EFFECT OF FLY ASH AND RICE HUSK ASH ON STRENGTH CHARACTERISITCS OF PAVEMENT QUALITY CONCRETE

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the award of the degree of

MASTERS OF ENGINEERING
IN
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DECLARATION

The author hereby declares that this thesis entitled **"Effect of Fly Ash and Rice Husk Ash on Strength Characteristics of Pavement Quality Concrete"**, in whole or part, has not been used to obtain any degree in this, or any other, institute. Except where references have been given in text, it is entirely the authors own work.

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CERTIFICATE

This is to certify that the thesis entitled **"Effect of Fly Ash and Rice Husk Ash on Strength Characteristics of Pavement Quality Concrete"** being submitted by **Mr. Amrinder Pal Singh, Roll No 801122002** in partial fulfillment for the award of degree of **Masters of Engineering in Structural Engineering** at **Thapar University, Patiala** is a bonafide work carried out by him under our guidance and supervision and that no part of this thesis has been submitted for the award of any other degree

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ABSTRACT

There is growing interest in the construction of concrete pavements, due to its high strength, durability, better serviceability and overall economy in the long run.

The thrust nowadays is to produce thinner and green pavement sections of better quality, which can carry the heavy loads. The high strength is a concrete having compressive strength greater than 40MPa, made of hydraulic cements and containing fine and coarse aggregates; The present study aims at, developing pavement quality concrete mixtures incorporating fly ash and rice husk ash partial replacement of cement. The aim is to the design of slab thickness of PQC pavement using the achieved flexural strength of the concrete mixtures. In this study, compressive and flexural strength for pavement quality concrete mixtures for different percentage replacement of cement are reported. It is found that it is possible to achieve savings in cement by replacing it with fly ash.

This study also shows that in view of the high flexural strength, high values of compressive strength the 20% replacement of cement with flyash is ideal for design of Pavement Quality Concrete (PQC).

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CHAPTER-1

INTRODUCTION

1.1 GENERAL

Concrete is basically a mixture of two components: Aggregates and Paste (or binder). The paste comprises cement, supplementary cementing or supplementary cementitious materials and water. It binds the aggregates (sand and gravel or crushed stone) into a rock-like mass. The purpose is to fill up the voids and come with a dense and strong materials. The fine aggregates fill up the voids formed by the coarse aggregates; and cement fills up the voids of the fine aggregates. Lesser the voids more would be the strength of concrete. The chemical reaction of the cementitious materials and water, is called hydration. It is the process by which paste hardens and binds the aggregates.

The high modulus of elasticity and rigidity of concrete compared to other road making materials provides a concrete pavement with a reasonable degree of flexural or beam strength. This property leads to a wider distribution of externally applied wheel loads. This in turn limits the pressures applied to the sub-grade. The major portion of the load carrying capacity of a concrete pavement is therefore provided by the concrete layer alone. Its thickness is primarily determined by the *flexural strength* of the concrete and by the magnitude of the wheel or axle loads. Sub-bases do not make a significant structural contribution to concrete pavements.

By contrast, a flexible pavement is a structure comprising a number of layers of bound or unbound materials which can have a variety of surface treatments and in which the intensity of stresses from traffic loads requires a lot more depth to diminish. Both the base and sub-base layers in flexible pavements contribute significantly to the structural properties of the pavement. Concrete acts more like a bridge over the sub-grade. Much less pressure is placed on the material below the concrete, than bituminous pavements. Since the first strip of concrete pavement was completed in 1893, concrete has been now extensively used for paving the highways and airports as well as business and residential streets.

1.2 CONCRETE PAVEMENTS

A concrete pavement is a structure comprising of a layer of Ordinary Portland Cement Concrete which is usually supported by a sub-base layer on the sub-grade. Concrete pavements may be either unreinforced (plain) or reinforced depending on how the designer prefers to control shrinkage cracking, which will occur in the pavements..

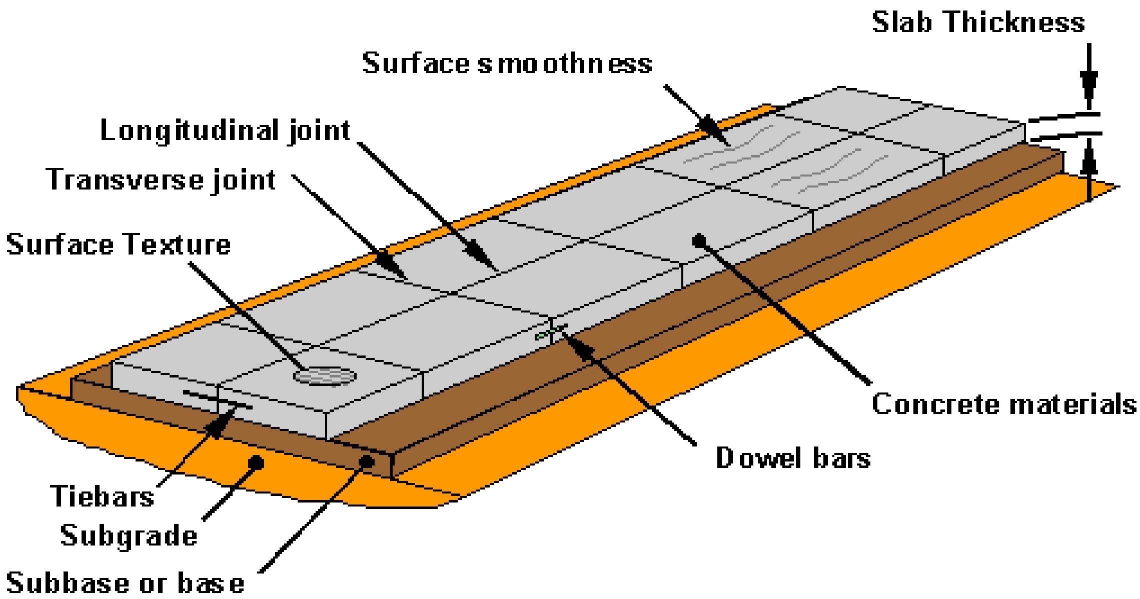

Fig1.1 Concrete Road Pavement Structure

1.2.1 Types of Concrete Pavements

1) Jointed Plain (unreinforced) Concrete Pavement – JPCP
2) Jointed Reinforced Concrete Pavement – JRCP
3) Continuously Reinforced Concrete Pavement – CRCP
4) Steel Fiber Reinforced Concrete Pavement – SFCP

a) ***Jointed Plain (unreinforced) Concrete Pavement – JPCP***

Jointed Plain Concrete Pavement (JPCP) uses contraction joints to control cracking and does not use any reinforcing steel. Transverse joint spacing is selected such that temperature and moisture stresses do not produce intermediate cracking between joints. This typically results in a spacing no longer than about 6.1 m (20 ft.). Dowel bars are typically used at transverse joints to assist in load transfer. Tie bars are typically used at longitudinal joints.

Crack control	-	Contraction joints, both transverse and longitudinal
Joint spacing	-	Typically between 3.7 m (12 ft.) and 6.1 m (20 ft.). Due to the nature of concrete, slabs longer than about 6.1 m (20 ft.) will usually crack in the middle. Depending upon environment and materials slabs shorter than this may also crack in the middle.

Reinforcement	-	None
Load transfer	-	Aggregate interlock and dowel bars. For low-volume roads aggregate interlock is often adequate. However, high-volume roads generally require dowel bars in each transverse joint to prevent excessive faulting.

b) ***Jointed Reinforced Concrete Pavement – JRCP***

JRCP uses contraction joints and reinforcing steel to control cracking. Transverse joint spacing is longer than that for concrete pavement contraction design (CPCD), it typically ranges from 30 ft. to 60 ft. This rigid pavement design option is no longer sanctioned by the department because of past difficulties in selecting effective rehabilitation strategies. However, there are several remaining sections in service.

Crack control	-	Contraction joints, Dowel bars as well as reinforcing steel.
Joint spacing	-	Longer than JPCP and up to a maximum of about 15 m (50 ft.). Due to the nature of concrete, the longer slabs associated with JRCP will crack
Reinforcement Steel	-	A minimal amount is included mid-slab to hold cracks tightly together. This can be in the form of deformed reinforcing bars or a thick wire mesh.
Load transfer	-	Dowel bars assist in load transfer across transverse joints while reinforcing steel assists in load transfer across mid-panel cracks

c) ***Continuously Reinforced Concrete Pavement - CRCP***

Continuously Reinforced Concrete Pavement (CRCP) provides joint-free design i.e. it does not require any construction joint. The formation of transverse cracks at relatively close intervals is a distinctive characteristic of CRCP. These cracks are held tightly by the reinforcement and should be of no concern as long as the cracks are uniformly spaced, do not spall excessively, and a uniform non-erosive base is provided. Research has shown that the maximum allowable design crack width is about 0.5 mm (0.02 inches) to protect against spalling and water penetration.

Crack control	-	Reinforcing Steel
Joint spacing	-	Not applicable. No transverse contraction joints are used.
Reinforcement Steel	-	about 0.6 - 0.7 percent of the cross-sectional pavement area and is located near mid-depth in the slab
Load transfer	-	Reinforcing steel

d) ***Steel Fiber Reinforced Concrete Pavement – SFCP***

Steel fibers are used in the pavement quality concrete 0.5% to 1% weight of the cement. With the inclusion of steel fibers the tensile and flexural strength of concrete increases considerably. It is determined that there is considerable increase in flexural strength with the use of short, small diameter of the steel fibers in the first crack ultimate flexural strength and flexural strength of plain concrete. With the use of steel fibers, the ultimate strength can be increased up to 3 times the strength of plain concrete.

It is determined that in the normal third-point bending test, the flexural strength of SFRC is about 50 to 70 percent more than that of the plain concrete mix. So with the inclusion of steel fibers the slab thick gets reduced and hence is very economical.

The most significant influence of the incorporation of steel fibers in concrete is to delay and control the tensile cracking of the composite material. This positively influences mechanical properties of concrete. These improved properties resulting SFRC being a feasible material for concrete road pavements.

1.2.2 Benefits of Concrete Pavements

The beneficial attributes of concrete pavements can be summarized as below:

1) Long lasting – 40 year Design Life
2) Heavy duty Pavements have generally the lowest cost.
3) Pavement maintenance costs are up to 10 times cheaper than the same for flexible pavements.
4) Minimum maintenance requirements result in less traffic disruption, minimum congestion time and results Work Zone safety.
5) Lowest Life Cycle Cost of all Heavy Duty pavements and highest salvage value.
6) Can be constructed over poor sub-grades.
7) Thinner overall pavement thickness = lower consumption of raw materials.
8) Resistant to abrasion from turning actions.
9) Not susceptible to high or low temperatures.
10) Not affected by weather, inert to spills and fire.
11) Completely recyclable.
12) High abrasion durability.
13) Profile durability.

14) Safer because it maintains its shape, no deformation, resistance to rutting and potholes and excellent skid resistance.
15) High sustainability rating through use of local materials.
16) Use of waste products like fly-ash and slag.
17) Riding quality does not deteriorate.
18) Can be slip formed up to 13 m.
19) Saving of fuel costs of at least 1.1% over asphalt (VTI Sweden – 1.1% 2008, NRC Canada – 0.8 to 6.9%).
20) Light colour enhances night visibility.
21) Less energy for street lighting (up to 30%).
22) Less heat-sink effect (avg. 8°C lower than asphalt = less air conditioning energy in urban areas).
23) Longitudinal diamond grinding, called Next Generation Concrete Surfacing (NGCS) now provides quieter surface than for example Open Graded Asphalt overlay.

1.2.3 Demerits of Concrete Pavements

1) They involve heavy initial investment.
2) Lots of joints are need to provide which prove additional places of weakness.
3) 28 days curing is required after completion before they can be opened to traffic.
4) It is not possible to adopt stage construction programmed in these roads.
5) Cement concrete road surface after some time of use becomes very smooth and slippery.
6) It is a noisy road, as bullock carts or steel tyred vehicles cause lot of noise while moving on them.

1.3 PAVEMENT QUALITY CONCRETE (PQC)

BRIEF AND DETAILED SPECIFICATIONS OF PAVEMENT QUALITY CONCRETE (PQC) AS PER MINISTRY OF ROAD TRANSPORT AND HIGHWAYS [MORT&H]

The purpose of this Quality Plan for Construction of Pavement Quality Concrete (PQC) is to provide detailed Construction Methodology, Materials used, Resources deployed.

Cement Concrete Pavement

The work shall consist of construction of un-reinforced, dowel jointed, plain cement concrete pavement in accordance with the requirements of these Specifications and in conformity with the lines, grades and cross sections shown on the drawings. The work shall include furnishing of all plant and equipment, materials and labour and performing all operations in connection with the work, as approved by the Engineer.

1.3.1 **Cement content** :

When Ordinary Portland Cement (OPC) is used the quantity of cement shall not be less than 360 kg/cum. In case fly ash grade-I (as per IS:3812) is blended at site as partial replacement of cement, the quantity of fly ash shall be up to 20 percent by weight of cement and the quantity of OPC in such a blend shall not be less than 310 kg/cum. The minimum of OPC content in case ground granulated Portland blast furnace is used, shall also not be less than 310 kg/m^3.

1.3.2 **Concrete strength:**

The characteristic flexural strength of concrete shall not be less than 4.5 MPa (M 40 Grade). Target mean flexural strength for mix design shall be more than 4.5 MPa + 1.65*s, where s is standard deviation of flexural strength derived by conducting test on minimum 30 beams. While designing the mix in the laboratory, correlation between flexural and compressive strengths of concrete shall be established on the basis of at least thirty tests on samples. However, quality control in the field shall be exercised on the basis of flexural strength. The water content shall be the minimum required to provide the agreed workability for full compaction of the concrete to the required density and the maximum free water cement ratio shall be 0.45 when only OPC is used and 0.50 when blended cement (Portland Pozzolana Cement or Portland Slag Cement or OPC blended with fly ash or Ground Granulated Blast Furnace Slag at site) is used.

