STUDY OF COARSE RAP USING DIFFERENT MINERAL ADMIXTURES IN PAVEMENT QUALITY CONCRETE

A Dissertation submitted in fulfillment of the Requirement for the award of the degree of

MASTER OF ENGINEERING

In INFRASTRUCTURE ENGINEERING

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DECLARATION

I, Imran Altaf Wasil hereby declare that the work presented in this thesis entitled **"STUDY OF COARSE RAP USING DIFFERENT MINERAL ADMIXTURES IN PAVEMENT QUALITY CONCRETE"** in fulfillment of the requirement for the award of degree of Master of Engineering in Infrastructure Engineering submitted at Civil Engineering Department, Thapar Institute of Engineering & Technology (Deemed to be University), Patiala is an authentic record of work carried out under supervision of Dr. Prem Pal Bansal (Professor & HOD, Civil Engineering Department, Thapar University), Dr. Tanuj Chopra (Assistant Professor, Civil Engineering Department, Thapar University) & Sh. Dinesh Ganvir (Principal Scientist & HOD, Rigid Pavement Division, CSIR-CRRI) from 2021 to 2022 . The matter presented in this has not been submitted either in part or full to any other university or institute for the award of any other degree.

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ABSTRACT

Due to a scarcity of virgin aggregates, the use of reclaimed asphalt pavement (RAP) as a substitute for natural aggregates has gained popularity. The present study includes the use of new processing technique in order to remove the agglomerated particles and reduce the binder content present in RAP aggregates. Therefore, experimentally investigating the strength, durability, and time dependent properties of coarse RAP for use in Pavement quality concrete (PQC) and to study the properties of PQC made with different percentages of RAP along with the effect of various supplementary cementitious material like Silica Fume (SF) and Ground Granulated Blast Furnace slag (GGBS) on various properties of RAP. A total of 12 concrete mixes with various proportions of coarse RAP (20%, 40% and 60%) and a fixed percentage (10% and 40%) of SF and GGBS were prepared to enhance the interfacial transition zone in PQC. Study of fresh properties like slump value and compaction factor, hardened properties like density, mechanical properties like compressive strength, split tensile strength and flexural strength, durability properties such as water absorption, abrasion resistance and sand blasting, time dependent properties such as shrinkage and non-destructive properties like rebound hammer and ultrasonic pulse velocity (UPV) of PQC was carried out in the present study. The new processing technique enhanced the properties of RAP inclusive PQC mixes and the strength of all the concrete mixes was above the design target strength. It was observed that there was a reduction of more than 60% in binder content of RAP aggregates as a result of processing of RAP. Partial replacement of cement by SF and GGBS also enhanced the properties of concrete mixes. However, with the increase in the percentage of replacement levels the properties of concrete mixes got reduced but the reduction was very less as compared to the previous researches. As per the study RAP can be use in PQC up to 60% after processing technique is done. Hence, this study proves to be effective in utilization of RAP aggregates as a replacement of Natural aggregates in PQC.

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LIST OF ABBREVIATIONS

RAP	Reclaimed Asphalt Pavement
NA	Natural Aggregate
NC	Natural Coarse
NF	Natural Fine
CRAP	Coarse Reclaimed Asphalt Pavement
FRAP	Fine Reclaimed Asphalt Pavement
SF	Silica Fume
GGBS	Glass Granulated Blast Furnace Slag
PQC	Pavement Quality Concrete
AB&AT	Abrasion and Attrition Technique
SP	Superplasticizer
SSD	Saturated Surface Dry Condition
UPV	Ultrasonic pulse velocity
BPT	British Pendulum Test

LIST OF SYMBOLS

P	Maximum load (N)
A_c	Area of cross-section
l	Span on which beam is supported
a	Distance between line of fracture and nearest support (mm)
b	Width of specimen (mm)
d	Depth of specimen (mm)
D	Diameter of specimen (mm)
L	Length of specimen (mm)
V	Pulse Velocity (km/s)
L’	Path Length (km)
T	Transmission time (seconds)

CHAPTER 1

1 INTRODUCTION

1.1 GENERAL

Urbanization and the expansion of highway infrastructure have resulted in a depletion of available natural aggregates. With the shift toward green and sustainable development and the government's prohibition on quarrying, the use of alternative materials for construction work has become an inevitable requirement (Al-Mufti and Fried, 2018; Dubey et al., 2020; Mohammadinia et al., 2015; Pranav et al., 2020). Hence, it is essential to make use of waste material in order to reduce the use of natural resources (Ameli, n.d.; Settari et al., 2015; Singh et al., 2019; Tamanna et al., 2020; Tavakoli et al., 2020). On the other hand, global production of solid waste from construction demolition (roads and buildings related structures) has peaked in the last decade, and its disposal has become a global issue (Kabir et al., 2016; Singh et al., 2020).

RAP is one such solid waste product that can be used in highway construction as aggregate. It is increasingly being used for a variety of pavement applications and is quickly becoming a global trend (Aurangzeb et al., 2014; Debbarma et al., 2020b; Shi et al., 2020; Su et al., 2009). However, in developing countries such as India, its effective utilisation is limited due to hesitation among highway engineers as there is lack of proper codal provisions despite its numerous benefits, which include a reduction in landfill matter and carbon footprint, cost-effectiveness, and the preservation of virgin aggregates for future need (Debbarma et al., 2020a, 2019a, 2019b; Shi et al., 2018; Singh and R.N., 2020). RAP is a by-product of the milling of distressed flexible asphalt pavement (Arulrajah, 2014; Debbarma et al., 2019a, 2020a; Farina, 2016; Kumari et al., 2018b; Mohammadinia et al., 2015; Monu et al., 2019; Singh et al., 2020; Taha et al., 1999; Ullah et al., 2018; Ullah and Tanyu, 2019). It is also obtained as a by-product of road/highway rehabilitations and resurfacing in cases of the exploration of beneath-surface layers (Dubey et al., 2020; Pokorný et al., 2020). RAP is not only cost-effective, but it also aims to meet the demands of sustainability and green technology (Dubey et al., 2020; Saliani et al., n.d.; Shi et al., 2019b). Some of the demolished RAP has been used in flexible pavements as granular and surface courses. However, there is still a significant amount of material that has not been used as a result, its use in concrete pavement mixes has skyrocketed in recent years. (Dubey et al., 2020; Shi et al., 2020, 2018). If these wastes are not properly managed and repurposed, they will result in an unprecedented increase in waste generation, posing a number of environmental concerns (Debbarma and Ransinchung R.N., 2021; Shi et al., 2021). Although recycled asphalt pavement has been used in some applications, a large amount of it is either stockpiled or discarded in landfills, which does not reflect the true value of RAP as a construction material (Khodair and Raza, 2017).

The amount of waste produced annually in the European community is estimated to be 3.0 billion tonnes, with construction and demolition waste (C&DW) accounting for 40% of the total; C&DW is primarily composed of concrete, asphalt, and masonry (Settari et al., 2015; Silva et al., 2014). Asphalt pavement makes up about 80% of the world's total road networks, with India having the second-largest network after the United States. According to the Federal Highway Administration, resurfacing flexible pavement roads results in the accumulation of 91 million metric tonnes of RAP aggregate in the United States (Singh et al., 2020; Xiao et al., 2009). In India, the total road network is approximately 4.69 million kilometres. According to MORTH, 2.53 million kilometres of roads have flexible pavement (2012). According to (Cardoso et al., 2016; Singh et al., 2020), flexible pavements contain 90% Natural Aggregate, and once the flexible pavement ages and deteriorates, the aggregates are discarded as solid waste.

To produce concrete mixes, RAP has been used in three different fractions: coarse RAP, fine RAP, and combined RAP, respectively. Fine RAP is defined as RAP that passes the 4.75 mm standard sieve, whereas coarse RAP is defined as RAP that is larger than 4.75 mm but smaller than 25 mm. A small number of researchers have looked into the use of combined RAP, which combines the coarse and fine fractions. Regardless of the type of RAP fractions used, all researchers reported that the concrete's mechanical strength could be reduced by about 10-81 percent (Abraham and Ransinchung, 2018; Brand et al., 2014; Debbarma and Ransinchung R.N., 2021; El Euch Ben Said et al., 2018; Erdem and Blankson, 2014; Hassan et al., 2000; Hossiney et al., 2010; Huang et al., 2006).

RAP is made up of high-quality, well-graded aggregates coated in asphalt cement when properly crushed and screened (Kumar et al., 2019). The main cause of the reduction in concrete strength has been attributed to the lower specific gravity of RAP and the formation of poor bonding between RAP and mortar(Ben Saïd et al., 2017; Debbarma and Ransinchung R.N., 2021). RAP aggregates typically contain large amounts of dust, agglomerate particles, and aged asphalt, which prevent binder material from binding with them; thus, contaminants can be removed using mechanical roughening techniques such as abrasion and attrition, or chemical solvents such as turpentine (Al-Mufti and Fried, 2017; Singh et al., 2020; Thakur et al., 2012). Furthermore, a bitumen extractor can be used to remove asphalt content in aggregate; however, this requires a significant amount of energy and capital. The dust and aged asphalt agglomerated particles are the result of the oxidation process during stockpiling (De Lira et al., 2015; Singh et al., 2020). The main impediment to RAP's effective use is a lack of RAP aggregate–mortar bonding, which allows for the formation of a porous interfacial transition zone (ITZ) and poor interlocking between mortar paste and RAP, resulting in inferior properties (Brand and Roesler, 2017; Debbarma et al., 2020a; Huang et al., 2006, 2005; Shi et al., 2019a).

1.2 NEED FOR RESEARCH

Based on the results of comprehensive literature review, researchers have used RAP as a partial replacement for NA in cement concrete mixes, either directly or simply washed. Although there have been few or no attempts in the past to remove or puncture the asphalt encasing the RAP aggregates. Also, because of the drastic reduction in properties of RAP concrete compared to conventional concrete, the majority of the authors have recommended using it for non-structural applications. However, a few researchers have proposed using RAP in the surface course, but with a 50 percent capping limit. However, because of the poor bonding between RAP aggregates and cementitious matrix, SF inclusion in RAP concrete is not recommended. However, the present study consists of using an improvement technique in order to remove the asphalt film from RAP aggregates. As a result, the incorporation of mineral admixtures may lead to further improvements in properties. Also, many studies have been conducted on the use of mineral admixtures in natural aggregates concrete and recycled concrete aggregates, but their beneficial effects for RAP inclusive concrete have not been thoroughly investigated. Also, till now no research has been done on the use of GGBS in RAP inclusive concrete. Therefore, this dissertation focuses on the study of use of beneficiated RAP as coarse aggregate in cement concrete pavement with addition of SF and GGBS.

1.3 OBJECTIVES

The objective of present study is to experimentally investigate the strength, durability and long-term properties of coarse RAP for use in PQC.

1.4 SCOPE

To achieve the above objective, the scope of the present study comprises of:

1. Characterization of coarse RAP.
2. Mix design of Pavement Quality Concrete with addition of different percentages of RAP.

3. To study the mechanical, durability and long-term properties of PQC made with different percentages of RAP.
4. To study the effect of various supplementary cementitious material i.e., SF and GGBS on mechanical, durability and long-term properties of RAP.

1.5 METHODOLOGY

The methodology followed in this study is stated below:

1. Production of RAP from a large stockpile obtained through ripping and crushing from distressed old flexible pavement (more than 25 years old) with a mixture of both wearing course and bituminous macadam.
2. Introducing a new processing technique in order to remove the contaminants like dust film, asphalt film and agglomerated particles from RAP aggregate. This beneficiation technique is termed as "Abrasion and Attrition Technique".
3. Experimentally characterizing the coarse RAP produced and comparison of the same with that of NA. The properties include particle size distribution and gradation, specific gravity, water absorption, bulk density, abrasion resistance, impact resistance, crushing resistance, asphalt content.
4. Mix design of PQC by modifying the NA concrete mix designed according to IRC: 44 – 2017.
5. Production of concrete with various proportions of coarse RAP (20%, 40% and 60%) and a fixed percentage (40% and 10%) of SF and GGBS to enhance the interfacial transition zone in PQC.
6. Study of fresh properties like slump value and compaction factor, hardened properties like density, mechanical properties like compressive strength, split tensile strength and flexural strength, durability properties such as water absorption, abrasion resistance and sand blasting, time dependent properties such as shrinkage and non-destructive properties like rebound hammer and ultrasonic pulse velocity (UPV) of PQC and comparing these properties with those of NA concrete.
7. Study the effect of supplementary cementitious material like SF and GGBS on mechanical, durability and long-term properties of PQC with a fixed percentage of 10% and 40% respectively along with different replacement level of coarse RAP (20%, 40% and 60%).

CHAPTER 2

2 LITERATURE REVIEW

2.1 INTRODUCTION

Due to a scarcity of virgin aggregates, the use of reclaimed asphalt pavement (RAP) as a substitute for natural aggregates has gained popularity. Despite the fact that RAP is recycled in asphalt pavement, there is still excess RAP, and its use in concrete pavements has expanded in recent years (Debbarma and Ransinchung R.N., 2021). According to a survey, 98 percent of India's pavements are flexible. As a result, the maintenance and reconstruction of such pavements generate RAP, which can be reused in concrete pavements as well as surface course, base course, and subbase of flexible pavements (Abraham and Ransinchung, 2018). Various studies on the properties of reclaimed asphalt pavement and its optimal requirements for usage in concrete has been conducted throughout the years.

In this chapter review of various aspects such as removal techniques of Asphalt pavement, properties of RAP are presented.

2.2 Beneficiation Technique used in RAP: Attrition and Abrasion Technique (AB & AT):

RAP was found to be made up of three distinct layers of contaminant, including asphalt film, stiff dust film, and loose dust film, according to the laboratory analysis. To provide a strong bonding between mineral aggregate and hydrated cement paste, these contamination layers must be removed. In this context, it was proposed to devise a "(Abrasion and Attrition (AB&AT)" technique for removing the aforesaid films enclosed around the aggregate to a large extent. Furthermore, this method allows for effective puncturing of coated asphalt sheet, ensuring good bonding between aggregate particles and cement matrix. The schematic view of the AB&AT set-up is depicted in Figure 5 (Singh et al., 2017).

2.2.1 Principle

The hardening/ stiffening of the asphalt layer caused by the ageing of the binder during its life and stockpiling(De Lira et al., 2015) results in its removal when a small force is applied, and this principle is used to remove some patches of asphalt coating around the RAP aggregates (Ransinchung et al., 2019)**.**

2.2.2 Machinery Details

The 'titling drum type mixer' was chosen for beneficiation since mixers are readily available at all building sites. The mixer had a 160 kg capacity and a 0.45 m diameter opening (charging and discharging through the same hole) with a 0.6 m drum length. The mixer was powered by an electric motor with a capacity of 5 HP (Horse Power). As an abrasive charge, steel balls with an average diameter of 4.5 cm and a weight of 380 g were utilised to create mechanical stresses on the surface of RAP aggregates(Singh et al., 2017).

2.2.3 Optimisation of Method

The amount of aggregates fed into the mixer is determined by the angle of tilt, mixer speed, mixing time, and material passing a 4.75 mm sieve, while the number of charge balls is determined by the gradation of the processed material and total asphalt removed. To ensure maximum contact between the aggregate surface and the abrasive charge, the angle of tilt of the mixture was kept constant at 10 degrees. This ensured a high level of attrition and abrasion. The mixer could rotate at a rate of 35 times per minute and was held at that speed throughout the study. The mixing time was kept constant (15 minutes) because a longer time could result in broken and spherical aggregates, which would reduce load transfer efficiency(Ransinchung et al., 2019).

The percentage passing (dust + agglomerated particles) through a 4.75 mm sieve due to attrition between CRAP aggregates is shown in the Table 2.1 The maximum attrition between the aggregates occurred when the input quantity of CRAP aggregates in the drum mixer was 25 kg, as shown in Table 2.1, and was kept constant for determining the optimum number of charging balls. As determined by ASTM D2172, asphalt content remained after processing with various numbers of charge balls (0–20). It was discovered that increasing the number of charging balls past 15 had no influence on the asphalt composition. Based on the above findings, it was determined that 25 kg of DRAP and 15 charging balls were the best amounts to feed into the mixer. The CRAP was processed using the AB&AT procedure, which totally removed both top layers and punctured the asphalt covering at several areas. RAP's asphalt content has been found to be reduced by more than half after processing.(Singh et al., 2017). Figure 2.1 and Figure 2.2 shows a schematic diagram of Abrasion and Attrition RAP and graph between Asphalt content and No. of Charge Balls. Figure 2.3 shows the variation of CRAP with percentage passing through 4.75 mm sieve.

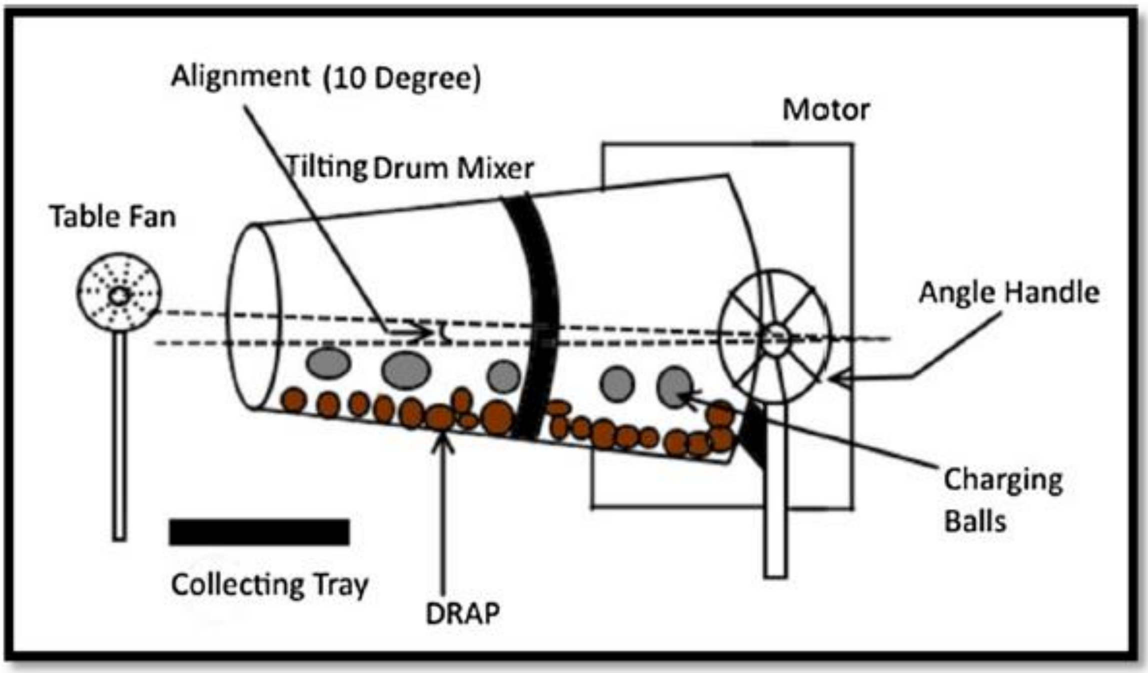

Figure 2.1 AB & AT set up (Source: (Ransinchung et al., 2019))

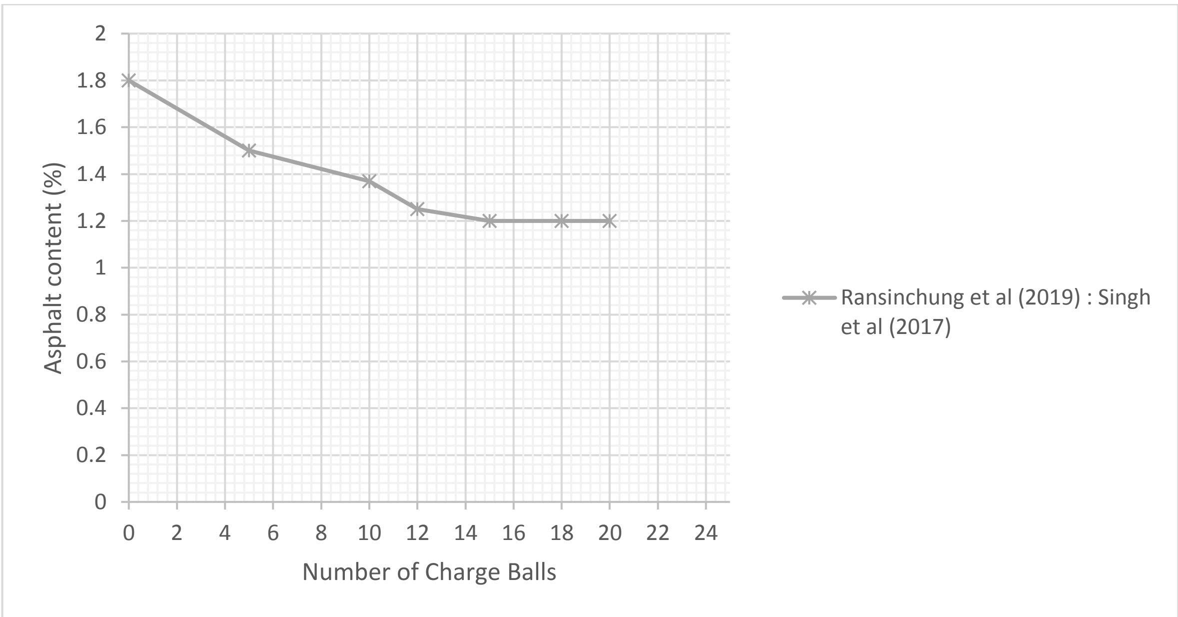

Figure 2.2 Asphalt content vs No. of Charge Balls (Source: (Ransinchung et al., 2019))

Table 2.1 Percentage passing through 4.75 mm sieve

CRAP (Kg)	Passing through 4.75 mm sieve
15	4.35
20	5.97
25	7.03
30	6.26

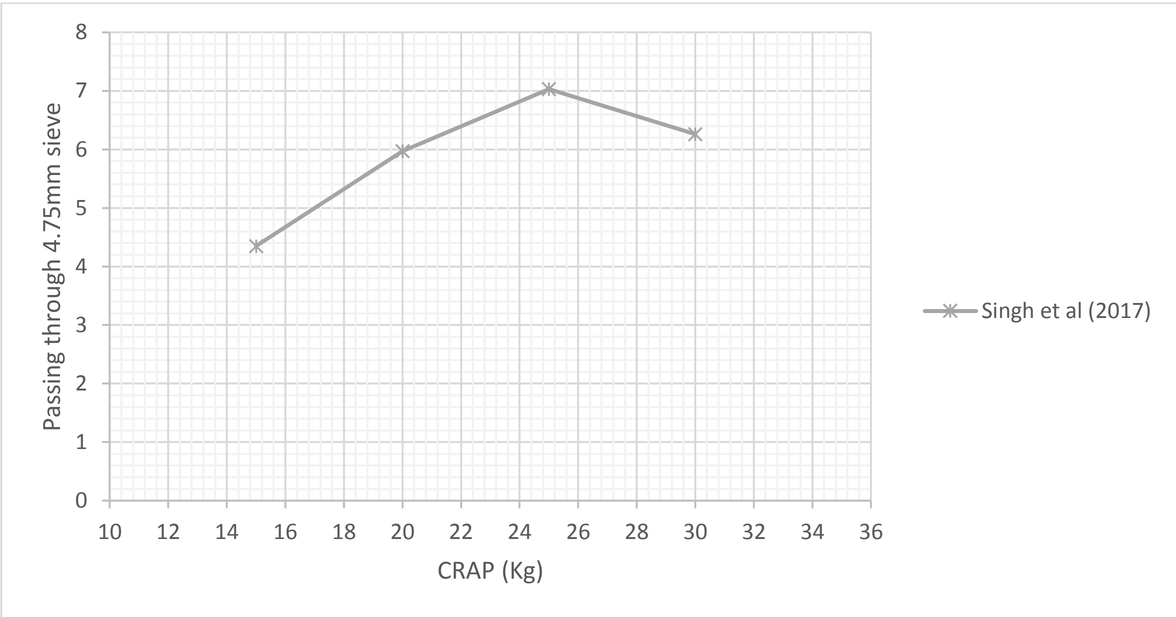

Figure 2.3 CRAP vs % passing through 4.75 mm sieve

2.3 Silica Fume

The silicon and ferrosilicon alloy industries produce SF as a by-product. Because it contains more than 85% reactive silica, it is considered the most reactive pozzolanic mineral admixture. The high specific surface area (>18000 cm^2/gm) of amorphous silica, combined with the abundant amount of amorphous silica, densifies the microstructure of concrete, making it suitable for high strength concrete applications (Ransinchung et al., 2017). While replacing OPC with SF reduces workability, it has been shown to greatly improve the mechanical and durability qualities of concrete. It not only improves the durability of concrete, but it also improves its abrasion resistance. Mineral admixtures improve the properties of concrete in two ways: first, by producing more CSH gel as a result of the reaction between calcium hydroxide (CH) (hydration by product) and reactive silica from admixtures; and second, by providing a filler effect due to finer particle size than OPC, which further fills the space left after the hydration process, densifying the ITZ (Ransinchung et al., 2017).

The advantages of using SF for concrete has threefold benefits:

a. Densification of the microstructure
b. Creating additional CSH gel by reacting with portlandite.
c. Densifying the Interfacial Transition Zone (ITZ).

Silica fume is defined by the American Concrete Institute (ACI) as an "extremely fine non-crystalline silica produced in electric arc furnaces as a by-product of the manufacturing of elemental silicon or silicon alloys." It's typically a grey powder that looks like Portland cement or some fly ashes. Both pozzolanic and cementitious qualities can be found in it (Siddique and Iqbal Khan, 2011).

Silica fume has been identified as a pozzolanic admixture capable of significantly improving mechanical characteristics. It is relatively easier to obtain compressive strengths of the order of 100–150 MPa in the laboratory by combining silica fume with superplasticizers (Siddique and Iqbal Khan, 2011).

2.3.1 Availability and Handling

Dry and wet silica fumes are available. Dry silica can be supplied as produced or densified, with or without dry admixtures, and kept in silos and hoppers. Chemically admixed silica fume slurry is available in low and high dosages. Tanks hold slurried products (Siddique and Iqbal Khan, 2011).

2.3.2 Physical Properties of Silica Fume

The particles in silica fume are relatively small, with more than 95% of them being finer than 1 µm. The table below lists its normal physical features. The colour of silica fume is either premium white or grey. Table 2.2 represents various physical properties of Silica Fume.

Table 2.2 Physical Properties of Silica Fume

Property	**Value**
Particle Size	1 µm
Bulk Density (As produced)	130-430 kg/m^3
Bulk Density (Slurry)	1320-1440 kg/m^3
Bulk Density (Densified)	480-270 kg/m^3
Specific Gravity	2.22

Surface Area	13,000-30,000 m^2/kg

2.3.3 Chemical composition

Silica fume is mostly made up of non-crystalline pure silica. The material is essentially vitreous silica, mostly of the cristobalite form, according to X-ray diffraction studies of different silica gases. Silica fume contains a lot of amorphous silicon dioxide and is made up of tiny spherical particles. In most cases, silica fume includes more than 90 percent SiO_2. Iron, magnesium, and alkali oxides are also found in small concentrations (Siddique and Iqbal Khan, 2011).

2.4 PROPERTIES OF COARSE AND FINE RECLAIMED ASPHALT PAVEMENT (RAP)

2.4.1 Physical properties

RAP is considered as a 4 phase system which consist of aggregate, asphalt film, stiff dust film and loose dust film making a two transition phase, one from aggregate to asphalt coating and second from asphalt film to dust layer (Singh et al., 2017). On the other hand, Natural aggregates are considered as a single-phase system.

2.4.1.1 Specific gravity and water absorption

For comparison, the specific gravity and water absorption of CRAP and FRAP as determined by various studies, as well as the corresponding value of NA, are plotted in the Figure 2.4 and Figure 2.5. The average specific gravity of CRAP was found to be 2.45 while that of NA varies between 2.6 to 2.8. The average specific gravity of FRAP was found to be 2.10 while that of NF varies between 2.6 to 2.75. The specific gravity of NA was found to be higher than that of CRAP and FRAP, as expected. This is due to the presence of agglomerated particles and a lower density asphalt covering in RAP aggregate (Kumari et al., 2018a).

As seen from the Figure 2.6 and Figure 2.7 the average water absorption of CRAP and FRAP was found to be 1.21% and 1.22%. In some cases, the water absorption of RAP is higher than that of NA and NF. This is due to the fact that there is a presence of dust layer around the RAP aggregates that soaked more water (Kumari et al., 2018a). Whereas in some cases the water absorption of RAP is lower than that of NA and NF. The reason behind this is hydrophobic nature of asphalt coating of RAP aggregates (Kumari et al., 2018a).

