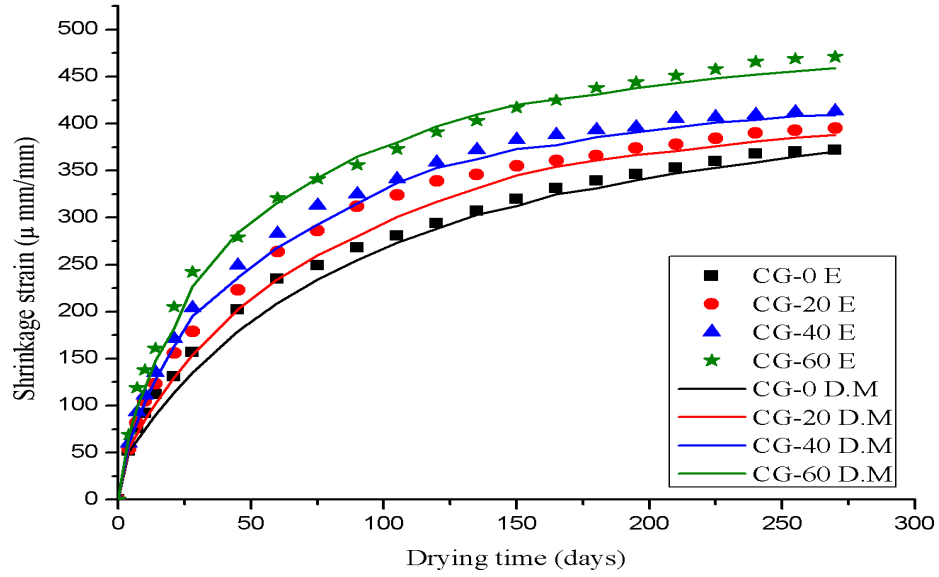


Fig. 5.19: Experimental and predicted shrinkage strain with proposed model for crushed gravel aggregates with variation in GGBS content

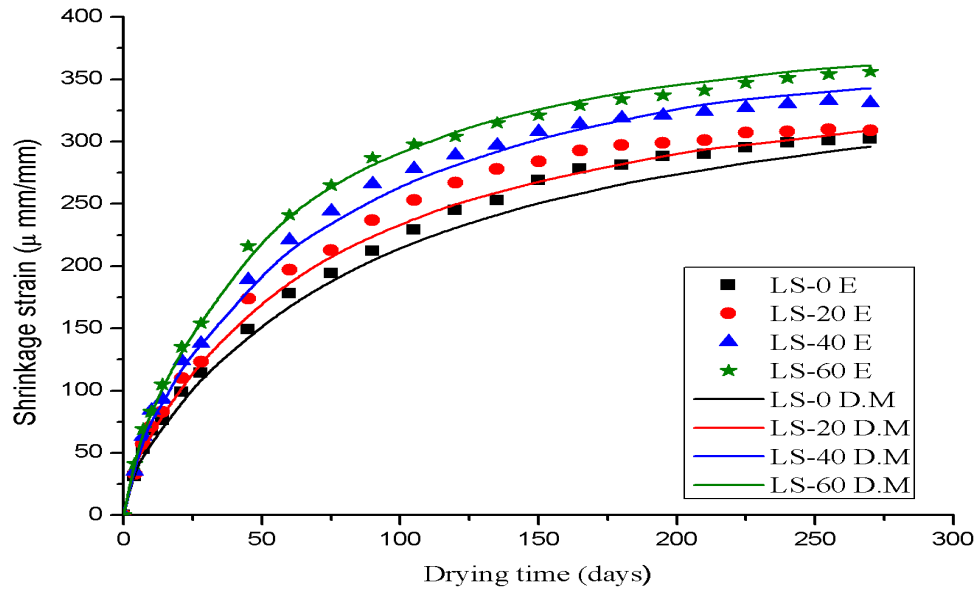


Fig. 5.20: Experimental and predicted shrinkage strain with proposed model for limestone aggregates with variation in GGBS content

The observed higher shrinkage strain of mixes incorporating GGBS can be attributed to higher volume of paste and denser matrix produced by the GGBS particles (*Shariq et al., 2016; Yuan et al., 2015*). Since the specific gravity of GGBS (2.83) is lower than cement (3.12), it led

to production of higher volume of paste in GGBS mixes when cement is directly replaced by GGBS on weight basis. Higher volume of paste leads to higher free shrinkage. Similar observation was made by *Tazawa et al. (1989)* while studying drying shrinkage in conventional concrete. Also, introduction of very fine GGBS created denser concrete with finer capillary voids in the cementitious matrix. Due to finer capillary pores, lesser amount of free water is available and hence greater capillary pore water pressure is created, leading to higher shrinkage strain. The observed shrinkage behaviour is in good agreement with the findings made by other researchers. *Shariq et al. (2016)* observed higher shrinkage strain with higher replacement level of cement with GGBS. *Mokarem et al. (2005)* reported higher shrinkage in mixes containing supplementary cementitious materials as compared to ordinary Portland cement concrete.

When the shrinkage development from 28 days to 270 days is compared, it is observed that mixes containing GGBS have lesser increase in shrinkage strain as compared to the control mixes, irrespective of the type of aggregates used. For instance, shrinkage development from 28 days to 270 days was 136.9% (CG-0), 120.6% (CG-20), 102.4% (CG-40) and 94.6% (CG-60) for mixes incorporating crushed gravel. Similarly, in limestone RCC mix, increase in shrinkage strain was observed to be 164.9%, 151.2%, 139.8% and 131.1% in LS-0, LS-20, LS-40 and LS-60 respectively. Thus, it can be concluded that GGBS has a greater influence on the shrinkage strain during the initial drying period of 28 days.

5.5.4 Statistical Analysis of Drying Shrinkage Results

Till date, a number of well established models are available for the prediction of shrinkage strain of conventional concrete. The details of all these models are presented in Table 5.2. Models sequenced 1 to 4 in Table 5.2 are applicable only for ordinary cement concrete without considering the effect of any supplementary cementitious materials. However, model number 5, proposed by *Shariq et al. (2016)* considered the effect of GGBS as supplementary cementitious materials in the exponential form. However, in all models, the effect of aggregate type was not considered, which can be quite significant.

Table 5.2: Details of existing well established shrinkage prediction models

S. No.	Model Name	Model equation	Parameter description	Applicability
1.	ACI-209 Model (Mokarem et al., 2005; ACI, 1992)	$\varepsilon_{sh}(t) = \frac{(t - t_{sh,0})}{35 + (t - t_{sh,0})} \varepsilon_{sh\infty}$	$\varepsilon_{sh}(t)$: shrinkage strain at time t $t_{sh,0}$: time at start of drying (days) $\varepsilon_{sh\infty}$: ultimate shrinkage strain (in/in) t : time (days)	• Type I&III cement as per ASTM • Normal and light weight concrete • Moist and steam curing conditions • Slump: 70mm • Air content: <6% • Cement content: 279 to 446(kg/m³)
2.	Bazant B3 Model (Mokarem et al., 2005; Bazant and Baweja, 1995)	$\varepsilon_{sh}(t) = -\varepsilon_{sh\infty} K_h S(t)$ $\varepsilon_{sh\infty} = (26(w)^{2.1} (f'_c)^{-0.28} + 270)10^{-6}$ $K_h = 1 - h^3$ $S(t) = \tanh \sqrt{\frac{t - t_0}{T_{sh}}}$	$\varepsilon_{sh}(t)$: Shrinkage strain at time t (in/in) $\varepsilon_{sh\infty}$: ultimate shrinkage strain (in/in) w : water content (lb/ft ³) K_h : cross-section shape factor h : relative humidity (%) t : age of concrete (days) t_0 : age of concrete at beginning of shrinkage $S(t)$: time function for shrinkage	• Cement content: 160 to 720 (kg/m³) • 28 days compressive strength (cylinder): 17 to 70 MPa • Water-cement ratio: 0.30 to 0.85

3.	CEB 90 Model (Mokarem et al., 2005; Shariq et al., 2016; Khay el al., 2010)	$\varepsilon_{cso} = \varepsilon_s (f_{cm})(\beta_{RH})$ $\varepsilon_s (f_{cm}) = (160 + 10\beta_{sc} (9 - f_{cm} / 1450))10^{-6}$ $\beta_{RH} = -1.55\beta_{ARH}$ $\beta_{ARH} = 1 - (RH / 100)^3$	ε_{cso} : drying shrinkage of Portland cement concrete ε_s : drying shrinkage obtained from RH-shrinkage chart β_{sc} : coefficient depends upon type of cement f_{cm} : 28-days compressive strength (psi)	• Ordinary concrete • 28 days compressive strength (cylinder): 12 to 80 MPa • Mean RH (%): 40 to 100 • Mean temperature (°C): 5-30
4.	Gardner/Lockman Model (Mokarem et al., 2005; Gardner and Lockman, 2001)	$\varepsilon_{sh} = \varepsilon_{shu} \beta(h) \beta(t)$ $\varepsilon_{shu} = 1000k \sqrt{\frac{4350}{f'_{cm28}}} 10^{-6}$ $\beta(h) = 1 - 1.18h^4$ $\beta(t) = \frac{(t - t_c)}{(t - t_c + 97(V / S)^{0.5}} * 10^{-6}$	ε_{shu} : ultimate shrinkage strain (in/in) $\beta(h)$: humidity correction factor $\beta(t)$: time correction factor t_c : drying age commenced (days) t : age of concrete (days)	• Plain cement concrete • 28 compressive strength: less than 70 MPa • Water-cement ratio: 0.4 to 0.6
5.	Model developed by Shariq et al. (Shariq et al., 2016)	$(\varepsilon_s)_t = \frac{\alpha}{\sqrt{f_c}} \left(\frac{35000t^{0.4}}{220 + t^{0.4}} \right)$ $\alpha = e^{(0.004 ps)}$	$(\varepsilon_s)_t$: Shrinkage strain at time t ps : % of GGBS f_c : 28-day cylinder compressive strength (MPa)	Ordinary concrete containing GGBS as supplementary cementitious material