1.3.3 **Transverse joints:**

Transverse joints shall be contraction and expansion joints constructed at the spacing described in the drawings. Transverse joints shall be straight within the following tolerances along the intended line of joints which is the straight line transverse to the longitudinal axis of the carriageway at the position proposed by the Contractor and agreed to by the Engineer, except at road junctions or roundabouts where the position shall be as described in the drawings:

i) Deviations of the filler board in the case of expansion joints from the intended line of the joint shall not be greater than ± 10 mm.

ii) The best fit straight line through the joint grooves as constructed shall be not more than 25 mm from the intended line of the joint.

iii) Deviations of the joint groove from the best fit straight line of the joint shall not be greater than 10 mm.

1.3.4 Contraction joints:

The contraction joints shall be placed transversely at pre-specified locations as per drawings/design using dowel bars. These joints shall be cut as soon as the concrete has undergone initial hardening and is hard enough to take the load of joint sawing machine without causing damage to the slab Contraction joints shall consist of a mechanical sawn joint groove, 3 to 5 mm wide and one-fourth to one-third depth of the slab ± 5 mm or as stipulated in the drawings and dowel bars complying with Clause 602.6.5 of MoRT&H Specifications. Contraction joint shall be widened subsequently accommodate the sealant as per Clause 602.11 of MoRT&H Specifications, to dimensions shown on drawings or as per IRC:57

1.3.5 Expansion joints:

The expansion joints shall consist of a joint filler board complying with Clause 602.2.7 of MoRT&H Specifications and dowel bars complying with Clause 602.6.5 of MoRT&H Specifications and as detailed in the drawings. The filler board shall be positioned vertically with the prefabricated joint assemblies along the line of the joint within the tolerances given in Clause 602.6.2.1 of MoRT&H Specifications and at such depth below the surface as will not impede the passage of the finishing straight edges or oscillating beams of the paving machines. The adjacent slabs shall be completely separated from each other by providing joint filler board. Space around the dowel bars, between the sub-base and the filler board shall be packed with a suitable compressible material to block the flow of cement slurry.

1.3.6 Transverse construction joint:

Transverse construction joint shall be placed whenever concreting is completed after a day's work or is suspended for more than 30 minutes. These joints shall be provided at location of constructing joints using dowel bars. The construction joints may preferably coincide with the pre-specified location of construction joints by properly planning the day to day concreting work of PQC. The joint shall be made butt type. At all construction joints, steel bulk heads shall be used to retain the concrete while the surface is finished. The surface of the concrete laid subsequently shall conform to the grade and cross sections of the previously laid pavement. When positioning of bulk head/stop-end is not possible, concreting to an additional 1 or 2 m length may be carried out to enable the movement of joint cutting machine so that joint grooves may be cut and the extra 1 or 2 m length is cut out and removed subsequently after concrete has hardened. Like contraction joint, the construction joint shall also be widened to dimensions as per IRC:57, not before 14 days curing of PQC.

1.3.7 Longitudinal joint:

1) The longitudinal joints shall be saw cut as per details of the joints shown in the drawing or as per dimensions given in IRC:57. The groove may be cut after the final set of the concrete. Joints should be sawn to at least rd the depth of the slab ±5 mm as indicated in the drawing.

2) Tie bars shall be provided at the longitudinal joints as per dimensions and spacing shown in the drawing and in accordance with Clause 602.6.6 of MoRT&H Specifications. Longitudinal joints shall also be widened to dimensions as per IRC:57, not before 14 days curing of PQC.

1.4 SELECTION OF MATERIALS IN PQC AS PER MORTH SPECIFICATIONS.

Source of materials : The Contractor shall indicate to the Engineer the source of all materials to be used in the concrete work with relevant test data sufficiently in advance, and the approval of the Engineer for the same shall be obtained at least 45 days before the scheduled commencement of the work in trial length. If the Contractor subsequently proposes to obtain materials from a different source during the execution of main work, he shall notify the Engineer, with relevant test data, for his approval, at least 45 days before such materials are to be used.

1.4.1 Cement : Any of the following types of cement capable of achieving the design strength may be used with prior approval of the Engineer.

Table 1.1 Type of cement confirming to Indian standard code

S. No.	Type	Confirming to:
1	Ordinary Portland Cement 43 Grade	IS:8112
2	Portland Blast Furnace Slag Cement	IS:455
3	Portland Pozzolana Cement	IS:1489-Part I
4	Ordinary Portland Cement 53 Grade	IS:12269

1) Fly ash up to 20 percent by weight of cement may be used in Ordinary Portland Cement 53 Grade. No fly ash shall be used in any other grade of Cement other than 53 Grade. The fly ash shall conform to IS:3812 (Part I).

2) Ground Granulated Blast Furnace Slag (GGBFS) obtained by grinding granulated slag conforming to IS:12089. GGBFS shall not be used in any other grade of cement except 53 grade. The content of GGBFS shall be up to 50 percent by weight of Ordinary Portland Cement 53 grade.

3) Site mixing of fly ash and ground granulated slag shall be permitted only after ensuring availability of the equipments at site for uniform blending through a specific mechanized facility with automated process control like batch mix plants conforming to IS:4925 and IS:4926. Site mixing will not be allowed otherwise.

4) Mix design will be done as per IRC:44. The OPC content shall not be less than 310 kg/cum in case of blending at site. The curing period may be suitably enhanced by at least about 2 days.

5) The Portland Pozzolana Cement produced in factory shall not have fly ash content more than 25 percent. The Portland Pozzolana Cement produced in factory with fly ash content more than 25 percent shall not be used. Certificate from the manufacturer to this effect shall be procured before use.

If the soil around PQC has soluble salts like sulphates in excess of 0.5 percent, the cement used shall be sulphate resistant and shall conform to IS:12330. Guidance may be taken from IRC:44 for ascertaining the compressive/flexural strength of cement concrete required to match with the prescribed design strength of concrete. Cement to be used may preferably be obtained in bulk form. If cement in paper bags is proposed to be used, there shall be bag-splitters with the facility to separate pieces of paper bags and dispose them off suitably. No paper pieces shall enter the concrete mix. Bulk cement shall be stored in accordance with Clause 1014 of MoRT&H Specifications. The cement shall be subjected to acceptance test just prior to its use.

1.4.2 Chemical Admixtures

Admixtures conforming to IS:9103 and IS:6925 shall be permitted to improve workability of the concrete or extension of setting time, on satisfactory evidence that they will not have any adverse effect on the properties of concrete with respect to strength, volume change, durability and have no deleterious effect on steel bars. The particulars of the admixture and the quantity to be used, must be furnished to the Engineer in advance to obtain his approval before use. Satisfactory performance of the admixtures should be proved both on the laboratory concrete trial mixes and in the trial length paving. If air entraining admixture is used, the total quantity of air in air-entrained concrete as a percentage of the volume of the mix shall be 5±1.5 percent for 31.5 mm nominal size aggregate.

1.4.3 Fibers:

Fibers may be used subject to the provision in the design/approval by the Engineer to reduce the shrinkage cracking and post-cracking. The fibers may be steel fiber as per IRC:SP:46 or polymeric Synthetic Fibres within the following range of specifications:

Effective Diameter	10 micron – 1.0 mm
Length	6-48 mm
Specific gravity	more than 1.0
Suggested dosage	0.6-2.0 kg/cum (0.2 -0.6 % by weight of cement in mix). [Usage will be regulated as stipulated in IRC:44/IS:456 or any other specialist literature.]
Water absorption	less than 0.45 percent

Melting point of this fiber shall not be less than 160°C and the aspect ratio generally varies from 200 to 2000. These synthetic fibers will have good alkali and UV light resistance.

1.4.4 Aggregates

Aggregates for pavement concrete shall be natural material complying with IS:383 but with a Los-Angeles Abrasion Test result not more than 35 percent. The aggregates shall be free from dirt, flint, chalcedony or other silica in a form that can react with the alkalis in the cement. In addition, the total chlorides content expressed as chloride ion content shall not exceed 0.06 percent by weight and the total sulphate content shall not exceed 0.25 percent by weight.

Coarse aggregates : Coarse aggregates shall consist of clean, hard, strong, dense, non-porous and durable pieces of crushed stone or crushed gravel and shall be devoid of pieces of disintegrated stone, soft, flaky, elongated, very angular or splintery pieces. The maximum size of coarse aggregate shall not exceed 31.5 mm for pavement concrete. No aggregate which has water absorption more than 2 percent shall be used in the concrete mix. The aggregates shall be tested for soundness in accordance with IS:2386 (Part-5). After 5 cycles of testing, the loss shall not be more than 12 percent if sodium sulphate solution is used or 18 percent if magnesium sulphate solution is used. The combined flakiness and elongation index of aggregate shall not be more than 35 percent.

Fine aggregates : The fine aggregates shall consist of clean natural sand or crushed stone sand or a combination of the two and shall conform to IS:383. Fine aggregate shall be free from soft particles, clay, shale, loam, cemented particles, mica and organic and other foreign matter.

1.4.5 Water:

Water used for mixing and curing of concrete shall be clean and free from injurious amount of oil, salt, acid, vegetable matter or other substances harmful to the finished concrete. It shall meet the requirements stipulated in IS:456.

1.4.6 Mild steel bars for dowels and tie bars :

i) Dowel Bar shall be of plain mild steel conforming to IS:432 and will have yield stress of Fe-240.

ii) Tie bar shall be of TMT steel conforming to IS:1786 and will have yield stress of Fe-500.

1.4.7 Fly Ash

The fly ash, also known as pulverized fuel ash, Coal Fly ash (FA) is a by-product of the combustion of pulverized coal in thermal power plants. It is removed by the dust collection systems from the exhaust gases of fossil fuel power plants as very fine, predominantly spherical glassy particles from the combustion gases before they are discharged into atmosphere. The size of particles is largely dependent on the type of dust collection equipment . Diameter of fly ash particles ranges from less than 1 μm to 150 μm. It is generally finer than Portland cement. The fly ash obtained from electrostatic precipitators may have a specific surface of about 350000 to 500000 mm^2/g, i.e. it is finer than Portland cement. The fly-ash obtained from cyclone separators is comparatively coarser and may contain larger amounts of un-burnt fuel. The chemical composition of fly ash is determined by the types and relative amounts of incombustible material in the coal used. The major chemical constituents in fly ash are silica, alumina and oxides of calcium and iron. Because of its fineness and pozzolanic and sometimes self-cementitious nature, fly ash is widely used in cement and concrete.

Depending upon the collection system, varying from mechanical to electrical precipitators or bag houses and fabric filters, about 85–99.9% of the ash from the flue gases in retrieved in the form of fly ash. Fly ash accounts for 75–85% of the total coal ash, and the remainder is collected as bottom ash or boiler slag . Fly ash because of its mineralogical composition, fine particle size and amorphous character is generally pozzolanic and in some cases also self cementitious whereas bottom ash and boiler slag are much coarser and are not pozzolanic in nature. Therefore, It is important to note that all the ash is not fly ash and the fly ashes produced by different power plants are not equally pozzolanic and, therefore, are not always suitable for use as mineral admixture in concrete. The major components of fly ash reported in oxide form are silica (SiO_2), alumina (Al_2O_3), and oxides of calcium and iron (CaO and Fe_2O_3). Fly ash composition varies with the source of coal.

a) ***Utilization of Fly Ash in Cement and Concrete*****:** Coal fly ash can be utilized in following ways.

High Volume Uses

High volume utilization of fly ash includes

- as structural fills in embankments, dams, dikes and levees, and
- as sub-base and base courses in road way construction.

Medium Volume Uses

This includes the use of fly ash

- as raw material in cement production
- as an admixture in blended cements and
- as replacement of cement or as a mineral admixture in concrete
- in addition coal ash including fly ash may be used as partial replacement of fine aggregate in concrete
- for production of lightweight aggregates for concrete and many other applications.

Low Volume Uses

This includes the coal ash utilization

- in high value added applications such as metal extractions. High value metal recovery of Aluminum (Al), Gold (Au), Silver (Ag), Vanadium (Va) and Strontium (Sr) fall in this category.
- Fly ash has potential uses for producing light weight refractory material and exotic high temperature resistant tiles
- Cenospheres or floaters in fly ash are used as special refractory material and also as additives in forging to produce high strength alloys.

Miscellaneous Uses

Based upon its physical properties, coal ash is used

- as land fill for land
- as filler in asphalt, plastics, paints and rubber products

- in water treatment and as absorbent for oil and chemical spills

Due to different densities of cement and fly ash 3100 to 3200 Kg/m^3 and 2200 to 2400 kg/m^3, respectively, a part replacement by equal mass increases the volume of cementitious material, where as replacement by equal volume reduces the mass. In practice the replacement is usually on a mass basis. The use of fly ash influences the volume yield of concrete. It has little effect on the drying shrinkage of concrete.

b) ***Classification of Fly Ash***

As per ASTM C 618 (1993) specification for "Fly ash and raw or calcined natural pozzolan for use as mineral admixture in Portland cement concrete," pozzolans are defined as "silicious and aluminous materials which in themselves possess little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with $Ca(OH)_2$ at normal temperatures to form compounds possessing cementitious properties."

ASTM C 618 (1993) categorizes natural pozzolans and fly ashes into the following three categories.

Class-N: Raw or calcined natural pozzolans such as some ditomaceous earths, opaline chert and shale, stuffs, volcanic ashes and pumice are included in this category. Calcined kaolin clay and laterite shale also fall in this category of pozzolans.

Class-F: Fly ash normally produced from burning anthracite or bituminous coal falls in this category. This class of fly ash exhibits pozzolanic property but rarely, if any, self hardening property.

Class-C: Fly ash normally produced from lignite or sub-bituminous coal is the only material included in this category. This class of fly ash has both pozzolanic and varying degree of self-cementitious properties. (Most Class-C fly ashes contained more than 15% CaO. But some Class-C fly ashes may contain as little as 10% CaO).