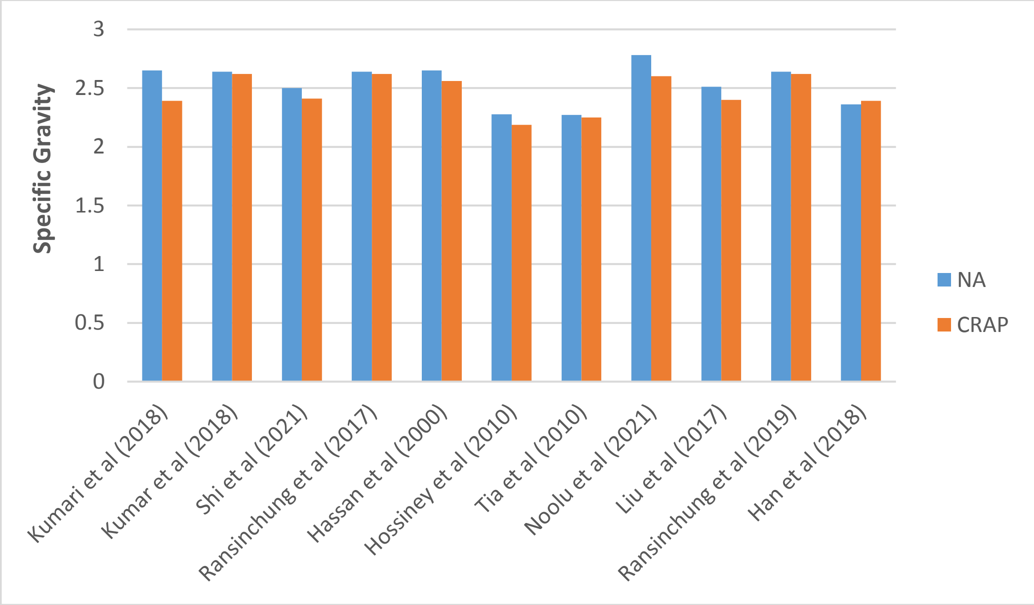

Figure 2.4 Specific gravity of CRAP

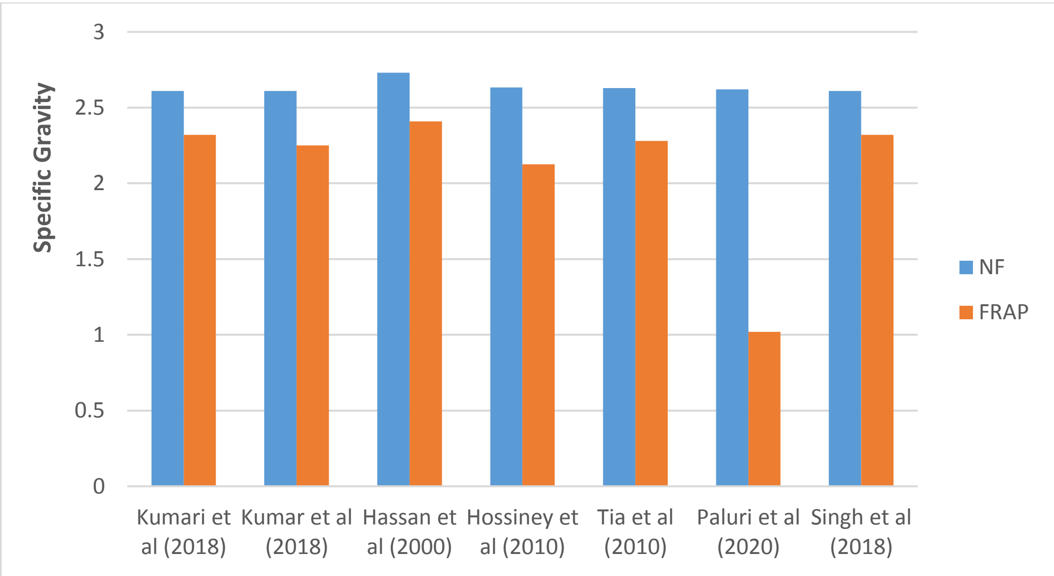

Figure 2.5 Specific gravity of FRAP

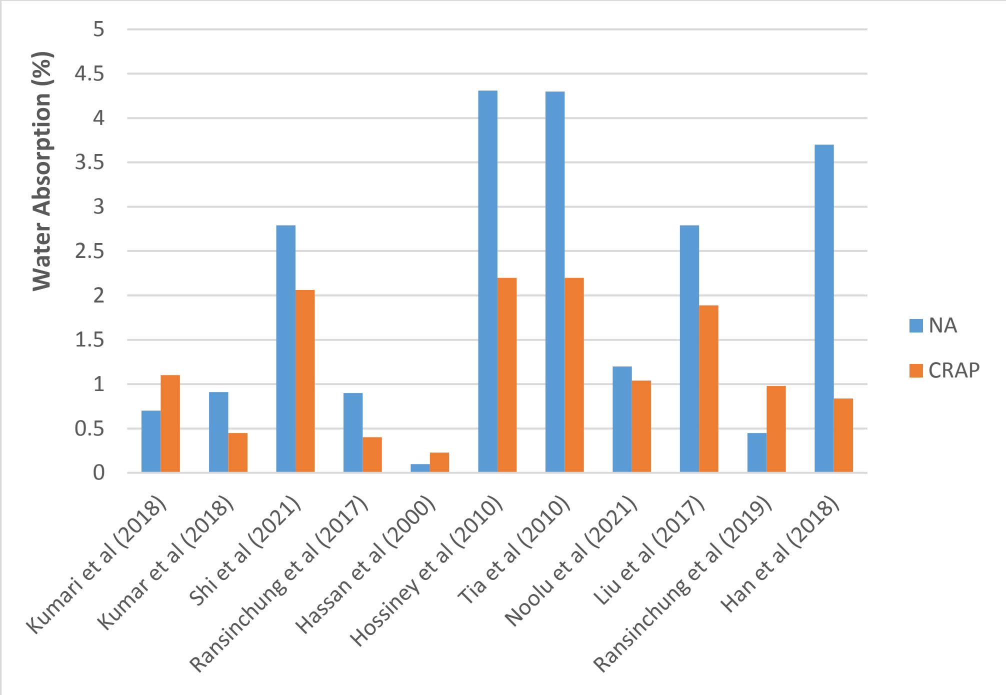

Figure 2.6 Water absorption of CRAP

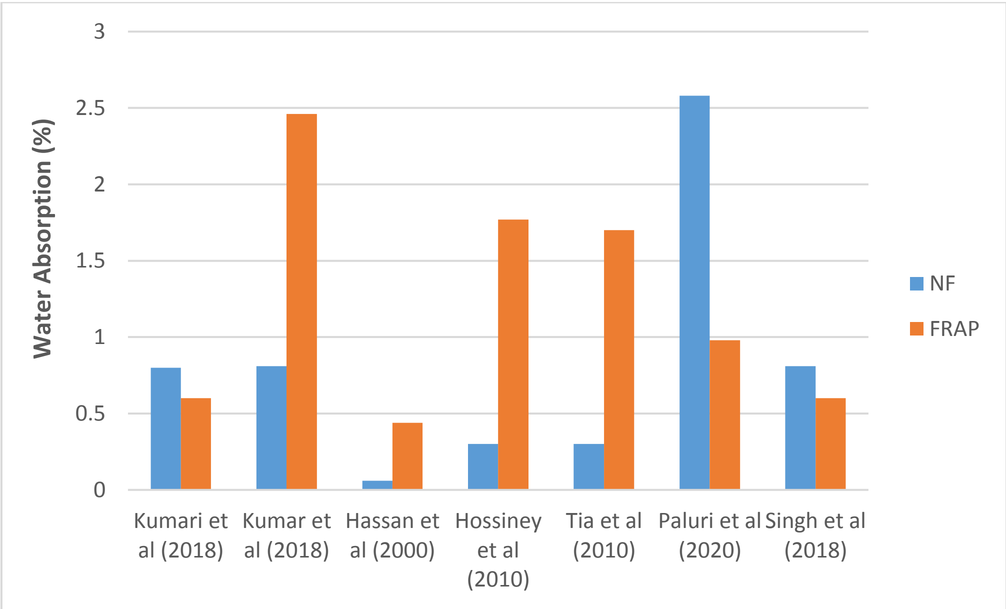

Figure 2.7 Water absorption of FRAP

The inverse relationship between water absorption and specific gravity is well known, and the same holds true for CRAP and FRAP. In addition, the Figure 2.8 and Figure 2.9 depicts the relationship between the data in the literature. The scatter in the plot could be attributable to an unexplained difference in the porosity of the cement asphalt bond.

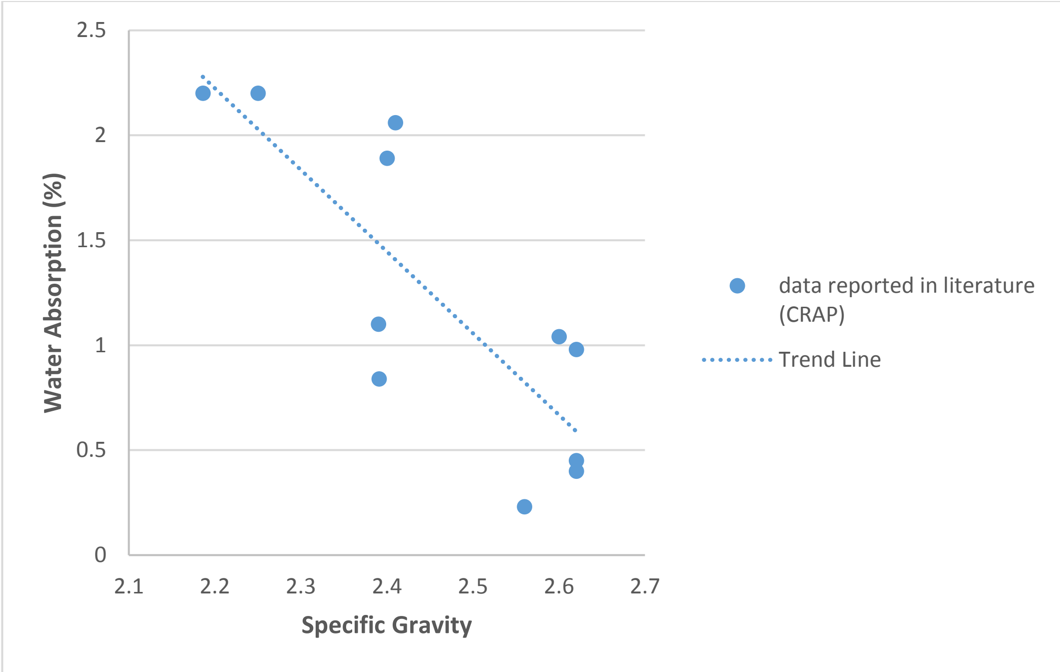

Figure 2.8 Trend Line of CRAP

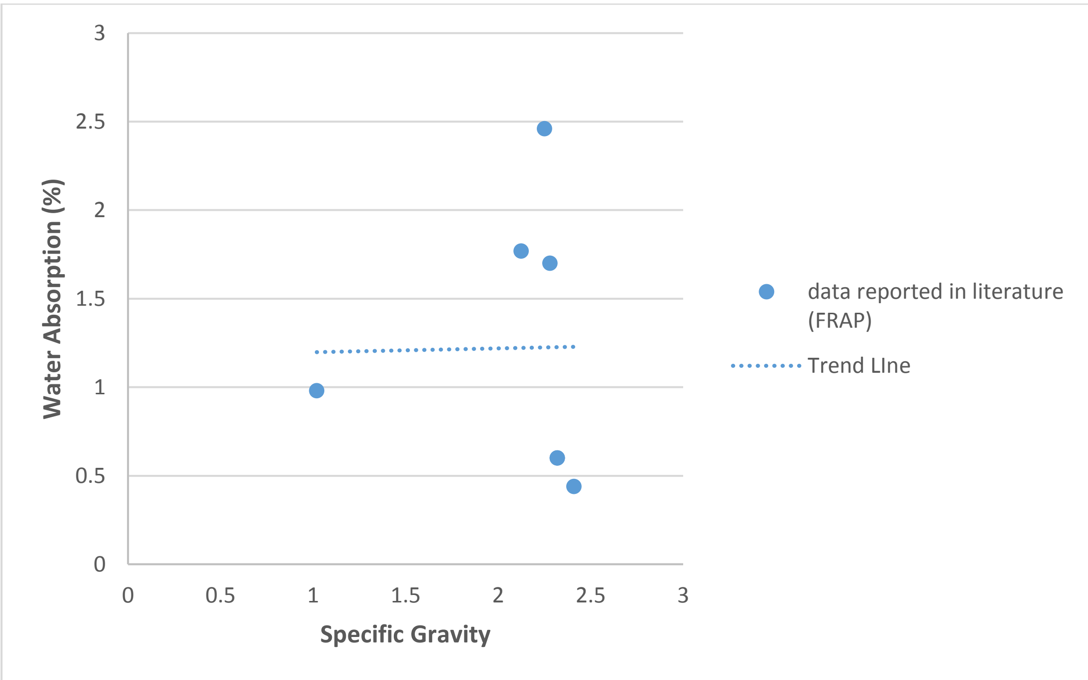

Figure 2.9 Trend Line of FRAP

2.4.1.2 Asphalt Content

AS per study done by various researchers the asphalt content of CRAP and FRAP is shown in the Figure 2.10. It is observed that the asphalt content of FRAP is more than that of CRAP. It is due to the fact that the thickness of the asphalt film on the FRAP is more than that of CRAP. Also, the average asphalt content of CRAP and FRAP was found to be 3.4% and 6.85%.

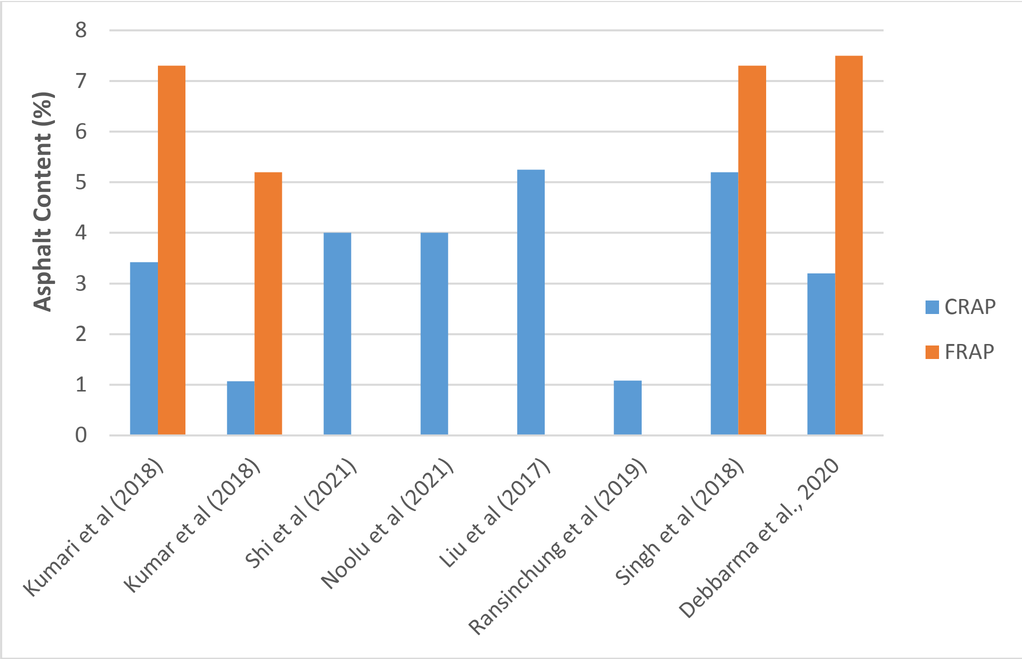

Figure 2.10 Asphalt Content of CRAP and FRAP

2.4.1.3 Particle shape and gradation of RAP

The standard RAPs have a higher sphericity and include less flat and elongated particles than the virgin coarse aggregate, according to the test results. It should be mentioned that the presence of less flat and elongated particles in dense-graded PQC mixtures makes it easier to achieve superior workability and mechanical qualities. However, some RAPs had lesser sphericity, higher flat and elongated distribution values, and the highest angularity, which could be attributable to a higher amount of agglomerated particles in this RAP (Liu et al., 2017).

2.4.1.4 Bulk Density

The Figure 2.11 depicts a comparison of NA and CRAP bulk densities as reported by several studies. CRAP had lower bulk densities than NA, ranging from 1350 to 1410 kg/m3 for CRAP and 1480 to 1640 kg/m3 for NA. The presence of large number of agglomerated particles in CRAP resulted in decrease in density and an increase in void content. Due to coarser NF grading, which led in reduced density and increased void content, identical outcomes were obtained in the case of FRAP (Kumari et al., 2018a).

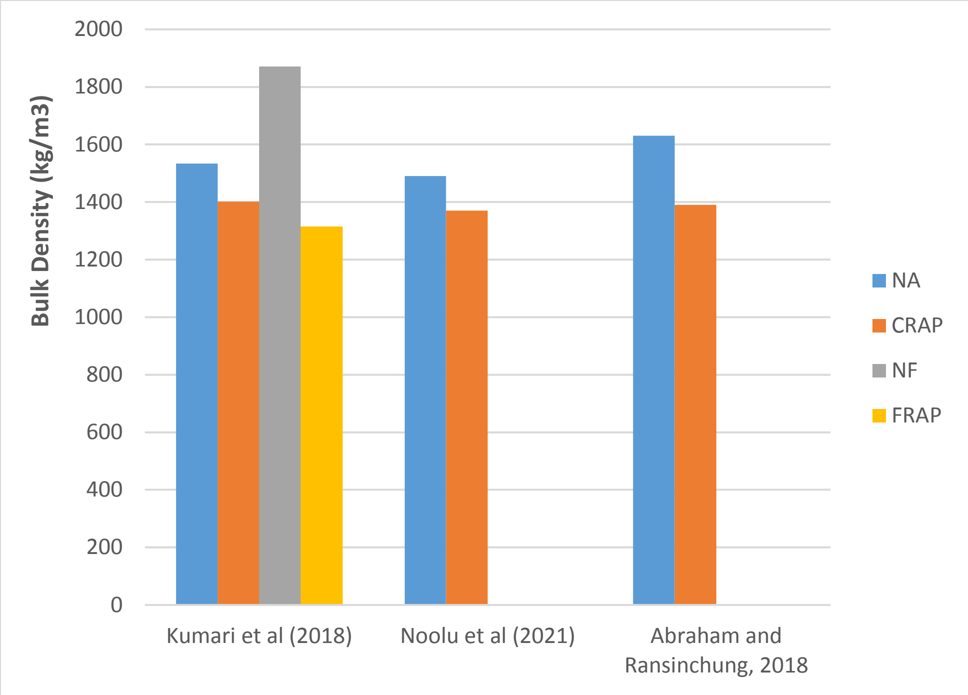

Figure 2.11 Bulk density of CRAP and FRAP

2.4.1.5 Impact Value

The presence of a less brittle asphalt film around RAP aggregates can absorb more impact load, resulting in a considerable increase in impact resistance as compared to NA. Due to the brittleness of the asphalt layer, the Aggregate Impact Value of NA was found to be higher than CRAP (Kumari et al., 2018a) as shown in Figure 2.12.

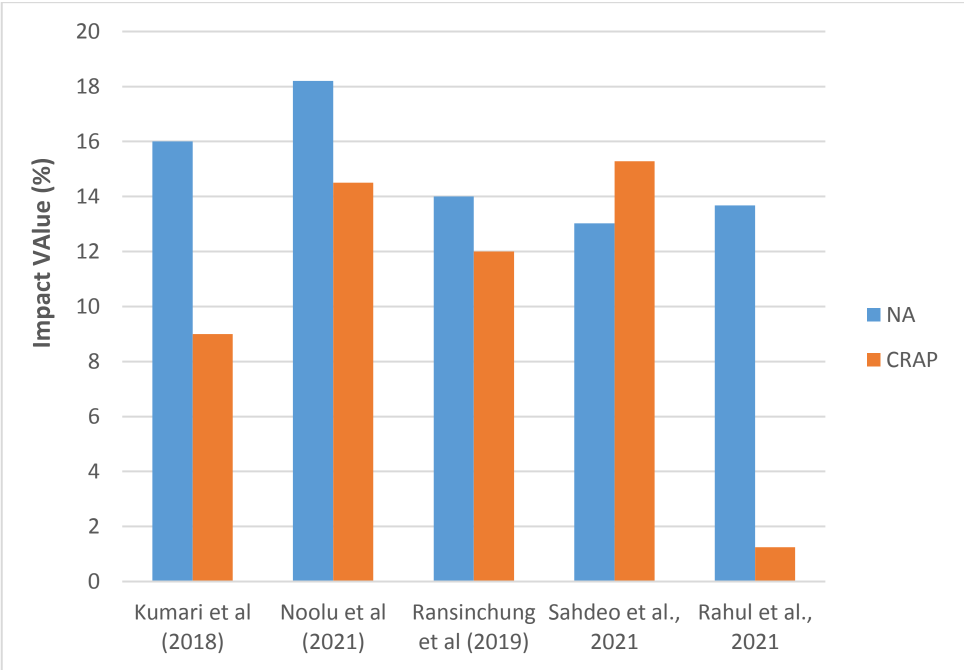

Figure 2.12 Impact Value of CRAP

2.4.1.6 Abrasion Value

Figure 2.13 depicts research on the abrasion value of CRAP and NA. When comparing NA to CRAP, it was discovered that NA has a slightly greater Abrasion value. The presence of a dust layer and the brittleness of the asphalt layer around CRAP are the reasons for this (Kumari et al., 2018a).

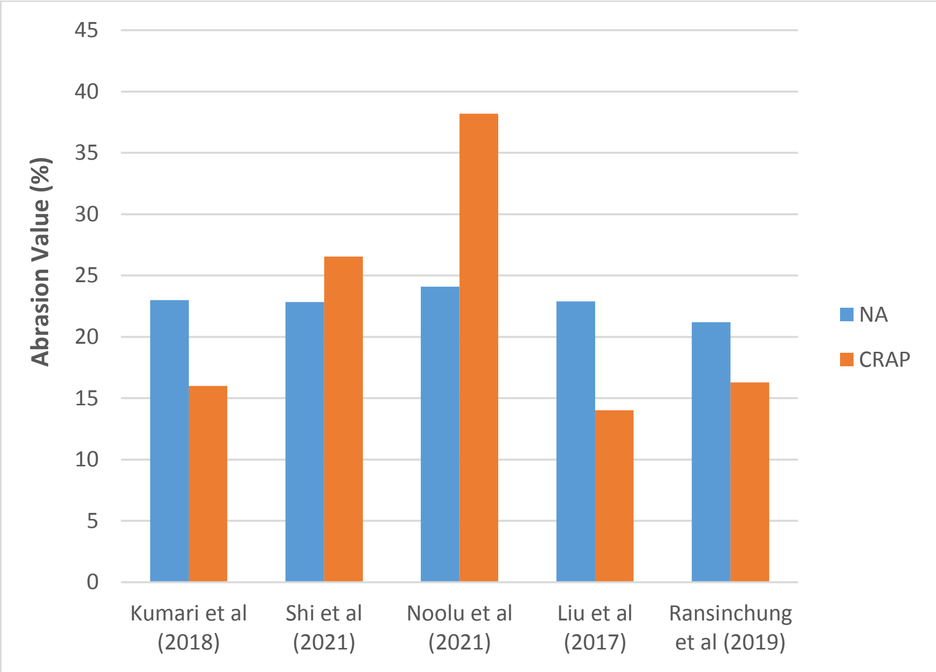

Figure 2.13 Abrasion value of CRAP

2.4.1.7 Crushing Value

The Aggregate Crushing Value was determined to be lower for CRAP than NA, similar to the Aggregate Impact Value results as shown in Figure 2.14. This is owing to the presence of a relatively unoxidized soft asphalt layer that tends to cover the NC particles when confining pressure is applied, resulting in the crushed aggregates being bound into a single solid mas (Kumari et al., 2018a).

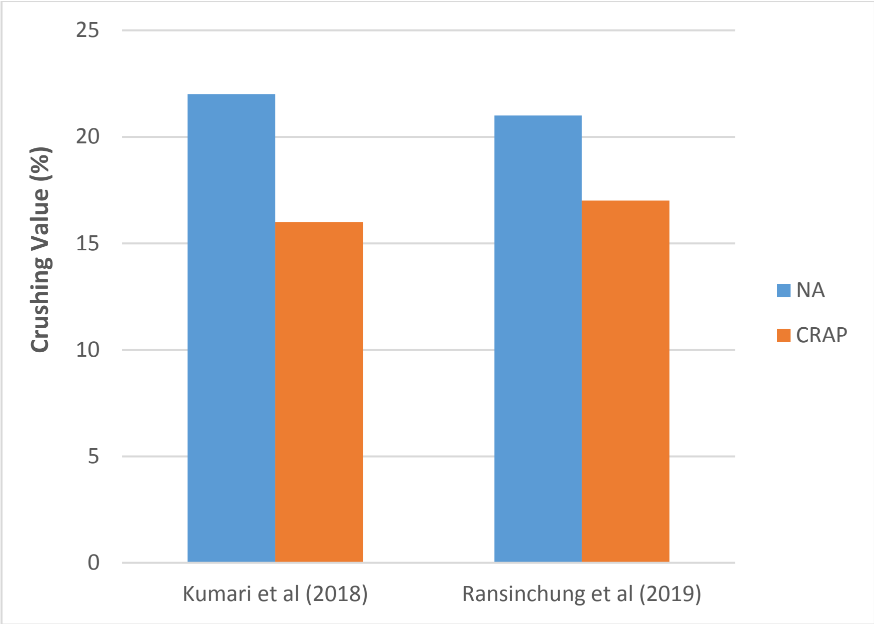

Figure 2.14 Crushing Value of CRAP

2.4.2 Properties of Fresh Mix

2.4.2.1 Workability

Slump, density and compaction factor tests were performed on RAP. The partial replacement of natural coarse aggregates with processed coarse RAP aggregates resulted in an increase in initial slump values, whereas fine RAP mixtures showed the opposite tendency. The beneficiation process entirely re-moved the dust layer clinging to the surface of the aggregates, whereas a high concentration of the same, absorbed the mixing water from FRAP mixes, resulting in an increase in fresh workability after incorporation of CRAP aggregates. Furthermore, the hydrophobic characteristic of asphalt coating might have contributed in improving the CRAP mixtures' fresh workability (Kumar et al., 2018) . Concretes comprised entirely of coarse or fine RAP had a lower slump than control concrete. This could be due to the asphalt binder's high viscosity. Surprisingly, the slump of both coarse and fine RAP concrete was greater than that of the control mix. The cause for this is most likely due to the asphalt covering that surrounds both coarse and fine RAP to prevent water from being absorbed by the aggregates (Huang et al., 2005). The existence of a dust layer, which tends to absorb more water from the mix, is sometimes accounted for the lower initial slump value of CRAP (Ransinchung et al., 2019). The addition of FRAP to concrete considerably reduced the initial slump. The combined effect of angular shape and high-water absorption of FRAP is responsible for such a significant reduction in the workability of fresh concrete. Fresh density of mixtures was observed to decrease as FRAP inclusion increased, similar to slump value. The lower specific gravity of FRAP compared to NF could reflect the decreased density of FRAP-containing mixtures (Singh et al., 2018). Figure 2.15 and Figure 2.16 shows slump value for various replacements of CRAP and FRAP respectively.