In the present study, three models namely, ACI 209 model (S.No 1), Bazant B3 model (S.No. 2) and Shariq model (S.No.5) were applied to the observed shrinkage strain values in control RCC mixes and are presented in Figs. 5.21 and 5.22. These models are chosen because of being popular among design engineers for their ease of application due to use of lesser input variables. Figs. 5.21 and 5.22 illustrate the comparison of shrinkage strains observed by various

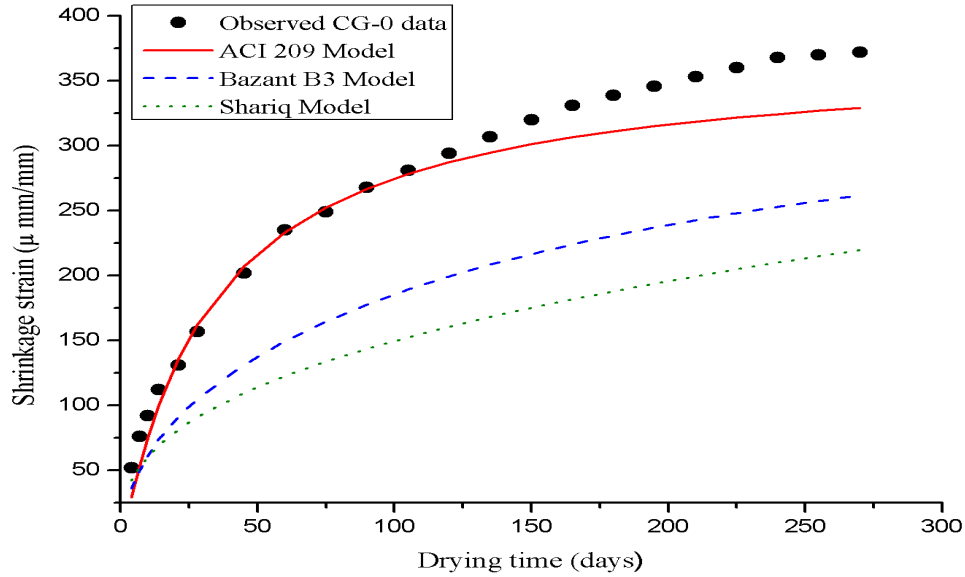


Fig. 5.21: Observed drying shrinkage data for control mix CG-0 and its comparison with established prediction models

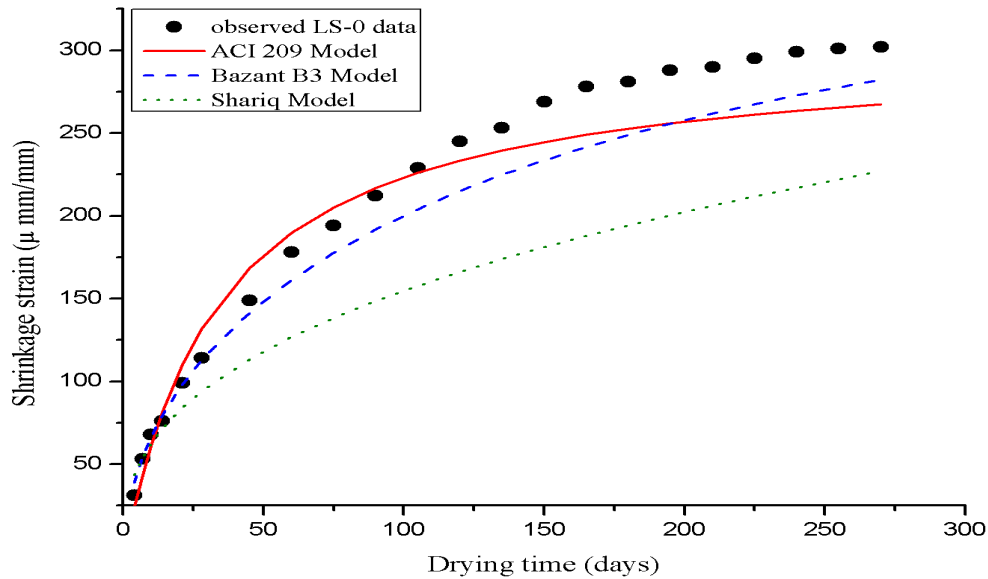


Fig. 5.22: Observed drying shrinkage data for control mix LS-0 and its comparison with established prediction models

models with the data obtained for control RCC mixes incorporating crushed gravel and limestone aggregates, respectively. From the figures, it was observed that Bazant B3 and Shariq models predict much lower values of shrinkage strain than the experimental values observed in RCC mixes. ACI 209 model fitted well with the experimental data till the initial 120 days of drying; thereafter the model presents lower values. From both the figures, it can be concluded that the

existing models which fitted well in conventional concrete cannot be used to accurately predict the shrinkage strain of RCC. It can be due to the difference in micro structure of RCC from the conventional concrete. Therefore, a multiple variable regression model was attempted for predicting drying shrinkage of RCC and is presented in the succeeding section.

From the experimental results in the present study, it is observed that the main factors affecting the drying shrinkage of RCC are incorporation level of GGBS percentage, aggregate type and drying duration. To determine the significance of these variables to predict drying shrinkage, statistical analysis of variance (ANOVA) was performed by using SPSS software. The contribution of each individual variable was determined and is presented in Table 5.3. The p-value represents significance level of each variable; variables having p-value lower than 0.05 have a significant effect on the drying shrinkage at 95% confidence level (*Videla and Aguilar, 2006*). The observed p-values for each variable is lower than 0.05, indicating that all these parameters viz. GGBS percentage replacement level, drying duration and aggregate type influence the drying shrinkage behaviour of RCC.

The ANOVA results confirm that replacement level of cement with GGBS, aggregate type and drying time has a significant role in drying shrinkage strain of the RCC mix. Therefore, a multiple variable regression model was developed considering all the significant parameters. Since shrinkage increases asymptotically with time, thus non-linear curve fitting analysis was performed to obtain the regression model. The proposed model is presented in Eq. (5.5).

$$\varepsilon_{sh}(t) = K \left[\varepsilon_{\min} + (\varepsilon_{\max} - \varepsilon_{\min}) \frac{\alpha}{(12) + \alpha} \right]^x \quad \dots(5.5)$$

$$\alpha = \frac{e^p}{\sqrt{f_c}} \times t$$

where: $\varepsilon_{sh}(t)$ = shrinkage strain of RCC at time t (μ mm/mm)

$\varepsilon_{\min}$ = minimum shrinkage strain of RCC (μ mm/mm)

$\varepsilon_{\max}$ = maximum shrinkage strain of RCC (μ mm/mm)

p = percentage replacement of cement with GGBS (%)

f_c = 28 days cylinder compressive strength (N/mm^2)

t = time (days)

K and x are constants, depending on the type of aggregates

In the present study, for crushed gravel aggregates a value of 0.30 (for K) and 1.2 (for x) fitted well with high correlation coefficient (R^2) of 0.988; while for limestone aggregates, correlation coefficient (R^2) of 0.996 was obtained with K and x values of 0.42 and 1.16 respectively. The model is proposed for the prediction of shrinkage strain in RCC mix with cement content and water-cement ratio ranging between 300-350 kg/m³ and 0.35-0.45 respectively.

Table 5.3: Analysis of variance (ANOVA) for drying shrinkage test results

Source of variation	SS	df	MS	F	p-value	F _{cr}	Significance criteria
1. Crushed gravel effect of drying shrinkage (drying time considered from 0 to 270 days)							
GGBS	80368.48	3	26789.49	231.45	0.000	4.12	Significant
Drying time	1144075.83	21	54479.80	470.69	0.000	2.20	Significant
Error	7291.76	63	115.74				
Total	1231736.08	87					
2. Limestone effect of drying shrinkage (drying time considered from 0 to 270 days)							
GGBS	30580.22	3	10193.40	154.26	0.000	4.12	Significant
Drying time	822577.37	21	39170.35	592.77	0.000	2.20	Significant
Error	4163.03	63	66.08				
Total	857320.63	87					
3. Aggregate type effect on drying shrinkage at 270 days of drying							
GGBS	6603.37	3	2201.13	12.08	0.035	9.28	Significant
Aggregate type	15576.12	1	15576.12	85.52	0.003	10.13	Significant
Error	546.37	3	182.12				
Total	22725.87	7					

Based upon the established relationship, the predicted values of shrinkage strain are plotted earlier along with the experimental values in Figs. 5.19 and 5.20. From both figures, it was observed that the values obtained by using proposed model differ from the experimental obtained values in a narrow range. Further, a comparison was made between shrinkage strains predicted by the proposed model with the experimentally observed strain values and is presented

in Fig. 5.23 for all data points. Fig. 5.23 clearly indicates that most of the predicted values lie within the 95% prediction band. The effectiveness of developed model is further explored by performing an error percentage analysis (Mokarem et al., 2005). Error percentage is calculated by using Eq. (5.6):

$$\text{Error Percentage (\%)} = \frac{(\text{Residual value} * 100)}{\text{Measured drying shrinkage}} \quad \dots\dots(5.6)$$

where, Residual value is the difference between the predicted drying shrinkage and the experimentally measured drying shrinkage