According to IS : 3812-1981 fly ash shall be supplied in the following grade corresponding to the properties specified in tables.

Grade 1: For incorporation in cement mortar and concrete and in lime pozzolana mixture, and for manufacture of Portland pozzolana cement.

Grade 2: For incorporation in cement mortar and concrete and in line pozzolana mixture.

c) ***Reaction Mechanism:*** Setting or hardening of OPC concretes occurs due to the hydration reaction between water and cementitious compounds in cement which give rise to several types of hydrates of calcium silicate (CSH), calcium aluminate (CAH) besides calcium hydroxide (CH). These hydrates are generally called as "Tobermorite gel". The adhesive and cohesive properties of the gel bind the aggregate particles. Calcium hydroxide is a by-product of cement hydration. When fly ash is incorporated in concrete, the calcium hydroxide liberated during hydration of OPC reacts slowly with the amorphous alumino-silicates, the pozzolanic compounds, present in the fly ash. The products of these reactions, termed as pozzolanic reaction products, are time dependent but are basically of the same type and characteristics as the products of the cement hydration. Thus additional cementitious products become available which impart additional strength to concrete.

The following equations illustrate the pozzolanic reaction of fly ash with lime to produce additional calcium silicate hydrate (C–S–H) binder:

Cement reaction: C_3S + H --------hydration------> C-S-H + CaOH

Pozzolanic reaction : CaOH + S -------------> C-S-H

(silica from ash constituents)

Table 1.2 Chemical Requirements of fly ash (IS: 3812-1981)

S.No	Characteristic	Requirement
1	Silicon dioxide (SiO2) plus aluminum oxide (Al2O3) plus iron oxide (Fe2O3), per cent by mass, Min.	70
2	Silicon dioxide (SiO2), per cent by mass, Min	35
3	Magnesium oxide (MGO), per cent by mass, max	5
4	Total sulphur trioxide (SO3), per cent by mass, max	2.5
5	Available alkalis as sodium oxide Na2O, per cent by mass, max	1.5
6	Loss in ignition, per cent by mass, max	12

1.4.8 RICE HUSK ASH

Rice husk is one of the main agricultural residues obtained from the outer covering of rice grains during the milling process. It constitutes 20% of the 500 million tons of paddy produced in the world. The rice husk ash had no useful application and had usually been dumped into water streams and caused pollution and contamination of springs until it was known to be a useful mineral admixture for concrete.. Various experiments were carried out to determine properties of concretes incorporating optimum RHA. Tests include compressive strength, splitting tensile strength, workability, water permeability and flexural strength.

Table 1.3 - Physical Requirements Of Fly ash (IS: 3812-1981)

S.No	Characteristics	Requirement	
		Grade of flyash	
		Grade-1	Grade-2
1	Fineness, specific surface in m^2/kg by Blaine's permeability method, Min	320	250
2	Lime reactivity Average compressive stress in N/mm^2, Min	4	3
3	Compressive strength at 28 days in N/mm^2, Min	Not less than 80% of the strength of corresponding plain cement mortar cubes	
4	Drying shrinkage, per cent, max	0.15	0.10
5	Soundness by autoclave test expansion of specimens, per cent, Max.	0.8	0.8

Table 1.4 Chemical composition of Indian fly ash

Chemical composition	Percentage %
Silica (SiO_2)	49-67
Alumina (Al_2O_3)	16-29
Iron Oxide (Fe_2O_3)	4-10
Calcium Oxide (CaO)	1-4
Magnesium Oxide (MgO)	0.2-2
Sulphur (SO_3)	0.1-2
Loss of Ignition	0.5-3.0

In the case of RHA, the compressive strength of blended concrete structures has been shown to be enhanced and water permeability to be decreased chemically and physically. Rice Husk is one of the waste materials in the rice growing regions. This not only makes the Purposeful utilization of agricultural waste but it will also reduce the consumption of energy used in the production of cement. Therefore Rice Husk is an agro based product which can be used as a substitute of cement without sacrificing the strength and durability. Generally the Rice Husk Ash is used while burning the raw clay bricks in the Brick Kilns. The calorific value varies with rice variety, moisture and bran content but a typical value for husks with 8-10% moisture content and essentially zero bran is 15 MJ/kg. Rice Husk Ash is obtained from burning of Rice Husk, which is the by-product of rice milling. It is estimated that 1000 kg of rice grain produce 200 kg of Rice Husk; after Rice Husk is burnt, about 20 percent of the Rice Husk or 40 kg would become RHA. Rice Husk Ash contains as much as 80-85% silica which is highly reactive, depending upon the temperature of incineration. However, the strength characteristics are considered adequate for general masonry work. The utilization of rice husk ash as a pozzolanic material in cement and concrete provides several advantages, such as improved strength and durability properties, reduced materials costs due to cement savings, and environmental benefits related to the disposal of waste materials and to reduced carbon dioxide emissions .Portland Rice Husk Ash cements containing up to 50% Ash by weight showed compressive strengths which were considerably higher than the control Portland cements even at early ages of 3 and 7 days. The cements containing Rice Husk Ash possess excellent resistance to dilute organic and mineral acids. The water demand for normal consistency tends to increase with increasing Ash content of the blended cements. However, this can be corrected by application of certain water reducing admixtures. The investigations as outlined above point towards encouraging trend. Reactions that take place in the preparation of Rice Husk Ash concrete are; Silicon burnt in the presence of Oxygen gives Silica.

$Si + O_2$ --------► SiO_2

C_3 S (Cement) + H_2O -----------► CSH + Ca $(OH)_2$

The highly reactive silica reacts with Calcium hydroxide released during the hydration of cement, resulting in the formation of Calcium Silicates responsible for strength.

SiO_2 + Ca $(OH)_2$ -----► CSH + SiO_2

Physical Properties

Completely burnt rice-husk is grey to white in color, while partially burnt Rice Husk Ash is blackish. Rice husk ash is a very fine material. The RHA samples are black with some gray particles, resulting from different stages of the carbon combustion during burning of rice husk. The active silica obtained after the heating and grinding presents reduced size of particles and grey coloration due to the lower content carbonaceous material. At 700^{0}C for 6 hours, the thermal treatment yielded bright white silica. Average particle size of rice-husk ash ranges from 3 to 10 μm.

Table 1.5 Chemical composition of RHA

Constituents	**Percentage**		
	[*Khani et al.* (2009)]	[*Habeeb et al. (*2010)]	[*Givi et al.* (2010)]
Silica (SiO_2)	89.61	88.32	87.86
Alumina(Al_2O_3)	0.04	0.46	0.68
Iron oxide(Fe_2O_3)	0.22	0.67	0.93
Calcium Oxide (CaO)	0.91	0.67	1.30
Magnesium oxide (MgO)	0.42	0.44	0.35
Sodium oxide (Na_2O_3)	0.07	0.12	0.12
Potassium oxide(K_2O)	1.58	2.91	2.37
LOI Loss on ignition	5.91	5.81	-

Advantages of Using RHA

Rice-husk ash is a very fine pozzolanic material. The utilization of rice husk ash as a pozzolanic material in cement and concrete provides several advantages such as improved strength, enhanced durability properties, reduced materials costs due to cement savings, and environmental benefits related to the disposal of waste materials and to reduced carbon dioxide emissions.

Applications of RHA.

RHA can be used as aggregates and fillers for concrete and board production, are an economical substitute for micro silica /silica fumes , absorbents for oils and chemicals, soil ameliorants (An ameliorant is something that helps improve soil drainage, slows drainage, breaks up soil or binds soil, feeds and improves structure etc.), as insulation powder in steel mills, as repellents in the form of "vinegar-tar", as a release agent in the ceramics industry, as an insulation material for homes and refrigerants

CHAPTER-2

LITERATURE REVIEW

The relevant literature pertaining to the use of Fly-Ash(FA) and Rice Husk Ash(RHA) in concrete carried out in India and abroad has been reviewed and presented as under:-

2.1 FLY-ASH

2.1.1 Fresh Properties:

Low calcium Class F fly ash normally acts as a fine aggregate of spherical form in early stages of hydration whereas high calcium Class-C fly ash may contribute to the early cementing reactions in addition to its presence as fine particulate in the concrete mix. Hydration of cement is an exothermic reaction and the released heat causes a rise of temperature of fresh concrete.

Brown, J.H. (1982) in his paper " **The strength and workability of concrete with Fly Ash substitution**" conducted several studies with fly ash replacing cement and fine aggregate at levels of 10-40% by volume. He concluded that for each 10% of ash substituted for cement, the compacting factor or workability changed to the same order as it would by increasing the water content of the mix by 3-4%. When fly ash was substituted for sand or total aggregate, workability increased to reach a maximum value at about 8% ash by volume of aggregate. Further substitution caused rapid decrease in workability.

Gebler and Klieger, (1983) in their paper " **Effect of fly ash on the air void stability of concrete** " investigated the requirements of Air Entraining Agent (AEA) for Class-C and Class-F fly ashes. They reported that (1) concretes made with Class C fly ash generally require less AEA than those made with Class F fly ashes; (2) for 6% air content in concrete, the AEA varied from 126 to 173% for fly ashes having more than 10% CaO, whereas it was in the range of 177 to 553% for fly ashes containing less than 10% CaO; and (3) increase in both total alkalies and SO_3 contents in fly ash affect the air entrainment favorably. A concrete containing a Class F fly ash that has relative high CaO content and less organic matter or carbon tends to be less vulnerable to loss of air.

Owens, (1989) in his paper" **Fly ash and its usage in concrete**" reported that with the use of fly ash containing large fraction of particles coarser than 45μm or a fly ash with high amount of unburned carbon, exhibiting loss on ignition more than 1%, higher water demand was observed.

Sivasundram, et al. (1990) in their paper " **Selected properties of high volume fly ash concretes**" investigated the setting time of high-volume fly ash concrete mixes, and concluded that the initial setting time of 1.50 hours was comparable to that of the control concrete, whereas the final setting time was extended by about 3 hours as compared to that of the control concrete.

2.1.2 Hardened Properties:

Carette and Malhotra, (1983) in their research paper "**Characterization of Canadian fly Ashes and their Performance in Concrete"** studied the effect of Canadian fly ashes on the compressive strength of concrete mixes. Cement was replaced with 20% fly ash in all the mixes. Compressive strength was measured up to the age of 365 days. It was seen that compressive strength continued to increase with age, indicating pozzolanic action of fly ashes.

Joshi and Lohtia, (1993) in their paper " **Effects of premature freezing temperatures on compressive strength, elasticity and microstructure of high volume fly ash concrete**" tested a large number of fly concrete mixes made by using three different fly ashes containing about 10% calcium oxide. The replacement level varied between 40 and 60% by weight of cement. The mixes were super-plasticized and air-entrained to obtain 100 to 120 mm slump and 6 ± 1% air content. The cementitious material content varied from 380 to 466 kg/m^3, water to cementitious material ratio from 0.27 to 0.37, coarse aggregate ranged from 1,012 to 1,194 kg/m^3, and fine aggregate or sand varied from 712 to 643 kg/m^3. They reported that (1) at 7 days, the fly ash concretes obtained strength between 27.9 and 41.0 MPa compared to 44.1 MPa of control concrete. However at the age of 28 days, the fly ash concretes developed strength varying from 37.6 to 50.7 MPa against 58.7 MPa for control concrete. At 120 days, strength of fly ash concrete ranged from 54.8 to 74.6 MPa whereas it was 74.6 MPa of control concrete.

Lohtia et al. (1996) in his paper " **Creep of fly ash concrete** " studied the creep and creep recovery of plain and fly ash concretes at stress-strength ratios of 20 and 35%. Fly ash content was varied between 0 and 25%. They concluded that (1) replacement of 15% of cement with fly ash was optimum with respect to strength, elasticity, shrinkage and creep of fly ash concrete; (2) creep–time curves for plain and fly ash concretes were similar, and creep linearly related to the logarithm of time; (3) with fly ash content up to 15%, increase in creep was negligible. However, slightly higher creep occurred with fly ash content more than 15%; (4) creep coefficients were similar for the materials with fly ash content in the range of 0–25%; and (5) creep recovery was found to vary from 22 to 43% of the corresponding 150-day creep. For replacement beyond 15%, the creep recovery was smaller. No definite trend of creep recovery as a function of stress-strength ratio was observed.

Saraswathy et al. (2003) in their paper **" Influence of activated fly ash on corrosion–resistance and strength of concrete**" investigated the influence of activated fly ash on the compressive strength of concrete. Various activation techniques, such as physical, thermal and chemical were adopted. Concrete specimens were prepared with 10, 20, 30 and 40% of activated fly ash replacement levels with cement. Compressive strength was determined at 7, 14, 28 and 90 days. They concluded that (1) activation of fly ash improved the strength of concrete. However, the compressive strength of fly ash concrete was less than that of ordinary portland cement (OPC) even after 90 days of curing; and (2) among the activation systems, chemically activated coal fly ash (CFA) improved the compressive strength to a certain extent, only with 10 and 20% replacements. Since the CFA surface layer is etched by a strong alkali to facilitate more cement particles to join together and also the addition of CaO which is further promoting the growth of CSH gel and $Ca(OH)_2$ which is more advantageous to enhance the strength development.