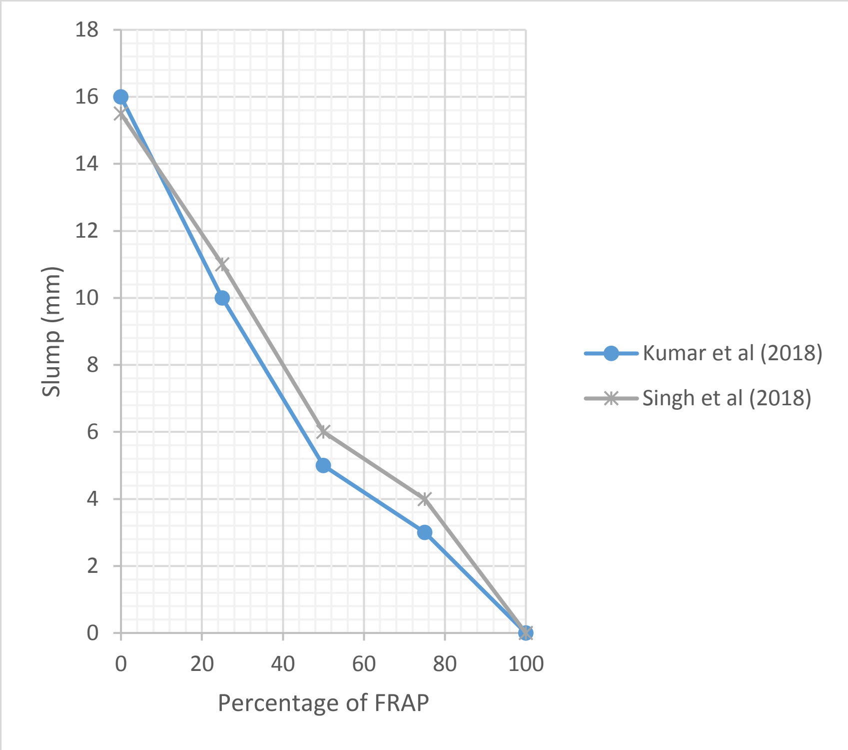

Figure 2.15 Slump Value of FRAP

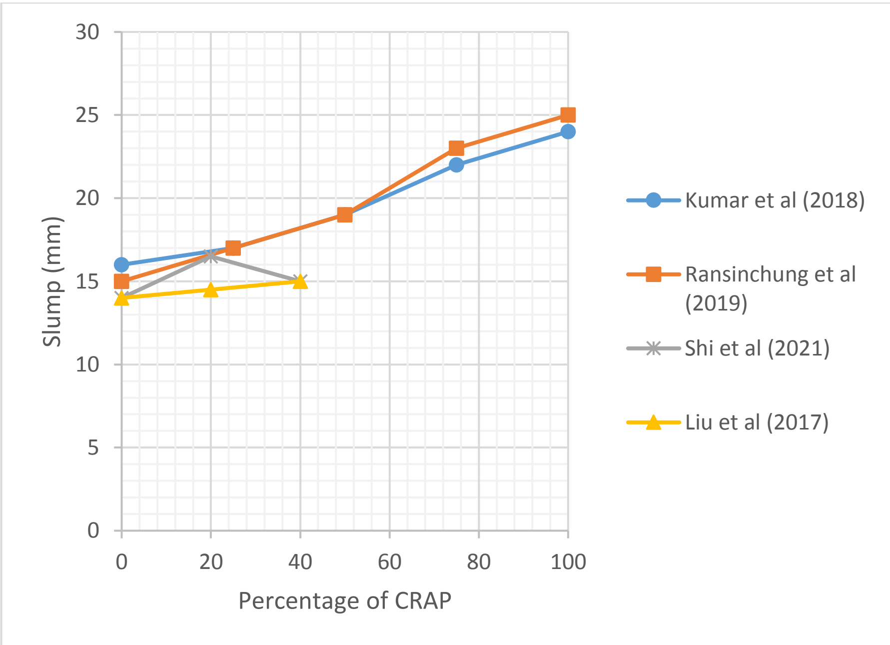

Figure 2.16 Slump value of CRAP

2.4.3 Properties of Hardened Mix

2.4.3.1 Compressive Strength

When comparing the compressive strength of concrete mixes made with RAP to that of control concrete, the results showed a systematic decline in compressive strength. Among the three RAP combinations, the strength of concrete formed with both coarse and fine RAP decreased the most. The concrete's strength was the least affected by CRAP. The compressive strength reduction of concrete using FRAP was comparable to that of CRAP and complete (coarse and fine) RAP mixes. Because the asphalt coating surrounding the aggregate particle was significantly softer than the concrete matrix and aggregate, the drop in strength was comprehensible. The presence of a soft asphalt binder in the concrete matrix may have caused stress concentration and micro cracking. A weak bond between the asphalt coating and the concrete matrix/aggregate could also be a factor (Hassan et al., 2000; Huang et al., 2005; Paluri et al., 2020; Tia et al., 2010). The presence of fewer agglomerated particles and good bonding between the relatively clean and hard surface of CRAP aggregates and the hydrated cement matrix resulted in higher compressive strength. The combined effect of an increase in water–cement ratio due to reduced water absorption of CRAP aggregates and the presence of a punctured asphalt layer can be linked to the drop in strength with increased CRAP aggregates (Ransinchung et al., 2019). The presence of asphalt, which hindered the development of bonding between cement and aggregates, may be responsible for the decrease in compressive strength values as FRAP content increases. Similarly, a decrease in workability associated with an increase in FRAP content could be attributed (Singh et al., 2018). Figure 2.17 and Figure 2.19 shows variation of compressive strength of various grades of concrete from different studies with respect to the percentage of CRAP and FRAP.

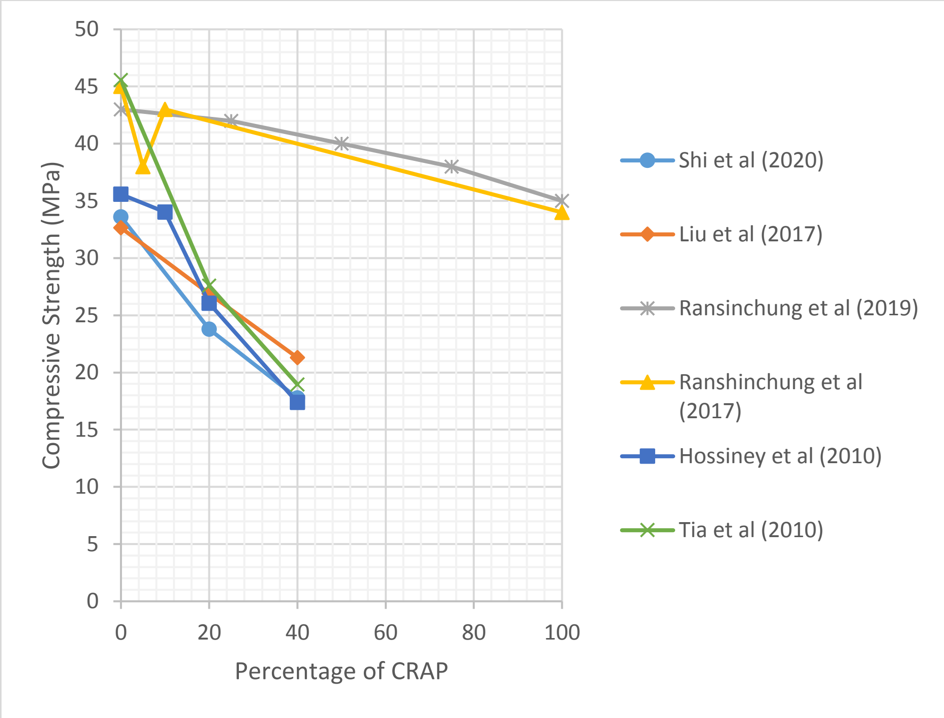

Figure 2.17 Compressive Strength of CRAP

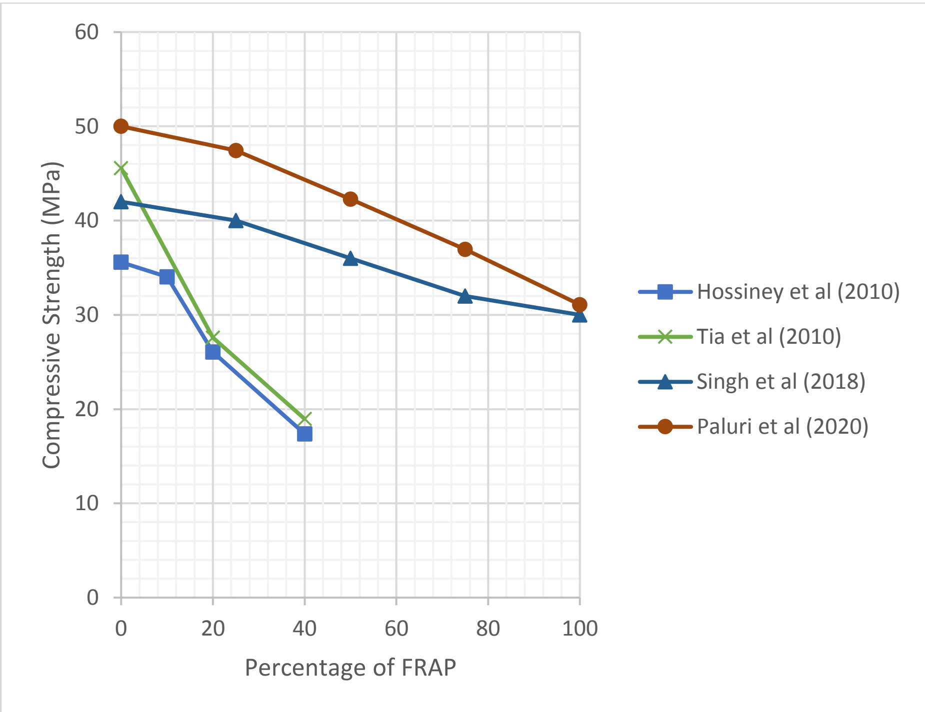

Figure 2.18 Compressive Strength of FRAP

2.4.3.2 Flexure Strength

The flexural strength of concrete was found to be lowered as partial substitution of NA by RAP aggregates increased. When comparing regular RAP to beneficiated RAP, it was discovered that beneficiating RAP increased the flexural strength of concrete (Ransinchung et al., 2019). The flexural strength of concrete was found to be lowered as partial substitution of NA by RAP aggregates increased. When comparing regular RAP to beneficiated RAP, it was discovered that beneficiating RAP increased the flexural strength of concrete. Flexural strength tends to decrease when FRAP concentration increases on all curing days. However, unlike compressive strength, where replacement of NF up to 50% resulted in reductions of less than 25%, flexural strength showed larger reductions (Singh et al., 2018). Figure 2.19 and Figure 2.20 shows flexural strength of various replacement of CRAP and FRAP.

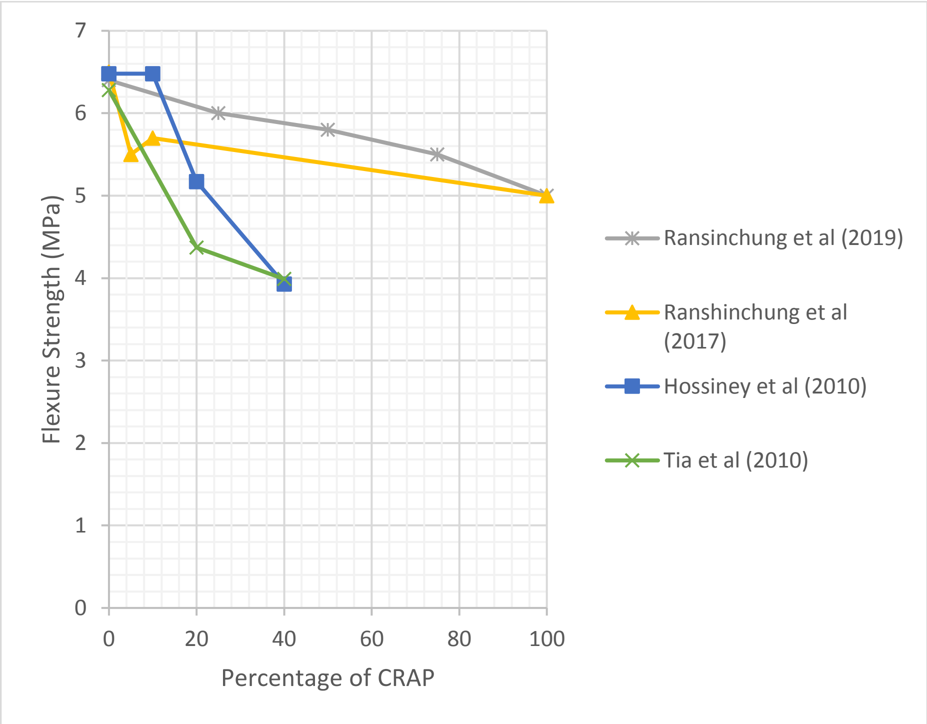

Figure 2.19 Flexure Strength of CRAP

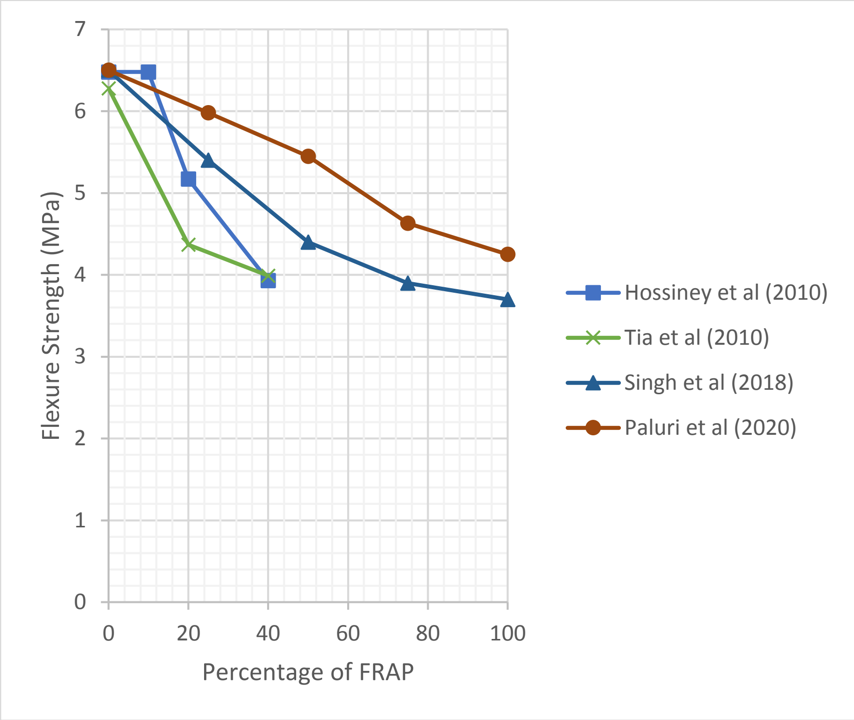

Figure 2.20 Flexure Strength of FRAP

2.4.3.3 Split Tensile Strength

The split tensile strength showed a similar strength drop trend to the compressive strength. The rate of strength reduction in the split tensile strength of CRAP mixes, on the other hand, was much lower than the compressive strength (Huang et al., 2005). Regardless of curing days, incorporating FRAP into concrete lowered spilt tensile strength. When compared to the control mix, a 25% replacement level resulted in strength reductions of less than 20%, while greater incorporations resulted in strength reductions of more than 30%. The production of excessive voids due to stiff mix and the presence of asphalt, which formed a weak cement mortar paste, may be responsible for the loss in split tensile strength of CRAP and FRAP included concrete (Singh et al., 2018). Figure 2.21 and Figure 2.22 shows Split tensile strength of various replacement level of CRAP and FRAP.

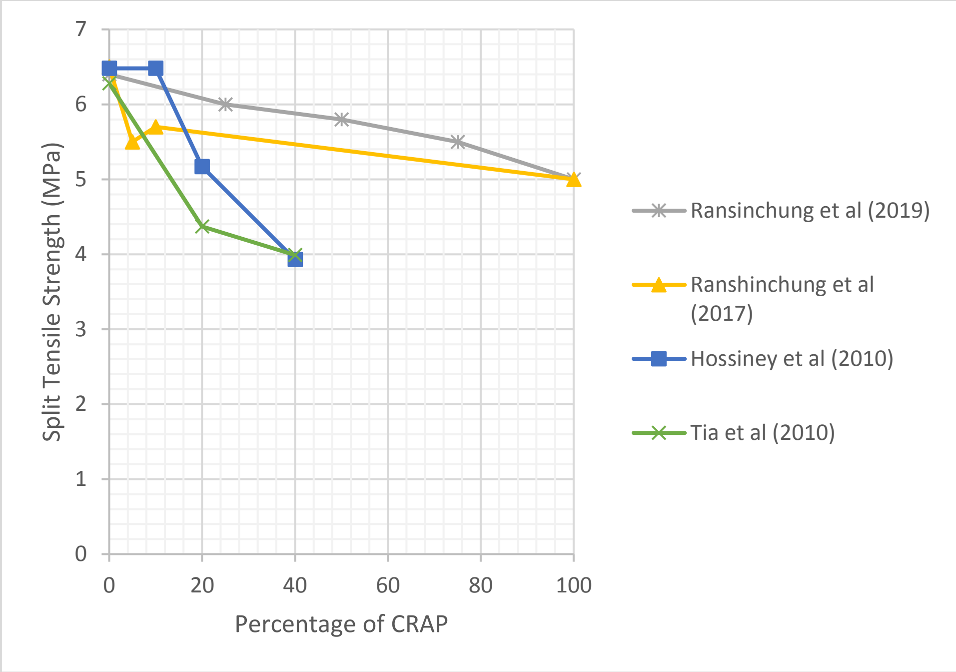

Figure 2.21 Split Tensile Strength of CRAP

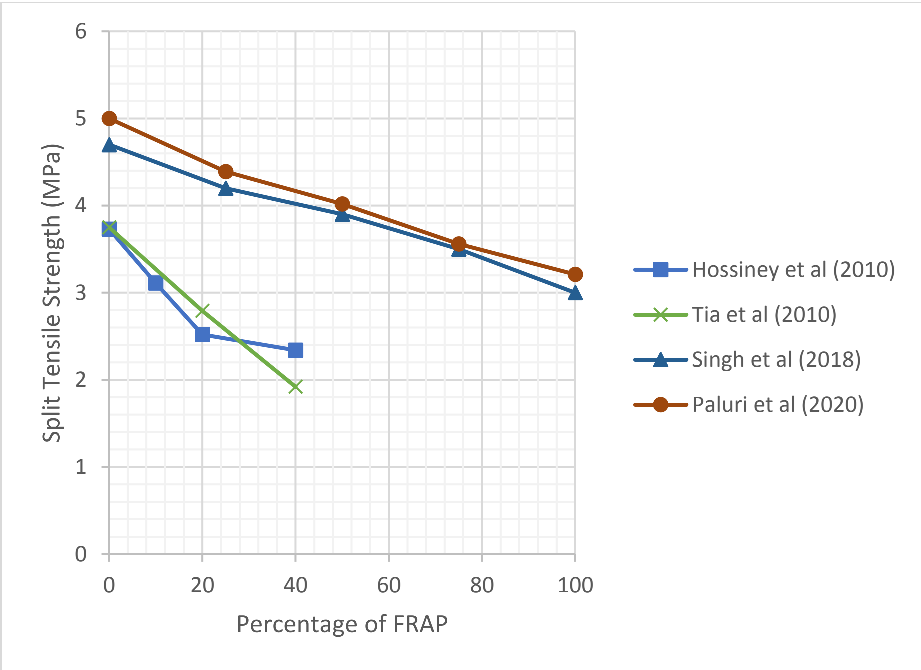

Figure 2.22 Split Tensile Strength of FRAP

2.4.3.4 Shrinkage

The effects of RAP concentration on the shrinkage strain of concrete were discovered to have no distinct trend as was seen in other mechanical properties of RAP (Tia et al., 2010). Figure 2.24 shows the variation of RAP with drying shrinkage.

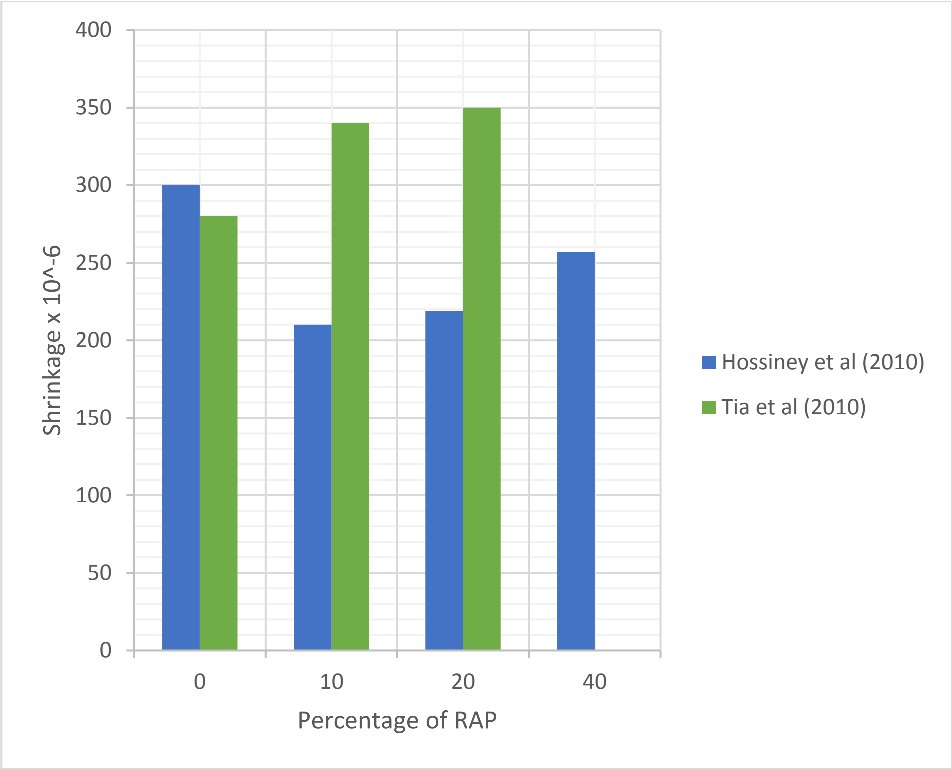

Figure 2.23 Shrinkage of RAP

2.4.4 Durability Properties

2.4.4.1 Water absorption

At 28 and 91 days of moist curing, the reduction of CRAP in comparison to the NC mix was 16.1% and 21.1 %, respectively. This reduction is related to the melted asphalt layer filling voids when exposed to an oven, allowing for improved refinement of the macro and micro pores of the concrete. The increase in water absorption of concrete containing up to 50% CRAP aggregates is mostly attributable to an increase in the water–cement ratio (additional water saved due to CRAP aggregates, lower water absorption than NA). However, when NA is partially substituted with CRAP aggregates to a level greater than 50%, the asphalt coating that surrounds the aggregates supersedes over the water–cement ratio (Ransinchung et al., 2019). It was observed that using FRAP might considerably increase the water absorption of hardened concrete at later ages. Up to 50% replacement, there was a slight increase in water absorption values, but after that, there were significant increases. Higher water absorption of FRAP aggregates could explain the increase in water absorption values for RAP inclusive concrete (Singh et al., 2018). Figure 2.24 presents the variation of water absorption with different replacement of RAP.

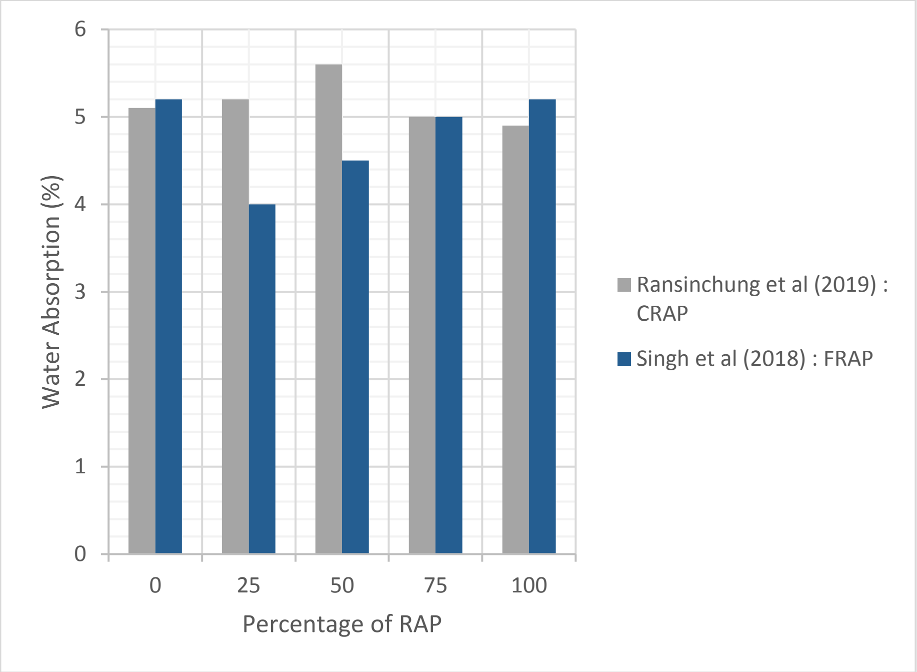

Figure 2.24 Water Absorption

2.4.4.2 Total Permeable Voids:

CRAP aggregates significantly enhanced the total permeability voids in lean concrete when compared to NA mix. For instance, with a 100 percent CRAP mix, permeability voids rose by less than 20%. This increase in permeability gaps after RAP aggregates were added, could be attributed to agglomerated particles retaining more water in the mix. When RAP was used at a ratio of more than 75%, however, there was a reduction in permeable voids. This could be related to the melting of the asphalt layer on the boundary aggregates due to the heat generated during the oven drying process. This melted asphalt layer may have clogged surface spaces, thus reducing the permeable voids. Also, due to the existence of a punctured asphalt film around its periphery, CRAP aggregates may reduce the permeability spaces in concrete, which melts and fills the pore voids when exposed to high temperatures (Ransinchung et al., 2017). When FRAP aggregates were added to CRAP inclusive mixes, the number of permeable voids increased significantly. Permeable voids increased from 10.5% to 13.57 % after partial substitution of NF with FRAP aggregates from 25% to 100%, a 29 percent increase above the control mix. The slightly coarser grading of FRAP aggregates may account for this increase in permeable voids (grading zone I) (Kumari et al., 2018a). Figure 2.25 and Figure 2.26 represents variation of permeable voids with replacement of CRAP and FRAP respectively.

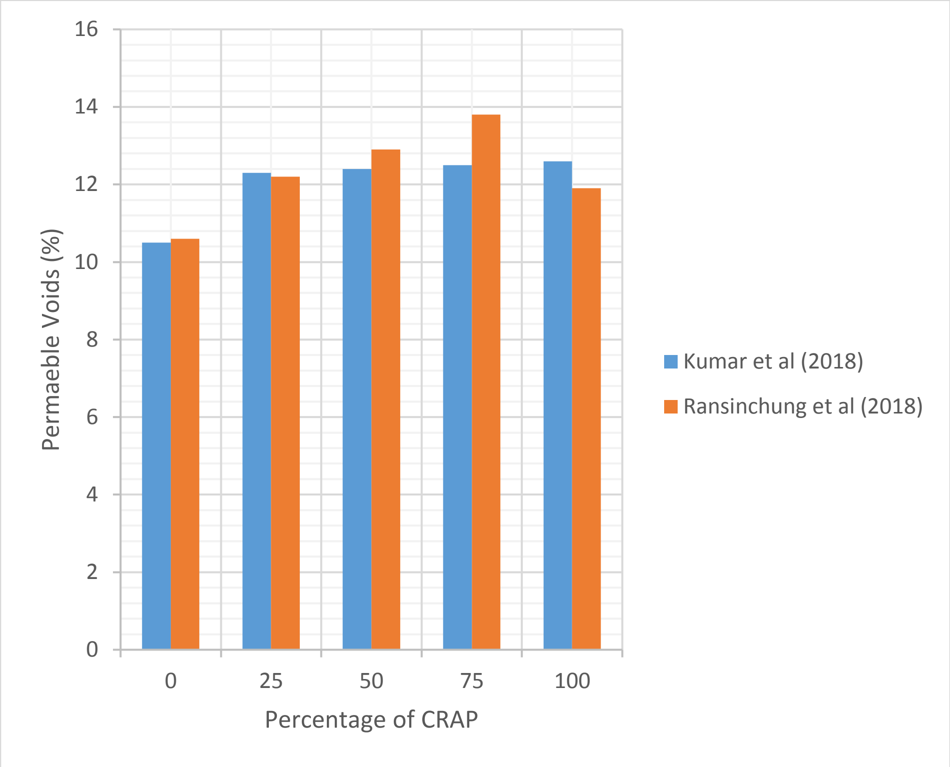

Figure 2.25 Permeable voids of CRAP

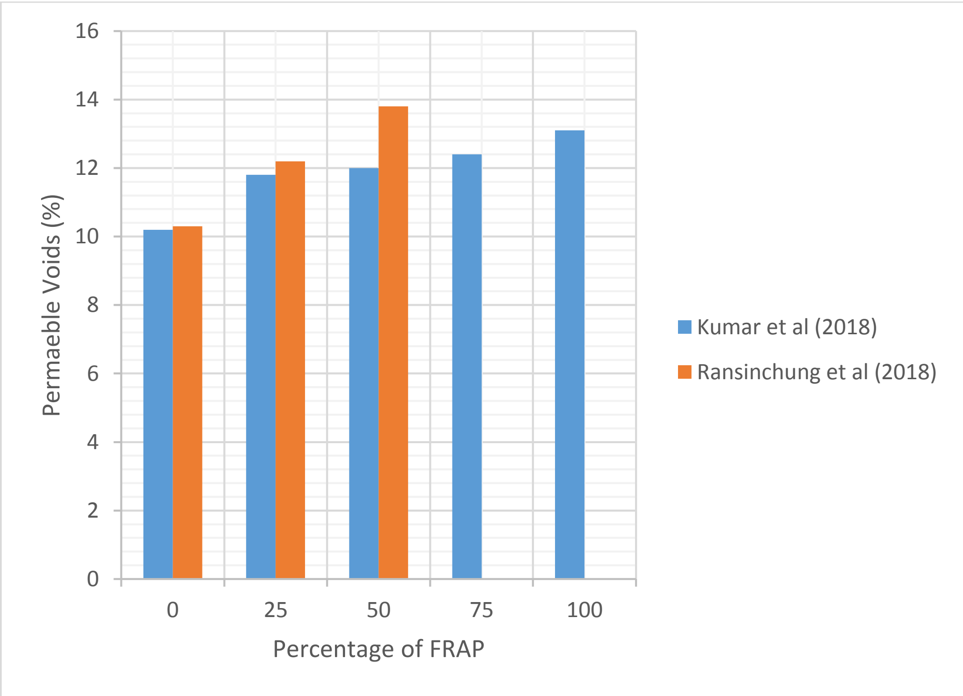

Figure 2.26 Permeable voids of FRAP

2.4.4.3 Abrasion Resistance

The mass loss for the CRAP mix was found to be significantly higher than that of the NC mix (about 66 percent). The mass loss of 25CRAP mix and NC mix (0.24 percent) was the same.