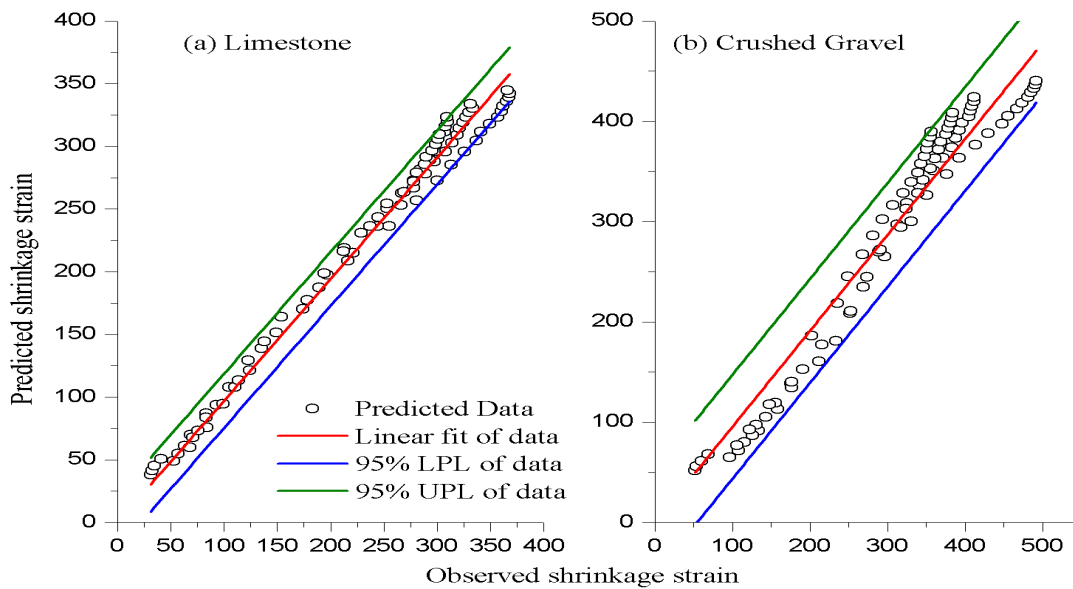


Fig. 5.23: Comparison between observed and predicted shrinkage strain using proposed model at 95% prediction band

Table 5.4 presents the percentage error calculated by using Eq. (5.6) for both types of aggregates with all replacement levels of cement with GGBS. From the table, it was observed that percentage error for the prediction of shrinkage strain by the developed model is very low for longer drying durations. On the basis of above discussion, it can be concluded that the drying shrinkage strain of RCC can be evaluated by using the developed prediction model, incorporating GGBS and made by using either crushed gravel or limestone aggregates. It is to mention here that the present model is developed for RCC mixes having a particular set of materials, viz. GGBS, limestone aggregates, gravel aggregates etc. The intent of the proposed

model is to study the influence of GGBS and different types of aggregates on drying shrinkage strain of RCC.

Table 5.4: Error percentage value of shrinkage strain predicted by using developed model

Drying time (days)	Percentage error (%)							
	Crushed gravel				Limestone			
	CG-0	CG-20	CG-40	CG-60	LS-0	LS-20	LS-40	LS-60
7	-17.1	-14.6	-10.7	-16.8	-11.3	-7.1	-6.3	-4.3
14	-19.6	-17.1	-5.1	-8.1	-7.8	-3.6	-4.3	-2.8
28	-14.0	-13.9	-4.4	-6.1	-3.5	0	-0.7	-0.6
45	-11.4	-9.4	-5.2	1.7	-4.0	-7.5	-4.2	-3.7
60	-11.1	-11.3	-5.3	-1.5	-5.6	-5.1	-3.2	0
75	-6.0	-9.1	-6.4	0.3	-3.1	-2.3	-4.1	-0.4
90	-4.8	-10.2	-3.1	2.5	-3.3	-5.4	-4.8	-1.7
105	-2.8	-7.1	-1.2	1.8	-4.4	-5.9	-3.2	-1.0
120	-2.1	-6.4	-1.7	1.5	-5.7	-6.3	-2.7	1.3
135	-1.3	-5.7	-2.7	1.7	-4.7	-6.8	-1.7	0.9
150	-2.5	-2.2	-2.6	0.7	-6.6	-5.6	-1.9	1.5
165	-1.8	-1.9	-2.8	0.2	-7.2	-6.1	-1.3	1.2
180	-2.4	-1.4	-1.8	-1.6	-5.3	-5.0	-0.6	1.5
195	-1.7	-1.9	-1.3	-1.3	-5.5	-3.6	0.9	2.1
210	-1.6	-2.9	-2.2	-1.7	-4.4	-2.3	1.8	2.0
225	-1.9	-1.8	-1.5	-2.2	-4.1	-3.2	2.1	1.4
240	-2.4	-2.3	-1.2	-3.0	-4.0	-2.2	2.1	1.4
255	-1.3	-2.0	-0.9	-2.7	-2.9	-1.6	2.1	1.4
270	-0.5	-1.7	-0.9	-2.5	-1.9	0	3.2	1.4

5.5.5 Thermal Shrinkage Strain at Maximum Ambient Temperature

The effect of coarse aggregate type, heating duration and GGBS content on the thermal shrinkage strain of RCC mixes exposed to a constant maximum ambient temperature of 70 ± 2 °C for duration of 270 days has been studied. Along with this, thermal shrinkage strain is compared

with the corresponding drying shrinkage strain, to get better insight information between these two parameters.

(a) Effect of aggregate type and heating duration on thermal shrinkage strain

The development of thermal shrinkage strain, with the variation in coarse aggregate type and heating duration is presented in Fig. 5.24. It can be observed that thermal shrinkage strain follows the behaviour similar with the drying shrinkage behaviour i.e. crushed gravel aggregates presents higher thermal shrinkage strain than the limestone aggregate at all heating durations. At the end of heating duration of 270 days, 24.4% higher shrinkage strain was observed in mixes incorporating crushed gravel aggregates as compared to the corresponding mixes incorporating limestone aggregates. The observed lower thermal shrinkage strain in limestone aggregates can be attributed to the lower coefficient of thermal expansion. Literature suggests that coefficient of thermal expansion ranges between 9.76 to 10.44 $\mu\epsilon / ^\circ\text{C}$ for limestone aggregates; while for gravel aggregates coefficient of thermal expansion lies between 11.68 to 13.73 $\mu\epsilon / ^\circ\text{C}$ (*Buch and Jahangirnejad, 2008*).

To further examine the relative strain development with heating duration, the second ordinate of the Fig. 5.24 was constructed by assuming the final thermal shrinkage strain as 100%. From the second ordinate of the figure, it was observed that during the initial 4 days of heating, 37.4% of total shrinkage strain was developed in crushed gravel aggregate mix; while in limestone mix, this percentage was 25.7%. In general, approximately, 70-75% of total strain developed during the heating period of initial 45 days. However, under the atmospheric temperature, the same percentage of strain was developed during 90 days of drying.

Fig. 5.25 compares the thermal and drying shrinkage strain values obtained in the control RCC mixes made with crushed gravel and limestone aggregates. From the figure, it was observed that continuously heating the specimens at $70 \pm 2^\circ\text{C}$ accelerate the shrinkage strain at early duration of heating. At higher temperature, most of the shrinkage strain developed within first 28 days of heating. While the later duration shrinkage strain at higher temperature was lower than the corresponding drying shrinkage strain. At the end of testing duration, approximately

25% lower thermal shrinkage strain was observed as compared to drying shrinkage strain, irrespective of the type of aggregates.

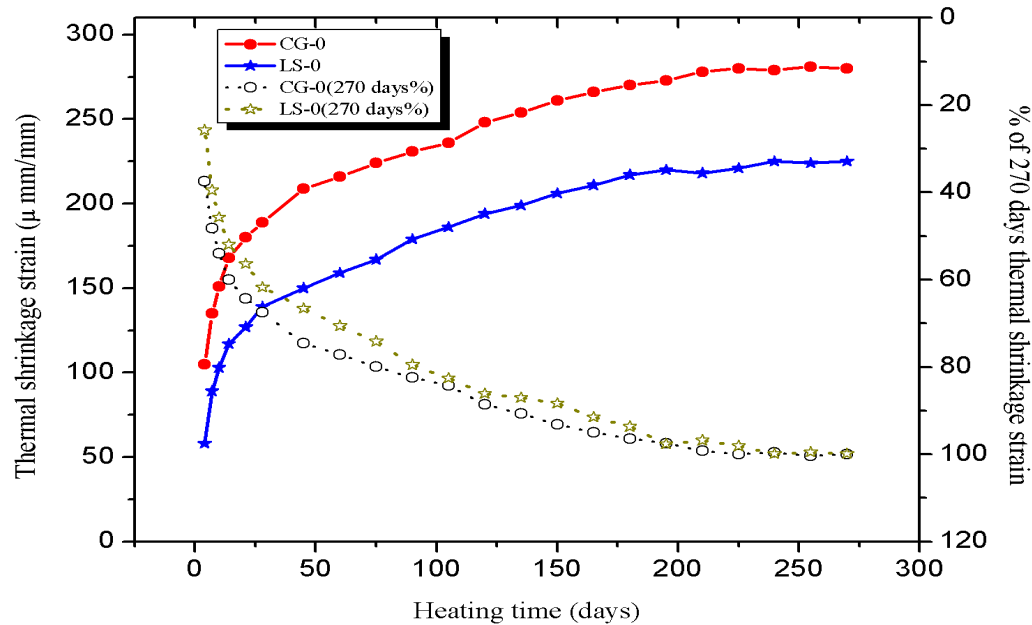


Fig. 5.24: Effect of aggregate type on development of thermal shrinkage strain in RCC