Siddique, R. (2003) in his paper "**Effect of fine aggregate replacement with class F fly ash on the mechanical properties of concrete**", studied the effect of partial replacement of fine aggregate (sand) with varying percentages of Class F fly ash on the compressive strength, splitting tensile strength, flexural strength and modulus of elasticity of concrete up to the age of 365 days. Fine aggregate (sand) was replaced with five levels of percentages (10, 20, 30, 40, and 50%) of Class F fly ash by weight. Control mix (without fly ash) was proportioned to have a 28-day cube compressive strength of 26.4 MPa. Based on the results, it was concluded that (1) compressive strength of fine aggregate (sand) replaced fly ash concrete specimens was higher than the plain concrete (control mix) specimens at all the ages. The strength differential between the fly ash concrete specimens and plain concrete specimens became more distinct after 28-days; (2) compressive strength continued to increase with age for all fly ash replacement levels; (3) The maximum compressive strength occurs with 50% fly ash content at all ages. It was 40.0 MPa at 28-day, 51.4 MPa at 91-day, and 54.8 MPa at 365-day. (4) splitting tensile strength, and flexural strength of fine aggregate (sand) replaced fly ash concrete specimens was higher than the plain concrete (control mix) specimens at all the ages. The strength differential between the fly ash concrete specimens and plain concrete specimens became more distinct after 28-days; (5) both splitting and flexural strengths continued to increase with age for all fly ash percentages; (6) at all the ages, the maximum splitting tensile strength was observed with 50% fly ash content. It was 3.5 MPa at 28-day, 4.3 MPa at 91-day, and 4.4 MPa at 365-day; (7) maximum flexural strength was found to occur with 50% fly ash content at all ages. It was 4.3 MPa at 28-day, 5.2 MPa at 91-day, and 5.4 MPa at 365-day. (8) modulus of elasticity of fine aggregate (sand) replaced fly ash concrete specimens was higher than the plain concrete (control mix) specimens at all the ages. The differential between the fly ash concrete specimens and plain concrete specimens became more distinct after 28-days; (9) modulus of elasticity of fine aggregate (sand) replaced fly ash concrete continued to increase with age for all fly ash percentages; and (10) at all ages, the maximum value of modulus of elasticity occurs with 50% fly ash content. It is 24.5 GPa at 28-day, 28.0 GPa at 91-day, and 29.0 GPa at 365-day.

Atis et al. (2004) in their paper " **Strength and shrinkage properties of mortar containing a nonstandard high-calcium fly ash**" assessed the drying shrinkage of mortar mixtures containing high calcium non standard fly ash up to the age of 5 months. Five mortar mixtures including control Portland cement and fly ash mortar mixtures were prepared. Fly ash replaced cement on mass basis at the replacement ratios of 10, 20, 30 and 40%. Water–cementitious materials ratio was 0.4. Mixtures were cured at 65% relative humidity and 20 ± 2^0 C. They reported that shrinkage of Portland cement mortar at 5 months was 0.1228%. Shrinkage of fly ash mortar decreased with the increase in fly ash content. Shrinkages of mortar containing 10, 20 and 30% fly ash were 25, 37 and 43%, lower than the shrinkage of Portland cement mortar at the end of 5 months. The reduction in shrinkage with the use of fly ash in mortar could be explained by the dilution effect of fly ash. The expansive property of fly ash most probably contributed to the reduction in drying shrinkage.

Demirboga et al. (2007) in their paper "**Thermo-mechanical properties of concrete containing high-volume mineral admixtures**" investigated the Thermal Conductivity (TC) of HVFA concrete at the age of 28 days. Cement was replaced with 0, 50, 60, and 70% of Class C fly ash. They concluded that TC of concrete decreased to 32, 33, and 39% for 50, 60 and 70% fly ash replacement, respectively.

2.1.3 Durability Properties:

Ho and Lewis, (1983) in their paper **" Carbonation of concrete incorporating fly ash or a chemical admixture**", investigated the carbonation rates of three types of concrete mixes (1) plain concrete; (2) the second containing a water reducing admixture; and (3) third in which fly ash was used to replace part of the cement. Accelerated carbonation was induced by storing specimens in an enriched CO2 atmosphere (4%) at 20_C and 50% RH for 8 weeks. One week under these conditions was approximately equivalent to 1 year in a normal atmosphere (0.003% CO2). They concluded that (1) concretes having the same strength and water-to-cement ratio do not necessarily carbonate at the same rate; (2) concrete containing fly ash showed significant improvement in quality when curing was extended from 7 to 90 days. This improvement was much greater than that achieved for the plain concrete; and (3) depth of carbonation is a function of the cement content for concretes moist-cured for 7 days. However, with further curing to 90 days, concrete containing fly ash showed a slower rate of carbonation as compared to plain and water-reduced concretes.

Virtanen, J. (1983) in his paper **" Freeze–thaw resistance of concrete containing blast furnace slag, fly ash or condensed silica fume**" evaluated the freezing and thawing resistance concrete made with fly ash. They concluded that (1) air content has the greatest influence on the freeze–thaw resistance of concrete; (2) addition of fly ash had no major influence on the freeze–thaw resistance of concrete if the strength and air content are kept constant.

Naik et al., (1994) in their research **" Permeability of concrete containing large amounts of fly ash"** evaluated the influence of addition of large amounts (50 and 70% cement replacement) of Class C fly ash on the chloride permeability of concrete. Concrete mixtures were designated as C-3 (0% fly ash), P4-7 (50% fly ash) and P4-8 (70% fly ash). Chloride permeability was determined in accordance with ASTM C1202, Chloride permeability decreased with age. At the age of 2 months, all concrete mixtures except the 70% fly ash mixture exhibited moderate (2,000–4,000 C) permeability in accordance with ASTM C1202 specifications. The 50% fly ash concrete mixture showed lower permeability relative to the no-fly ash concrete at all ages. The 70% fly ash mixture also performed better than that of the no-fly ash concrete after 3 months.

Mehta, P.K. (2000) in his paper " **Sulfate Attack on concrete**." concluded that fly ashes are amongst the group of pozzolans that significantly increase the life expectancy of concrete exposed to sulfate attack. In general, Class F type fly ash meeting the specification requirements will improve the sulfate resistance of any concrete/mortar mix in which it is included, although the degree of improvement may vary with either the cement used or the fly ash. The situation with Class C fly ash is different. A few studies indicated that some Class C fly ashes may rather reduce sulfate resistance when used in normal proportions

Siddique, R. (2003) in his paper **" Effect of fine aggregate replacement with class F fly ash on the abrasion resistance of concrete** " studied the abrasion resistance of concrete proportioned to have four levels of fine aggregate replacement (10, 20, 30 and 40%) with Class F fly ash. A Control mixture with ordinary Portland cement was designed to have 28 days compressive strength of 26 MPa. Concrete specimens of size 65 9 65 9 60 mm were made for the purpose. The abrasion resistance of concrete mixtures was determined at the ages of 28, 91, and 365 days in accordance with Indian Standard Specifications. It was measured in term of depth of wear. The variation of depth of wear versus percentage of fine aggregate replacement with Class F fly ash, at 60 min of abrasion time concluded that with the increase in fly ash content, depth of wear decreased, which indicated that the abrasion resistance of concrete increased with the increase in fly ash content. This showed that for a particular percentage of fine aggregate replacement with fly ash, depth of wear decreased with increase in age, which means that abrasion resistance increased with age. This could be primarily attributed to the increase in compressive strength resulting from increased maturity of concrete with age.

Chalee et al. (2007) in their paper " **Effect of W/C ratio on covering depth of fly ash concrete in marine environment**" studied the effect of W/C ratio on covering depth required against the corrosion of embedded steel of fly ash concrete in marine environment up to 4-year exposure. Fly ash was used to partially replace Portland cement type I at 0, 15, 25, 35, and 50% by weight of cementitious material. Water-to-cementitious material ratios (w/c) of fly ash concretes were varied at 0.45, 0.55, and 0.65. Tests were conducted for corrosion of embedded steel bar after being exposed to tidal zone for 2, 3, and 4 years. Based on the tests, they concluded that (1) covering depth required for the initial corrosion of embedded steel bar in concrete could be reduced with fly ash; (2) decrease in W/C ratio resulted in reducing the covering depth required for initial corrosion, and generally affected the cement concrete rather than the fly ash concrete; (3) fly ash concretes with 35 and 50% replacements and W/C ratio of 0.65, provided the result of corrosion resistance at 4-year exposure as good as cement concrete with W/C ratio of 0.45; and (4) concrete with compressive strength of 30 MPa could reduce the covering depth from 50 to 30 mm by using fly ash to replace Portland cement of 50%.

2.2 RICE HUSK ASH:

2.2.1 Fresh Properties:

Zhang and Malhotra, (1996) in their paper " **High-performance concrete incorporating rice husk ash as a supplementary cementing material**" investigated the influence of RHA on the air-entraining admixture (AEA) requirement of concrete mixtures made with RHA (0, 5, 8, 10 and 15%) as partial replacement of cement. It was observed that AEA requirement increased with the increase in RHA content possibly because of high specific surface area of RHA in comparison to cement.

Bui et al. (2005) in their paper " **Particle Size Effect On The Strength Of Rice Husk Ash Blended Gap-Graded Portland Cement Concrete**." investigated the workability of concrete and super plasticizer content to be added when cement is replaced by RHA in gap graded concrete. Two kinds of an ordinary Portland cement were employed, i.e., PC30 and PC40. Twenty four concrete mixtures were made, 12 mixtures for each of the two kinds of PC. Three levels of the water to binder ratio were investigated, i.e., 0.30, 0.32 and 0.34. The mixtures with water to binder ratio of 0.30 were made with a binder content of 550kg/m^3 concrete. The binder content of all other mixtures was

500kg/m^3. Rice husk ash was used to replace 10%, 15% and 20% by mass of PC. The super plasticizer was added to all mixtures for obtaining high workability. In the mixtures with water to binder ratio of 0.34, the amount of super plasticizer was kept constant to investigate the influence of RHA on workability All the mixtures have high slump values and they all are stable. A reduction in slump of the blended concrete mixtures is found when a part of the PC is replaced by RHA at equal super plasticizer content. This decline in slump increases with replacement level of cement by RHA. The finer cement (PC40) requires a higher super plasticizer dosage for achieving equal slump. For the plain (PC30) concrete, a super plasticizer dosage of 1% by mass of the cement is required to attain a slump of 210mm. In case of the finer cement (PC40), the super plasticizer dosage is increased to 1.28% for obtaining approximately the same slump.

Ganesan et al. (2008) in his paper **" Rice husk ash blended cement: assessment of optimal level of replacement for strength and permeability properties of concrete**" reported the effect of cement replacement with RHA on the consistency and setting times of cement. Percentages of cement replacement were 0, 5, 10, 15, 20, 20, 25, 30, and 35. They observed that consistency of control mixture was approximate 32%, however, water required for standard consistency linearly increased with an increase in RHA content. The standard consistency with 35% RHA content was 44%. As ashes are hygroscopic in nature and the specific surface area of RHA is much higher than cement, it needs more water. The results of initial and final setting times are shown in Fig. 5.10. It was observed that up to 15%, RHA level increased the initial setting time. At 20, 25, 30 and 35%, there was reduction in initial setting time. The initial setting time measured for RHA blended cements up to 35% was higher than that of control OPC. On the other hand, the final setting time decreased with the increase in RHA up to 35%.

Khani et al. (2009) in their paper **" The Effect of Rice Husk Ash on Mechanical Properties and Durability of Sustainable Concretes**." studies the surface water absorption in rice husk ash concrete in comparison to controlled concrete**.** A total of 4 concrete mixtures were made; one corresponding to a control concrete (CTL) and three others with 7%, 10% and 15% RHA Replaced with cement by weight. Concrete cubes of 100×100×100 mm dimension were cast for water penetration tests. In this test, water was forced into the concrete samples from one side for three days and under constant pressure of 0.5 MPa. Then, the samples were split in a plane parallel to the direction of water penetration, and the greatest depth of water penetration into the concrete sample was measured. Depth of water penetration of concrete specimens decreased significantly with an increasing in RHA content and curing period.

Givi et al. (2010) in their paper "**Assessment of the effects of rice husk ash particle size on strength, water permeability and workability of binary blended concrete**." investigated the workability of concrete by partial replacement of cement with agro-waste rice husk ash. Standard slump tests were used to determine the workability of the concrete. Two types of rice husk ash with average particle size of 5 micron (ultra fine particles) and 95 micron and with four different contents of 5%, 10%, 15% and 20% by weight were used. Result shows the influence of RHA content and particle fineness on the workability of mixtures at constant water to binder ratio of 0.40. Unlike the C0 series, all investigated RHA blended mixtures had acceptable workability with high slump (more than 6.5 cm) values. Partial replacement of cement by RHA improved workability of fresh concrete for both used average particle sizes. However, the RHA with average particle size of 95 μm gave rise to higher slump values for comparable cases.

Habeeb et al. (2010) in their paper "**Study on Properties of Rice Husk Ash and Its Use as Cement Replacement Material"** investigates the workability, fresh density and super plasticizer content of rice husk ash (RHA) produced by using a Ferro-cement furnace. The slump was in the range of 200 – 240 mm, bleeding was negligible for the control mixture. For concretes incorporating RHA, no bleeding or segregation was recorded. The fresh density was in the range of 2253 to 2351 $kg.m^{-3}$, the lowest density values were for 20F1 mixture. The concretes incorporating finer RHA resulted in denser concrete matrix. SP content has to be increased along with the RHA fineness and content due to the high specific surface area of RHA which would increase the water demand, therefore, to maintain high workability, SP content rose up to 2% for the 20F3 mixture.

Uduweriya et al. (2010) in their paper " **Investigation of Compressive Strength of Concrete Containing Rice-Husk-Ash** " investigated surface absorption of rice husk ash concrete (20% RHA), fly ash concrete (20% FAC) and combination of rice husk and fly ash in concrete (10%RHA + 10%FAC). The surface water absorption test was carried out with a concrete cube of 150x150x150 mm size. The W/C ratio of 0.75 was used for concrete mixture. The surface water absorption of concrete was tested at 10, 30 and 60 min after the water head was released to the concrete surface.