The abrasion resistance decreased when the inclusion level was increased, but not to an extreme extent. When compared to the NC mix, the increase in mass loss was only 8.3%, 16.7%, and 20.8 percent for 50CRAP, 75CRAP, and 100CRAP mixes, respectively (Ransinchung et al., 2019). The most significant aspect for the building of concrete pavements and floors is abrasion resistance. Heavy loads with frequent skidding and rubbing is expected because cement concrete pavements are built for commercial vehicles. The abrasion induced by these processes causes the pavement surface to deteriorate. As a result, materials that can endure this deterioration must be carefully chosen. The control mix proved to be the most abrasion-resistant of the three. Incorporating 25% FRAP into concrete resulted in a mass loss of roughly 33% when compared to the control mix. Concrete's resistance to abrasion was shown to decrease significantly when more FRAP was incorporated. These findings suggest that replacing NF with FRAP could increase the wear and tear on the concrete surface, causing it to fail (Singh et al., 2018). The variation of loss in mass with respect to different replacement of RAP is shown in Figure 2.27.

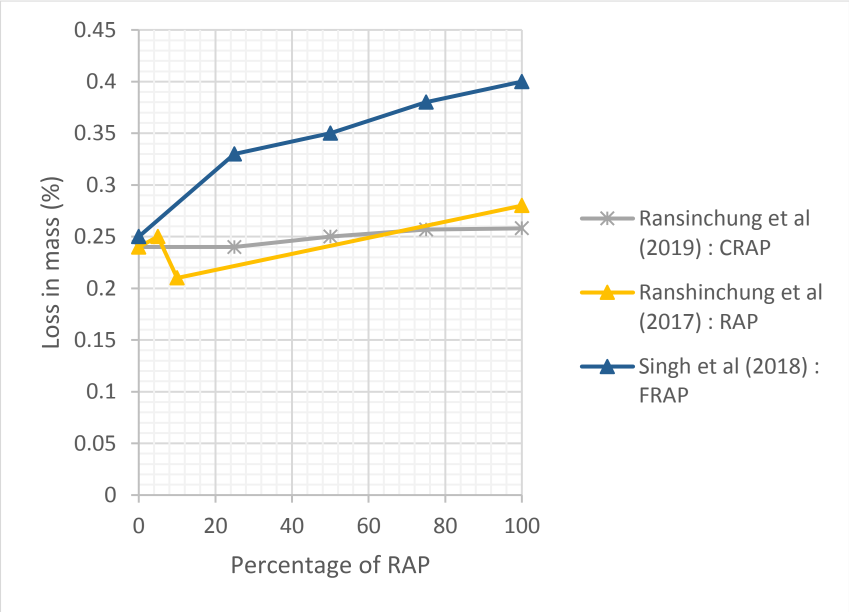

Figure 2.27 Abrasion Resistance of RAP

2.4.4.4 Acidic Environment (Loss in mass after exposure to HCL and H_2SO_4 solution):

The results of specimens treated to a 2.5 percent HCL solution revealed that CRAP inclusive concrete is less resistant to HCL acid attack than NC concrete. As the fraction of NA replaced by CRAP aggregates in concrete increases, the mass of specimen's decreases. The oven dried specimens were shown to be more sensitive to HCL acid than SSD condition samples. In the Oven Dried condition, mass losses compared to the NC mix were 9.65 percent, 26.78 percent, 34.43 percent, and 36.07 percent for 25CRAP, 50CRAP, 75CRAP, and 100CRAP, respectively. CRAP mixes were less resistant to H_2SO_4 solution than NC mixes. In both situations (SSD and OD), the losses in mass of specimens were shown to increase as the substitution level increased. The Oven Dried specimens, on the other hand, were more vulnerable than the SSD specimens in the case of the HCL solution (Ransinchung et al., 2019). The weight reductions after being exposed to HCL solution were 4.61, 5.24, 5.77, 6.39, and 6.75 percent for the mixes containing 0, 25, 50, 75, and 100 percent FRAP, respectively, whereas the weight reductions after being exposed to H_2SO_4 solution were 7.28, 8.78, 9.42, 10.25, and 11.26 percent. The findings suggest that incorporating FRAP into concrete might significantly increase the susceptibility of concrete to chemical acid. Increased permeability voids (owing to lower workability) allowed for easy transportation of acid ions in the specimen and subsequent leaching of calcium salts, which resulted in increased weight loss with increased FRAP concentration (Singh et al., 2018). Figure 2.28 and Figure 2.29 shows variation of CRAP and FRAP after exposure to HCL and H_2SO_4.

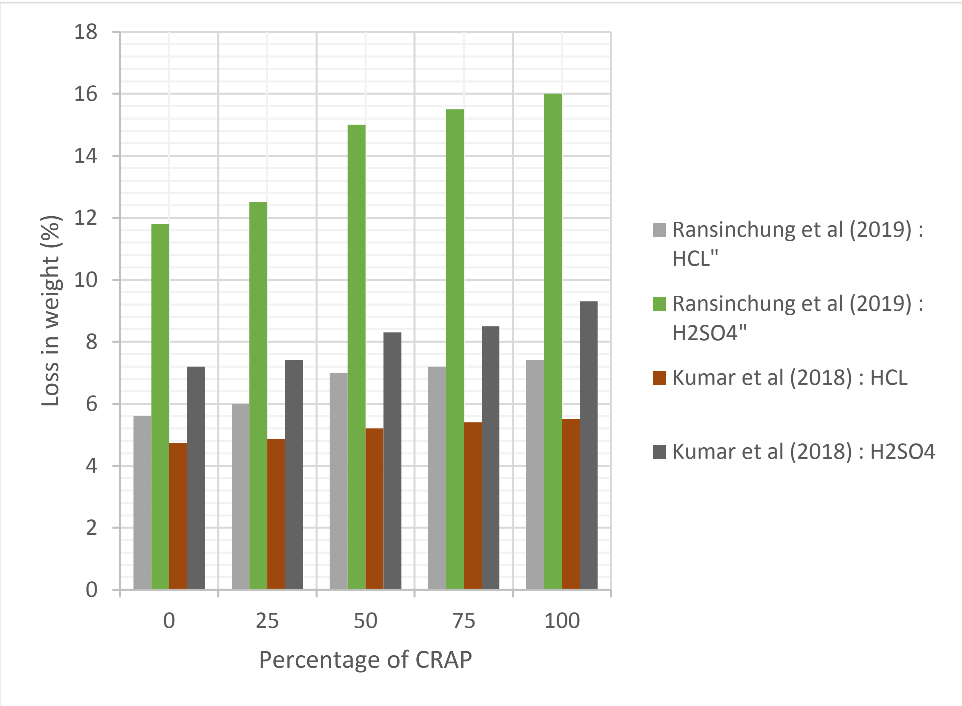

Figure 2.28 Effect of Acidic Environment on CRAP

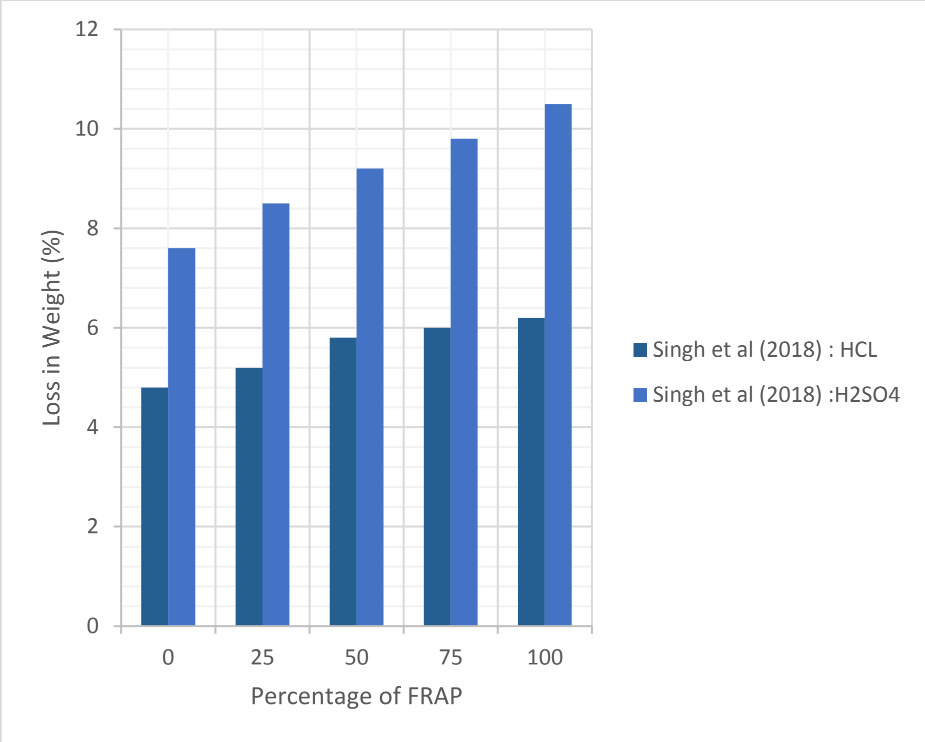

Figure 2.29 Effect of Acidic Environment on FRAP

2.4.4.5 Sorptivity:

With the addition of CRAP aggregates, the coefficient of sorptivity was found to decrease. Incorporating CRAP aggregates enhanced the coefficient of sorptivity up to 50 percent as compared to the NC mix on both curing days. However, as portion replacement increased beyond 50 percent, the coefficient of sorptivity began to decline, and this value was discovered to be even lower than the NC mix for the mix containing 100 percent CRAP aggregates (Ransinchung et al., 2019). The effect of Coefficient of sorptivity with various replacement of RAP is shown in Figure 2.30.

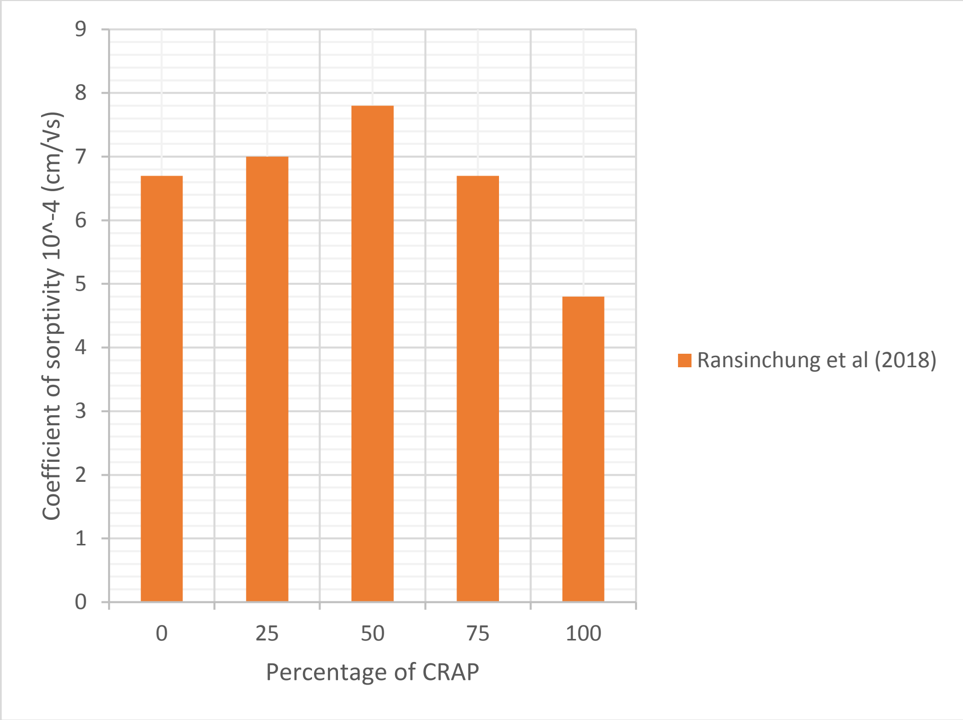

Figure 2.30 Coefficient of Sorptivity of RAP

2.4.4.6 Toughness:

The addition of RAP to concrete increased the hardness of the treated concrete in general. The concrete mixture with only FRAP, on the other hand, demonstrated hardness similar to the control combination Concrete mixtures including CRAP and RAP in its whole (both) RAP (coarse and fine) has much higher energy levels absorption (the region between the load-deformation curve and the load-deformation curve and the abscissa) than the concrete in the control(Huang et al., 2005).

The load head displacement was used to determine the vertical deformation. It was discovered that mixes containing RAP could retain peak load even after a reasonably long displacement. The failure mode of the specimens under compressive strength tests also indicated improved toughness. Concrete specimens made with RAP did not disintegrate abruptly at failure, but control concrete did. When RAP fail, only a few cracks appeared on the surface of concrete specimens (Huang et al., 2005). Figure 2.32 shows the toughness index of RAP mixes with age.

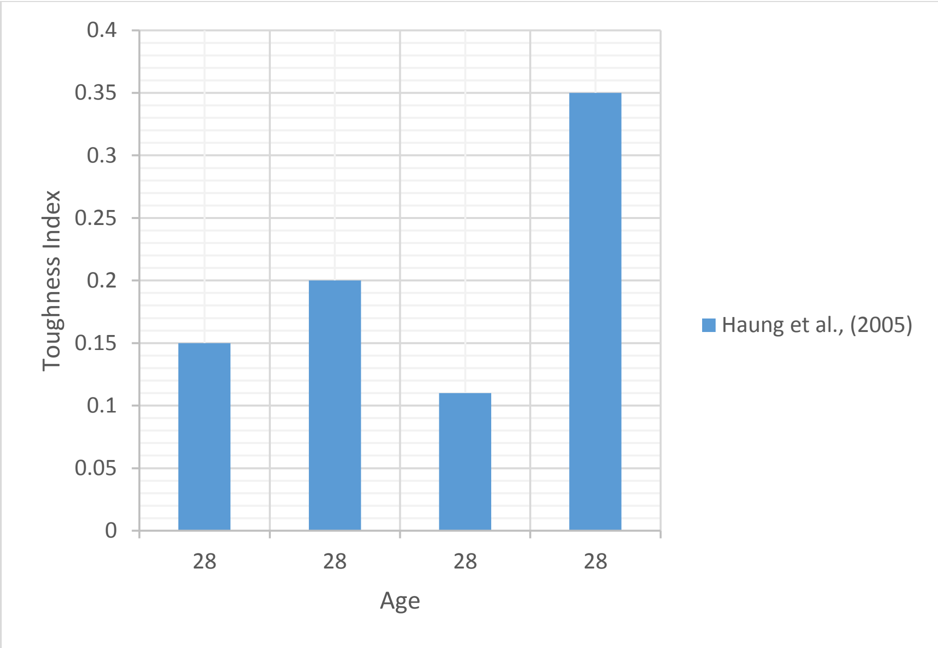

Figure 2.31 Toughness Index of RAP

2.4.4.7 Alkalinity and carbonation:

The pH value of concrete indicates how far it has degraded. The pH level of durable concrete should be between 12.5 and 13. The pH value of all the considered mixes was found to be greater than the stipulated minimum pH value. In comparison to the NC mix, when the CRAP aggregates increase, the pH value tends to increase (Ransinchung et al., 2019)

The addition of FRAP to concrete lowered its alkalinity. The reductions in pH value compared with the control mix were observed to be 0.3, 1.3, 3.3, and 3.8 percent for the mixes containing 25, 50, 75, and 100 percent FRAP, respectively. However, the pH values were found to be greater than 12, indicating that the use of FRAP is not responsible for corrosion or carbonation(Kumari et al., 2018a).

The results of a carbonation test, in which the colour of the exposed surface remained unchanged after being exposed to phenolphthalein solution (no carbonation was detected after 120 days of wet curing), backed up this theory (no carbonation was observed after 120 days of moist curing)(Singh et al., 2018). Figure 2.32 shows effect of alkalinity of mix with various replacement of RAP.

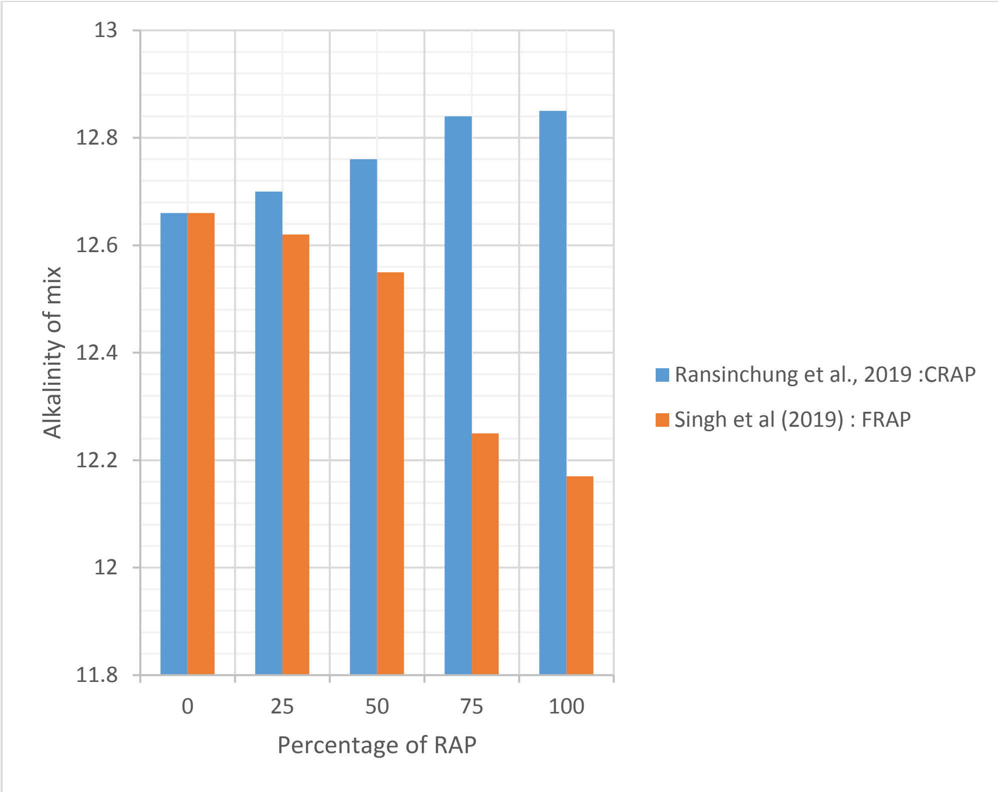

Figure 2.32 Effect of Alkalinity on RAP

2.5 GAPS IDENTIFIED IN LITERATURE

Based on the available literature review presented above, the following gaps have been identified:

i. In most of the research RAP aggregate was manufactured in the laboratory which does not relate to the field condition.

ii. From the study of literature review it was observed that in most cases no processing technique was used in order to remove the asphalt layer from the RAP aggregates. However, in case of some researchs, the processing technique used was found to be less effective in removing the binder content as the concrete mixes prepared could not reach the design target value of 40 MPa.

iii. GGBS as a supplementary cementitious material was not studied in any of the previous research containing replacement of natural aggregates by RAP aggregates.

iv. Detailed study on mechanical, durability and time dependent properties of pavement quality concrete with inclusion of RAP aggregates was not carried out in earlier research, with patrial replacement of cement by GGBS.

CHAPTER 3

3 CHARCTERIZATION OF MATERIALS, MIX DESIGN AND CASTING OF TEST SPECIMENS

3.1 INTRODUCTION

The experimental study for the present work has been planned as per the objectives stated in chapter 1 and it includes characterization of various materials used in the study, proportioning pavement quality concrete (PQC) mixes and casting and testing of proportioned mixes. Accordingly, the materials used in this study have been tested to determine the appropriate engineering, mechanical and durability properties. Following the characterization of materials, concrete mix of 40 MPa grade with desired workability has been proportioned according to IRC: 44 – 2017 method. Hence, the characterization of materials and mix design of the 12 concrete mixes viz. 0% RAP(CM1), 20% RAP (20R), 40% RAP (40R), 60%RAP (60R), 0% RAP with 40% GGBS (CM2G), 20% RAP with 40% GGBS (20RG), 40% RAP with 40% GGBS (40RG), 60% RAP with 40% GGBS (60RG), 0% RAP with 40% GGBS and 10% SF(CM3GS), 20% RAP with 40% GGBS and 10% SF (20RGS), 40% RAP with 40% GGBS and 10% SF (40RGS) and 60% RAP with 40% GGBS and 10% SF (60RGS) are presented in this chapter. The casting of various concrete test specimens for the experimental studies is also discussed.

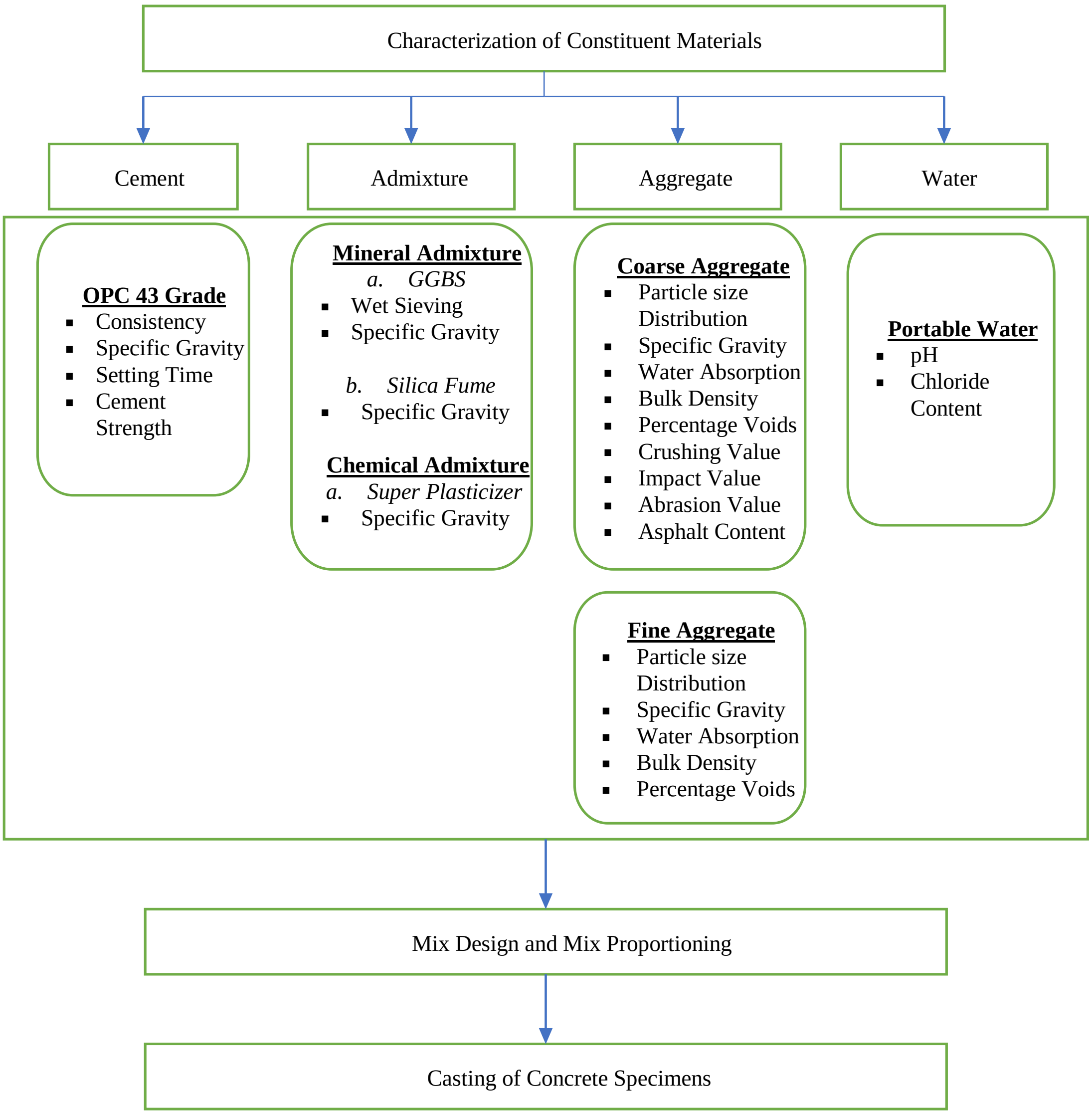

Figure 3.1 Outline of Experimental Programme for characterization of constituent Materials

3.2 CHARACTERIZATION OF CONSTITUENT MATERIALS

The materials used in this study are coarse aggregate i.e., natural coarse aggregate (20mm and 10mm) and coarse reclaimed asphalt pavement (RAP), fine aggregate, cement, silica fume (SF) and Glass granulated blast furnace slag (GGBS) were tested and their properties were determined.

3.2.1 Coarse Aggregate

In this study crushed granite natural aggregate (NA) of size 20 mm and 10 mm available in Delhi region have been used. Fig 3.2 and 3.3 shows the natural aggregates of size 20 and 10 mm respectively.

RAP was obtained from a big stockpile of distressed old flexible pavement through ripping and crushing (more than 20 years old). RAP is dumped in a stack-yard as a result of this operation as shown in Fig. 3.6. The above-mentioned RAP was composed of a combination of wearing course and Bituminous Macadam. Bituminous Macadam is composed of all-in coarse aggregates (size > 2.36 mm) and fine aggregates (size 2.36–0.075 mm) combined with asphalt (3.0%–3.5% by weight of dry aggregates) to create a well-stable mixture that can be utilised for the bound base course of flexible pavements. Demolition of the same would result in a well-graded mixture of all aggregates, as discovered in the current study. Hence, RAP was screened through a 19 mm sieve and mechanically separated into coarse (d > 4.75 mm) and fine (d < 4.75 mm) RAP. For the laboratory experiment, only the coarse fraction was considered. Fig 3.4 and 3.6 shows coarse RAP and agglomerated particles present in RAP.

Figure 3.2 Natural Aggregate 20 mm

Figure 3.3 Natural Aggregate 10 mm

Figure 3.4 Coarse RAP aggregates

Figure 3.5 Agglomerated particles

Figure 3.6 Stockpiled RAP

a. Processing of RAP

The presence of a significant amount of dust on the surface of the RAP aggregates can be attributed to the stockpile life during which it was exposed to all seasons. Dust may have formed a bond with the soft asphalt layer during the summer. The thickness of this dust layer is mostly influenced by the layer of pavement to be removed, the method of removal, and the stockpile's duration and exposure environment. When compared to RAP extracted from the wearing course, RAP from the base layer may contain a significant amount of dust. In comparison to ripping and crushing, milling removes very little dust contaminant from RAP. Due to the large quantity of RAP it is very difficult to store RAP in a protected way in order to prevent any foreign stuff from entering. RAP is removed in large scale in developing nations like India by ripping the entire pavement, crushing it, and stockpiling it in the open for a long time.

This results in a thick dust film surrounding the RAP aggregates, with the dust layer around the asphalt coating firmly adherent, but the distant dust layer is loose and can be easily removed by sieving or washing. Furthermore, depending on the size of the aggregates and the type of removal procedure utilized, some agglomerated particles may be present on the surface of RAP aggregates. RAP is a four-phase aggregate that includes aggregate, asphalt film, stiff dust film, and loose dust film. It has two transition phases: one from aggregate to asphalt coating, and the other from asphalt film to dust layer as shown in Fig. 3.7. Producing concrete with normal RAP aggregates might result in a weak bond between mortar and dust layer hence reducing the concrete's properties. To improve the bonding, this dust film must be removed. Similarly, entirely removing or breaking the asphalt film's continuity may improve the odds of better bonding as shown in Fig. 3.8. Keeping both of these factors in mind, a new method is proposed that produced better bonding than earlier experiments. This approach makes it easier to puncture a covered asphalt layer properly, ensuring strong bonding between aggregate particles and the cement matrix.

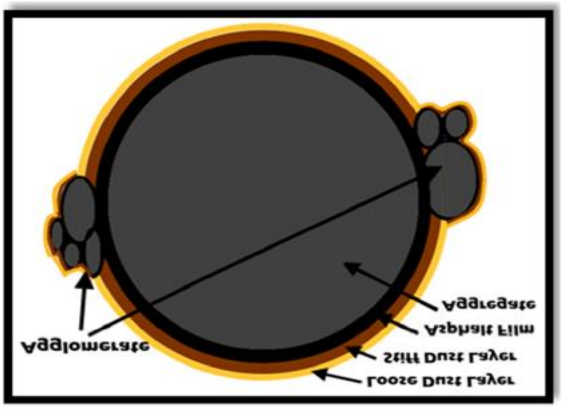

Figure 3.7 Normal RAP

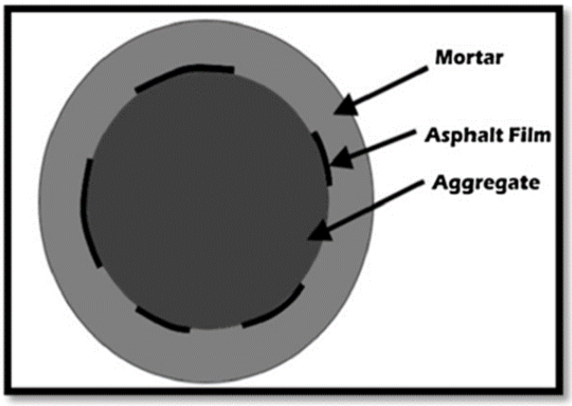

Figure 3.8 AB&AT RAP

Need for new processing technique

From previous study it was found that tilting drum type mixer was not that much effective in removing the contaminant layers from the RAP aggregates. The reason behind this was, there was not maximum abrasion and attrition between the RAP aggregates and the charge balls since the mixer was inclined at an angle of 10˚ hence the asphalt content reduction was also less than 45%. Therefore, in this study Los Angeles Abrasion machine was used. The reason behind using this machine was to get maximum contact of charge balls with the RAP aggregates so that maximum abrasion and attrition of RAP aggregates takes place. This was possible due to 360° revolution of Los Angeles machine resulting in maximum contact between the abrasive charge balls and RAP aggregates and therefore resulting in removal of contaminant layers and reduction in asphalt content. Using this method, it was found that reduction in asphalt content was more that 60%. Hence it proved to be an effective way in order to remove the contaminant layers from RAP aggregates.