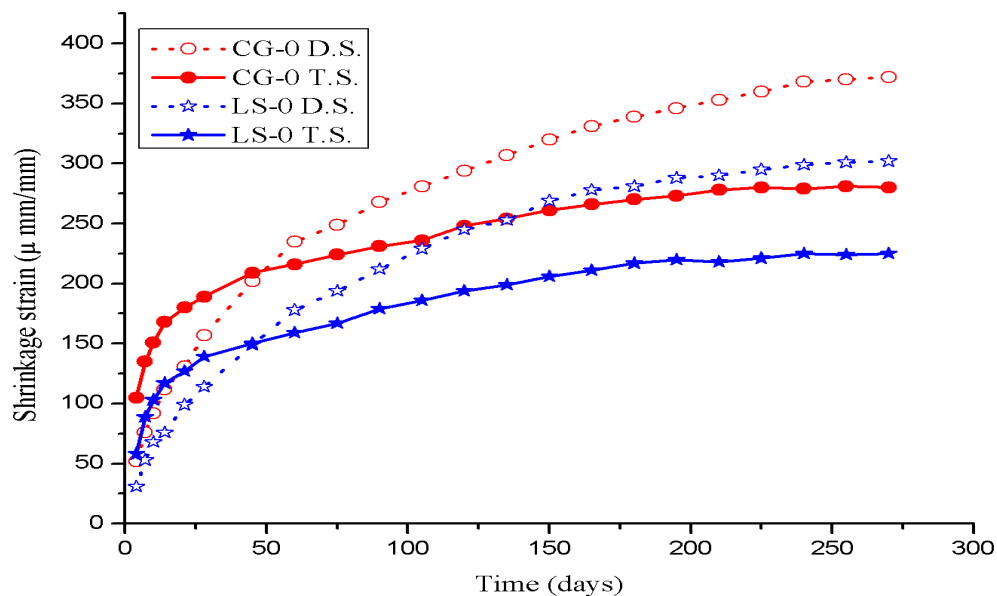


Fig. 5.25: Comparison between drying and thermal shrinkage strain of RCC mixes incorporating either crushed gravel or limestone aggregates
(D.S.: Drying shrinkage; T.S.: Thermal shrinkage)

The observed behaviour can be attributed to the acceleration in hydration reaction of cement at higher temperature. It is well established that shrinkage strain occurs firstly due to consumption of water during hydration reaction of cement; and secondly due to moisture evaporation because of relative humidity gradient inside and outside the concrete matrix (*Yang et al., 2014*). Acceleration in hydration reaction at early age, leads to the formation of more hydration products by making the alteration in composition and internal microstructure of concrete. Further, decrease in internal relative humidity at later age because of moisture evaporation leads to formation of stable microstructure. Previous investigators; *Yang et al. (2014)*; *Chu et al. (2012)* also reported a higher early drying and autogenous shrinkage strains at elevated temperatures followed by lower shrinkage strain values thereafter. Thus, it can be concluded that the change in temperature had a distinct influence on the shrinkage strain development.

(b) Effect of GGBS content on thermal shrinkage strain

The progressive variation in thermal shrinkage strain with variation in GGBS content is shown in Fig. 5.26 and 5.27 for RCC mixes incorporating crushed gravel aggregates and limestone aggregates respectively. Similar to the drying shrinkage strain behaviour, thermal shrinkage strain also increases with the addition of GGBS irrespective of aggregate type. During the initial hour of heating, the thermal shrinkage strain increases with the progressive increase in GGBS content, but the increasing trend gets altered towards the end of heating duration.

In RCC mix containing crushed gravel aggregates, at 28 days of heating duration, 17.9%, 22.2% and 34.4% higher shrinkage strain in mixes; namely; CG-20, CG-40 and CG-60 was observed as compared to control CG-0 mix. Similarly, this percentage increase in shrinkage strain was limited to 13.5% (CG-20), 8.5% (CG-40) and 6.4% (CG-60) at end of testing duration of 270 days. Similarly, RCC mix containing limestone aggregates with 20% GGBS content has marginally higher thermal shrinkage strain as compared to the control mix LS-0. In LS-40 and LS-60 mix 11.5% and 22.2% higher thermal shrinkage strain was observed as compared to control mix (LS-0) at 270 days of testing duration.

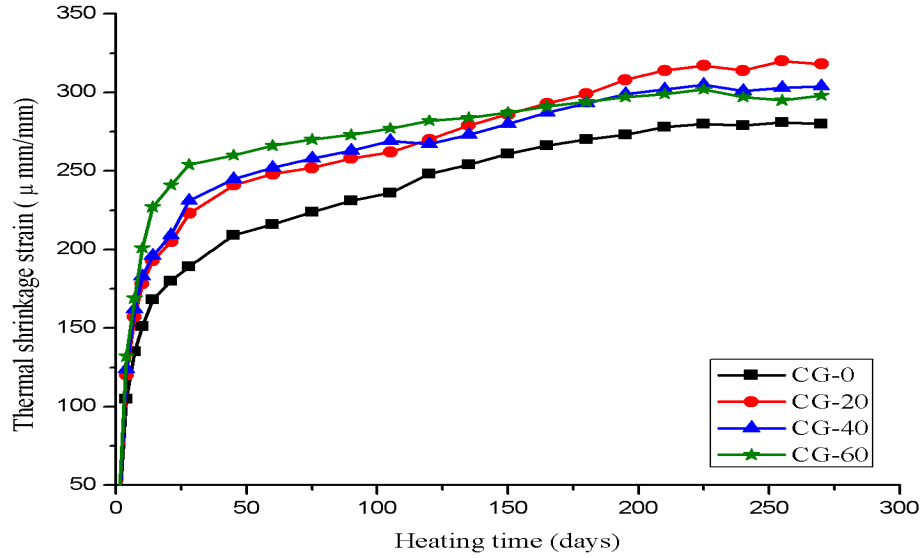


Fig. 5.26: Effect of GGBS content on development of thermal shrinkage strain with crushed gravel aggregates

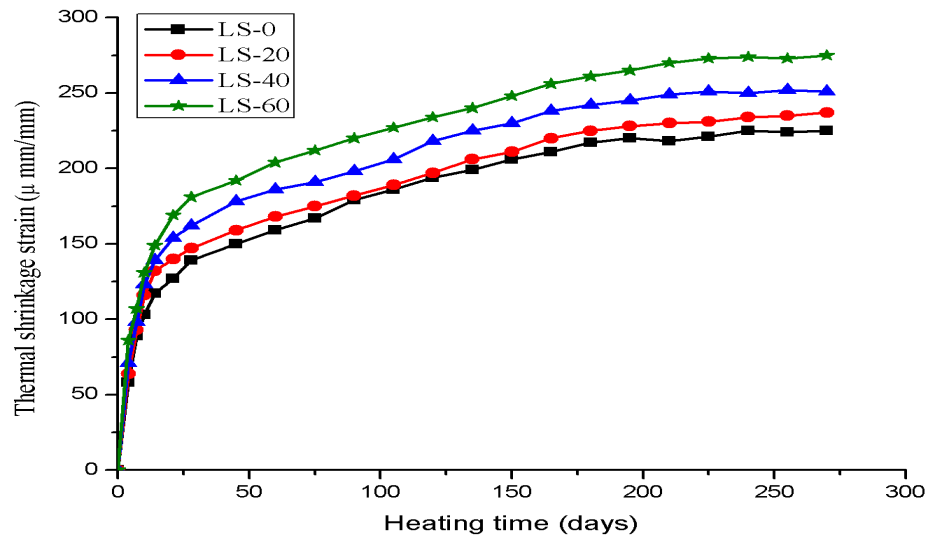


Fig. 5.27: Effect of GGBS content on development of thermal shrinkage strain with limestone aggregates

Further, the relative strain development was calculated by assuming the final thermal shrinkage strain as 100% corresponding to each mix and is presented in Table 5.5. It was observed that during 28 days of heating, 67.5%, 70.2%, 76.0% and 85.2% of total thermal shrinkage strain was developed in CG-0, CG-20, CG-40 and CG-60 mixes respectively. Similarly, in mixes, namely, LS-0, LS-20, LS-40 and LS-60 the strain development was respectively, 61.7%, 59.5%, 58.9% and 61.5% of total thermal shrinkage strain. Thus, GGBS had

a greater influence on the strain development in case of RCC containing crushed gravel aggregates than on corresponding mixes made by using limestone aggregates.

Table 5.5: Relative thermal shrinkage strain development in all RCC mixes

Drying time (days)	Thermal shrinkage strain development relative to total strain (%)							
	Crushed gravel				Limestone			
	CG-0	CG-20	CG-40	CG-60	LS-0	LS-20	LS-40	LS-60
4	37.5	37.8	40.7	44.2	25.7	27.0	28.2	31.2
7	48.2	49.4	53.3	56.6	39.5	39.2	39.0	38.9
10	53.9	56.1	60.2	67.4	45.7	48.9	49.0	47.6
14	59.9	60.7	64.4	76.1	51.9	55.7	55.3	54.1
21	64.2	64.5	68.7	80.8	56.4	59.1	61.3	61.4
28	67.4	70.2	75.9	85.2	61.7	62.0	64.5	65.8
45	74.6	75.9	80.6	87.2	66.6	67.1	70.9	69.8
60	77.1	78.1	82.8	89.2	70.6	70.8	74.1	74.1
75	79.9	79.3	84.8	90.5	74.2	73.8	76.1	77.0
90	82.4	81.2	86.5	91.6	79.5	76.8	78.8	80.0
105	84.2	82.5	88.4	92.9	82.6	79.7	82.1	82.5
120	88.5	85.1	87.8	94.6	86.2	83.1	86.8	85.0
135	90.6	87.8	89.7	95.2	88.4	86.9	89.6	87.2
150	93.1	90.1	92.1	96.3	91.5	89.1	91.6	90.1
165	94.9	92.1	94.4	97.6	93.7	92.8	94.8	93.0
180	96.4	94.2	96.4	98.6	96.4	94.9	96.4	94.9
195	97.5	97.0	98.3	99.6	97.7	96.2	97.6	96.3
210	99.2	98.9	99.3	100.3	96.8	97.0	99.2	98.1
225	99.9	99.8	100.3	101.3	98.2	97.4	100.0	99.2
240	99.6	98.9	98.9	99.6	99.9	98.7	99.6	99.6
255	100.3	100.8	99.6	98.9	99.5	99.1	100.3	99.2
270	100.1	100.2	99.9	100.1	99.9	100.0	100.0	100.0

(c) Comparative effect of GGBS content on drying and thermal shrinkage strain

The effect of GGBS content on the development of shrinkage strain under normal atmospheric condition (drying shrinkage) and at high temperature (thermal shrinkage) is shown in Figs.5.28 and 5.30 for RCC mixes incorporating crushed gravel and limestone aggregates, respectively. From both the figures, it was observed that irrespective of GGBS content, high temperature results in higher early age thermal shrinkage strain as compared to drying shrinkage strain. With the enhancement in heating duration, the thermal shrinkage strain becomes lower than the drying shrinkage strain at all replacement levels of cement with GGBS.