The concrete containing RHA has shown a considerable reduction in surface water absorption in concrete. Further addition of fly ash has given better result than the 20% RHA replacements

Tashima et al. (2010) in their paper " **The Possibility Of Adding The Rice Husk Ash (RHA) To The Concrete.**", evaluates how different contents of rice husk ash (RHA) added to concrete may influence its physical and mechanical properties like surface water absorption. Samples with dimensions of 10 X 20 cm were tested, with 5% e 10% of RHA, replacing in mass the cement. Three mixtures were made i.e. Mixture D (controlled mix), Mixture E (5% replacement) , Mixture F (10% replacement). The results reveal that higher substitution amounts results in lower water absorption values, it occurs due to the RHA that is finer than cement. Adding 10% of RHA to the concrete, a reduction of 38.7% in water absorption is observed when compared to mixture D.

Nagrale et al. (2012) in their paper **" Utilization of Rice Husk Ash**" studied various properties like workability of the concrete mix when we replace cement by 15% RHA compared to our normal ordinary Portland cement concrete mix. The fine aggregate used is natural sand conforming to Zone II & standard sand of Grade 2. The coarse aggregate used are of sizes 20 mm and 12.5mm. Cement used is of 43 grade. The mix proportion used is M20. Three w/c ratios were used i.e. 0.45, 0.5, 0.6. The Concrete Slump values decreases with the addition of RHA. This means that a less workable (stiff) mix is obtained when RHA is used as a cement blender. More water is therefore required to make a workable mix. The increased fines in the concrete due to excess RHA is partly responsible for this increased demand of water. Water Absorption tests reveal that higher substitution amounts results in lower water absorption values which is due to RHA being finer than cement. The use of RHA considerably reduces the water absorption of concrete. Thus, concrete containing RHA can be effectively used in places where the concrete can come in contact with water or moisture.

2.2.2 Hardened Properties

Zhang et al. (1996) in their paper " **Rice husk ash blended cement: assessment of optimal level of replacement for strength and permeability properties of concrete**", this paper presents an experimental study on the effects of the incorporation of rice-husk ash (RHA) in cement concrete on the compressive strength of concrete is discussed, and the results are compared with those obtained with the control Portland cement concrete and concrete incorporating silica fume. Three types of mixes were made. The RHA and the control concrete had similar one-day strengths, but the RHA concrete had Somewhat higher strength than the control concrete thereafter up to 180 days. Compared with the silica fume concrete, the compressive strength of the RHA concrete was lower up to 28 days, but similar at 90 and 180 days.

Bui et al. (2004) in their paper " **Concrete Incorporating Rice-Husk Ash: Compressive Strength and Chloride-Ion Penetrability** " investigated the compressive strength of the mix when cement is replaced by RHA in gap graded concrete. Two kinds of an ordinary Portland cement were employed, i.e., PC30 and PC40. Twenty four concrete mixtures were made, 12 mixtures for each of the two kinds of PC. Three levels of the water to binder ratio were investigated, i.e., 0.30, 0.32 and 0.34.

The mixtures with water to binder ratio of 0.30 were made with a binder content of 550kg/m3 concrete. The binder content of all other mixtures was 500kg/m3. Rice husk ash was used to replace 10%, 15% and 20% by mass of PC. Cubes of 100mm size were cast and compacted in two layers on a vibrating table. Compressive strength of the concretes was determined at 1, 3, 7, 28 and 90 days, using three specimens per test. Compressive strength of RHA blended concrete is higher than those of the plain cement concrete, irrespective of water to binder ratio and age. Compressive strength increases with blending percentage at corresponding values of water to binder ratio and age. Higher contents of RHA can be used without a strength loss. This will, however, cause an increase in the super plasticizer dosage required. The RHA blending increases the relative strength at all ages, however, most pronounced is the increase in the first 7 days. Concretes made with the fine cement (PC40) revealed considerable higher strengths than those made with the coarser cement (PC30). Partial cement replacement by RHA also significantly improved strength, although the effect was pronounced only at later ages. The compressive strength values of RHA blended concrete to be lower than those of plain cement concretes at ages up to 3 days. However, at later age, say from 7 days onward, the blended concretes have higher compressive strength than those of the control concretes.

Khani et al. (2009) in their paper " **The Effect of Rice Husk Ash on Mechanical Properties and Durability of Sustainable Concretes**", investigated split tensile strength of the concrete incorporating rice husk ash of 7%, 10% and 15% by weight of cement. Two 150 X 300 mm cylindrical concrete specimen was prepared for the test. Concrete containing RHA has a greater splitting tensile strength than that the control concrete at all ages. It is clear that, as the amount of RHA increases, the tensile strength increases up to 20%. For instance, at 90 days the 15%RHA concrete had a tensile strength of 5.62 MPa compared with 4.58 MPa for the control concrete. Concrete cubes of 100×100×100 mm dimension were cast for compressive strength. RHA concrete had higher compressive strengths at various ages and up to 90 days when compared with the control concrete. The results show that it was possible to obtain a compressive strength of as high as 46.9 MPa after 28 days. In addition, strengths up to 63.2 MPa were obtained at 90days

Rukzon et al. (2010) in their paper **" Strength and Carbonation Model of Rice Husk Ash Cement Mortar with Different Fineness**", studied the compressive strength properties of mortar mix with the addition of rice husk ash. Three finenesses of rice husk ashes, viz., original rice husk ash RAO with 70% retained on sieve number 325, medium rice husk ash RA1 with 15–20% by weight retained on a sieve No. 325, and fine rice husk ash RA2 with 1–3% by weight retained on a sieve No. 325. Ordinary Portland cement was partially replaced with RAO, RA1, and RA2 with 0 to 40 % . The compressive strengths were tested at the age of 3, 7, 28, 60, 90, and 180 days. The compressive strength of mortar containing RAO (20RAO and 40RAO) was lower than that of the control mortar (OPC) at all ages. The compressive strength of mortar was reduced with an increase in dosage of rice husk ash. The use of RA1 resulted in an increase in the strength as compare to RAO at all ages. For RA2, the results showed that higher compressive strength was developed due to the high fineness of RA2 and a decrease in the water requirement. the replacement of portland cement Type I with rice husk ash decreases the compressive strength . An increase of rice husk ash fineness increases the strength.

Habeeb et al. (2010) in their paper **" Study on Properties of Rice Husk Ash and Its Use as Cement Replacement Material** " investigates the compressive strength of rice husk ash (RHA) produced by using a Ferro-cement furnace. Mixture proportioning was performed to produce high workability concrete (200 – 240 mm slump) with target strength of 40 MPa for the control mixture. A total of 13 concrete mixtures were casted to study compressive strength of RHAC and results were seen for 1, 3, 7 and 28 days. F1, F2, F3 represents the grinding time that is 180 min, 270 min and 360 min respectively . The results showed that at early ages the strength was still comparable, while at the age of 28 days, finer RHA concrete exhibited higher strength than the concrete with coarser RHA. The 5% replacement level achieved slightly lower values of compressive strength at early ages for up to 7 days except for the 05% replaced F3 mixture where the compressive strength was higher due to the increased reactivity and the filler effect of RHA. Based on that, it can be noticed that the amount of RHA present when 5% replacement used is not adequate to enhance the strength significantly. The strength increased with RHA for up to 10% which resulted in achieving the maximum value. When 20% of OPC was replaced for RHA, the strength of concrete achieved equivalent values to the OPC control mixture.

Givi et al. (2010) in their paper " **Assessment of the effects of rice husk ash particle size on strength, water permeability and workability of binary blended concrete**." studies the compressive strength of concrete by partial replacement of cement with agro-waste rice husk ash. Two types of rice husk ash with average particle size of 5 micron (ultra fine particles) and 95 micron. A total of three series of mixtures were prepared in the laboratory trials. The control mixtures were made of natural aggregates, cement and water. RHA with average particle sizes of 95 and 5 μm (ultra fine particles), the mixtures were prepared by replacing 5%, 10%, 15% and 20% of cement with RHA. Cubes of 100 mm edge were cast and compacted in two layers on a vibrating table. The results show that the RHA-blended concrete had higher compressive strength at 90 days in comparison with that of the concrete without RHA, although at 7 and 28 days different behaviors were observed between the concretes with the two RHA considered. It is found that the cement could be advantageously replaced by RHA up to maximum limit of 15% and 20% with average particle sizes of 95 and 5 μm, respectively. Although, the optimal level of RHA content for both average particle sizes were achieved with 10% replacement.

Uduweriya et al. (2010) in their paper " **Investigation of Compressive Strength of Concrete Containing Rice-Husk-Ash** " investigated the results of three different replacement percentages of RHA in concrete (10%, 20% and 30% by mass of cement) were compared with the concrete that does not contain RHA. Concrete cylinders of 150 mm diameter and 300 mm height were casted by using concrete with W/C ratio of 0.75. Mechanical vibrator was used to compact the concrete during casting. The average value of tensile strength was obtained by testing of three specimens. There is significant increment in the tensile strength in concrete containing RHA. The maximum tensile strength is resulted with 20% replacement. Therefore tendency of cracking of concrete containing RHA can be considered as low compared to the normal concrete.

Kishore et al. (2011) in their paper **" Study on Strength Characteristics of High Strength Rice Husk Ash Concrete** " has investigated the compressive strength, splitting tensile and flexural strength of high strength concrete with different replacement levels of ordinary Portland cement by Rice Husk Ash. . The standard cubes (150mmX150mmX150mm) and the standard cylinders (150mm diaX300mm height) were caste. In all specimens with M40 and M50 grade mix cases were caste and tested. The strength effect of High-strength concrete of various amounts of replacement of cement viz., 0%, 5%, 10%, 15% with Rice Husk Ash of both the grades were compared with that of the high-strength concrete without Rice Husk Ash. As the replacement level increases there is decrease in splitting tensile strength at 28 days age of curing for both M40 and M50 grades of concrete by 5 to 10%. The splitting tensile strength for both M40 and M50 grade of concrete was 3.98MPa and 4.19MPa respectively at 15% replacement.

Table 2.1 Compressive Strength of M50 Grade Rice Husk Ash Concrete. (Kishore et al., 2011)

Rice husk ash	**Compressive strength (MPa) of M50**		
	7 days	**28 days**	**90 days**
0	48.31	59.37	62.50
5	42.00	56.40	58.36
10	38.40	53.43	56.40
15	37.37	50.46	52.50

It shows that the splitting tensile strength at 15% replacement decreased by 5.1% for M40 grade of concrete and 9.1% for M50 grade of concrete, when compared with that of the conventional concrete. Maximum value is obtained by M50 grade concrete with 5% replacement. The compressive strength at 7, 28 and 56 days have been obtained. Replacement of cement with Rice Husk Ash leads to decrease in the compressive strength for both M40 and M50 mixes. He observed that for both grade of concretes the flexural strengths were decreased at 15% replacement of rice husk ash with cement, but obtained target strength at 10% replacement.

Khan et al. (2012) in their paper **" Reduction in environmental problems using rice-husk ash in concrete** " studied the flexural strength of RHAC. Test was carried out on RHAC concrete beams containing 0%, 25%, 30% and 40% of RHA as a replacement of OPC All the concrete beams were cast in 1:2:4 mixture design ratio of Cement: sand: coarse aggregate, respectively. A midpoint loading was applied. The load taken by pure OPC concrete beams was greater than the RHAC concrete beams both at first crack development and failure. The deflection at mid span decreased with the increase of RHA content in RHAC concrete beams both at first crack development and at failure. For 25% RHAC concrete beam the mid span deflection at failure load is 32 mm as compared to 38 mm for OPC concrete beam. For 25% OPC replacement with RHA, the beam performed very well in flexure. The failure load for 25% RHAC is 51 KN which is quite close to 54 KN for OPC concrete beam.

Tashima et al. (2012) in their paper " **The Possibility Of Adding The Rice Husk Ash (RHA) To The Concrete** " evaluates how different contents of rice husk ash (RHA) added to concrete may influence its physical and mechanical properties like splitting tensile strength. Samples with dimensions of 10 X 20 cm were tested, with 5% e 10% of RHA, replacing in mass the cement. Three mixtures were made i.e. Mixture D (controlled mix) , Mixture E (5% replacement) , Mixture F (10% replacement). All the replacement degrees of RHA researched, achieve similar results in splitting tensile strength. According to the results, may be realized that there is no interference of adding RHA in the splitting tensile strength. There is slight increase in the split tensile strength if compared to controlled mix. The addition of RHA causes an increment in the compressive strength due to the capacity of the pozzolan, of fixing the calcium hydroxide, generated during the reactions of hydrate of cement. All the replacement degrees of RHA increased the compressive strength. For a 5% of RHA, 25% of increment is verified when compared with mixture D.

CHAPTER-3

MATERIAL AND DESIGN METHODOLOGY

3.1 GENERAL

The present chapter deals with the presentation of results obtained from various tests conducted on materials used for developing concrete. In order to achieve the objectives of present study, an experimental program was planned to investigate the effect of Fly Ash, Rice Husk Ash on Compressive Strength, and Flexural Strength of pavement quality concrete.

3.2 MATERIALS

The properties of material used for making concrete mix are determined in laboratory as per relevant code of practice. Different materials used in present study were cement, coarse aggregates, fine aggregates, fly ash and rice husk ash. Description of various materials which were used in this study is given below:

3.2.1 Cement

Although all materials that go into concrete mix are essential, cement is very often the most important because it is usually the delicate link in the chain. The function of cement is first of all to bind the sand and stone together and second to fill up the voids in between sand and stone particles to form a compact mass. It constitutes only about 20 percent of the total volume of concrete mix; it is the active portion of binding medium and is the only scientifically controlled ingredient of concrete. Any variation in its quantity affects the compressive strength of the concrete mix. Portland cement referred as (Ordinary Portland Cement) is the most important type of cement and is a fine powder produced by grinding Portland cement clinker. The OPC is classified into three grades, namely 33 Grade, 43 Grade, 53 Grade depending upon the strength of 28 days. It has been possible to upgrade the qualities of cement by using high quality limestone, modern equipments, maintaining better particle size distribution, finer grinding and better packing. Generally use of high grade cement offers many advantages for making stronger concrete. Although they are little costlier than low grade cement, they offer 10-20% saving in cement consumption and also they offer many hidden benefits. One of the most important benefits is the faster rate of development of strength.