Machinery details

This testing machine consists of a hollow steel cylinder, closed at both ends, having an inside diameter of 700 mm and inside length of 500 mm. The cylinder is mounted on a sturdy frame on ball bearings. The opening is closed dust tight with a removable bolted cover in place. A detachable shelf which extends through the drum catches the abrasive charge and does not allow it to fall on the cover. The drum is rotated at a speed of 30-33 RPM by an electric motor through a heavy-duty reduction gear, fitted with revolution counter and push button starter and supplied complete with a tray for collection of the material. Abrasive charge consists of set of 12 Nos. cast iron spheres or steel spheres (hardened steel balls) approximately 48 mm diameter each weigh between 390-445 gram. These abrasive charge balls were used to produce mechanical stresses on the surface of RAP aggregates. Fig. 3.9 shows the Los Angeles Abrasion machine used for beneficiation process.

Optimization of method

The amount of aggregates fed into the mixer depends on the angle of tilt, mixer speed, mixing time, and material passing through 4.75 mm sieve, while the number of charge balls is determined by the gradation of the processed material and total asphalt removed. The machine revolve fully 360° resulting in maximum contact between the RAP aggregates and abrasive charge balls and hence in turn resulting in maximum abrasion and attrition process. The mixer could rotate at a rate of 33 revolutions per minute and was held at that speed throughout the study. Fig. 3.10 and 3.11 shows the comparison between normal RAP and Abraded RAP. The mixing time was kept constant (15 minutes) because as longer time could result in broken and spherical aggregates, which would reduce load transfer efficiency. Table 3.1 shows the percentage of particles passing through a 4.75 mm sieve (dust + agglomerated particles) due to attrition between normal RAP aggregates. Table 3.1 shows that when the input quantity of normal RAP aggregates in the Los Angeles abrasion machine was 25 kg, maximum attrition between the aggregates occurred, and as such this quantity(25 kg) was kept constant in order to determine the optimum number of charge balls. Fig. 3.12 showed the variation of Asphalt content (determined by ASTM D2172) with various number of charge balls (0-14) keeping the quantity of RAP constant i.e., 25 kg. It was found that the decrease in asphalt content was insignificant after 12 charge balls. Based on the above findings, it was found that 25 kg of normal RAP and 12 charging balls were the best amounts to feed into the mixer. The AB&AT method of processing conventional RAP totally removed both top layers and punctured the asphalt coating at numerous spots. RAP's asphalt content has been shown to be reduced by more than 60% after processing.

Figure 3.9 Los Angeles Abrasion Machine

Figure 3.10 Normal RAP

Figure 3.11 Processed RAP

Table 3.1 Percentage passing through 4.75 mm sieve

RAP (kg)	Passing through 4.75 mm sieve (%)
10	9.558
15	12.563
20	13.768
25	16.106
30	14.632

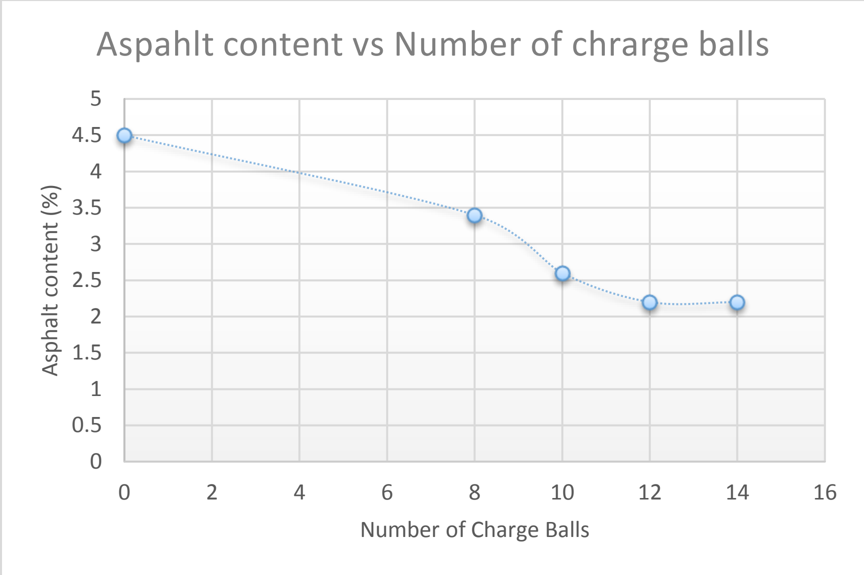

Figure 3.12 Asphalt Content v/s Charge Balls

b. Particle size distribution

Fig. 3.13 shows the particle size distribution curve of both NA and abraded RAP. It was found that in this study that both the coarse aggregates i.e., NA and RAP showed similar particle size distribution as can be seen in the figure. The particle size distribution of both NA and RAP used are completely within the limits specified in IRC: 44 – 2017.

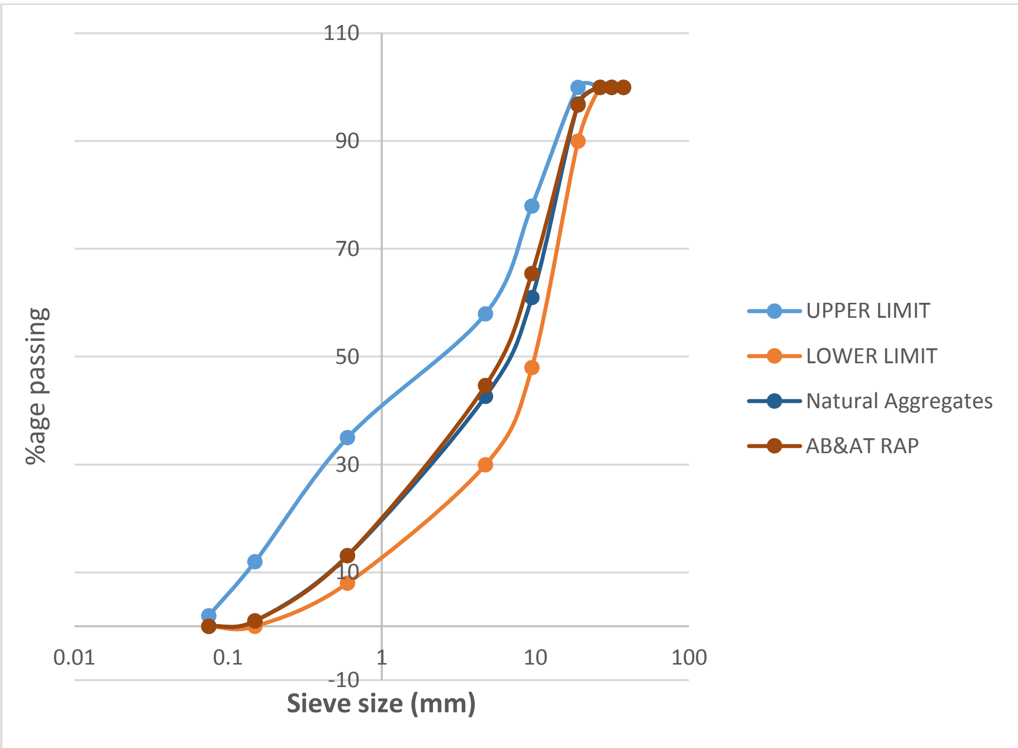

Figure 3.13 Particle size distribution curve

c. Specific gravity and water absorption

The specific gravity and water absorption of the aggregates has been determined as per IS 2386 (Part 3). The specific gravity of abraded RAP was found to be 2.464 which is lower as compared to the NA having specific gravity of 2.793 for 20 mm aggregate and 2.846 for 10 mm aggregate. However, the water absorption of abraded RAP was found to be 0.58% which is higher as compared to the NA having water absorption of 0.2845 for 20 mm aggregate and 0.236% for 10 mm aggregate. The reason behind lower specific gravity of abraded RAP is due to the presence of lower density asphalt coating and agglomerated particles in RAP aggregates. However, the reason behind higher water absorption of RAP as compared to the NA is possibly due to the dust layer around RAP aggregates that soaked more water.

d. Bulk density and percentage voids

The bulk density of NA and abraded RAP was determined by using procedure given in IS 2386 (Part 3). The bulk densities of NA was found to be 1676 kg/m^3 for 20 mm aggregate and that of 10 mm aggregate was found to be 1713 kg/m^3. However, the bulk density of abraded RAP was found to be 1732 kg/m^3 which is higher as compared to the NA. The reason behind higher density of RAP is due to the removal of most of the agglomerated particles present in RAP as a result of Abrasion and Attrition Technique. The percentage void of RAP aggregate was found to be 29.707% which is lower as compared to the NA having percentage void of 39.992% and 39.81% for 20 mm and 10 mm aggregate respectively. The reason behind lower percentage of voids is due to the presence of negligible amount of agglomerated particles present in RAP and most of the RAP aggregates were 12.5 mm passing size.

e. Crushing Value

The crushing test was performed by using procedure specified in IS 2386 (Part 4). Aggregates passing through 12.5 mm sieve and retained on 10 mm sieve were taken. The crushing value of abraded RAP and NA were found to be 21.72% and 24.361% respectively. These values are within the limits of 30% for wearing surface and 50% for normal concrete as per IS 383. The reason behind lower crushing value of RAP is due to the presence of relatively un-oxidized soft asphalt coating which tends to cover the NA aggregates under the application of confining pressure thereby binding the crushed aggregates into a single solid mass.

f. Impact Value

The impact value of aggregate has been determined as per the procedure given in IS 2386 (Part 4). The impact value of abraded RAP was found to be 14.6% and that of NA was found to be 15.351% for 20 mm and 22.808% for 10 mm aggregate respectively. However, the limits specified in IS 383 for concrete to be used in wearing surface and in normal concrete are

30% and 45%. RAP as well as NA conforms to the criteria suggested by IS 383 as the values are within the limits. In the present study, the abraded RAP exhibited better impact resistance than the NA. This is due to the fact that RAP underwent abrasion and attrition technique which removed the contaminant layers from the aggregate hence resulting in better impact value. Also, presence of less brittle asphalt film around RAP aggregates could absorb more impact load thereby increases the impact resistance of the same significantly compared to NA.

e. **Abrasion value**

The abrasion value of the aggregates was performed following the procedure given in IS 2386 (Part 4) using Los Angeles Abrasion testing machine. A total of 5 kg mass of aggregate comprising of 2.5 kg of 20 mm passing and 12.5 mm retaining and 2.5 kg of 12.5 mm passing and 10 mm retaining aggregates have been taken. These aggregates are subjected to 500 revolutions with 11 charge balls. The abrasion value of abraded RAP was found to be 25.654% and that of NA was found to be 27.34% respectively. These values are found to be within the limits specified in IS 383 i.e., less than 30% for concrete used in wearing surfaces and 50% for aggregates in normal concrete. The abrasion value of RAP aggregates was found to be lower as compared to the NA, the reason behind this was the removal of dust layer and asphalt film around the RAP aggregates due to processing technique used.

g. Asphalt content

The asphalt content of RAP is determined by following the procedure given in ASTM D2172. In this a 500g sample of coarse RAP is taken and it is placed in the bitumen extractor i.e., centrifuge machine revolving at a speed of 3600 rpm/min. A bitumen removing solvent i.e., trichloroethylene is used to remove the binder content from the RAP aggregates. The asphalt content of normal RAP was found to be 4.5% while that of abraded RAP was found to be 2.2%. The difference in the asphalt content was due to the processing technique that was used to remove the contaminant layers as well as reduce the binder content of the RAP aggregates. It was found that there was a decrease of more than 60% in the binder content value of abraded RAP as compared to the normal RAP. Fig. 3.14 shows the centrifuge machine for bitumen extraction and Fig. 3.15 shows the RAP aggregates after bitumen is extracted.

Figure 3.14 Centrifuge Machine

Figure 3.15 RAP aggregate after bitumen is extracted

Therefore, the properties of both abraded RAP and NA discussed above are summarized and compared with the values specified in some of the existing specifications and are presented in Table 3.2. It was found that the aggregate used in the study satisfies the quality requirements specified in various codes as can be seen in Table 3.2.

Table 3.2 Properties of coarse aggregate used in present study

Property	Natural Aggregate		Abraded RAP	Acceptance Criteria
	20 mm	10 mm		
Specific Gravity	2.793	2.846	2.464	2.3 - 2.9 (ACI E1 - 07)
Water Absorption (%)	0.284	0.236	0.58	2.0 (IS 2386 (Part 5))
Bulk Density (kg/m^3)	1.676	1.713	1.732	1280 - 1920 (ACI E1 - 07)
%age Voids (%)	39.992	39.81	29.707	-
Crushing Value (%)	24.361		21.72	30 (IS 383)
Impact Value (%)	15.351	22.808	14.6	30 (IS 383)

Abrasion Value (%)	27.34		25.654	30 (IS 383)
Asphalt Content (%)	-	-	2.2	-

3.2.2 Fine Aggregate

Crushed stone sand available in Delhi is used as fine aggregate in this study. The various properties of sand obtained from the laboratory study are presented below.

a. Particle Size Distribution

The particle size distribution of the fine aggregate used in this study had a fineness modulus of 2.95 hence was conformed to the grading zone 1 as shown in Fig. 3.16.

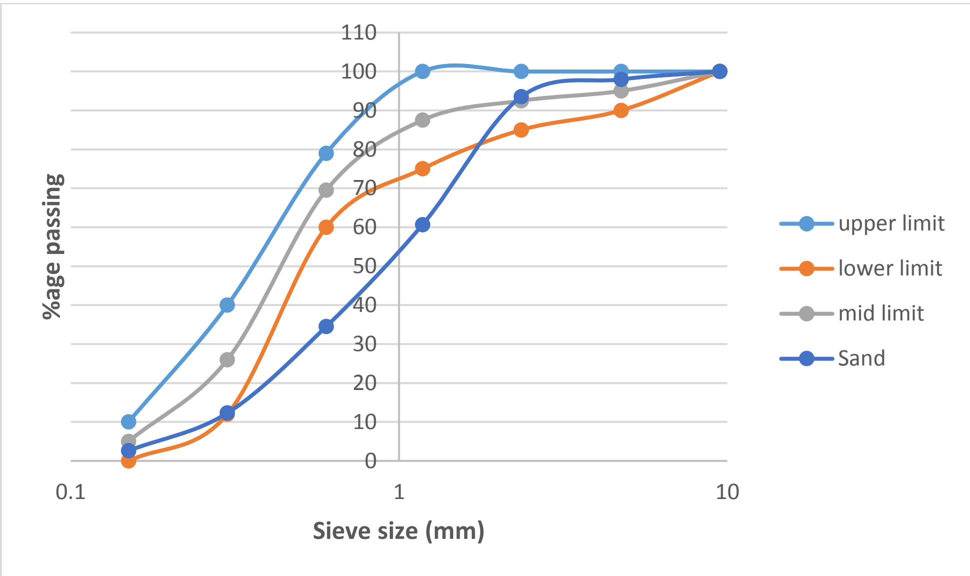

Figure 3.16 Particle size distribution curve of fine aggregate

b. Specific Gravity and water absorption

The specific gravity and water absorption of fine aggregates was determined as per IS 2386 (Part 3). The specific gravity of fine aggregates was found to be 2.55 and the water absorption of value obtained was 1.38%.

c. Bulk Density and percentage voids

The bulk density of sand was determined by using procedure given in IS 2386 (Part 3). It was found that the bulk density of sand was 2257 kg/m^3 and the percentage voids was found to be 12.83%.

3.2.3 Cement

Ordinary Portland cement of grade 43 conforming to IS 8112 is used. The various properties of the cement are discussed below.

a. Consistency

The consistency of cement was determined as procedure given in IS 4031 (Part IV) using Vicat's apparatus. The standard consistency of cement used in the present study was found to be 28.5%.

b. Specific Gravity

The specific gravity of cement was determined as per procedure given in IS 4031(Part 11) using Le-Chatelier appararus following Le-Chatelier principle. It was found that specific gravity of cement was 3.147 which was used in the present study.

c. Setting time

The initial and final setting time of cement was determined as per procedure given in IS 4031 (Part V). In order to determine the initial setting time the rod bearing needle was used and to determine the final setting time needle with annular attachment was used. The initial and final setting time of the cement used in the present study is given below.

Initial setting time	:	2 hours and 54 minutes
Final setting time	:	6 hours and 43 minutes

d. Cement Strength Test

The cement strength was determined as per procedure given in IS 4031 (Part VI) and the results of the same are presented in the Fig 3.17. However, these values obtained complies with the values specified in IS 8112.

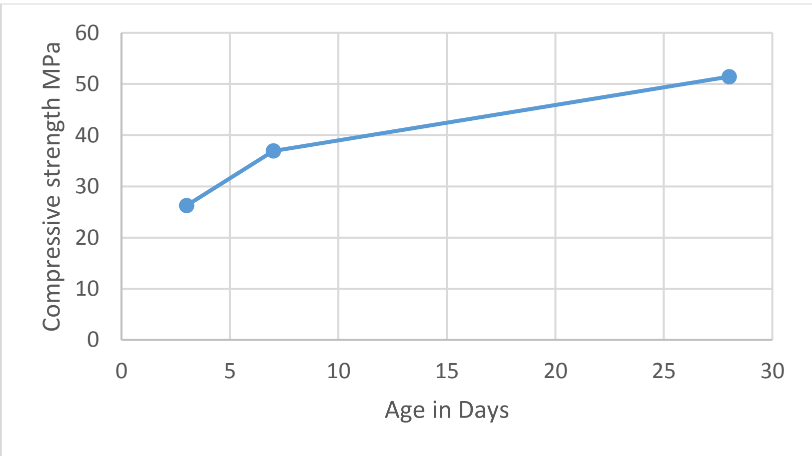

Figure 3.17 Compressive Strength of cement in days

3.2.4 Ground Granulated Blast Furnace Slag (GGBS)

The GGBS used in this study was manufactured at JSW cement Ltd. in Andhra Pradesh and it was collected from Hyderabad.

a. Wet sieving

The wet sieve analysis was performed as per procedure given in IS 1727 (1967) and it was found that the particles retained on 45-micron sieve was found to be 4.1% which is well within the limits as specified in IS 3812 (Part 2). Hence, GGBS is suitable for use in concrete as an admixture or filler material. Fig. 3.18 shows the wet sieving of GGBS.

Figure 3.18 Wet sieving of GGBS

b. Specific gravity

The specific gravity of GGBS was determined as per procedure given in IS 1727 (1967) using Le-Chatelier appararus following Le-Chatelier principle. It was found that specific gravity of GGBS was found to be 2.91 which was used in the present study.

3.2.5 Silica Fume

The silica fume used in this study was of Grade 920D. The specific gravity of silica fume was determined as per procedure given in IS 1727 (1967) and it was found to be 2.22 which was used in the present study.

3.2.6 Superplasticizer

The plasticizer used in the present study is Glenium SKY 8233, based on second generation poly carboxylic ether polymers, and developed using Nanotechnology. The specifications of the super plasticizer as given by the manufacturer as presented in the Table 3.3.

Table 3.3 Specifications of Super Plasticizer

Aspect	Reddish Brown Liquid
Relative Density	1.08 ± 0.02 at 25° C
pH	≥6
Chloride ion content	< 0.2%

3.2.7 Water

The water used for mixing was free from oil, organic material and other impurities. The pH of water was found to be 7.2 with no chloride content. Hence, the water was suitable for curing as well as concrete mixing.

3.3 MIX DESIGN

The properties of various materials used in this study have been discussed in Section 3.2. Mix proportioning of concrete mixes for the necessary characteristic compressive, flexure, and split tensile strength is carried out utilising the above materials, and the details are presented in this section.

12 concrete mixes of target strength 40 MPa (Compressive) and 4.5 MPa (Flexure) were prepared and are shown below.

1. 0% RAP(CM1)
2. 20% RAP (20R)
3. 40% RAP (40R)
4. 60%RAP (60R)
5. 0% RAP with 40% GGBS (CM2G)
6. 20% RAP with 40% GGBS (20RG)
7. 40% RAP with 40% GGBS (40RG)
8. 60% RAP with 40% GGBS (60RG)
9. 0% RAP with 40% GGBS and 10% SF(CM3GS)
10. 20% RAP with 40% GGBS and 10% SF (20RGS)
11. 40% RAP with 40% GGBS and 10% SF (40RGS)
12. 60% RAP with 40% GGBS and 10% SF (60RGS)

3.3.1 Mix Design by IRC 44 - 2017

Firstly, the design of both the control mix and RAP mix has been done as per the procedure given in IRC 44 – 2017 based on flexure strength of 4.5 MPa and slump value ranging between 25 to 35 mm. The input parameters and various steps in the mix design are given below.

A1 Stipulations for proportioning

Grade designation	4.5 N/mm^2 Flexural Strength
Type of cement	OPC 43 grade conforming to IS: 269
Maximum nominal size of 20mm aggregate	19 mm
Maximum nominal size of 10 mm aggregate	9.5 mm
Minimum cement content (as per IRC: 15)	360 kg/m^3
Maximum water-cement ratio (as per IRC: 15)	0.40
Workability	25 ± 10 mm (slump)
Degree of supervision	Good
Type of aggregate	Crushed angular aggregate
Maximum cement content	450 kg/m^3
Chemical admixture type	Superplasticizer

A2 Test data for materials

Cement Used	OPC 43 grade conforming to IS: 269
Specific Gravity	
Coarse Aggregate (20 mm)	2.793 (Section 3.2.1c)
Coarse Aggregate (10 mm)	2.846 (Section 3.2.1c)
Fine Aggregate	2.55 (Section 3.2.2b)
Water Absorption	
Coarse Aggregate (20 mm)	0.284 (Section 3.2.1c)
Coarse Aggregate (10 mm)	0.236 (Section 3.2.1c)

Fine Aggregate	1.38 (Section 3.2.2b)
Free Surface moisture	
Coarse Aggregate (20 mm)	Nil
Coarse Aggregate (10 mm)	Nil
Fine Aggregate	Nil
Sieve Analysis	
Coarse Aggregate (20 mm)	As per Table 1 of IRC 44
Coarse Aggregate (10 mm)	As per Table 1 of IRC 44
Fine Aggregate	Zone 1 of Table 2 of IRC 44

A3 Design flexural strength for mix proportioning

Target strength using both equations i.e.

1. $f'_{cr} = f_{cr} + 1.65\ S_f$ $= 4.5 + 1.65 \times 0.40$
   $= 5.16\ N/mm^2$

2. $f'_{cr} = f_{cr} + 0.45$ $= 4.5 + 0.55$
   $= 5.05\ N/mm^2$

The higher value is to be adopted. Therefore, target strength will be $5.16\ N/mm^2$ as $5.16\ N/mm^2 > 5.05\ N/mm^2$.

A4 Approximate air content

From Table 7 of IRC 44, the approximate amount of entrapped air to be expected in normal (non-air entrained) concrete is 1.0 percent for 9.5 mm nominal maximum size of aggregate.

A5 Selection of water-cement ratio

From various trails performed in this study the water cement ratio selected was taken as 0.38.

A6 Selection of water content

Water content $= 425 \times 0.38$

$= 161.5\ kg/m^3$

The above proposed water content is with the addition of superplasticizer. Since when superplasticizer is proposed then reduction in water content is taken as 20% as per IRFC 44.

Therefore, after performing various trails with the addition of 0.25% of Superplasticiser, it was found that the water cement ratio is taken as 0.38.

A7 Calculation of cement content

From various studies it was proposed that the cement content to be taken in this study shall be 425 kg/m^3. This cement content is well within the range of maximum and minimum cement content as per IRC 15.

A8 Proportion of volume of coarse aggregate and fine aggregate content

Since the proportioning of coarse aggregate and fine aggregate is done as per Table 11 of IRC 44. But in the present study the blending of aggregates was done according to the gradation performed in the laboratory and it was found that the gradation met the required limits as per IRC 44.

Volume of coarse aggregate per unit volume of total aggregate (20 mm)	=	0.63
Volume of coarse aggregate per unit volume of total aggregate (10 mm)	=	0.37
Volume of coarse aggregate content per unit total volume of aggregate (20 mm)	=	0.3906
Volume of coarse aggregate content per unit total volume of aggregate (10 mm)	=	0.2294
Volume of fine aggregate content per unit total volume of aggregate	=	0.38

A9 Mix calculations

a) Absolute Volume of concrete = 1- volume of air

= 1- 0.001

= 0.99 m^3

b) Volume of cement $= \frac{\text{Mass of cement}}{\text{Specific gravity of cement}} \times \frac{1}{1000}$

$= \frac{425}{3.147} \times \frac{1}{1000}$

= 0.1349 m^3

c) Volume of water $= \frac{\text{Mass of water}}{\text{Specific gravity of water}} \times \frac{1}{1000}$

$= \frac{161.5}{1} \times \frac{1}{1000}$

= 0.1615 m^3

d) Volume of superplasticizer = $\frac{\text{Mass of SP}}{\text{Specific gravity of SP}} \times \frac{1}{1000}$

= $\frac{1.0625}{1.08} \times \frac{1}{1000}$

= 0.001 m^3

e) Volume of all in aggregate = {a - (b + c + d)}

= 0.99-(0.1349+0.1615+0.001)

= 0.6926 m^3

f) Mass of coarse aggregate (20 mm) = (e) × volume of coarse aggregate × Specific gravity of coarse aggregate × 1000

= 0.6926× 0.3906 × 2.793 × 1000

= 755.59 kg/m^3

g) Mass of coarse aggregate (10 mm) = (e) × volume of coarse aggregate × Specific gravity of coarse aggregate × 1000

= 0.6926× 0.2294 × 2.846 × 1000

= 452.18 kg/m^3

h) Mass of fine aggregate = (e) × volume of fine aggregate × Specific gravity of fine aggregate × 1000

= 0.6926 × 0.38 × 2.55 × 1000

= 661.94 kg/m^3

A10 Mix Proportions based on Aggregate in Dry Condition

Cement = 425 kg/m^3

Water = $128 + 8.53^* + 2.14^{**} + 1.06^{**}$

= 173.23 kg/m^3

Chemical Admixture = 1.0625 kg/m^3

Fine Aggregate $= \frac{\text{Mass of fine aggregate in SSD condition}}{1+\frac{water\ absorption}{100}}$

$= \frac{670.47}{1+\frac{1.38}{100}}$

= 661.94 kg/m^3

Coarse Aggregate (20 mm) $= \frac{\text{Mass of coarse aggregate in SSD condition}}{1+\frac{water\ absorption}{100}}$

$= \frac{755.59}{1+\frac{0.284}{100}}$

= 754.45 kg/m^3

Coarse Aggregate (10 mm) $= \frac{\text{Mass of coarse aggregate in SSD condition}}{1+\frac{water\ absorption}{100}}$

$= \frac{452.18}{1+\frac{0.236}{100}}$

= 451.12 kg/m^3

*Extra water to be absorbed by dry fine aggregate = 670.47 – 661.94

= 8.53 kg/m^3

** Extra water to be absorbed by dry coarse aggregate (20mm)

= 755.59 – 754.45

= 2.14 kg/m^3

** Extra water to be absorbed by dry coarse aggregate (10mm)

= 452.18 – 451.12

= 1.06 kg/m^3

Thus, the above procedure gives the mix design for the control concrete CM1. In the other mixer in which RAP is added with different proportions as well as GGBS and Silica Fume is added as supplementary cementitious material similar procedure for the mix design is followed. Table 3.4 gives the mix proportions for the various concrete mixes shown in the table.

Table 3.4 Mix proportion in the study

Mix	Cement kg/m³	Water kg/m³	CA kg/m³			FA kg/m³	GGBS kg/m³	SF kg/m³	SP kg/m³
			NA 20mm	NA 10mm	RAP				
CM1	425	161.5	753.45	451.12	-	661.94	-	-	1.064
20R	425		602.76	360.89	210.39	661.94	-	-	
40R	425		452.07	270.67	420.79	661.94	-	-	
60R	425		301.39	180.44	631.18	661.94	-	-	
CM2G	255		682.85	408.85	-	606.66	170	-	
20RG	255		551.77	330.36	192.59	606.66	170	-	
40RG	255		413.83	247.77	385.19	606.66	170	-	
60RG	255		275.88	165.18	577.78	606.66	170	-	
CM3GS	212.5		753.45	451.12	-	661.94	170	42.5	
20RGS	212.5		602.76	360.89	210.39	661.94	170	42.5	
40RGS	212.5		452.07	270.67	420.79	661.94	170	42.5	
60RGS	212.5		301.39	180.44	631.18	661.94	170	42.5	

3.4 DETAILS OF THE TEST SPECIMEN

The details such as property investigated, the size and number of specimens are presented in Table 3.5.