Fig. 5.28 shows that at early testing duration of 4 days, 101%, 122%, 106% and 91% higher thermal shrinkage strain was observed as compared to drying shrinkage strain for the mixes, CG-0, CG-20, CG-40 and CG-60 respectively. The long term thermal shrinkage decreases with the increase in GGBS content and got stabilization at 40% and 60% replacement level of cement with GGBS. On the other hand, long term drying shrinkage strain increased with the progressive increase in GGBS content. At 270 days of testing duration, in mixes, namely, CG-0, CG-20, CG-40 and CG-60 thermal shrinkage strain decreased by 25%, 19%, 26% and 36% respectively on comparison with drying shrinkage strain. The progressive variation between thermal and drying shrinkage strain with variation in GGBS content is presented in Fig. 5.29.

Similarly, in RCC containing limestone aggregates, initially higher thermal shrinkage strain than the drying shrinkage strain followed by alteration in behaviour was registered. At 270 days of testing duration, approximately 22% to 25% decrease in thermal shrinkage strain was registered as compared to drying shrinkage strain with all GGBS incorporation levels (Fig. 5.30). The progressive variation between thermal and drying shrinkage strain is shown in Fig. 5.31. Fig. 5.31 shows that in case of RCC containing limestone aggregates, GGBS content has a negligible influence on the variation in shrinkage strain after 28 days of testing duration; while in crushed gravel aggregates GGBS content shows the large influence on this variation.

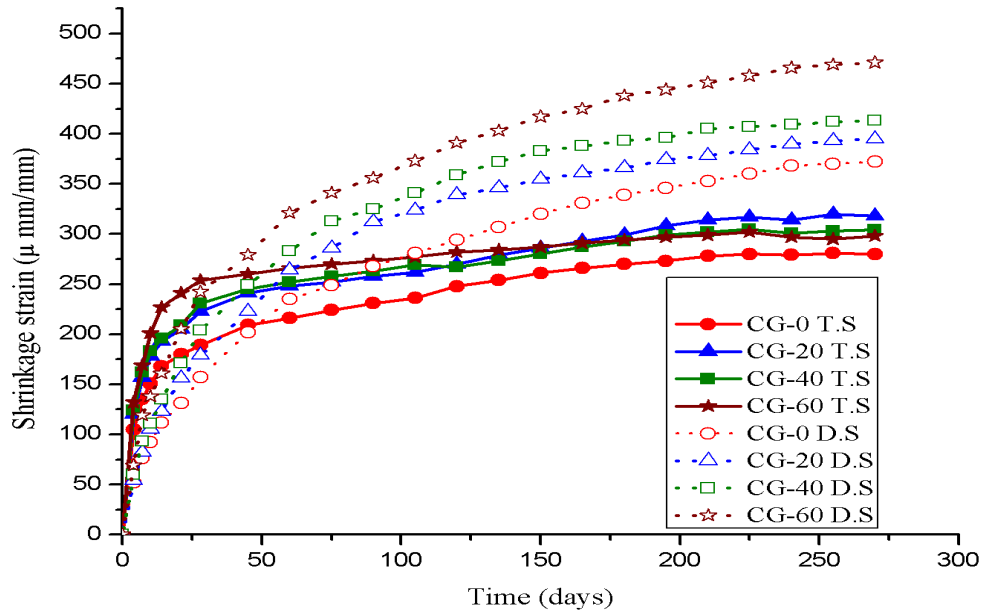


Fig. 5.28: Comparison between thermal and drying shrinkage development with crushed gravel aggregates and GGBS
(T.S: Thermal shrinkage; D.S: Drying Shrinkage)

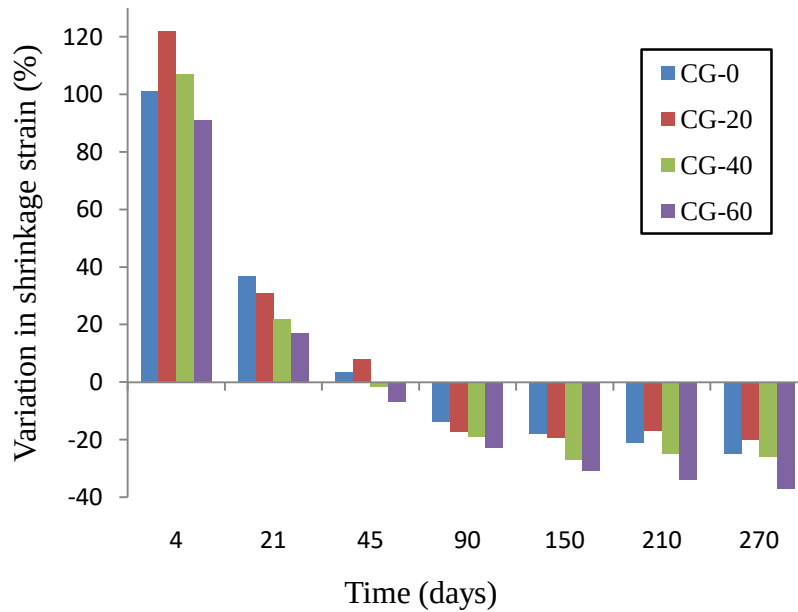


Fig.5.29: Variation in thermal and drying shrinkage strain for crushed gravel aggregates RCC mix with variation in GGBS content

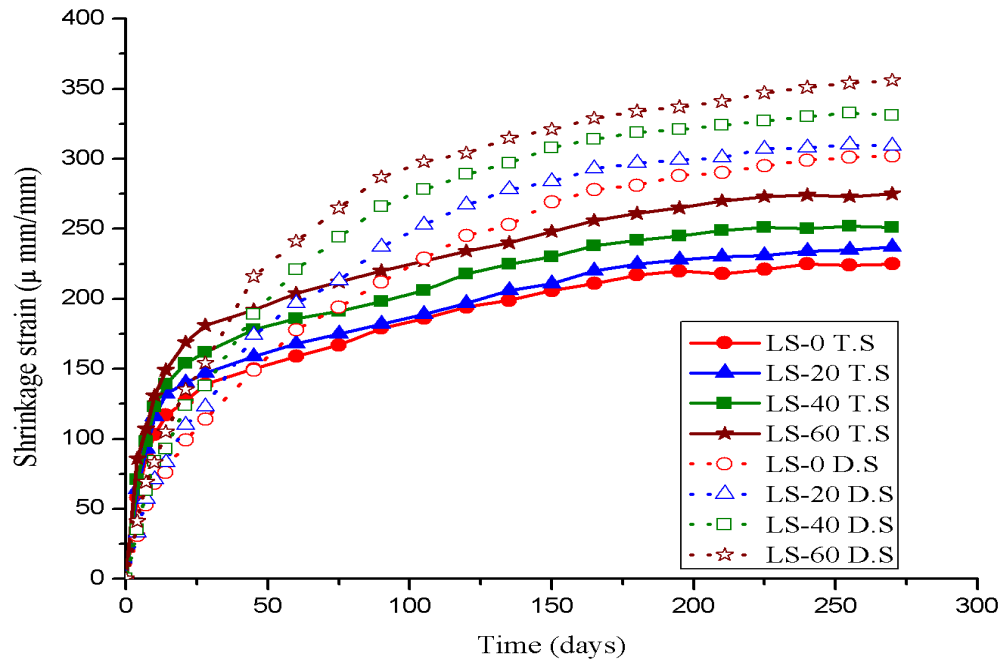


Fig. 5.30: Comparison between thermal and drying shrinkage development with limestone aggregates and GGBS
(T.S: Thermal shrinkage; D.S: Drying Shrinkage)

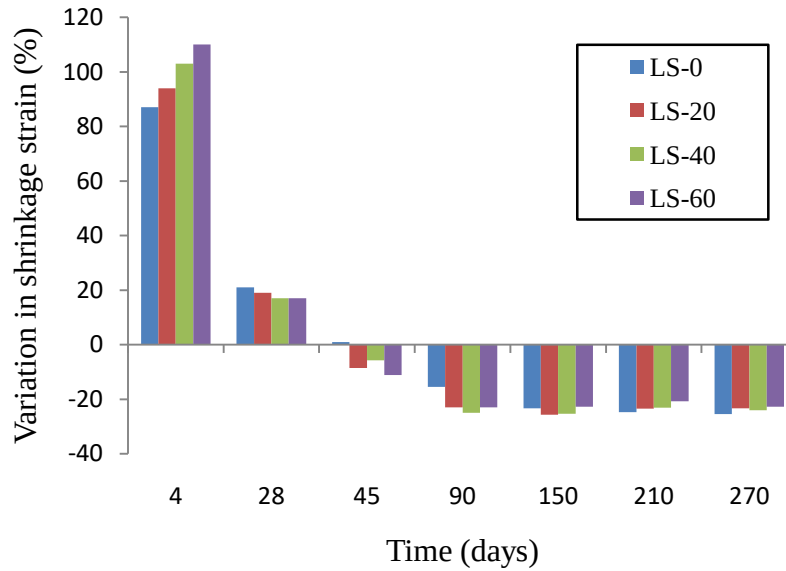


Fig.5.31: Variation in thermal and drying shrinkage strain for limestone aggregates RCC mix with variation in GGBS content

5.5.6 Thermal Shrinkage Strain with Temperature Cycles

In order to study the effect of heating and cooling cycles on the thermal shrinkage strain, all the RCC mixes were subjected to temperature cycles for a period of one month. One temperature cycle consisted of 12 hours heating at $70 \pm 2^\circ\text{C}$ followed by a cooling at $10 \pm 2^\circ\text{C}$ for a period of 12 hours. For this, prism specimens of size $75 \times 75 \times 290$ mm were cast and change in length of specimens were noted after every third cycle of heating and cooling. Also, with the use of observed thermal strain, thermal stresses were calculated as per *IS: 456-2000 (BIS, 2000)*. The thermal stresses are calculated as:

$$\sigma = \varepsilon * E$$
$$E = 5000\sqrt{f_{ck}} \quad \dots\dots 5.7$$

where, σ = thermal stress (N/mm^2)

ε = thermal strain ($\mu \text{ mm/mm}$)

E = modulus of elasticity (N/mm^2)

f_{ck} = 28 days compressive strength (N/mm^2)

The variation in thermal strain and corresponding stress with the variation in heating and cooling cycles, aggregate type and GGBS content are discussed in succeeding sections.