Ordinary Portland Cement (OPC) of 43 Grade from a single lot was used throughout the course of the investigation. It was fresh and without any lumps. The physical properties of the cement as determined from various tests conforming to Indian Standard IS: 8112:1989 are listed in Table 3.1.

Cement was carefully stored to prevent deterioration in its properties due to contact with the moisture.

Table 3.1 Properties of OPC 43 Grade Concrete

Sr.No.	Characteristics	Values Obtained Experimentally	Values Specified By IS 8112:1989
1.	Specific Gravity	3.10	-
2.	Standard Consistency	27%	-
3.	Initial Setting Time, minutes	149	30 minutes (minimum)
4.	Final Setting Time, minutes	257	600 minutes (maximum)
5.	Compressive Strength 3 days 7 days 28 days	 27.8 N/mm^2 36.5 N/mm^2 48.6 N/mm^2	 23 N/mm^2 33 N/mm^2 43 N/mm^2

3.2.2 Coarse Aggregates

Aggregates constitute the bulk of a concrete mixture and give dimensional stability to concrete. To increase the density of resulting mix, the aggregates are frequently used in two or more sizes. The most important function of the fine aggregate is to assist in producing workability and uniformity in mixture. The fine aggregate assist the cement paste to hold the coarse aggregate particles in suspension. This action promotes plasticity in the mixture and prevents the possible segregation of paste and coarse aggregate, particularly when it is necessary to transport the concrete some distance from the mixing plant to placement. The aggregates provide about 75% of the body of the concrete and hence its influence is extremely important. They should therefore meet certain requirements if the concrete is to be workable, strong, durable and economical. The aggregates must be proper shape, clean, hard, strong and well graded.

The coarse aggregate used were a mixture of two locally available crushed stone of 20 mm and 10 mm size in 70:30 proportion. The aggregates were washed to remove dirt, dust and then dried to surface dry condition.

Physical properties of coarse aggregates are given in Table 3.2. The sieve analysis of coarse aggregate was done. Table 3.3 & Table 3.4 show the result of sieve analysis. Proportioning of coarse aggregates was done and fineness modulus was obtained.

Table 3.2 Physical properties of Coarse Aggregates (20mm and 10mm)

Characteristics	Value
Colour	Grey
Shape	Angular
Maximum Size	20 mm/10mm
Specific Gravity	2.73/2.72
Water Absorption	0.20%/0.35%

Table 3.3 Sieve Analysis of Coarse Aggregate (20 mm)

Weight of sample taken = **3000gm**					
Sr. No	**IS-Sieve(mm)**	**Wt. Retained (gm)**	**%age Wt. Retained**	**%age passing**	**Cumulative % retained**
1	100	0.00	0.00	100.00	0.00
2	80	0.00	0.00	100.00	0.00
3	40	0.00	0.00	100.00	0.00
4	20	53.00	1.77	98.23	1.77
5	10	2938.50	97.95	0.28	99.72
6	4.75	5.50	0.18	0.10	99.90
7	PAN	3.00	0.10	0.00	
	Total	3000.00	**SUM = 500 + 201.38**		701.38
				FM =7.01	

Table 3.4 Sieve Analysis of Coarse Aggregate (10 mm)

Weight of sample taken = 3000gm.					
Sr. No	IS-Sieve(mm)	Wt. Retained (gm)	%age Wt. Retained	%age passing	Cumulative % retained
1	100	0.00	0.00	100.00	0.00
2	80	0.00	0.00	100.00	0.00
3	40	0.00	0.00	100.00	0.00
4	20	0.00	0.00	100.00	0.00
5	10	2012.00	67.07	32.93	67.07
6	4.75	958.00	31.93	1.00	99.00
7	PAN	30.00	1.00	0.00	
	Total	3000.00	**SUM = 500+166.07**		666.07
				FM =6.66	

3.2.3 Fine Aggregates

The aggregates most of which pass through 4.75 mm IS sieve are termed as fine aggregates. The fine aggregate may be of following types:

i) Natural sand, i.e. fine aggregate resulting from natural disintegration of rocks.

ii) Crushed stone sand, i.e. fine aggregate produced by crushing hard stone.

iii) Crushed gravel sand, i.e. fine aggregate produced by crushing natural gravel.

According to size, the fine aggregate may be described as coarse, medium and fine sands. Depending upon the particle size distribution IS: 383-1970 has divided the fine aggregate into four grading zones (Grade I to IV). The grading zones become progressively finer from grading zone I to IV.

In this experimental program**,** fine aggregates (stone dust) were collected from Jhelum Stone Crusher, Mirthal, Pathankot and conforming to grading zone II. It was coarse sand light grey in colour. The sand was sieved through 4.75 mm sieve to remove particles greater than 4.75 mm size. Physical properties and sieve analysis of fine aggregate are tested as per Indian Standards and results are shown in Table 3.5 and Table 3.6 respectively.

Specific gravity of fine aggregates was experimentally determined as 2.49. Sieve analysis of fine aggregates was performed to get Fineness Modulus.

Table 3.5 Physical Properties of fine aggregates

Characteristics	Value
Specific gravity	2.49
Bulk density,	1.3
Fineness modulus	2.52
Water absorption, %	0.89
Zone	II

TABLE 3.6 Sieve Analysis of Fine Aggregate

Weight of sample taken =1000 gm.					
Sr. No	**IS-Sieve**	**Wt. Retained (gm)**	**%age Wt. Retained**	**%age passing**	**Cumulative % retained**
1	4.75 mm	6	0.6	99.4	0.6
2	2.36 mm	59	5.9	93.5	6.5
3	1.18 mm	220	22	71.5	28.5
4	600 µ	159	15.9	55.6	44.4
5	300 µ	316.5	31.65	23.95	76.05
6	150 µ	196.5	19.65	4.3	95.70
7	PAN	43	4.3	0	
	Total	**1000.00**		**SUM**	**251.75**
				FM =	**2.52**

3.2.4 Fly Ash

Flyash used in the study was obtained from Guru Nanak Dev Thermal Power Plant, Bathinda. Its physical and chemical properties are given in Table 3.7 and Table 3.8 respectively.

Table 3.7 Physical properties of fly ash

Physical property	value
Colour	Whitish gray
Bulk density	1120 kg/m^3
Specific gravity	2.10
Fineness	2840 cm^2/gm

Table 3.8 Chemical properties of fly ash

Constituent	Component in %
Silica (SiO_2)	46.8
Alumina (Al_2O_3)	23.7
Ferric Oxide (Fe_2O_3)	13.2
Calcium Oxide (CaO)	1.2
Magnesia (MgO)	1.0
Loss on Ignition (LOI)	6.9

3.2.5 RICE HUSK ASH

Completely burnt rice husk ash was brought from rice mills from Rajpura. Its physical and chemical properties are given in Table 3.9 and Table 3.10 respectively.

Table 3.9 Physical properties of rice husk ash

Physical property	Value
Colour	gray with slight black
Bulk density	104.9 kg/m^3
Specific gravity	1.96
fineness	2775 cm^2/gm
Avg particle size	150.47µm
mesopores	78%
Heating value	9.68 MJ/kg

Table 3.10 Chemical properties of rice husk ash

Component	%
Silica	92.1
Alumina	0.51
Iron oxide	0.40
Calcium oxide	0.55
Potassium oxide	1.53
Titanium di oxide	0.02
Manganese oxide	0.08
Phosphorous penta oxide	0.08
Sulphur tri oxide	0.12

3.2.6 SUPERPLASTISIZER

The superplasticizer "GLENIUM™ B233" procured from SIKA India Pvt. Limited was used in present study. The technical data provided by manufacturer is given in Table 3.11

Table 3.11 Properties of Superplasticizer

Sr. No.	Characteristics	Value
1.	Type	Poly carboxylic ether (PCE)
2.	Form	Liquid
3.	Colour	Light Brown
4.	Specific Gravity	1.09
5.	Relative density	1.09 ± 0.01 at 25^0C
6.	PH Content	> 6
7.	Setting Time	There may be mild extension of initial or final set

The dosage of superplasticizer recommended is 0.6% to 2% by weight of cementitous material. 1% superplasticizer by weight of cementitious material was selected in this study to get the medium range of workability.

3.3 MIX DESIGN OF PAVEMENT QUALITY CONCRETE (PQC)

Step 1: As per clause **602** of MORT&H Specification

Cement – 43 grade OPC as per **IS 8112** as per **602.2.2**

Coarse aggregate – 20 mm and 10 mm as per **602.2.4**

Los angles Abrasion value not greater than 35%

Impact value not greater than 30%

Fine aggregate – Natural sand as per **IS 383**

Admixture – Conplast AEA (if required)

Air entrained concrete 5% maximum (optional)

Step 2: Design Parameter:

1. Characteristics flexural strength required at 28 days = 4.5 N/mm^2
2. Maximum water cement ratio = 0.40 as per clause **602.3.3.1**
3. Maximum size of coarse aggregate = 25 mm
4. Degree of quality control = Good

5. Minimum cement content = 350 kg/m^3 as per clause **602.3.2**
6. Maximum cement content = 425 kg/m^3 as per clause **602.3.2**

Step 3: Calculation of aggregate content:

After determining the weight per cubic meter of cement, water, coarse aggregate and percentage of air content, the fine aggregate is calculated so as to produce one cubic meter of concrete using absolute volume method. On converting the weight per cubic meter into volume, we have

(a) Volume of cement = $\frac{Weight\ of\ cement}{Specific\ gravity\ of\ cement\ X\ 1000}$

(b) Volume of coarse aggregate = $\frac{Weight\ of\ coarse\ aggregate}{Specific\ gravity\ of\ coarse\ aggregate\ X\ 1000}$

(c) Volume of water = $\frac{Weight\ of\ water}{1000}$

(d) Volume of fine aggregate = 1 – {Volume of cement + coarse aggregate + water + Air content}

(e) Weight of fine aggregate = Volume of fine aggregate × specific gravity ×1000

Now by following the above steps for mix design, the mix proportion for different compressive strength are given by using following data:

Specific gravity of cement	=	3.10
Specific gravity of fine aggregate	=	2.49
Specific gravity of coarse aggregate, 20mm	=	2.73
Specific gravity of coarse aggregate, 10mm	=	2.72
Slump value	=	50 to 70 mm

TABLE 3.12 Mix Design of Pavement Quality Concrete

Mean Target Flexural strength (MPa)	Max. Size of Aggregate, (mm)	Mix proportions (C : FA : CA-I : CA-II)	W/C Ratio	Materials for 1 m^3 in kg				
				Water	Cement	F.A	C.A-I (20mm)	C.A-II (10mm)
4.5	20	1 : 1.828 : 1.936 : 0.83	0.4	156	390	713	755.30	323.70
5	20	1 : 1.815 : 1.921 : 0.82	0.35	140	400	726	768.60	329.40
5.5	20	1 : 1.805 : 1.908 : 0.82	0.3	123	410	740	782.60	335.40

CHAPTER-4
RESULTS AND DISCUSSIONS

4.1 GENERAL

The present study was undertaken to investigate the compressive strength and flexural strength of concrete with different levels of replacement of cement with fly ash and rice husk ash in concrete mix. Cement was partially replaced by fly ash at three different levels of replacement i.e. 10%, 20% and 30% and same with rice husk ash. Concrete mixtures were also cast with combined replacements of flyash and rice husk ash. Tests were performed after 7 and 28 days of curing of concrete. Cubes and beams were prepared for determining compressive strength and flexural strength of concrete with different water-cement ratio as 0.30, 0.35 and 0.40 for minimum required flexural strengths of 5.5 N/mm^2 5 N/mm^2 4.5N/mm^2, respectively. Super-plasticizer was used in all the mixes at 1% level by weight of cementitious material.

4.2 COMPRESSIVE STRENGTH

4.2.1 General

In most structural applications, concrete is employed primarily to resist compressive stresses. When a plain concrete member is subjected to compression, the failure of the member takes place, in its vertical plane along the diagonal. The vertical crack occurs due to lateral tensile strains. A flow in the concrete, which is in the form of micro crack along the vertical axis of the member will take place on the application of axial compression load and propagate further due to the lateral tensile strains.

4.2.2 Test Procedure and Results

Test specimens of size 150mm x 150mm x 150mm were prepared for testing the compressive strength. In this study, the mix was done manually. The cement and fine aggregate were first mixed dry to uniform colour and then coarse aggregate was added and mixed with the mixture of cement and fine aggregates. Water was then added and the whole mass mixed. The interior surface of the moulds and the base plate were highly oiled before concrete was placed. After this the specimens were removed from the moulds and placed in clean fresh water at for 28 days curing. For testing in compression, no cushioning material was placed between the specimen and the plates of the machine. The load was applied axially without shock till the specimen was crushed. Test results of compressive strength test at the age of 28 days are given in the Table 4.1. Table 4.2 and Table 4.3.