Table 3.5 Details of Test Specimens

Property	Test Age (Days)	Specimen size (mm)	N0. of Specimens (per mix - per test age)
Destructive Testing			
Compressive strength	7	Cube Size: 100	3
	28		3
	90		3
Flexural Strength	7	Beam Size: 500x100x100	3
	28		3
	90		3

Split Tensile Strength	7	Cylinder Diameter: 150 Height: 300	3
	28		3
	90		3
Drying Shrinkage	28	Prism Size: 300x80x80	3
Water Absorption	28	Cube Size: 100	3
Abrasion Resistance	28	Slab Size: 500x400x100	1
Sand Blasting	28	Cube Size: 100	3
Skid Resistance	28	Slab Size: 500x400x100	1
Non-Destructive Testing			
Ultrasonic Pulse Velocity	28	Cube Size: 100	3
Rebound Hammer Number	28	Slab Size: 500x400x100	1
Total Number samples per mix			**42**

3.5 CASTING OF THE TEST SPECIMEN

The concrete specimens were casted for the mixes CM1, 20R, 40R, 60R, CM2G, 20RG, 40RG, 60RG, CM3GS, 20RGS, 40RGS and 60RGS to evaluate the strength and durability of the Pavement quality concrete containing different proportions of RAP along with GGBS and SF. The various steps involved in the preparation of the specimen are discussed in this section.

3.5.1 Preparation of moulds

a. Cubes, Beams, Cylinders and Prisms

The cube moulds of size 100 mm × 100 mm × 100 mm size, beams moulds of size 500 mm × 100 mm × 100 mm, cylinder moulds of size 50 mm diameter and 200 mm height and 150 mm diameter and 300 mm height, prism moulds of size 300 mm × 80 mm × 80 mm conforming to IS: 10086 – 1982 have been used. Prior to the casting the interior surface of the moulds were initially cleaned and coated with oil in order to prevent adhesion of concrete.

Figure 3.19 Prepared moulds of Cylinders and Cubes

Figure 3.20 Prepared moulds of Beams

b. Slabs

The slabs that were used at the time of casting were of size 500 × 400 × 100 mm. the slabs were cleaned, tightened and placed over the vibrating table in order to form the base of the mould.

Figure 3.21 Prepared mould of slab

3.5.2 Mixing of concrete

The batch mixer of capacity 300 kg was used in order to mix the concrete prepared in the laboratory as shown in the Fig 3.22. The temperature and relative humidity during the mixing and casting process were 25±3°C and 60±10% respectively. The coarse aggregate was first loaded in the batch mixer followed by fine aggregate, abraded RAP and then cement, GGBS, SF were added. The mixer was allowed to operate for about 5 minutes in order to get a uniform dry mass of material. After that half of the water was added and the mixer was allowed to mix for 4 minutes. The mixer was stopped for 3-4 minutes, and super-plasticizer dosage was added to balance amount of water, which was then added to the mixer and the rotation was continued for another 4 minutes. In case of silica fume mixes the rotation of the mixer was reduced along as silica fume has the tendency to absorb more water.

Figure 3.22 Laboratory Batch Mixer

3.5.3 Placing of Concrete

Concrete was filled in the prepared moulds in two layers and vibrated using table vibrator. The following precautions were taken at the time of casting of the test specimens:

a. The moulds were placed on the vibrating table and after filling the moulds with concrete, at the time of vibration care was taken that none of the specimen fell of the table.
b. The slab moulds were placed on levelled surface and then filled with concrete in layers and proper compacted of concrete is ensured due to vibration. Fig 3.26 shows the compaction of fresh concrete in slabs.

After filling, the moulds were kept on levelled surface and excess concrete was struck off and the specimen surface was levelled and finished with a trowel. Fig 3.25 shows finished and levelled specimen.

Figure 3.23 Concreting of Beams and Cubes

Figure 3.24 Concreting of Cylinders and Prisms

Figure 3.25 Finished and levelled specimens

Figure 3.26 Finished and levelled slab

3.5.4 Demoulding and Curing of Concrete Specimens

The casted specimens were covered with jute bags to prevent evaporation until demoulding was done. The demoulded specimens were marked with the mix identification and date of casting and placed in a water tank for curing as shown in Fig. The test specimens were water cured for a maximum of 28 days at a curing temperature of 22°C.

Figure 3.27 Curing Tank for specimens

3.6 SUMMARY

The summary of the work presented in this chapter is as follows:

1. The properties of the RAP, NA, fine aggregate, cement, GGBS, silica fume and water used in the present study were experimentally determined.
2. Based on the established properties mix design was carried out to achieve a concrete of desired characteristic compressive, flexure and split tensile strength as per IRC 44 – 2017.
3. Beneficiation technique "Abrasion and Attrition Technique" was used in order to remove the contaminant layers from the RAP aggregates.
4. Addition of supplementary cementitious materials SF and GGBS was used in order to enhance the mechanical and durability properties of RAP inclusive PQC.
5. The tests on the specimens and observed properties are discussed in Chapter 4.

CHAPTER 4

4 MECHANICAL, DURABILITY AND TIME DEPENDENT PROPERTIES OF PAVEMENT QUALITY CONCRETE WITH INCULSION OF RAP

4.1 INTRODUCTION

The casted and cured concrete specimens of the mixes CM1, 20R, 40R, 60R, CM2G, 20RG, 40RG, 60RG, CM3GS, 20RGS, 40RGS and 60RGS were subjected to testing at different ages viz. 7, 28 and 90 days in order to study various structural properties. This chapter includes the testing of the above concrete specimen mixes for evaluating their respective strength, durability and time dependent properties along with failure pattern observed in various concrete mixes discussed above. Also, the results obtained for the RAP replaced PQC were compared with the results of NA concrete mixes.

4.2 WORKABILITY

4.2.1 Slump Test

The slump of the concrete mixes was determined as per procedure given in IS 1199 (1959). This test is performed in order to get the indication about the cohesiveness and workability of the concrete mix. The slump test apparatus consists of a copper mould in the shape of a frustum of cone with internal dimensions of 20 cm bottom diameter, 10 cm top diameter, and 30 cm height. The slump value used for Pavement Quality Concrete should be between 25 mm and 50 mm. Most of results of the slump value were in range of 25 to 50 mm and are presented in the Fig. 4.1 It was observed that the replacement of natural aggregates by abraded coarse RAP resulted in the increase in the slump value. However, it can be observed that the increase is very marginal and is almost same as that of the slump of control mix CM1. The reason behind increase in the slump value may be due to the removal of dust layer as a result of processing technique used in order to remove the contaminant layers present in RAP. Another reason behind increase in slump value can be due to the lower water absorption of coarse RAP as compared to the NA. Also, replacement of cement by 40% GGBS was found to increase the slump value of PQC, and it was found that increase in the slump value of RAP inclusive PQC along with 40% GGBS was 33.33% for 20RG, 60% for 40RG and 83.33% for 60RG as compared to the control mix CM2G. However, when 10% silica fume was added along with

40% of GGBS it was found that the slump of the control mix CM3GS was less as compared to the slump of control mixes CM1 and CM2G. The reason behind the lower slump value may be due to the high fineness and more specific surface area of micro silica resulting in consumption of more water present in the matrix. Also, it can be observed that the slump value of concrete mixes increases with the addition of 10% SF and 40% GGBS together as compared to the control mix CM3GS. The increase in the slump value of RAP inclusive PQC along with 40% GGBS and 10% SF was 13.33% for 20RGS, 46.66% for 40RGS and 93.33% for 60RGS as compared to the control mix CM3GS. Fig. 4.2 and 4.3 shows the slump test performed in the laboratory of PQC mix with 40% GGBS and normal PQC.

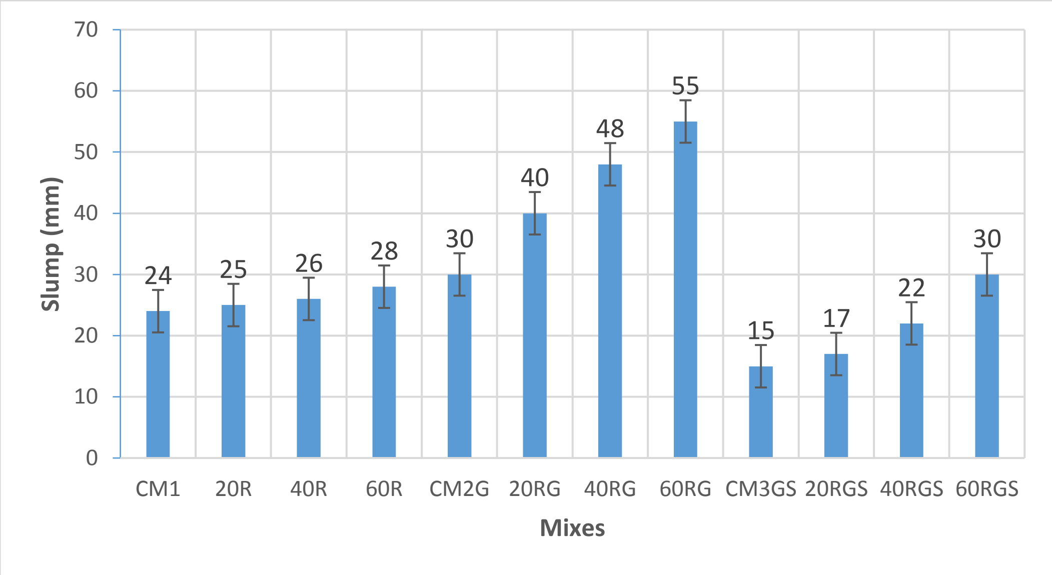

Figure 4.1 Slump Value of various mixes used in the present study

Figure 4.2 Slump of concrete mix with GGBS

Figure 4.3 Slump of normal concrete mix

4.2.2 Compaction Factor Test

The compaction factor test was performed as per procedure given in IS 1199 1959. This test is performed in order to determine the workability of concrete mixes when the nominal aggregate size does not exceed 38 mm. This test is performed only in laboratory however if the conditions permit it can be performed in the field also. This test is much more precise and accurate as compared to the slump test and is mostly used in case of concrete mixes with low workability. The compactor factor determined in this procedure is taken as the ratio of weight of partially compacted concrete to the weight of fully compacted concrete. The compaction factor of various concrete mixes is shown in Fig.4.4. It was observed that the compaction factor showed an increasing trend in all the concrete mixes. The increase in the compaction factor results in the increased workability of concrete. The reason behind the increase in the workability of RAP replaced PQC is due to the removal of various dust layers present in the RAP as a result of beneficiation technique that was used in order to remove various layers viz. loose dust film, stiff dust film and asphalt film. Also, the water absorption of RAP was found to be lower as compared to the NA which resulted in increase in workability. However most of the mixes depicted medium to low workability. It was found that the concrete mixes with 40% GGBS and 10% SF exhibited least workability as compared to the other mixes. The reason behind lower workability is due to fact that SF has higher specific surface area and absorbs more water present in the matrix. Fig. 4.5 shows the compaction factor apparatus used in laboratory.

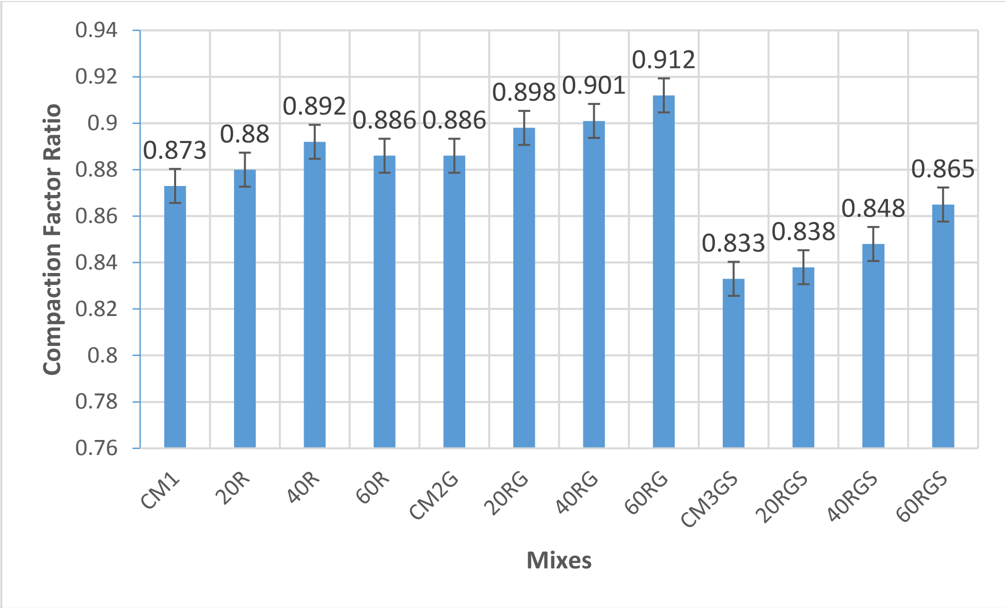

Figure 4.4 Compaction factor of various mixes used in the present study

Figure 4.5 Compaction factor apparatus used in laboratory

4.3 MECHANICAL PROPERTIES

4.3.1 Compressive Strength

The compressive strength of the hardened concrete specimens was determined at ages 7, 28 and 90 days by testing 100 mm cube specimens in accordance with IS 516 (BIS 1959). The compressive strength is computed by using the Eq. 4.1.

$$\text{Compressive Strength} = \frac{P}{A_c} \qquad 4.1$$

Where,

P = Maximum Load

A_c = Area of cross section

The test results of different concrete mixes are presented in Table 4.1. It can be found that most of the mixes surpassed the 7 days compressive strength i.e., 26 MPa except for the mix 40R and 60R having compressive strength slightly lower than 26 MPa. Also, it was found that the compressive strength of concrete mixes at 28 days were above 40 MPa except for mix 60R and 60RG with compressive strength slightly lesser than 40 MPa. However, 90 days compressive strength showed an increment of more than 10% as compared to strength obtained at 28 days.

Table 4.1 Compressive strength of concrete mixes used in present study

Mix Designation	Compressive Strength MPa		
	7 Days	**28 Days**	**90 Days**
CM1	29.29	50.59	66.50
20R	26.42	46.01	57.40
40R	25.40	40.29	50.12
60R	25.24	38.69	47.15
CM2G	28.05	48.20	60.33
20RG	27.02	47.23	53.43
40RG	26.76	41.09	49.23
60RG	26.07	39.23	45.77
CM3GS	29.60	50.40	65.88
20RGS	28.73	48.59	59.07
40RGS	27.43	45.60	54.52
60RGS	26.69	43.28	50.88

The variation of compressive strength of different concrete mixes used in this study with the age is presented in Fig. 4.6. From the figure it is seen that replacement of NA with RAP reduced the compressive strength of concrete. The similar trend was observed in the previous studies of literature review. However, in the present study the reduction was observed to be very less in comparison to other research. The reason behind lesser reduction in compressive strength was due to the processing technique that removed the asphalt layer along with other contaminant layers and also the presence of very less agglomerated particles resulted in improved strength of the mixes as compared to previous studies. One of the reasons behind improved strength as compared to previous studies is better bonding between the abraded Coarse RAP aggregates that are free from dust layers as a result of processing technique and hydrated cement paste matrix. It was observed at 7 days the reduction in the compressive strength was found to be 9.79% for 20R, 13.28% for 40R and 13.82% for 60R as compared to the control mix CM1. At 28 days the reduction in the compressive strength was found to be 9.05% for 20R, 20.3% for 40R and 22.9% for 60R as compared to the control mix CM1. However, addition of 40% GGBS did not have a significant effect on the increase in the strength of mixes. However, the 7 days strength of 40% GGBS with RAP mixes was higher as compared to normal RAP mixes. The increase in the strength was found to be 2.27% for 20RG, 5.35% for 40RG and 3.288% for 60RG as compared to the normal RAP concrete mixes. Also, the increase in the strength at 28 days was found to be 2.65% for 20RG, 1.98% for 40RG and 2.18% for 60RG as compared to normal RAP concrete mixes. As seen from the results above the increase in the strength with addition of 40% GGBS was marginal. However, addition of 10% SF along with 40% GGBS enhanced the compressive strength of concrete mix. It was found that the increase in the compressive strength at 7 days was 8.78% for 20RGS, 7.99% for 40RGS and 5.74% for 60RGS as compared to normal RAP concrete mixes. Also, the increase in compressive strength at 28 days was found to be 5.6% for 20RGS, 13.18% for 40RGS and 11.86% for 60RGS.As far as comparison with control mix is considered it was found that inclusion of 40% GGBS as well as 40% GGBS and 10% SF showed similar decreasing trend in compressive strength. With the inclusion of 40% GGBS the reduction at 7 days was found to be 3.67% for 20RG, 4.59% for 40RG and 7.05% for 60RG as compared to the control mix CM2G. Also, the decrease in strength at 28 days was found to be 2.01% for 20RGS, 14.7% for 40RGS and 18.60% for 60RGS when compared with control mix CM2G. As compared to the normal RAP mixes the decrease in reduction was found to be less. However, in case of 40% GGBS along with 10% SF the reduction was found to be least compared to all other mixes. At 7 days the reduction in concrete mixes when 40% GGBS and 10% SF is added was found to be 2.93% for 20RGS, 7.33% for 40RGS and 9.81% for 60RGS as compared with the control mix CM3GS. Also, at 28 days the reduction was found to be 3.59% for 20RGS, 9.52% for 40 RGS and 14.12% for 60RGS when compared with the control mix CM3GS. It was observed that all mixes crossed the design target strength of 40MPa at 28 days except for mixes 60R and 60RG which were slightly lesser than 40MPa. The substantial amount of activated silica that interacted with portlandite and created more CSH gel was responsible for the increase in compressive strength when 10% SF was used as a replacement of cement along with 40%

GGBS. Furthermore, SF particles with a large specific surface area filled the pores of ITZ and reinforced it, making it more dense and compacted.

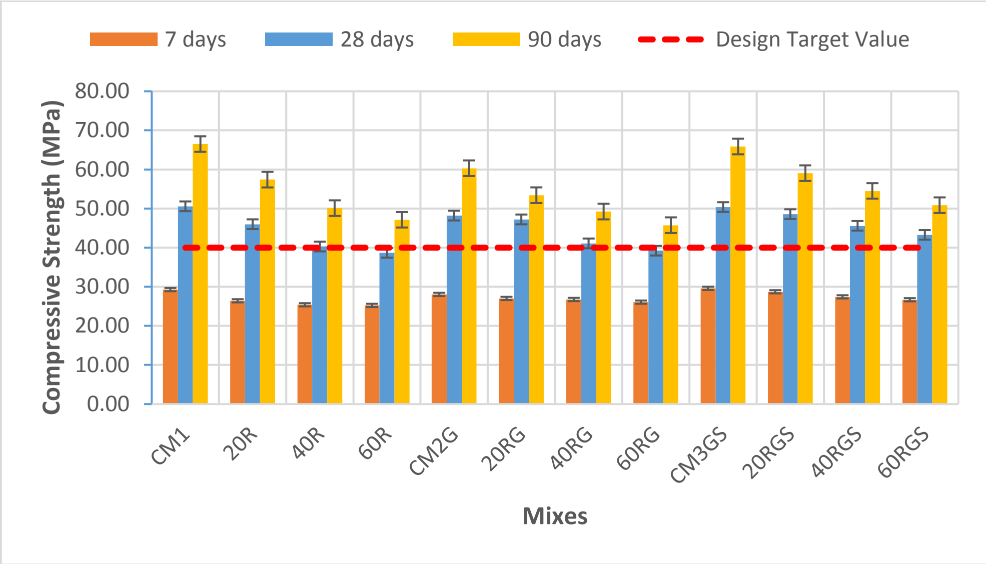

Figure 4.6 Compressive strength of various concrete mixes

Figure 4.7 Compressive strength Testing machine

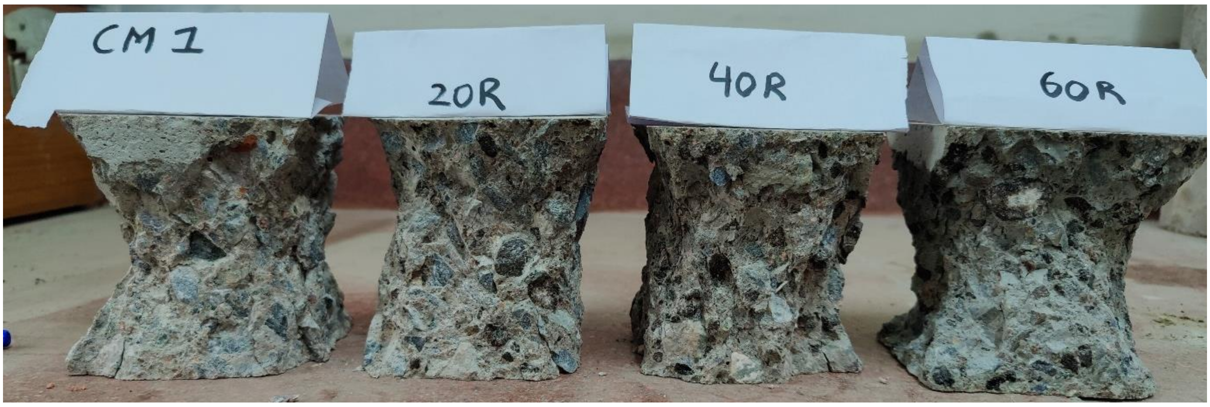

Figure 4.8 Failure Pattern of various concrete mixes under compression after 28 days

Fig. 4.7 shows the compression testing machine used to test the respective specimens and Fig 4.8 shows the tested concrete cubes under compression and it may be observed that all the mixes exhibited similar failure pattern.

4.3.2 Flexure Strength

The flexure strength of concrete was determined as per procedure given in IS 516 (BIS 1959) at 7, 28 and 90 days. The specimen of 500 × 100 × 100 mm size were evaluated using Eq. 4.2 or 4.3 depending upon the distance between the line of fracture and nearest support.

$$\text{Flexure Strength} = \frac{P \times l}{b \times d^2} \qquad 4.2$$

If, a < 13.33 cm

$$\text{Flexure Strength} = \frac{3P \times a}{b \times d^2} \qquad 4.3$$

If, a > 13.33 cm

Where,

P = maximum load (N)

l = span on which beam is supported (mm)

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

b = width of specimen (mm)

d = depth of specimen (mm)

The test results of different concrete specimens are present in the Table 4.2. It can be found that CM1 and CM3GS exhibited same flexure strength at 7 days however the strength of CM3GS was greater than as compared to CM1 at 28 days. Also, it was found that the flexure strength of all the concrete mixes at 28 days were above the target strength 4.5 MPa which is the minimum benchmark stipulated in the specifications of the Ministry of Road Transport and Highways (MORTH, 2013) for the PQC slab and the highest flexure strength was obtained for mix CM3GS. However, 90 days compressive strength showed an increment of more than 10% as compared to strength obtained at 28 days.

Table 4.2 Flexure Strength of concrete mixes used in the present study

Mix Designation	Flexure Strength MPa		
	7 Days	28 Days	90 Days
CM1	4.33	6.40	7.33
20R	4.20	5.93	6.80
40R	4.03	5.87	6.60
60R	4.00	5.80	6.53
CM2G	4.27	5.95	6.99
20RG	4.20	5.93	6.59
40RG	4.13	5.47	6.37
60RG	3.87	5.27	6.09
CM3GS	4.33	6.53	7.47
20RGS	4.27	6.33	7.10
40RGS	4.06	6.13	6.88
60RGS	3.93	6.00	6.63

The variation of flexure strength of different concrete mixes used in this study with the age is presented in Fig. 4.9. From the figure it is seen that replacement of NA with RAP showed similar trend as was observed in the compressive strength of concrete mixes. Also, it was found that in the present study the reduction in flexure strength was observed to be very less in comparison to other research. The reason behind lesser reduction in flexure strength was due to the beneficiation of RAP aggregates that removed the asphalt layer along with other contaminant layers and also removed the agglomerated particles present in RAP resulting in improved strength of the mixes as compared to previous studies. Another reason behind improved flexural strength of concrete mixes as compared to previous studies is better bonding between the abraded Coarse RAP aggregates and hydrated cement paste (The abraded coarse RAP aggregates after processing becomes free from dust layers which results in improved bonding). It was observed at 7 days the reduction in the flexure strength was found to be 3% for 20R, 6.93% for 40R and 7.62% for 60R as compared to the control mix CM1. At 28 days the reduction in the flexure strength was found to be 7.34% for 20R, 8.28% for 40R and 9.38% for 60R as compared to the control mix CM1. However, addition of 40% GGBS did not have a significant effect on the increase in the flexure strength of mixes similar to that of compressive strength. However, the 7 days strength of 40% GGBS with RAP mixes was higher in case of 40RG and same as that of normal RAP mix in case of 20RG. It was found that inclusion of 40% GGBS as well as 40% GGBS and 10% SF showed similar decreasing trend in flexure strength as was observed in compressive strength. With the inclusion of 40% GGBS the reduction at 7 days was found to be 1.64% for 20RG, 3.28% for 40RG and 9.37% for 60RG as compared to the control mix CM2G. Also, the decrease in strength at 28 days was found to be 0.34% for 20RGS, 8.37% for 40RGS and 11.43% for 60RGS when compared with control mix CM2G. The decrease in reduction in the flexure strength was found to be lesser as compared to the normal RAP mixes. However, in case of 40% GGBS along with 10% SF the reduction in flexure strength was found to be least compared to all other mixes at 28 days. At 7 days the reduction in concrete mixes when 40% GGBS along with10% SF is added was found to be 1.39% for 20RGS, 6.24% for 40RGS and 9.24% for 60RGS as compared with the control mix CM3GS. Also, at 28 days the reduction was found to be 3.06% for 20RGS, 6.13% for 40 RGS and 8.12% for 60RGS when compared with the control mix CM3GS. It was observed that all mixes crossed the design target strength of 4.5 MPa at 28 days. From the Fig 4.10 it is observed that concrete mixes with 40% GGBS plus 10% SF showed higher flexural strength as compared to normal RAP mix and RAP mixes with 40% GGBS. This improvement in flexural strength was due to the incorporation of 40% GGBS along with 10% SF resulting in formation of extra CSH gel due to reaction between silica from admixture and portlandite from hydration of cement. However, the substantial variation of the mix i.e., 40% GGBS along with 10% SF compared to the normal RAP mixes is discussed. It was observed that the increase in the flexural strength at 28 days was found to be 0.75% for 20RGS, 6.75% for 40RGS and 3.45% for 60RGS as compared to normal RAP concrete mixes. Therefore, addition of 10% SF along with 40% GGBS enhanced the flexural strength of concrete mix as compared to the normal concrete and concrete along with RAP mixes. Fig 4.10 shows the marking of the specimens before testing them for flexure strength and Fig. 4.11 shows the flexure testing machine used in the present study. Also, Fig 4.12, 4.13 and 4.14 shows various angles of crack pattern present in the beam specimen. It was observed that there the failure occurred between hydrated cement paste and aggregates present in the concrete mix. However, in some cases along with the above the failure only aggregate failure was also observed as shown in Fig 4.14.