(a) Effect of heating and cooling cycle on thermal strain with variation in aggregate type

The variation in thermal strain with the variation in coarse aggregate type is shown in Fig. 5.32 (a) under the 30 cycles of heating and cooling. Fig. 5.32 depicts that during the initial cycles of heating and cooling, a large variation in thermal strain was observed at each cycle of heating and cooling. This variation in strain goes on decreasing with the increase in heating and cooling cycles irrespective of aggregate type. For instance, at third cycle of heating and cooling, 38% variation in thermal strain was observed in CG-0 mix, which got reduced to 13% at 30th cycle. Similarly, in mix having limestone aggregates variation in thermal strain observed at end of 3rd and 30th cycle of heating and cooling was 44% and 18% respectively. It was well established that on first cycle of loading or heating the hysteresis is large, which decreases with the increase in number of cycles (*Neville and Brooks, 2010*).

On making a comparison based upon the aggregate type, it was observed that 10% lower thermal strain was shown by LS-0 mix as compared to CG-0 mix at the end of 3rd cycle of

heating, while the shrinkage strain was lower by 14% at 3rd cycle of cooling. The observed lower thermal strain in RCC mix containing limestone aggregates can be attributed to the lower coefficient of thermal expansion value of limestone over the crushed gravel. From the engineering point of view, thermal stress is important to check the warping and cracking produced in the concrete. Thus thermal stress is also calculated based upon the thermal strain, modulus of elasticity and 28 days compressive strength. Fig. 5.32 (b) depicts the thermal stress behaviour with both types of aggregates which is as obvious; similar with the thermal strain. The values of thermal stresses were further used to predict failure cycles as discussed later.

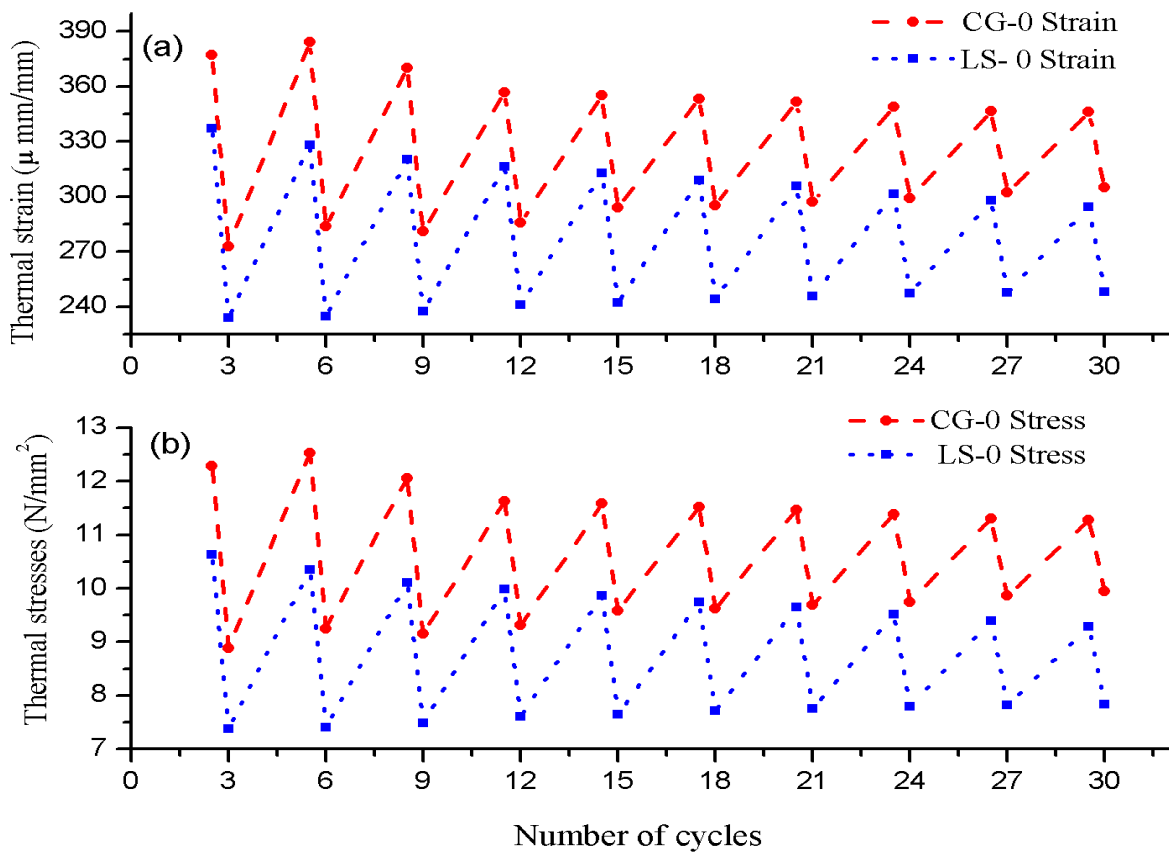


Fig. 5.32: Variation in (a) thermal strain and (b) corresponding stress with heating and cooling cycles in control RCC mix

(b) Effect of heating and cooling cycle on thermal strain with variation in GGBS content

The effect of heating and cooling cycles on thermal strain are shown in Figs 5.33 -5.35 with 20%, 40% and 60% replacement levels of cement with GGBS. With 20% replacement level of cement with GGBS, crushed gravel aggregates presents behaviour different than the limestone

aggregates. In CG-20 mix variation in strain remained constant at each cycle of heating and cooling while in LS-20 mix strain decreased with the enhancement in heating and cooling cycles. At the end of 30th cycle of heating and cooling the strain variation observed in CG-20 mix was 41%, while in LS-20 mix this variation was 28% (Fig. 5.33 (a)).

RCC mixes containing 40% and 60% GGBS shows the behaviour identical with the mix having 20% GGBS, but with the incremental increase in thermal strain irrespective of aggregate type. For CG-20 mix; maximum strain observed after heating was 399 μ mm/mm while in CG-40 and CG-60 mix this value was increased to 440 μ mm/mm and 487 μ mm/mm respectively. Similarly, in RCC mixes incorporating limestone aggregates maximum strain of 343 μ mm/mm, 351 μ mm/mm and 380 μ mm/mm was registered in LS-20; LS-40 and LS-60 mix respectively.

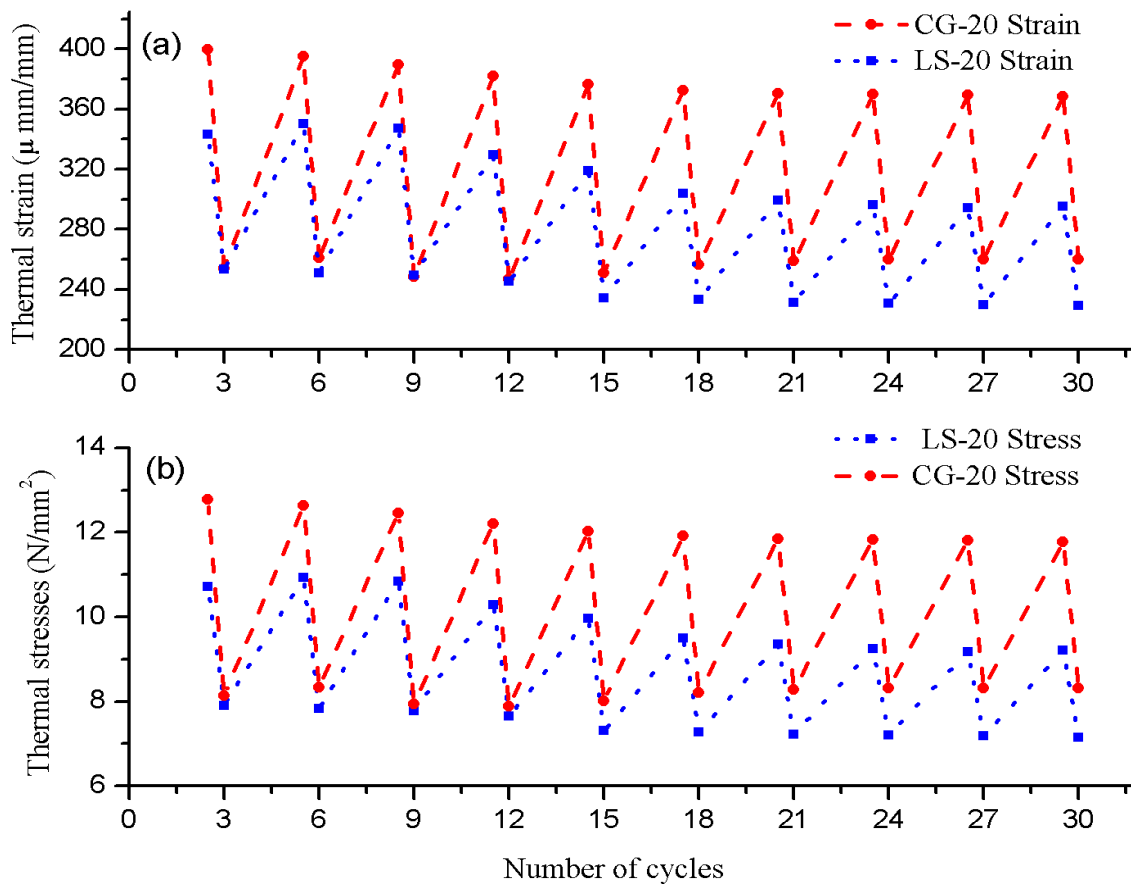


Fig. 5.33: Variation in (a) thermal strain and (b) corresponding stress with heating and cooling cycles in RCC mix containing 20%GGBS