Table 4.1 Compressive strength of 4.5 MPa flexure design (w/c = 0.4)

w/c = 0.4	7days			28days		
	Load (KN)	***Average (KN)***	***f_c (MPa)***	***Load (KN)***	***Average (KN)***	***f_c (MPa)***
Controlled **(FR00)**	863.7 889.9 825.6	859.73	38.21	1170 1246 1027	1147.66	51.01
10% F.A. **(FR10)**	809.9 861.4 873.6	848.33	37.70	1132 1075 1097	1101.33	48.94
20% F.A. **(FR20)**	791.7 729.2 799.1	773.333	34.3703	1034 1058 956.3	1016.1	45.16
30% F.A. **(FR30)**	638.1 660.8 672.3	657.066	29.2029	926.5 931.9 950	936.133	41.6059
10% R.H.A. **(FR01)**	496.3 541.6 488.1	508.67	22.60	731.8 713 758.3	734.36	32.63
20% R.H.A **(FR02)**	477.8 432.8 411.2	440.6	19.5822	621.2 655.9 661.3	646.133	28.7170
30% R.H.A. **(FR03)**	347.7 371.4 384.6	367.9	16.3511	560.1 549.7 617.9	575.9	25.5955
10% F.A 10%R.H.A. **(FR11)**	468.7 450.9 499.1	472.9	21.0177	689.6 765.7 728.5	727.933	32.3525
20% F.A. 10% R.H.A **(FR21)**	357.2 381.3 371.1	369.87	16.43	563.8 657.3 639.8	620.3	27.5688
10% F.A 20% R.H.A. **(FR12)**	343.6 358.1 376.3	359.33	15.97	596.2 572.3 579.2	582.56	25.89

Table 4.2 Compressive strength of 5.0 MPa flexure design (W/C = 0.35)

W/C=0.35	7days			28days		
	Load (KN)	***Average (KN)***	***f_c (MPa)***	***Load (KN)***	***Average (KN)***	***f_c (MPa)***
Controlled **(FROO)**	907.2 980.5 873.8	920.5	40.911	1246 1289 1304	1279.66	56.874
10% F.A. **(FR10)**	822.5 851.9 878.4	850.933	37.819	1058 1221 1081	1120	49.78
20% F.A. **(FR20)**	760.7 788.9 825.4	791.666	35.185	1056 1108 1086	1083.33	48.148
30% F.A. **(FR30)**	767.4 730.2 705.8	734.466	32.642	1051 976.7 862.2	963.3	42.813
10% R.H.A. **(FR01)**	563.1 552.1 516.4	543.86	24.17	737.1 762.8 775.8	758.56	33.71
20% R.H.A. **(FR02)**	468.4 482.8 509.3	486.833	21.637	687.3 699.3 751.3	712.633	31.672
30% R.H.A. **(FR03)**	450.3 469.2 387.1	435.533	19.357	630.1 610.4 572	604.166	26.851
10% F.A 10%R.H.A. **(FR11)**	521.3 511.6 580.4	537.766	23.900	801 813.6 745.3	786.633	34.961
20% F.A. 10% R.H.A **(FR21)**	456.2 453 501.2	470.133	20.8948	754.2 699.3 721	724.833	32.214
10% F.A 20% R.H.A. **(FR12)**	387.3 403.1 443.3	411.233	18.277	696.1 686.3 674.1	685.5	30.47

Table 4.3 Compressive strength OF 5.5 MPa flexure design (W/C = 0.3)

W/C = 0.3	7days			28days		
	Load (KN)	*Average (KN)*	*f_c (MPa)*	*Load (KN)*	*Average (KN)*	*f_c (MPa)*
Controlled **(FR00)**	1053 1069 1021	1047.66	46.5629	1380 1416 1368	1388	61.6888
10% F.A. **(FR10)**	963.1 988.2 912	954.433	42.4192	1349 1037 1212	1199.33	53.3037
20% F.A. **(FR20)**	1027 1009 963.4	999.8	44.4355	1208 1143 1230	1193.66	53.0518
30% F.A. **(FR30)**	748.1 766.5 700.1	738.233	32.8103	1017 967.2 1135	1039.73	46.2103
10% R.H.A. **(FR01)**	644.4 569.3 591.3	601.67	26.74	1024 986.4 796.6	995.67	44.25
20% R.H.A. **(FR02)**	542.4 506 531.8	526.73	23.61	744.9 701.5 730.7	725.7	32.2533
30% R.H.A. **(FR03)**	513.5 476.8 450.3	480.2	21.3422	588.7 593.4 647.3	609.8	27.1022
10% F.A 10%R.H.A. **(FR11)**	664.9 700.3 693.3	686.166	30.4963	1021 1014 981.8	1005.6	44.6933
20% F.A. 10% R.H.A **(FR21)**	563.6 589.5 601.1	584.733	25.9881	873. 893. 909.	892.16	39.6518
10% F.A 20% R.H.A. **(FR12)**	511.1 503.7 487	500.6	22.2488	836.4 800.2 792.5	809.7	35.98