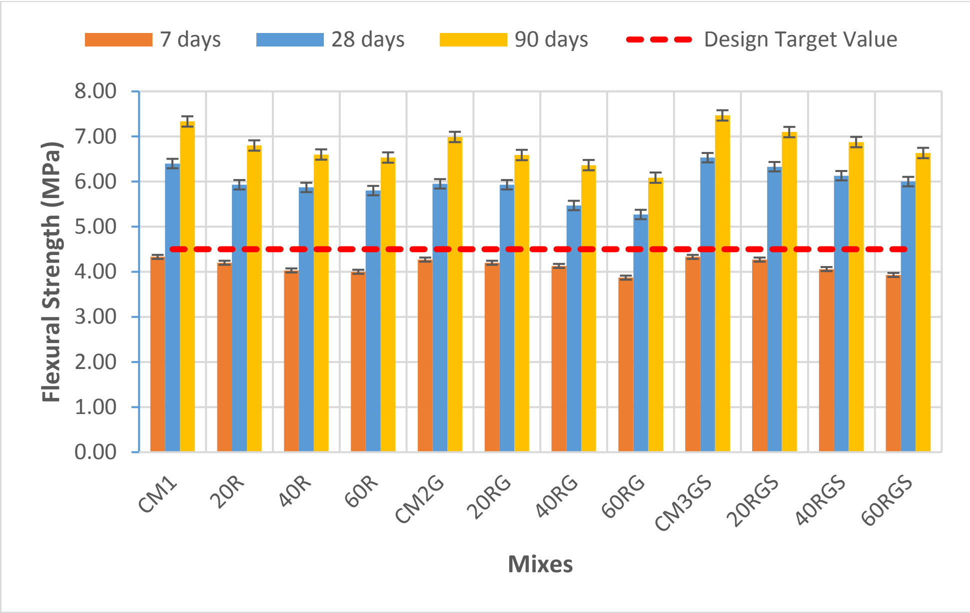

Figure 4.9 Variation in flexural strength of different concrete mixes

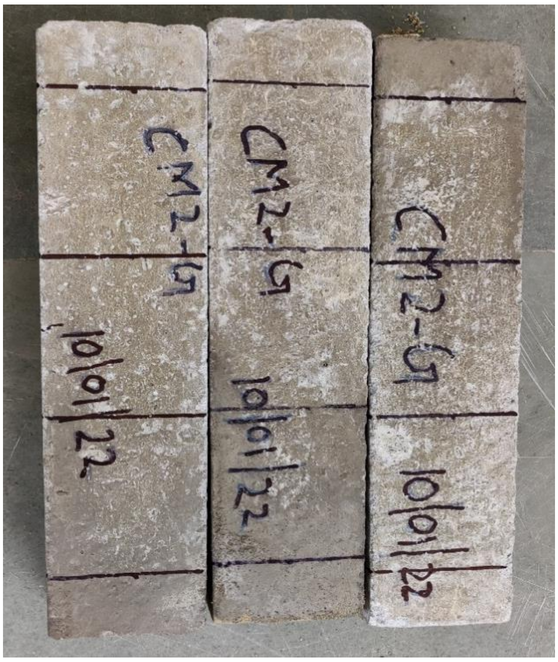

Figure 4.10 Marking of beam specimen before testing

Figure 4.11 Flexure Testing Machine

Figure 4.12 Side view of crack pattern in beam specimen used in present study

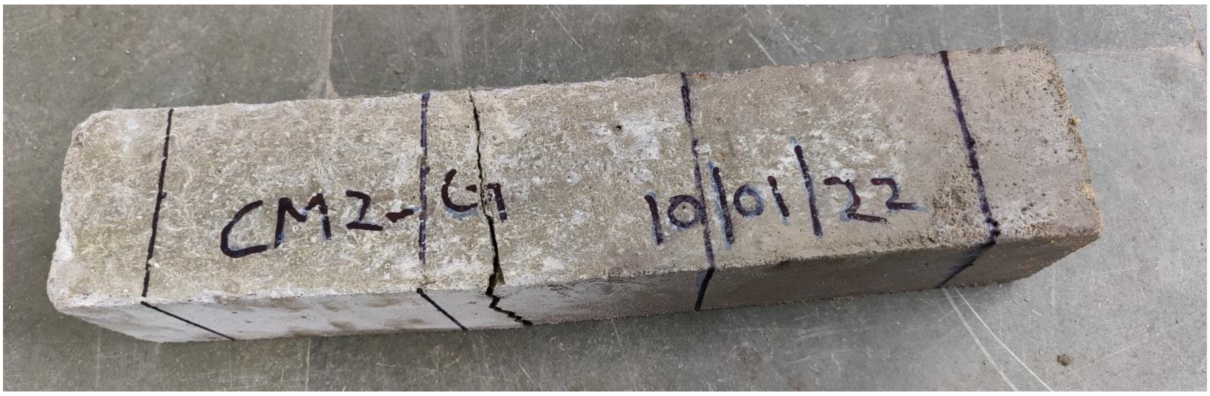

Figure 4.13 Top view of crack pattern in beam specimen used in present study

Figure 4.14 Front view of crack pattern in beam specimen used in present study

4.3.3 Split Tensile Strength

The split tensile strength of concrete was determined as per procedure given in IS 516 (BIS 1959) at 28 and 90 days. The specimen used were cylindrical concrete specimen of size 150 mm diameter and 300 mm length and 100 mm diameter and 200 mm length using Eq.4.4.

$$\text{Split Tensile Strength} = \frac{2P}{\pi LD} \quad 4.4$$

Where,

P = Maximum Load (N)

L = Length of the Specimen (mm)

D = Diameter of the specimen (mm)

The test results of different concrete specimens are present in the Table 4.3. It can be found that with increase in the replacement level of NA with abraded coarse RAP aggregate the split tensile strength got decreased. The trend was similar to that of compressive strength and flexure strength. However, it was seen that the concrete mixes with 40% GGBS along with 10% SF exhibited higher split tensile strength as compared to the rest of the concrete mixes. The reason behind higher split tensile strength of the above concrete mix is due to the formation of extra CSH gel owing to reaction between silica from admixture and portlandite from hydration of cement. However, it can be observed that the overall split tensile strength of the concrete mixes is much higher as compared to the previous studies. The reason behind the higher split tensile strength of the concrete mix is due to the processing technique used for the RAP in order to decrease the asphalt content present in the RAP and to improve the bonding between the RAP aggregates and hydrated cement paste present in the matrix.

Table 4.3 Split Tensile Strength of various concrete mixes used in the present study

Mix Designation	Split Tensile Strength MPa	
	28 Days	90 Days
CM1	4.78	5.01
20R	4.29	4.88
40R	4.01	4.65
60R	3.90	4.35
CM2G	4.88	5.23
20RG	4.51	4.97
40RG	4.45	4.68

60RG	4.20	4.44
CM3GS	5.13	5.69
20RGS	4.62	5.02
40RGS	4.57	4.97
60RGS	4.54	4.71

The variation of split tensile strength of different concrete mixes used in this study with the age is presented in Fig. 4.15. It was observed at 28 days the reduction in the flexure strength was found to be 10.25% for 20R, 16.11% for 40R and 18.41% for 60R as compared to the control mix CM1. It was found that inclusion of 40% GGBS as well as 40% GGBS and 10% SF showed similar decreasing trend in split tensile strength as was observed in compressive and flexural strength. With the inclusion of 40% GGBS the reduction at 28 days was found to be 7.58% for 20RG, 8.81% for 40RG and 13.93% for 60RG as compared to the control mix CM2G. This reduction was lesser as compared to the reduction observed in normal RAP mixes. However, in case of 40% GGBS along with 10% SF the reduction in split tensile strength was found to be least compared to all other mixes at 28 days. At 28 days the reduction in concrete mixes when 40% GGBS along with 10% SF is added was found to be 9.94% for 20RGS, 10.92% for 40RGS and 11.50% for 60RGS as compared with the control mix CM3GS. From the Fig. 4.15 it is observed that concrete mixes with 40% GGBS plus 10% SF showed higher split tensile strength as compared to normal RAP mix and RAP mixes with 40% GGBS. This improvement in flexural strength was due to the incorporation of 40% GGBS along with 10% SF resulting in formation of extra CSH gel due to reaction between silica from admixture and portlandite from hydration of cement. However, the substantial variation of the mix i.e., 40% GGBS along with 10% SF compared to the normal RAP mixes is discussed. It was observed that the increase in the split tensile strength at 28 days was found to be 7.7% for 20RGS, 13.96% for 40RGS and 16.41% for 60RGS as compared to normal RAP concrete mixes. Therefore, addition of 10% SF along with 40% GGBS enhanced the split tensile strength of concrete mix as compared to the normal concrete and concrete along with RAP mixes. Fig. 4.16 shows the split tensile machine used for testing of various specimens used in the present study and Fid. 4.17 shows the failure of the specimen during the loading. Also, Fig 4.18 and Fig. 4.19 shows the top and front view of specimens failed under split tensile test. Visual observation of testes specimen after failure showed that the surface of failure is rough and uneven and the specimen's failure surface passed through the ITZ and through the NA and RAP aggregates. However, 90 days compressive strength showed an increment of more than 5 to 10% as compared to strength obtained at 28 days.

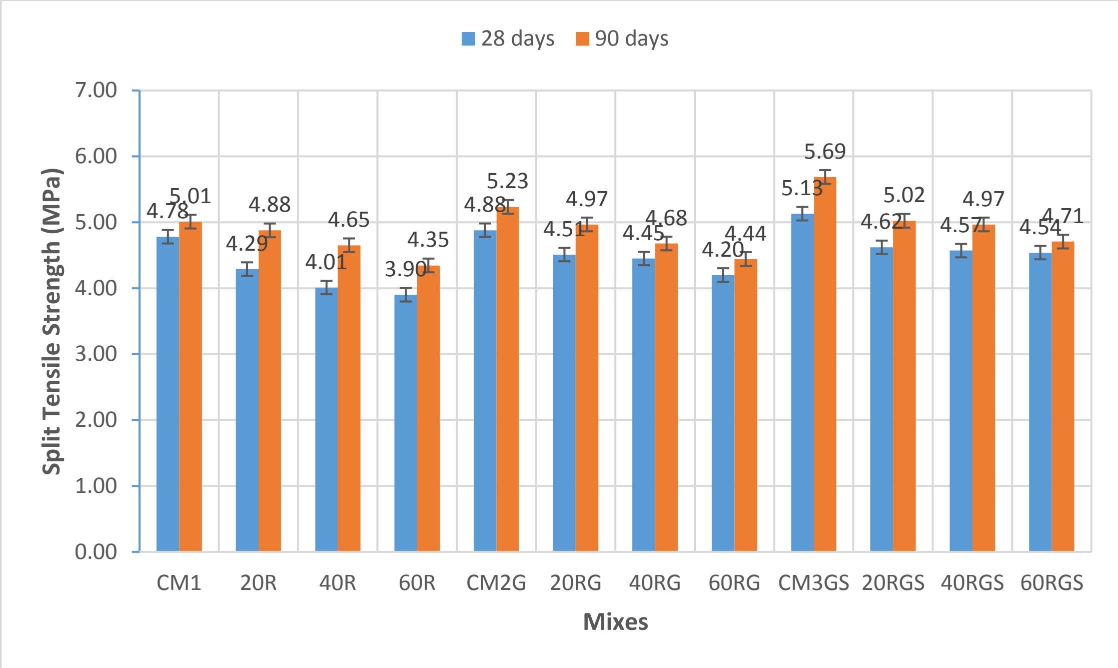

Figure 4.15 Variation in split tensile strength of different concrete mixes

Figure 4.16 Split Tensile Testing Machine

Figure 4.17 Failure of the specimen during loading

Figure 4.18 Top view of specimen failed in split tensile test

Figure 4.19 Front view of specimen failed in split tensile test

4.4 DURABILITY PROPERTIES

4.4.1 Hardened Density

The density of concrete specimens was determined as per procedure given in ASTM C 642 -97. This test method is important for generating the data needed for concrete mass and volume conversions. It can be used to determine whether concrete meets standards and to highlight variances between points within a mass of concrete. The results are present in Fig 4.20. It was observed that with the increase in the percentage of coarse RAP the density of the PQC mixes showed a decreasing trend as compared to the control mix CM1. The reason behind decrease in the density of concrete mixes was due to lower specific gravity of coarse RAP as compared to NA. The decrease in the density of RAP inclusive PQC mix was found to be 3.2% to 4.3% respectively as compared to control mix CM1. However, the decrease in the concrete mixes was found to be marginal and all of the concrete mixes had density above 2400 kg/m^3 except 60RG. The reason behind higher density of RAP inclusive PQC mixes was due to the processing technique that removed the agglomerated particles present in RAP and also due to the higher bulk density and percentage voids of RAP aggregates as compared to NA. Also, similar trend was observed with the addition of 40% GGBS and, 40% GGBS and 10% SF. It was observed that the decrease in the density of RAP inclusive PQC mix with addition of 40% GGBS was found to be 4.3% to 4.88% as compared to the control mix CM2G. Also, when 40% GGBS along with 10% SF was added to the RAP inclusive PQC mixes it was found that the decrease in the density of concrete mixes was in the range of 2.88% to 3.94% as compared to the control mix CM3GS.

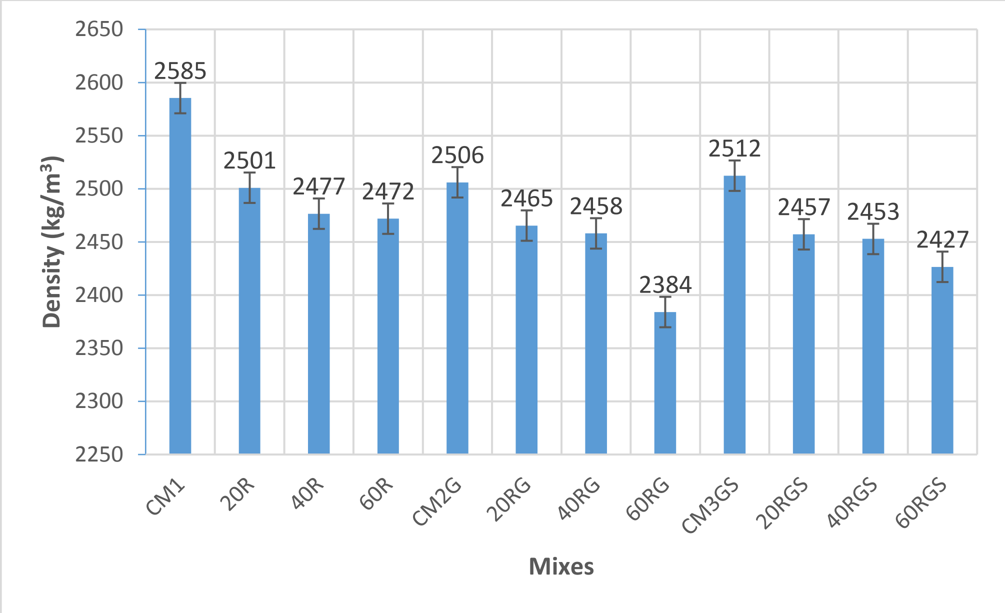

Figure 4.20 Density of various concrete mixes used in present study

4.4.2 Water Absorption

The variation of water absorption for various concrete mixes is shown below in the Fig. 4.21. The water absorption of the concrete mixes were taken after 28 days of moist curing. It can be observed that the water absorption increases with the increase in the proportion of abraded RAP aggregates. The increase in the water absorption of RAP inclusive concrete mixes was found to be 15.36% for 20R, 19.44% for 40R and 57.99% for 60R as compared to the control mix CM1. However, when 40% GGBS was added similar increasing trend was observed. The increase in the water absorption was found to be 18.52% for 20RG, 29.38% for 40RG and 30.37% for 60RG as compared to the control mix CM2G when 40% GGBS was added along with the inclusion of RAP aggregates in PQC. Also, when 10% SF is added along with 40% GGBS the waster absorption was found to be lesser as compared to other mixes however, it showed the same increasing trend. The increase in the water absorption was found to be 9.8% for 20RGS, 25.88% for 40RGS and 41.18% for 60RGS as compared to the control mix CM3GS when 10% SF was added along with 40% GGBS in RAP inclusive Pavement quality concrete. It is seen that higher water absorption of mixes is usually due to higher water absorption of aggregates as a result of porousness, but this does not apply in case of concrete mixes containing RAP aggregates. The main reason is due to the presence of asphalt film in the RAP aggregates that results in the increasing trend in the water absorption of RAP inclusive concrete mixes. Also, it is observed that with the addition of 10% SF along with 40% GGBS the water absorption of RAP inclusive concrete mixes is lesser as compared to other mixes. The reason behind lower water absorption is due to the fact that silica fume absorbs more water and filling of the voids by the additional calcium silicate hydrate (CSH) gel formed on account of reactions between calcium hydroxide (CH) and reactive silica.

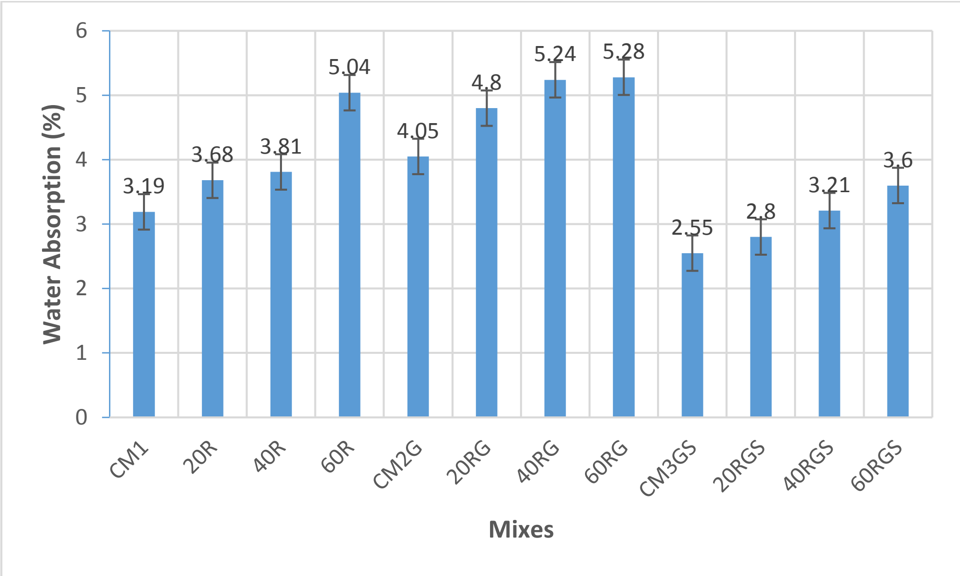

Figure 4.21 Variation of water absorption of different concrete mixes

4.4.3 Abrasion Resistance of Horizontal concrete surface (Slabs)

Since the surface of concrete pavements is in direct touch with moving tyres, there is a considerable risk of wear and tear from skidding, slipping, and rubbing. As a result, producing practical concrete mixes that not only have a good finish but also have sufficient abrasion resistance is a must. This test was conducted as per procedure given in ASTM C779. In this procedure the revolving disk machine operated by sliding and scuffing of the steel disks in conjunction with the abrasive grit. The disks revolve in a circular path on the concrete surface with a revolution speed of 12 rpm collectively however these disks revolve individually at a speed of 280 rpm on their own axis. The abrasive grit consists of 250 µ silicon carbide which is fed into the storage cup placed on the revolving circular plate through a 3 mm orifice. The flow of the abrasive shall be controlled at a rate of 4 to 6 g/min by adjusting the needle located in the orifice. The specimen used for the testing consists of 500 × 500 × 100 mm size. The machine was allowed to rotate on the surface for 5 minutes then the initial reading was taken and then it was allowed to rotate for 60 minutes after which the final readings were taken. The reading was taken by using a micrometre bridge. The average depth of wear of representative surfaces were compared at 5 minutes and 6ominutes respectively and the abrasion resistance was noted. The variation in loss in mass for various mixes is shown in the Fig. 4.22. It was observed that with the increase in the replacement level of NA by abraded RAP aggregates the loss in the mass of concrete mixes also increased. It was found that the increase in the mass loss in case of RAP inclusive mixes was found to be 10.26% for 20R, 38.46% for 40R and 46.15% for 60R as compared to the control mix CM1. However, when 40% GGBS was added to the RAP inclusive PQC it was found that the increment in the loss in mass was less as compared to the normal replacement mixes. It was found that the increase in the percentage level was 29.55% for 20RG, 34.09% for 40RG and 43.18% was 60RG as compared to control mix CM2G. Hence, with the addition of 40% of GGBS the resistance to abrasion got increased. However, it was found that when 10% SF along with 40% GGBS was added to RAP inclusive PQC mixes the percentage loss in mass was least as compared to the previous mixes. The increment in the percentage level was found to be 8.57% for 20RGS, 20% for 40RGS and 37.14% was 60RGS as compared to the control mix CM3GS. Hence, it was found that addition of SF enhanced the durability properties of RAP inclusive PQC mix. Fig. 4.23 shows various samples of concrete mixes tested and Fig. 4.24 shows the machine used to perform the abrasion resistance test.

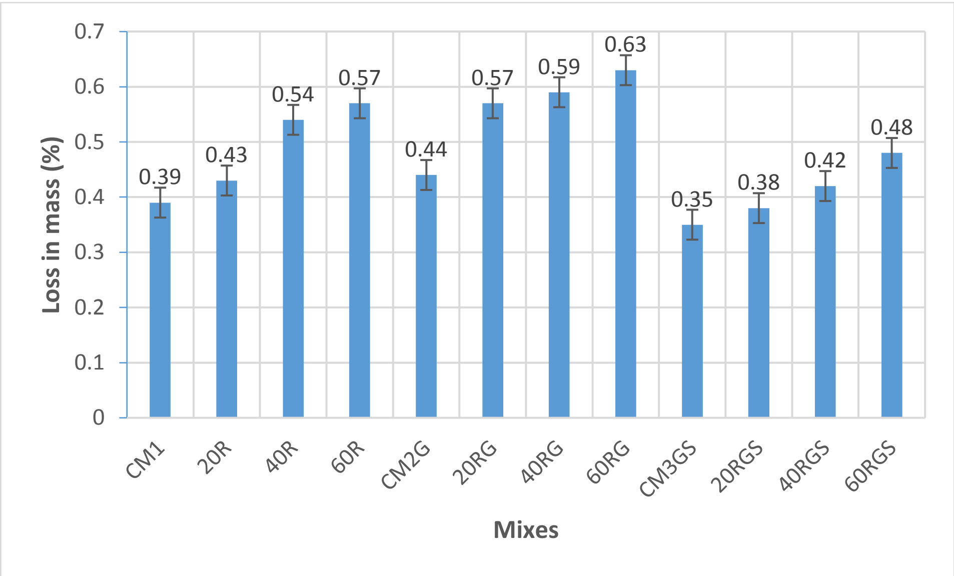

Figure 4.22 Abrasion Resistance for various concrete mixes

Figure 4.23 Samples of various concrete mixes tested for abrasion resistance

Figure 4.24 Revolving Disk abrasion machine

4.4.4 Abrasion Resistance of concrete specimen by sand blasting

Deterioration of concrete can occur due to abrasion by sliding, scraping, percussion or an action of abrasive materials carried by water. Hence, abrasion resistance of concrete specimen by sand blasting is one of the measures of durability property especially for concrete used for the construction of pavement (roads or airfields), bridge decks and footpaths. The above procedure of sand blasting is generally applicable for concrete with a density of 24 to 26 kN/m^3. The concrete specimens were tested as per procedure described in IS 9284 (BIS 1979) after 28 days of moist curing. The test was performed on a surface of 100 mm concrete cube specimen by subjecting it to air-driven silica sand as a result of which there is loss in the mass of the cubes and this loss in mass is taken as the abrasion loss of concrete. The operating pressure of 0.14 N/mm^2 was set up as per IS 9284 and quantity of abrasive sand was taken as 4000 g for each impingement as a result the loss in the mass of specimen was taken as the loss in mass in grams for two separate impressions on the same face of concrete cube under test. The test setup is shown in the Fig 4.25. It shall be noted that the abrasion loss of concrete was taken as the average of the results obtained for 4 surfaces each of 3 cubes as a percentage loss. The results of the loss in mass due to abrasion by sand blasting is shown in Fig 4.27. It was observed that with the increase in the replacement level of RAP aggregated the abrasion loss also increases but the increment was found to be marginal. However, it was observed that all values were below the maximum value for abrasion loss for pneumatic tyred traffic i.e. 0.24 as per IS 9284. This shows that the processing of RAP resulted in enhancement in the durability properties of RAP. This was due to decrease in the binder content of RAP and also removal of agglomerated particles due to abrasion and attrition of RAP aggregates. Compared to the control mix CM1 the increase in the abrasion loss was found to be 6.67% for 20R, 20% for 40R and 33.33% for 60R. However, when 40% GGBS was added to the concrete mix the increase was found to be 5.88% for 20RG, 17.65% for 40RG and 47.06% for 60RG as compared to the control mix CM2G. Also, when 10% SF along with 40% GGBS was added to the RAP inclusive concrete mix it was found that the increase was 13.3% for 20RGS, 20% for 40RGS and 33.33% was 60RGS as compared to the control mx CM3GS. The increment in the abrasion loss when 10% SF along with 40% GGBS was same as that of the normal RAP inclusive PQC mixes. However, when SF was added it was found that the abrasion loss percentage was least as compared to the other mixes. Hence, addition of silica fume along with GGBS resulted in enhancement of durability property of PQC mixes. Fig. 4.26 shows various cube specimens after sand blasting.

Figure 4.25 Sand Blasting test setup

Figure 4.26 Various concrete cubes after sand basting

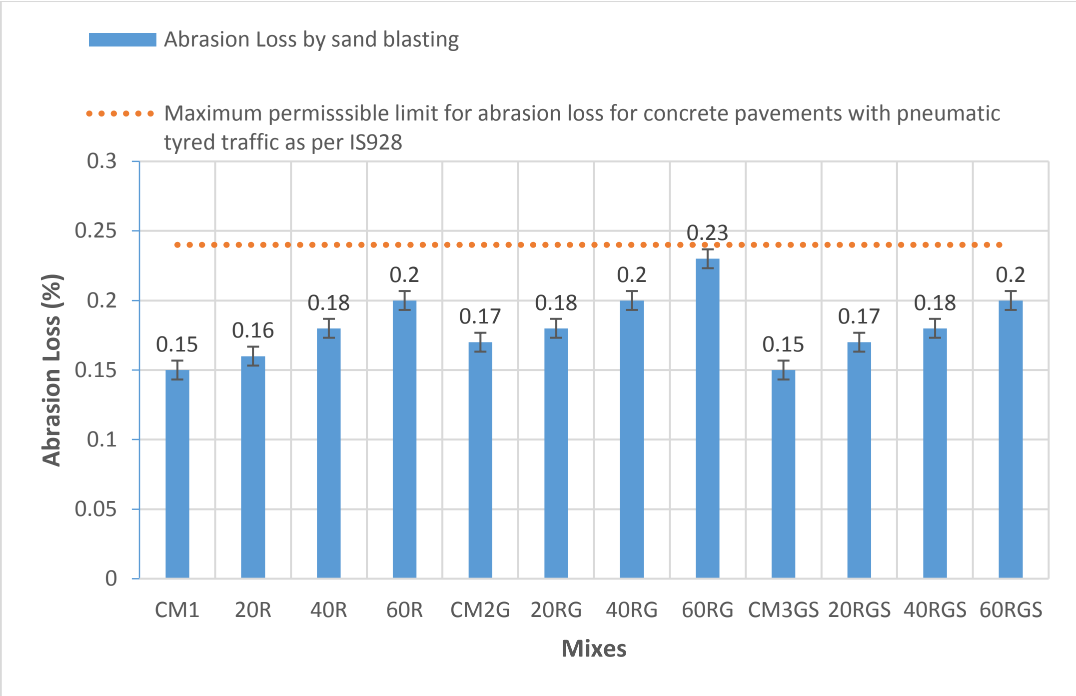

Figure 4.27 Abrasion Resistance of Concrete

4.4.5 Skid Resistance

This test is performed as per the procedure given in IRC SP: 83 (2018) after 28 days of moist curing. The test was performed on a concrete slab surface of 500 × 500 × 100 mm size. The test was performed by using a British pendulum tester as shown in Fig 4.28. The variation of British Pendulum Number (BPT) with various concrete mixes is shown in Fig 4.31. The skid resistance test is a standard approach for measuring the low-speed friction of a road surface material in the lab and in the field. The measured low-speed friction is substantially determined by the road material's surface micro-texture, and that the friction measurements made by the British pendulum test are an indirect form of measurement of the road material's available micro-texture. The British Pendulum Number is used to express the value measured by the tester (BPN). The British Pendulum Tester provides a better skid resistance rating than the dynamic tyre and trailer tests. The BPT number was taken two times first on dry surface and then on wet surface as shown in Fig. 4.29 and Fig. 4.30 respectively and a total of average of 3 readings was taken. It was found that the skid resistance of concrete specimen decreases with the increase in the replacement level of RAP. The skid resistance of dry surface was found to be higher as compared to the wet surface. It was found that CM3GS exhibited highest skid resistance value which indicates excellent skid resistance in all conditions. Almost all the concrete mixes showed acceptable skid resistance values in all conditions. However, some of the mixes like 60RG showed satisfactory surface in only favorable weather and vehicle conditions. It was found that with the addition of 40% GGBS there was a decrease in the skid resistance values as compared to the normal RAP concrete mixes. However, concrete mixes with 40% GGBS and 10% SF showed good to excellent skid resistance in all conditions and the skid resistance of these concrete mixes was higher as compared to the normal RAP concrete mixes.