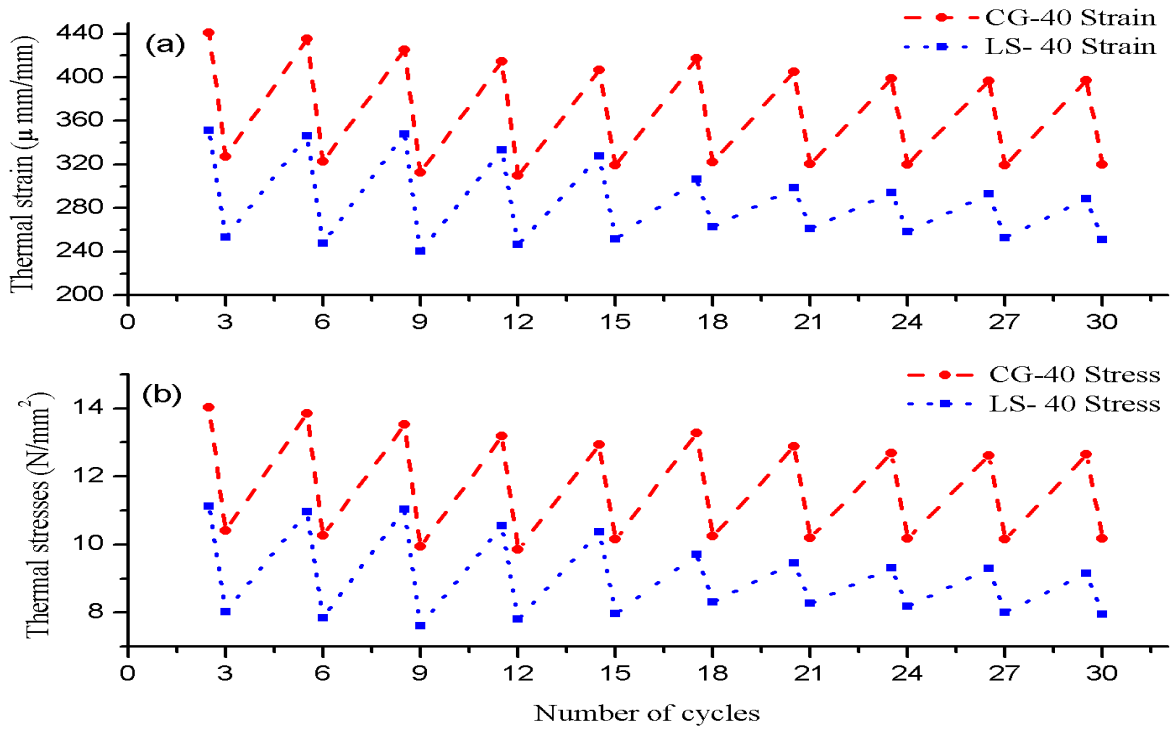


Fig. 5.34: Variation in (a) thermal strain and (b) corresponding stress with heating and cooling cycles in RCC mix containing 40%GGBS

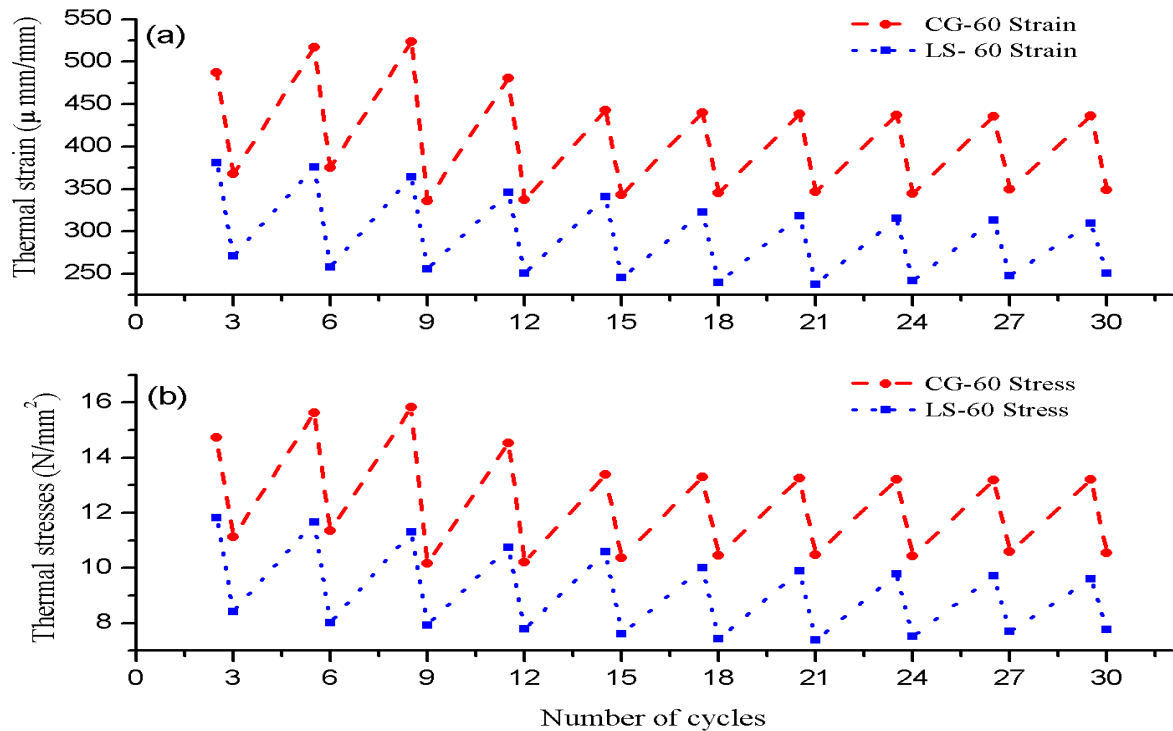


Fig. 5.35: Variation in (a) thermal strain and (b) corresponding stress with heating and cooling cycles in RCC mix containing 60%GGBS

(c) Prediction of failure cycle based upon thermal stresses

In actual practice, it is impossible to apply the heating and cooling cycles to concrete until the failure of the surface due to stochastic nature of the concrete. Therefore, the number of heating and cooling cycles required to produce the failure stress in concrete is calculated with the use of S-N curve (Neville and Brookes, 2010). In S-N curve, S represents the ratio of maximum stress to short term static strength and N represents the number of cycles at failure. In the present study, same S-N curve is used to predict number of failure cycles for RCC mixes produced by using different percentages of GGBS and two types of coarse aggregates. S value is calculated based upon the maximum stress value observed after 30th cycle to 7 days compressive strength value of each mix. The value of S so obtained is summarized in Table 5.6. The numbers of cycles required to failure of RCC surface are calculated with the use of S-N curve for concrete in compression and presented in Table 5.6. It was established that concrete does not have any endurance limit below which no failure can occur (Neville and Brookes, 2010). From the table it was clearly seen that in all RCC mixes failure can occur when the heating and cooling cycles exceed approximately one million of cycles. In case of RCC mixes containing limestone aggregates with the incorporation of GGBS, the failure cycles are more than the corresponding mixes made with crushed gravel aggregates.

Table 5.6: Number of failure cycles calculated based upon S value of different RCC mixes

Aggregate type	Description	Maximum Stress (N/mm²)	7 days C.S. (N/mm²)	S- value	Number of failure cycles (approximately)
Crushed Gravel	CG-0	11.2	30.8	0.36	8×10^8
	CG-20	11.7	28.7	0.41	10^8
	CG-40	12.6	26.7	0.47	5×10^6
	CG-60	13.2	22.0	0.59	10^6
Limestone	LS-0	9.3	34.3	0.27	10^9
	LS-20	9.2	29.4	0.31	9×10^8
	LS-40	9.1	28.1	0.32	9×10^8
	LS-60	9.6	25.5	0.37	10^8

5.6 CLOSING REMARKS

The present chapter focuses on the abrasion resistance, drying shrinkage and thermal shrinkage behaviour of RCC incorporating two types of coarse aggregates and GGBS as cementitious material. Abrasion resistance results show that curing age, aggregate type and percentage of GGBS incorporation into the RCC mix play an important role. Limestone aggregates presents marginally better abrasion resistance as compared to crushed gravel at all curing ages because of better mechanical interlocking of fine particles with the former. Incorporation of GGBS improved the abrasion resistance after 28 days of curing with the superior behaviour was shown by limestone aggregates with 40% GGBS content, which is attributed to combined action of porous structure of limestone aggregates and the fineness of GGBS which help in getting the refined concrete matrix.

Drying shrinkage results show that during the initial drying age, shrinkage occur at faster rate followed by second phase where, it increased with a decreasing rate for all RCC mixes. On the basis of aggregate type, 23% higher shrinkage was observed with crushed gravel aggregates on comparison with limestone aggregates at the end of drying duration of 270 days. GGBS incorporation into the RCC mixes exhibited higher shrinkage strain than the control mix, irrespective of the aggregate type. Due to presence of finer capillary pores in GGBS mixes, lesser amount of free water is available and hence greater capillary pore water pressure is created, leading to higher shrinkage strain.

Thermal shrinkage strain results show that heating the specimens at 70 ± 2 °C accelerated the shrinkage at early duration of heating. At higher temperature, most of the shrinkage strain was registered with in first month of heating, with the final thermal shrinkage strain lower than the corresponding drying shrinkage strain. However, the variation in aggregate type and GGBS incorporations presents the thermal shrinkage behaviour similar with the drying shrinkage behaviour. Prediction made from one month heating and cooling cycles represents that in all RCC mixes failure can occur when the heating and cooling cycles exceeds approximately one million of cycles.

On the basis of statistical analysis of results, an exponential relationship and multiple variable regression model is developed for the prediction of abrasion resistance and drying shrinkage respectively.

CHAPTER 6

CONCLUSIONS AND SCOPE FOR FUTURE WORK

6.1 GENERAL

The present research work aims at exploring the possibility of using supplementary materials as an alternate to conventional materials for roller compacted concrete (RCC). It will lead to overall sustainable development and will make RCC pavement construction more relevant to the local conditions. To achieve this objective, an industrial by- product i.e. ground granulated blast furnace slag was used as a replacement of cement; while locally available coarse aggregates i.e. limestone aggregates were attempted as a replacement of conventional crushed gravel aggregates. The cement was replaced with GGBS at replacement levels of 20%, 40% and 60% with both types of coarse aggregates. The effect of these replacements was studied in terms of various mechanical and in-service properties of RCC.

For both types of aggregates and three incorporation levels of GGBS, a total of eight RCC mixes were designed by using soil compaction approach, as per the provisions laid down in ACI 211.3R. The various properties studied for different RCC mixes include; compressive strength, split tensile strength, flexural strength, abrasion resistance, drying shrinkage and thermal shrinkage. For compressive strength and split tensile strength measurements; cylindrical specimens of size 150×300 mm were prepared; while flexural strength was measured on beam specimens of size 150×150×700 mm. For abrasion resistance measurements, RCC blocks of size 65×65×30 mm were cast. Drying shrinkage and thermal shrinkage was studied on prism specimens of size 75×75×290mm. During the preparation of each specimen, a surcharge of 4.9 kPa was used to facilitate compaction, as described in ACTM C 1176. The application of this surcharge corresponds to the effort applied by the rollers in the field.

To support the compressive strength behaviour, microstructure and stoichiometric analysis was carried out for all the mixes. Compressive strength being an essential property of concrete, thus regression models relating various mechanical properties to compressive strength of concrete were developed. Along with this, a non-linear multi regression model was developed for the prediction of drying shrinkage of RCC. The present chapter deals with the major conclusion drawn from the work.

6.2 ROLLER COMPACTED CONCRETE: MATERIAL DESIGN

- 1) Soil Compaction approach is comparable to the field application of RCC in pavements since the compaction applied by surcharge in laboratory can correspond to the effort applied by the rollers in the field.
- 2) The aggregate proportions consisting of 55% of fine aggregates, 30% of 10 mm coarse aggregates and 15% of 20 mm coarse aggregates achieved the maximum density with the higher correlation coefficient of 0.982 in the density gradation curve. Obtained higher correlation coefficient represents that the aggregate particles fit together in their densest possible arrangement along the gradation line.
- 3) Mix containing 16% of cement content as total dry mass of coarse and fine aggregates gave the highest value of 28-day compressive strength.
- 4) Compaction results show that limestone aggregates increased water requirement of the RCC mix as compared to the mixes containing crushed gravel aggregates because of higher water absorption of limestone aggregates. Further addition of GGBS as partial replacement of cement increased the optimum water content because of higher fineness modulus of GGBS.
- 5) All the designed mixes had 28 days compressive strength values greater than 27.6 MPa, indicating that all the prepared mixes can be used as a surface course.

6.3 STRENGTH PROPERTIES OF ROLLER COMPACTED CONCRETE

6.3.1 Compressive Strength

- 1) RCC mixes incorporating limestone aggregates register higher value of early age compressive strength as compared to the mixes incorporating crushed gravel aggregates. However, at later testing age of 365 days, mixes incorporating crushed gravel aggregates presents 9% higher strength as compared to RCC mixes with limestone aggregates. This behaviour can be attributed to the accelerated hydration occurring in mixes with limestone aggregates; while in mixes incorporating crushed gravel aggregates, hydration occurred at a steady rate.
- 2) Incorporation of GGBS into the RCC mixes led to lower strength at early ages, followed by strength higher than the control mix at later ages. In the case of mixes incorporating crushed gravel aggregates, addition of 20% and 40% GGBS gives the strength higher than the control

mix after 28 days of curing, while 60% GGBS incorporation registers strength comparable with the control mix. On the other hand, in RCC mixes incorporating limestone aggregates, strength higher than the control mix was registered at all replacement levels of cement with GGBS after 28 days of curing period.

- 3) The RCC mix having a combination of limestone aggregates and 40% GGBS presented the highest compressive strength value of 57.8 N/mm^2 i.e. 28% higher than the control mix.
- 4) The stoichiometric analysis showed that the upper limit of substitution of cement by GGBS can be taken as 49.4%, clearly indicating that GGBS acts as major contributor towards strength development by chemical reactions till 50% replacement level and afterward it will simply act as filler. The SEM analysis also support that the addition of GGBS influence the interfacial transition zone present between aggregates and hardened cement paste.

6.3.2 Split Tensile Strength and Flexural Strength

- 1) Split tensile strength and flexural strength results show that the variation in aggregate type did not have any significant effect on tensile and flexural strength of RCC. It is due to the fact that tensile strength is affected by the quality of paste and interfacial transition zone in concrete rather than on the properties of aggregates itself.
- 2) Similar to the compressive strength behaviour, addition of GGBS had a negative impact on split tensile strength and flexural strength of RCC during the initial curing age. With the enhancement in curing period, markedly higher strengths was observed with 40% GGBS incorporation level.
- 3) On the basis of statistical analysis, a power series relationship is proposed to correlate split tensile strength and flexural strength with the compressive strength of RCC mix. Both the proposed relationships have higher correlation coefficient indicating the effectiveness of the proposed relationships.

6.4 IN-SERVICE PERFORMANCE OF ROLLER COMPACTED CONCRETE

6.4.1 Abrasion Resistance

- 1) Abrasion resistance of RCC is influenced by various factors such as curing age, aggregate type, GGBS content and compressive strength of the mix.

- 2) Enhancement in curing period to 365 days reduced the depth of wear by approximately 75% as compared to depth of wear at 3 days of curing. Improvement in abrasion resistance with age can be primarily attributed to the increased maturity of concrete.
- 3) Due to better mechanical interlocking of fine particles with the limestone aggregates, RCC mixes incorporating limestone aggregates presents marginally better abrasion resistance as compared to the mixes incorporating crushed gravel aggregates, at all curing ages. At 365 days of curing, 20% higher depth of wear was registered by RCC mixes having crushed gravel aggregates as compared to limestone aggregates.
- 4) Improvement in abrasion resistance behaviour was observed at all replacement levels of cement with GGBS after 28 days of curing with the superior behaviour shown by RCC mix having limestone aggregate with 40% GGBS content. This behavior is attributed to combined action of porous structure of limestone aggregates and the fineness of GGBS which help in getting the refined concrete matrix.
- 5) An exponential relationship was established between compressive strength and depth of wear with correlation coefficient (R^2) higher than 0.9 for both types of aggregate. It indicates that abrasion resistance of RCC strongly depends upon the compressive strength.

6.4.2 Drying Shrinkage

- 1) Drying shrinkage results demonstrate that shrinkage strain development curve consists of two phases; in first phase, shrinkage occurred at faster rate (initial 90 days) followed by a second phase where, the shrinkage increased at the decreasing rate for all mixes.
- 2) RCC made with crushed gravel aggregates experienced 23% higher shrinkage strain as compared to the mixes with limestone aggregates, which can be attributed to lower modulus of elasticity of crushed gravel as compared to limestone.
- 3) GGBS incorporation into the RCC mixes exhibited higher shrinkage strain than the control mix, irrespective of the aggregate type. The least shrinkage strain was observed in the mix having limestone aggregates and 20% GGBS as partial replacement of cement.
- 4) A multiple variable regression model is developed predicting drying shrinkage of RCC incorporating parameters related to aggregate type, percentage level of GGBS and 28-day compressive strength of the mix. A high correlation coefficient (R^2) obtained between the experimental and predicted values by using developed model indicate the effectiveness of the

proposed model for RCC mix incorporation GGBS, a particular type of aggregates, with cement content ranging between 300-350 kg/m³ and at w/c ratio of 0.35-0.45.

6.4.3 Thermal Shrinkage

- 1) Thermal shrinkage strain results show that heating the specimens at 70±2°C accelerated the shrinkage at early duration of heating. Continuously heating of specimens lead to the early formation of the hydration products by making the alteration in composition and internal microstructure of concrete.
- 2) The variation in aggregate type and incorporation of GGBS affects the thermal shrinkage strain of RCC mixes. The addition of GGBS leads to higher thermal shrinkage strain as compared to drying shrinkage strain at early testing duration irrespective of aggregate type; while at 270 days of testing duration, approximately 20-35% lower thermal shrinkage strain was registered as compared to drying shrinkage strain.
- 3) The variation in thermal strain obtained after each heating and cooling cycle was reduced with the progressive increase in heating and cooling cycles irrespective of aggregate type and GGBS content. Further, the failure cycles predicted from the S-N curve shows that all RCC mixes could undergo one million heating and cooling cycles, indicating their effectiveness as RCC pavements.

6.5 SCOPE FOR FUTURE WORK

Some issues that require further investigation are summarized as under:

- 1) The incorporation of GGBS had a negative effect on all the properties of RCC at early age of curing, thus the possible ternary combination can be explored to improve early age properties of RCC.
- 2) In order to correlate the laboratory results with the field application of RCC, actual RCC pavement can be constructed.
- 3) The effect of waste generated from the demolished structures as an aggregate on various properties of RCC can be investigated.

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