Figure 4.28 British Pendulum Tester

Figure 4.29 BPT on Dry surface of slab specimen

Figure 4.30 BPT on wet surface of slab specimen

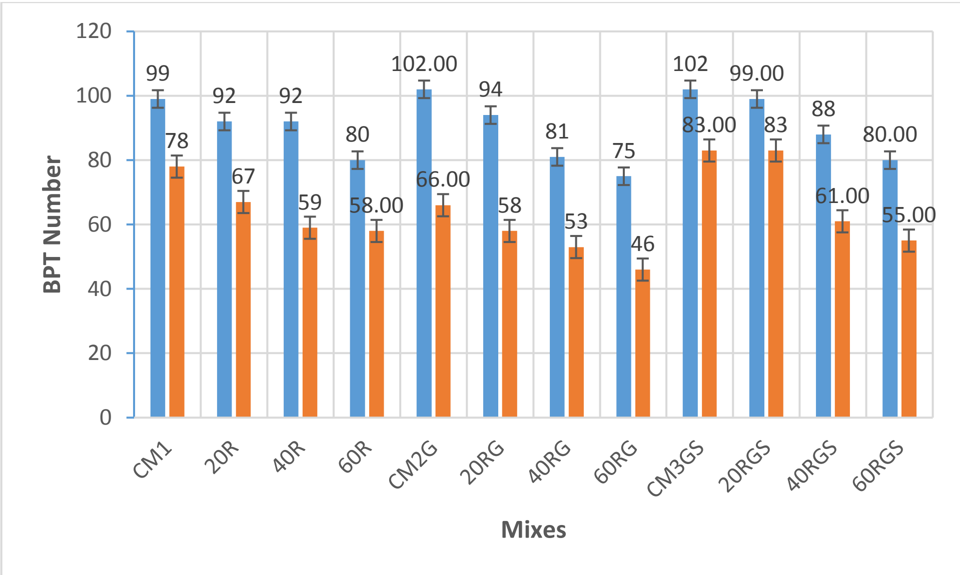

Figure 4.31 Skid Resistance Number of various concrete mixes

4.5 Time Dependent Concrete Properties

4.5.1 Drying Shrinkage

This method is used to determine the change in length of concrete specimens due to change in moisture content. This test is performed on specimens casted in laboratory or specimens cut from the structure or units when the maximum nominal size of aggregate is below 38 mm. the test was performed on the sample of size 300×75×75 mm after 28 days of water curing. The specimen were kept vertically at a room temperature of 23 ± 4˚C during the testing. The test was carried out according to the procedure followed in ASTM C490 and the plot of the change in length for various mixes is shown in Fig 4.34. It was found that with the increase in the replacement of NA by abraded RAP there was a decrease in the values of drying shrinkage similar to the trend observed in the mechanical properties of the concrete mix. The decrease was found to be 14.83% for 20R, 17.03% for 40R and 19.78% for 60R as compared to the control mix CM1. However, when 40% GGBS was added the decrease was found to be 13.33% was 20RG, 24.84% for 40RG and 35.15% for 60RG as compared to the control mix CM2G. Also, when 10%SF along with 40% GGBS was added the decrease was found to be 6.37% for 20RGS, 15.93% for 40RGS and 24.32% for 60RGS when compared to the control mix CM3GS. However, for the concrete mix with 10% SF along with 40% GGBS the drying shrinkage values were found to be higher as compared to the other mixes. It was found that the increase in the values of drying shrinkage was 37.91% for CM3GS as compared to CM1, 51.61% for 20RGS as compared to 20R, 39.73% for 40RGS as compared to 40R and 30.13% for 60RGS as compared to 60R. The increase in the values of drying shrinkage may be due to the high-water absorption due to presence of silica fume. Furthermore, SF particles with a large specific surface area filled the pores of ITZ and reinforced it, making it more dense and compacted.

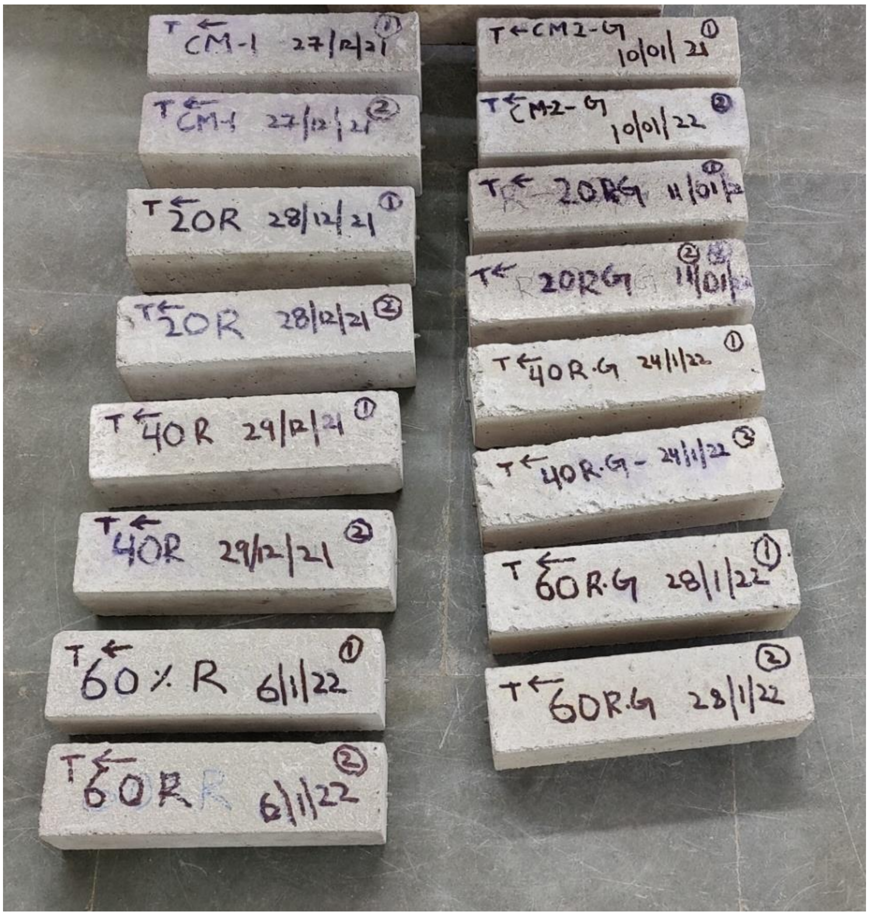

Figure 4.32 Casted specimens for drying shrinkage test

Figure 4.33 Apparatus for drying shrinkage containing sample for testing

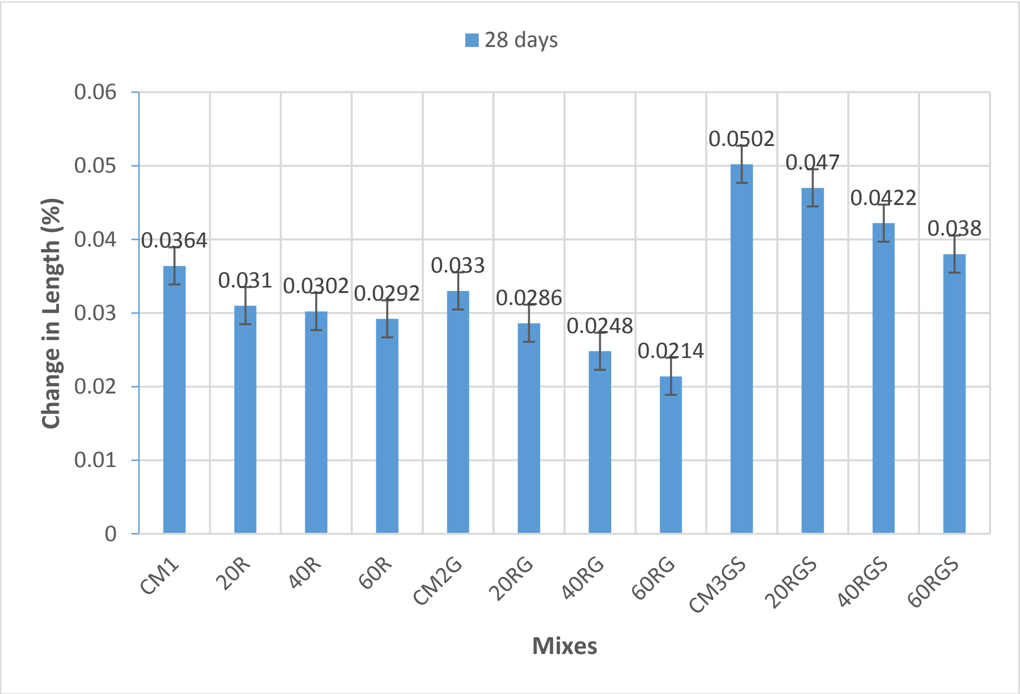

Figure 4.34 Variation of change in length with different percentages of concrete mix

4.6 Non-Destructive Testing

4.6.1 Rebound Hammer

This test is used in order to assess the compressive strength and uniformity of concrete. This method can be used with higher confidence to distinguish between problematic and acceptable parts of a structure, or to compare two structures relative to one another. The rebound hammer test is conducted as per the procedure given in IS 1311 Part II (BIS 1992). This test was performed on slabs of size 500×500×100 mm by giving vertical impacts on the horizontal surface of the concrete slabs as shown in Fig 4.35. Since the size of the sample is greater than 300 mm rebound hammer of high impact energy of 30 Nm is used for testing mass concrete including roads, airfield pavements and hydraulic structures. An N–type analogue Schmidt (Proceq make) rebound hammer was used and the rebound hammer test was carried on the slabs after 28 days of moist curing and the plot for the same is presented in Fig 4.36. It was observed that almost all the mixes were above the design target value except for 60RG whose rebound number was just below the target value. The reason behind the higher rebound number was due to the processing technique that was used in order to remove the asphalt film and other dust layers present in the RAP aggregates. It was observed that with the increment in the percentage of RAP aggregates there was a decrease in the rebound number similar to the trend observed in compressive strength. The decrease in the rebound number was found to be 3.13% for 20R, 6.25% for 40R and 10.42% for 60R as compared to the control mix CM1. Similarly, when 40% GGBS was added to the RAP inclusive PQC mix it was found that the decrease in the rebound number was 5.48% for 20RG, 10.87% for 40RG and 15.22% for 60RG as compared to the control mix CM2G. Also, when 10% SF along with 40% GGBS was added to the RAP inclusive PQC mix the decrease was found to be 4.82% for 20RGS, 9.62% for 40RGS and 12.5% for 60RG. Hence, it was found that the highest rebound number value was for the mix containing 10% SF along with 40% GGBS and therefore it was concluded that silica fume enhanced the non-destructive property of the concrete mix.

Figure 4.35 Vertical impact on the horizontal surface of the concrete slabs

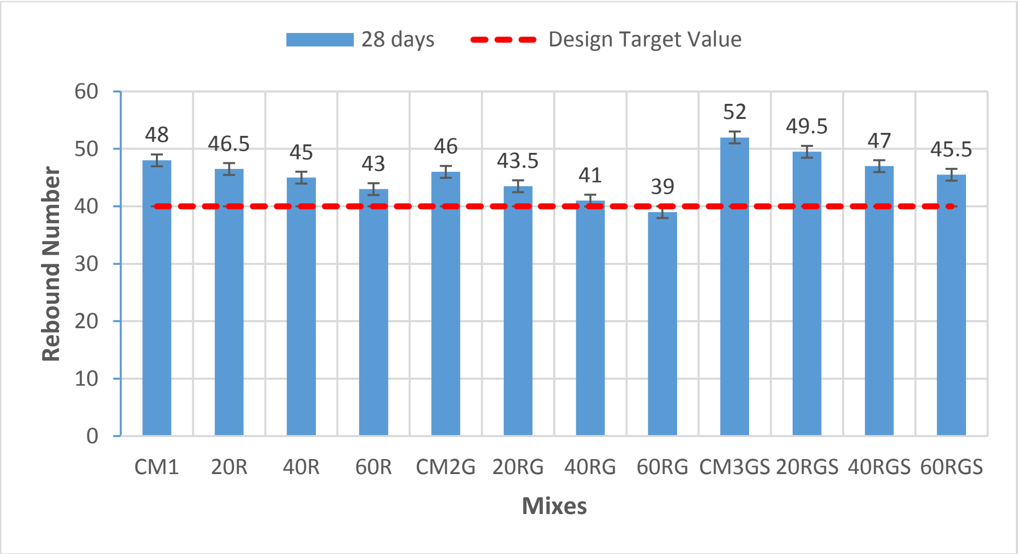

Figure 4.36 Variation of Rebound number of concrete mixes

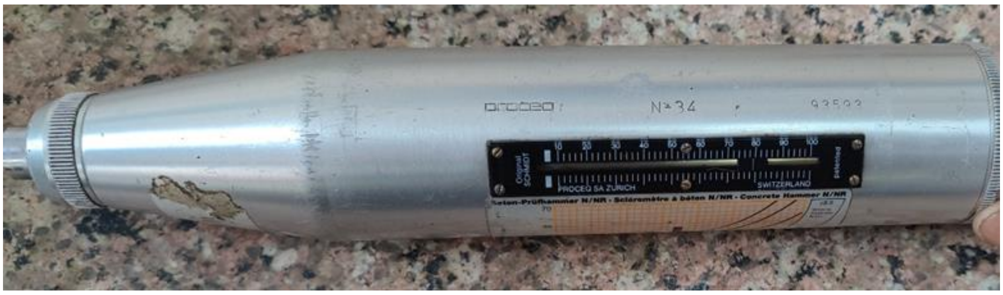

Figure 4.37 Rebound Hammer apparatus

4.6.2 Ultrasonic pulse velocity (UPV)

This test is used to determine the homogeneity in concrete and also if any cracks, voids and other imperfections are present in concrete. This test is also used to determine the dynamic elastic modulus of concrete. The test was performed as per procedure given in IS 13311 Part I (BIS 1992). The UPV apparatus used is a PUNDIT UPV tester (Proceq make) with transducers of operating frequency of 54 kHZ was used for non-destructive testing since the path length of the specimens ranges between 700-1500 mm. The test was carried out on concrete specimen in accordance with IS 516 Part V (BIS 2018). In this test direct transmission method was used in which transducers were placed parallel to each other on opposite surfaces of the specimen as shown in Fig. 4.39. The velocity of the pulse was calculated as per Eq. 4.5.

$$V = \frac{L'}{T} \qquad 4.5$$

Where,

V = pulse velocity (km/s)

L' = path length (km)

T = Transmission time (s)

The UPV test of the concrete specimen was taken after 28 days of moist curing and it present in the Table 4.4 given below and the plot for the same is presented in the Fig 4.40. The readings were take three times on each specimen and average of 3 specimens was taken for each mix.

Table 4.4 Ultrasonic Pulse Velocity of concrete specimens at different ages

Mix Designation	Average Pulse Velocity (km/s)	Concrete Quality Grading
CM1	4.61	Excellent
20R	4.62	Excellent
40R	4.59	Excellent
60R	4.54	Excellent
CM2G	4.76	Excellent
20RG	4.67	Excellent
40RG	4.66	Excellent

60RG	4.52	Excellent
CM3GS	4.79	Excellent
20RGS	4.75	Excellent
40RGS	4.71	Excellent
60RGS	4.68	Excellent

Figure 4.38 Ultra sonic pulse velocity apparatus

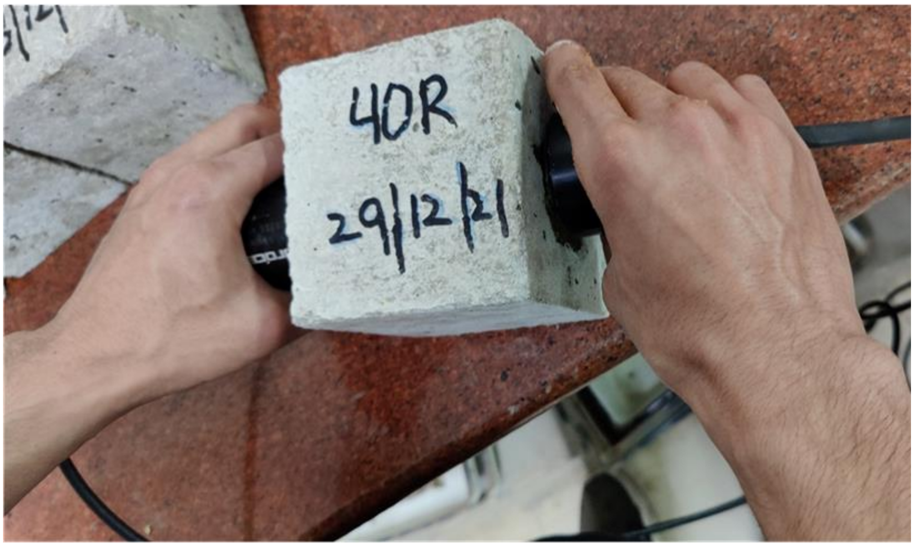

Figure 4.39 Testing of concrete specimen using UPV

It was observed that all the mixes had pulse velocity greater than 4.5 km/s which determines that the quality of the concrete mixes is excellent as per criteria given in IS 13311 Part I (BIS 1992). It was found that there was a similar trend observed in UPV as was seen in rebound hammer as well the compressive strength of the specimen. However, the concrete mixes with 10% SF along with 40% GGBS exhibited higher pulse velocity as compared to the rest of mixes. This shows that the PQC mixes with 10% SF and 40% GGBS have better quality of concrete as compared to the other mixes. The substantial amount of activated silica that interacted with portlandite and created more CSH gel was responsible for the increase in compressive strength when 10% SF was used as a replacement of cement along with 40% GGBS. It can be seen as the replacement level of RAP increases there is a slight decrease in the pulse velocity. However, this decrease is marginal.as compared to the decrease observed in other properties.

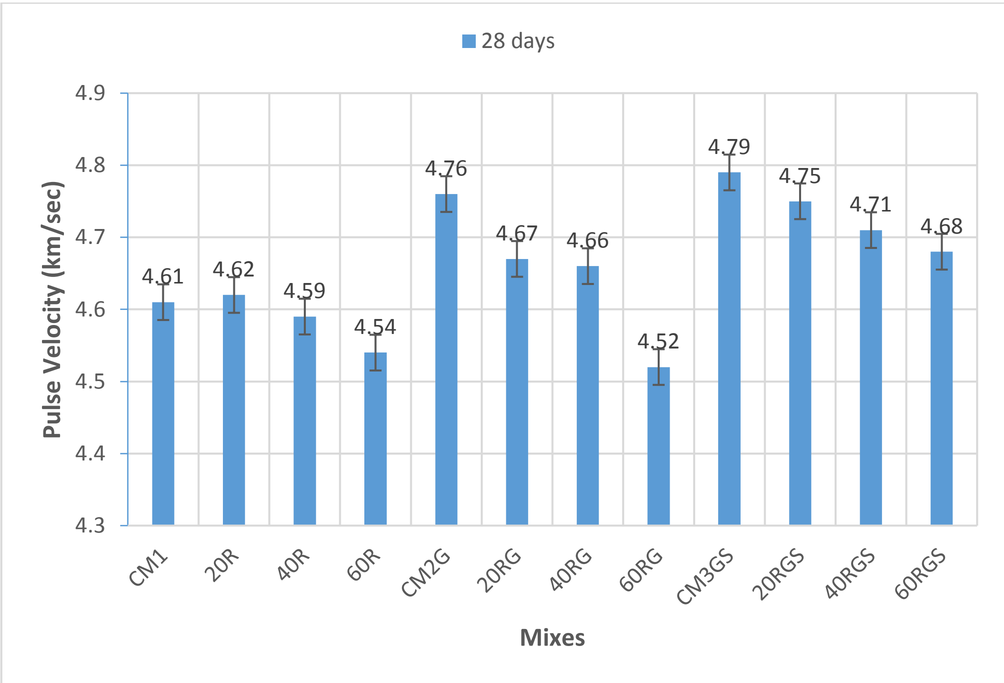

Figure 4.40 Variation of Ultrasonic Pulse Velocity of concrete at 28 days

4.7 SUMMARY

In this chapter various mechanical, durability and time dependent properties such as shrinkage were carried out and presented. The summary of the work is presented in this chapter as follows:

1. The fresh properties of the PQC mixes were also studied and their effect with SF and GGBS was presented along with the replacement levels of RAP.

2. The mechanical properties such as compressive strength, flexure strength and split tensile strength were tested and studied. It was found that with the increase in the replacement level of NA with abraded RAP aggregates there was a decreasing trend that was observed in all the mechanical properties. However, it was found that all the PQC mixes had strength above the design target value. This proved that the processing technique used for RAP improved the mechanical properties along with presence of silica fume and GGBS which played an important role in enhancement of the properties. Also, the PQC mixes with 10% SF along with 40% GGBS were found to exhibit highest improvement in the properties.
3. The durability properties such as water absorption. Abrasion resistance, Skid resistance and resistance against elevated temperature were also found to be improved with the processing technique used for RAP and with addition of 10% SF and 40% GGBS.
4. The time dependent properties such as drying shrinkage of various PQC mixes were found to be decreasing, which can be attributed to its improvement in mechanical properties.

CHAPTER 5

5 CONCLUSION

5.1 OVERVIEW

The present study includes a comprehensive experimental investigation in order to understand various mechanical, durability and time dependent properties of RAP inclusive PQC mix in comparison to the Natural aggregate mix i.e. control mix so as to make it suitable for the use in concrete pavements. Therefore, it comprised of the characterization of properties of various materials of concrete like Natural aggregate, abraded RAP as well as silica fume and GGBS. In this study coarse RAP was used after processing. The processing includes abrasion and attrition of RAP aggregates in order to remove the contaminant layers such as asphalt film, loose dust film and stiff dust film and to reduce the binder content. This processed RAP proved to be beneficial in enhancing all the properties of RAP inclusive PQC including mechanical, durability and time dependent properties. The mix design of natural aggregate PQC mix and RAP inclusive PQC mix was prepared as per IRC: 44-2017. Considering the above mix design 12 mixes were prepared while varying the replacement level of RAP (20, 40 and 60%) along with the replacement of OPC with 40% GGBS and 10% SF. The mixes were designated as 0% RAP(CM1), 20% RAP (20R), 40% RAP (40R), 60%RAP (60R), 0% RAP with 40% GGBS (CM2G), 20% RAP with 40% GGBS (20RG), 40% RAP with 40% GGBS (40RG), 60% RAP with 40% GGBS (60RG), 0% RAP with 40% GGBS and 10% SF(CM3GS), 20% RAP with 40% GGBS and 10% SF (20RGS), 40% RAP with 40% GGBS and 10% SF (40RGS) and 60% RAP with 40% GGBS and 10% SF (60RGS). All the mixes were designed as per the mix design in IRC: 44 – 2017 for a minimum characteristic compressive strength of 40 N/mm^2 at 28 days with a w/c_m of 0.39. Preliminary investigations were carried out on cement like initial setting time and final setting time along with specific gravity and consistency of cement. The concrete specimens were casted and a detailed experimental study for done. The workability of concrete mix was tested by performing slump and compaction factor test and the effect of SF and GGBS on the workability was studied. Also, concrete specimens were tested in order to obtain the hardened density, mechanical, durability and time dependent properties such as drying shrinkage and the effect of SF and GGBS on the various properties of PQC mix were studied. Based on these studies conclusions are drawn and suggestions for future work has been proposed.

5.2 CONCLUSION AND REMARKS

Based on the detailed study present in this dissertation, conclusions were drawn on the:

i. Properties of RAP.
ii. Mechanical, durability and Time dependent properties of PQC mixes.

These are presented in the following sections.

5.2.1 Properties of RAP

1. A processing technique was used in order to remove the contaminant layers such as asphalt film, loose dust film and stiff dust film present on the coarse RAP. Also, to reduce the binder content present in RAP the processing technique was used. This technique enhanced the properties of RAP as compared to the normal RAP.
2. The particle size distribution of RAP as well as NA was within the limits as per IRC: 44 – 2017.
3. The specific gravity of RAP was found to be lower than the NA and water absorption of RAP was found to be higher as compared to the NA.
4. The bulk density of abraded RAP was found to be higher as compared to the NA whole as the percentage voids were found to be lower as compared to NA.
5. The crushing value of the abraded RAP was found to be lower than NA and similar results were obtained for the impact value also. However, the values were well within the limits specified as per IS 2386 (Part 4).
6. The asphalt content of the abraded RAP was found to be lower as compared to the normal RAP and the reduction in asphalt content was found to be more than 60%.

5.2.2 Properties of RAP inclusive PQC mix

1. It was observed that the replacement of natural aggregates by abraded coarse RAP resulted in the increase in the slump value. Also, replacement of cement by 40% GGBS was found to increase the slump value of PQC and it was found that increase in the slump value of RAP inclusive PQC along with 40% GGBS was 33.33% for 20RG, 60% for 40RG and 83.33% for 60RG as compared to the control mix CM2G. However, when 10% silica fume was added along with 40% of GGBS it was found that the slump of the control mix CM3GS was less as compared to the slump of control mixes CM1 and CM2G. The increase in the slump value of RAP inclusive PQC along with 40% GGBS

and 10% SF was 13.33% for 20RGS, 46.66% for 40RGS and 93.33% for 60RGS as compared to the control mix CM3GS.

2. The compaction factor showed an increasing trend in all the PQC concrete mixes. The increase in the compaction factor results in the increased workability of concrete. However most of the mixes depicted medium to low workability. It was found that the concrete mixes with 40% GGBS and 10% SF exhibited least workability as compared to the other mixes.
3. The compressive strength, flexural strength and split tensile strength were found to show similar decreasing tend with the increase in the replacement level of NA by processed RAP. However, it was found that the decrease in the percentage level of strength is very less as compared to other research. It was observed that all the mixes crossed the design target strength of 40 MPa for compressive and 4.5 MPa for flexure. The reason behind improved strength was due better bonding between the abraded Coarse RAP aggregates that are free from dust layers as a result of processing technique and hydrated cement paste matrix. It was also seen that RAP inclusive PQC mixes exhibited failure pattern similar to NAC mixes under compression. However, when subjected to flexure it was observed that the failure occurred between hydrated cement paste and aggregates present in the concrete mix. However, in some cases only aggregate failure was observed.
4. It was observed that with the increase in the percentage of coarse RAP the hardened density of the PQC mixes showed a decreasing trend as compared to the control mix. The reason behind decrease in the density of concrete mixes was due to lower specific gravity of coarse RAP as compared to NA. The decrease in the density of RAP inclusive PQC mix was found to be 3.2% to 4.3% respectively as compared to control mix CM1. Also, when 40% GGBS along with 10% SF was added to the RAP inclusive PQC mixes it was found that the decrease in the density of concrete mixes was in the range of 2.88% to 3.94% as compared to the control mix CM3GS.
5. It was observed that the water absorption of PQC mixes increases with the increase in the proportion of abraded RAP aggregates. The increase in the water absorption of RAP inclusive concrete mixes was found to be 15.36% for 20R, 19.44% for 40R and 57.99% for 60R as compared to the control mix CM1. Also, when 10% SF is added along with 40% GGBS the water absorption was found to be lesser as compared to other mixes however, it showed the same increasing trend.

6. In case of abrasion resistance of horizontal concrete surfaces it was observed that with the increase in the replacement level of NA by abraded RAP aggregates the loss in the mass of concrete mixes also increased. It was found that the increase in the mass loss in case of RAP inclusive mixes was found to be 10.26% for 20R, 38.46% for 40R and 46.15% for 60R as compared to the control mix CM1. Also, with the addition of 40% of GGBS the resistance to abrasion got increased. However, it was found that when 10% SF along with 40% GGBS was added to RAP inclusive PQC mixes the percentage loss in mass was least as compared to the previous mixes.
7. In case of abrasion resistance of concrete specimen by sand blasting it was observed that with the increase in the replacement level of RAP aggregated the abrasion loss also increases but the increment was found to be marginal. However, it was observed that all values were below the maximum value for abrasion loss for pneumatic tyred traffic i.e. 0.24 as per IS 9284. This shows that the processing of RAP resulted in enhancement in the durability properties of RAP.
8. In case of drying shrinkage it was found that with the increase in the replacement of NA by abraded RAP there was a decrease in the values of drying shrinkage similar to the trend observed in the mechanical properties of the concrete mix. The decrease was found to be 14.83% for 20R, 17.03% for 40R and 19.78% for 60R as compared to the control mix CM1. However, for the concrete mix with 10% SF along with 40% GGBS the drying shrinkage values were found to be higher as compared to the other mixes.
9. In case of rebound hammer test it was observed that almost all the mixes were above the design target value except for 60RG whose rebound number was just below the target value. The reason behind the higher rebound number was due to the processing technique that was used in order to remove the asphalt film and other dust layers present in the RAP aggregates. It was observed that with the increment in the percentage of RAP aggregates there was a decrease in the rebound number similar to the trend observed in compressive strength.
10. In case of ultrasonic pulse velocity test it was found that all the mixes had pulse velocity greater than 4.5 km/s which determines that the quality of the concrete mixes is excellent as per criteria given in IS 13311 Part I (BIS 1992). It was found that there was a similar trend observed in UPV as was seen in rebound hammer as well the compressive strength of the specimen. However, the concrete mixes with 10% SF along with 40% GGBS exhibited higher pulse velocity as compared to the rest of mixes. This

shows that the PQC mixes with 10% SF and 40% GGBS have better quality of concrete as compared to the other mixes.

5.3 SUGGESTIONS FOR FUTURE WORK

From the present study it can be concluded that the mechanical properties such as compressive strength, flexure strength and split tensile strength and durability properties of RAP inclusive PQC mix can be improved by addition of GGBS and Silica Fume and by the processing technique used in order to reduce the binder content in RAP. The suggestions for future research are proposed and given below:

- Higher replacement level of RAP with NA and their effect with GGBS and SF needs to be studied.
- Study on the behaviour of RAP concrete specimens under modulus of elasticity, freeze and thaw and acid attack as well as study on time dependent properties for longer duration than that investigated in the present study.
- Study of various properties like Freeze and Thaw, Acid attack and carbonation of RAP inclusive concrete mixes containing GGBS needs to be done.
- Study of micro structural analysis such as SEM and XRD of the concrete mixes needs to be done.

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