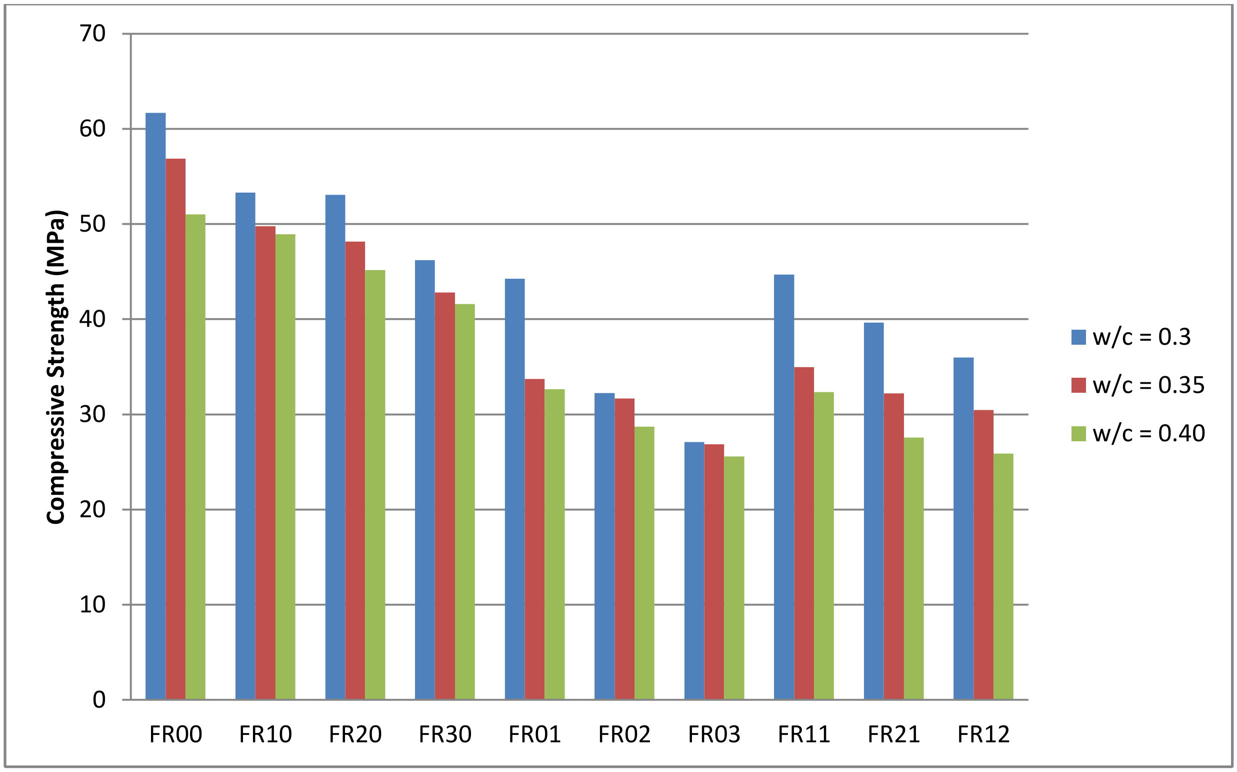

Fig 4.1 28-day compressive strengths for all water cement ratios

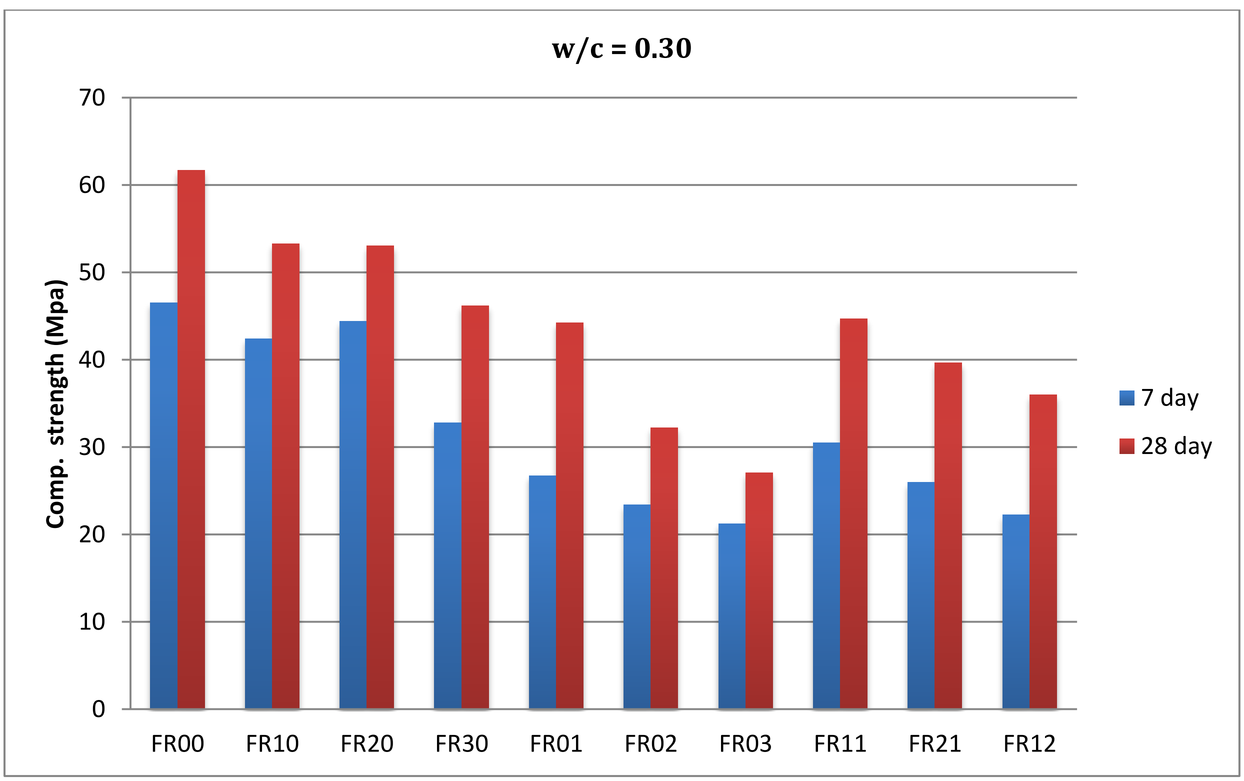

Fig 4.2 7-day and 28-day compressive strengths with w/c = 0.30

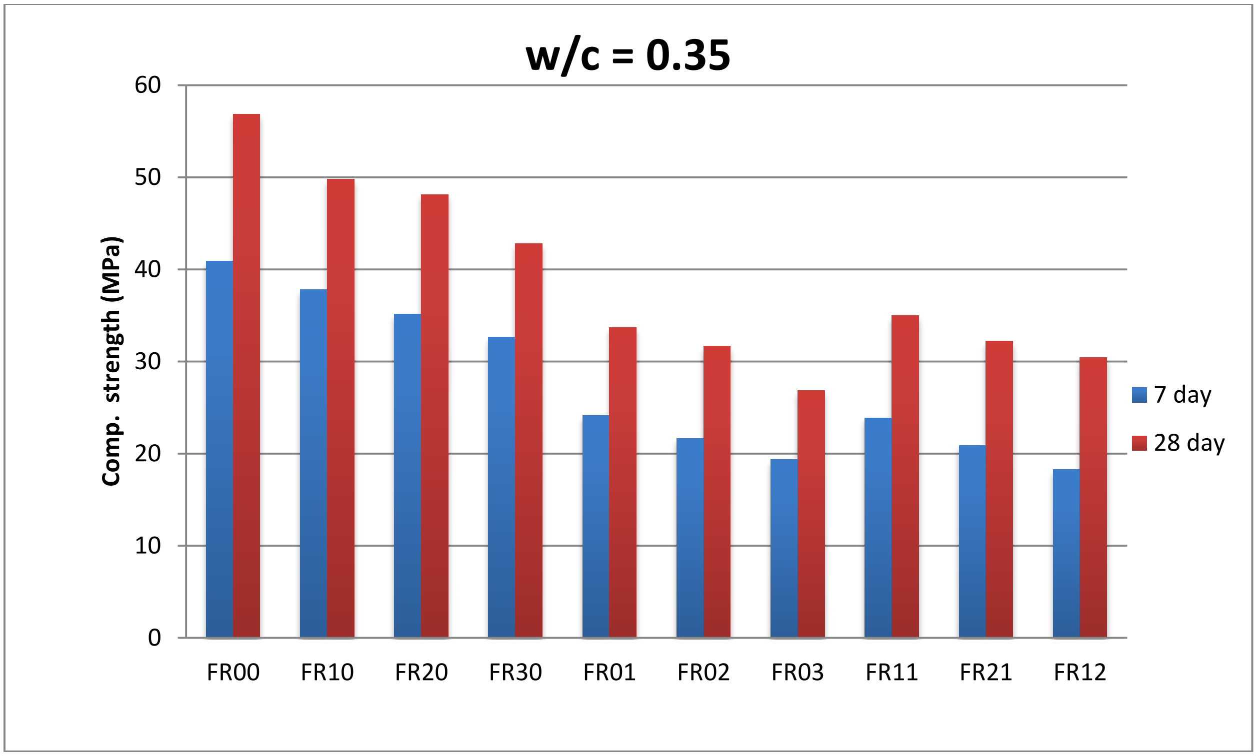

Fig 4.3 7-day and 28-day compressive strengths with w/c = 0.35

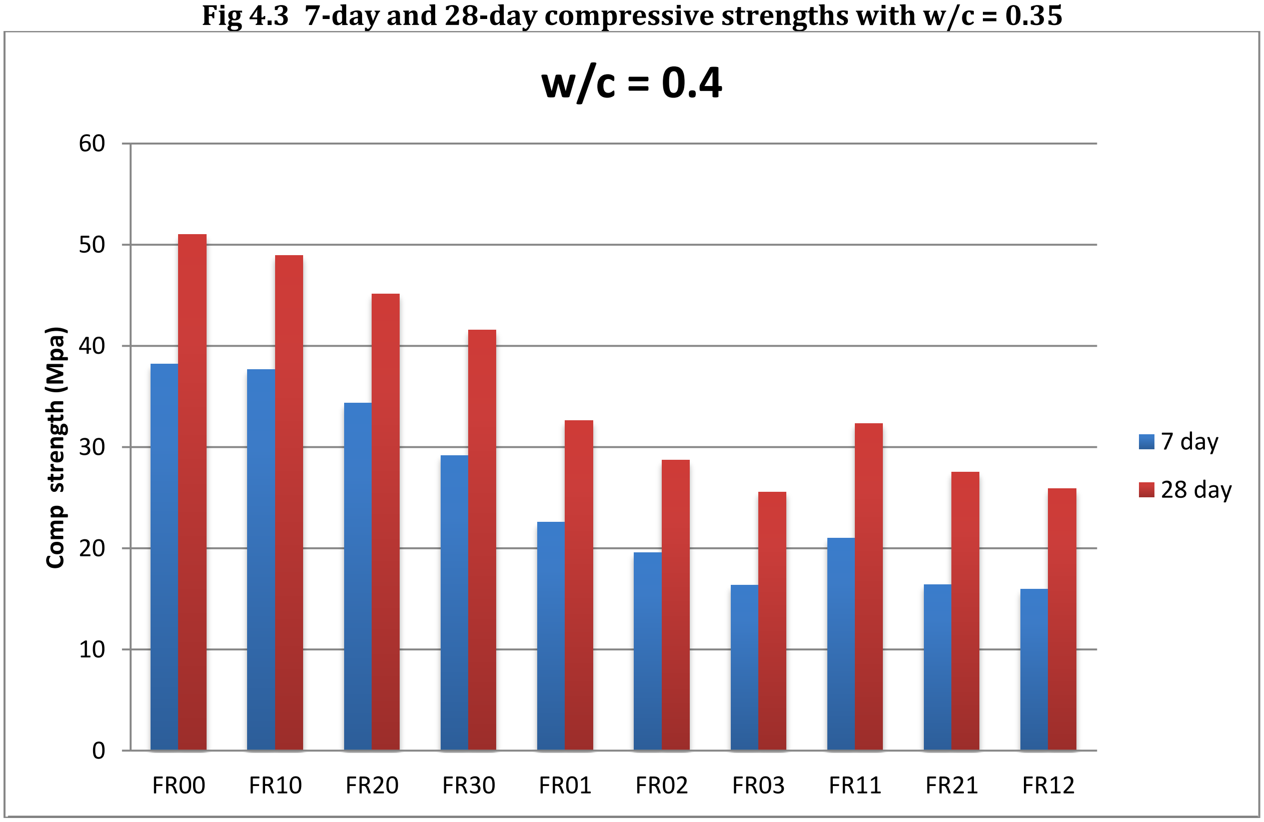

Fig 4.4 7-day and 28-day compressive strengths with w/c = 0.40

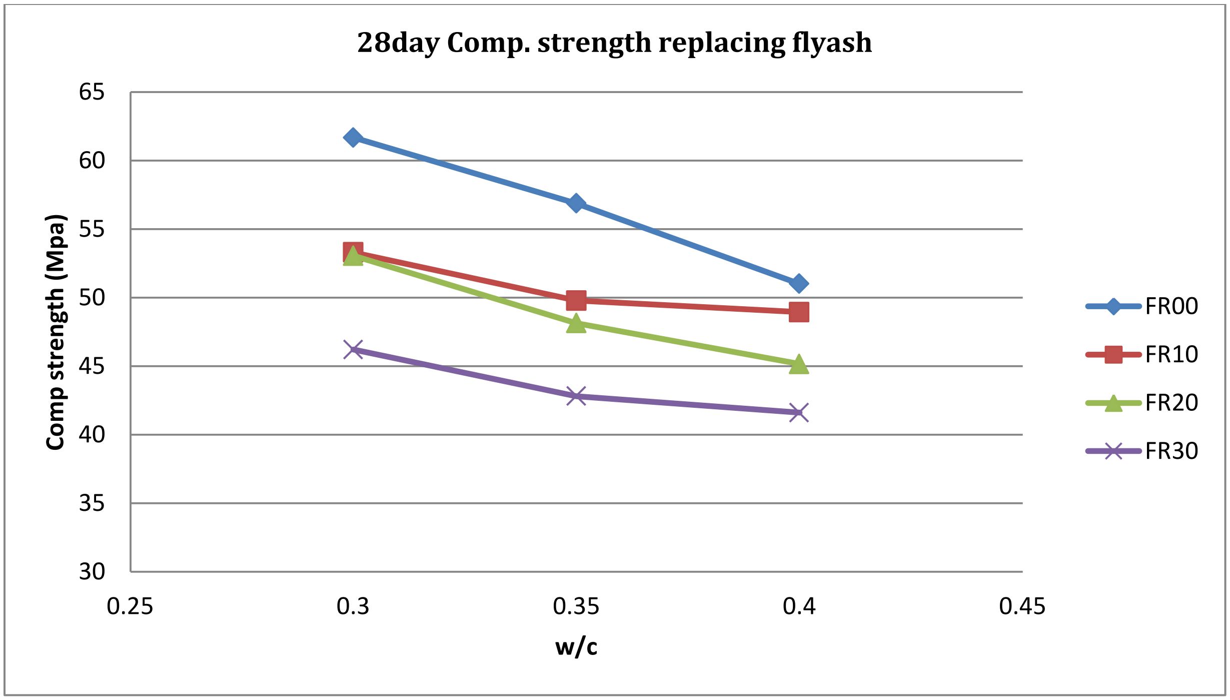

Fig 4.5 28-day compressive strength replacing fly ash

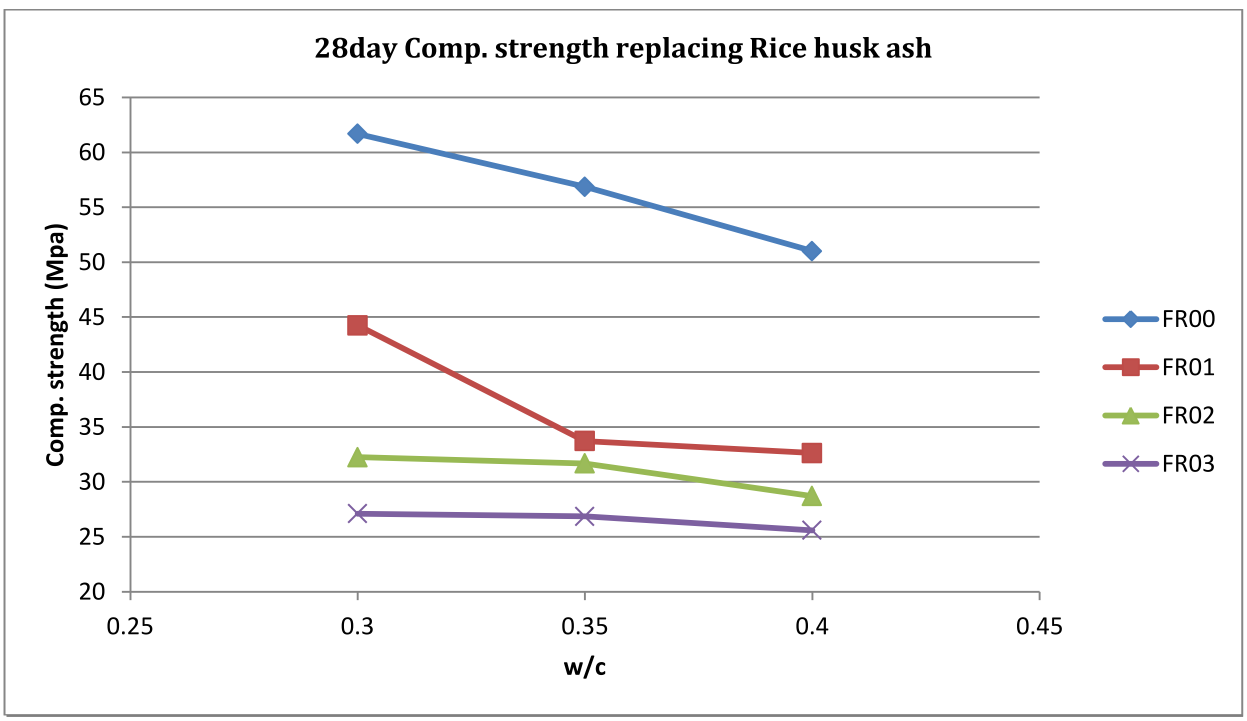

Fig 4.6 28-day compressive strength replacing rice husk ash

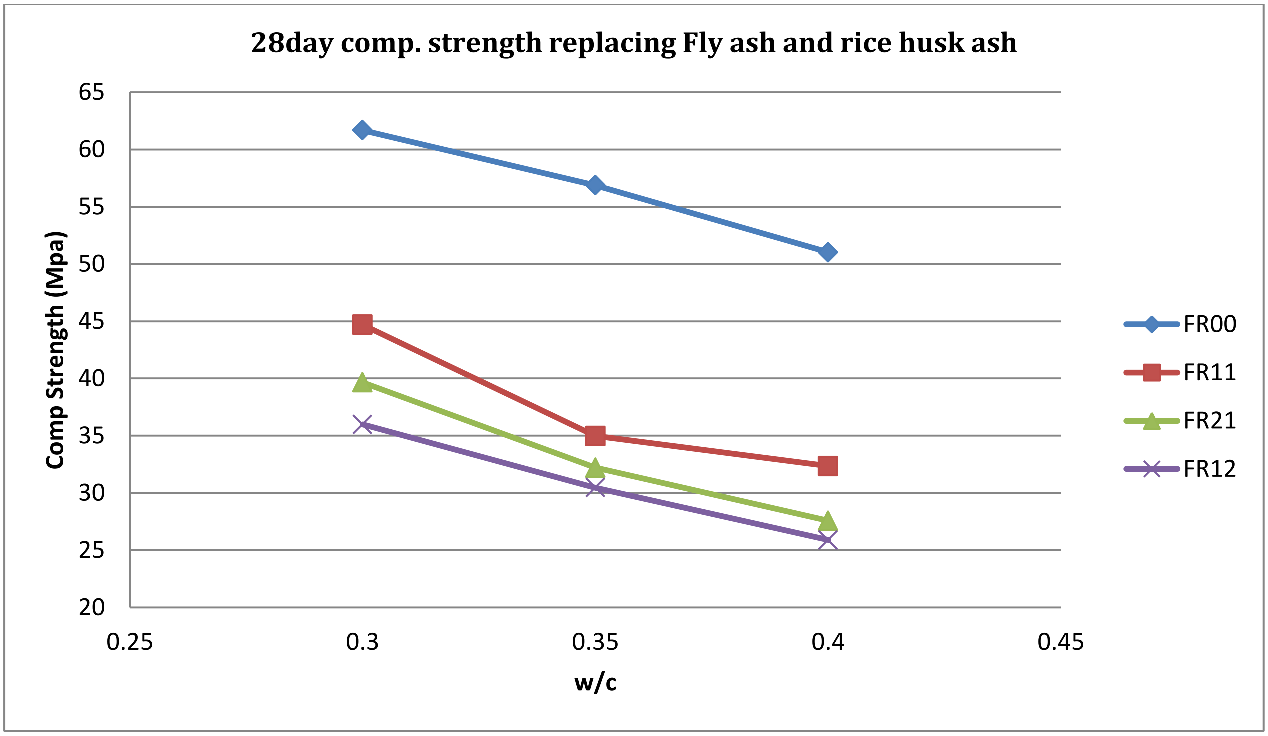

Fig 4.7 28-day compressive strength replacing fly ash and rice husk ash

4.3 FLEXURAL STRENGTH

4.3.1 General

The most common concrete structure subjected to flexure is a highway or airway pavement and strength of concrete for pavements is commonly evaluated by means of bending tests. When concrete is subjected to bending, then tensile and compressive stresses and in many cases direct shear stresses are developed.

4.3.2 Test Procedure and Results

Test specimens of beam size 150 mm × 150 mm × 700 mm were prepared for testing the flexural strength of unreinforced beams. The beam moulds containing the test specimens were placed in moist air for at least 90% relative humidity and a temperature of $27^0 \pm 2^0$ C for 24 hours $\pm$ 1/2 hour from the time of addition of water to the dry ingredients. After this the specimens were removed from the moulds and placed in clean fresh water at a temperature of $27^0 \pm 2^0$ C for 28 days curing. After 28 days of curing the specimens were tested in flexure on a Universal Testing Machine. Loads were applied at the one third points at a constant rate of 30 kg/minute. The distance between the centres of two rollers was kept 20 cm.

If the fracture occurred within the central one-third of the beam, the flexural strength was calculated on the basis of ordinary elastic theory using the following equations:

$F_b = \frac{PL}{BD^2}$, when 'a' is greater than 20 cm for 15 cm specimen

$F_b = \frac{3Pa}{BD^2}$, when 'a' is less than 20 cm but greater than 17 cm for 15 cm specimen

Where,

F_b = Flexural Strength of the specimen in N/mm^2

B = Width of the specimen (= 150 mm)

D = Depth of the specimen (= 150 mm)

L = Span of the specimen (= 700 mm)

P = Maximum load in Newton (N) applied to the specimen

A = Distance between the line of fracture and nearer support, measured on the centre line of the tensile side of the specimen in cm, shall be calculated to the nearest 0.5 kg/cm^2 .

If 'a' is less than 17 cm the results of a test shall be discarded. Test results of flexural test at the age of 28 days curing are given in table 4.4. The flexural strength results of concrete mix are also shown graphically.

Table 4.4 Flexural Strength(F.S.) of 5.5 MPa flexure design.(w/c =0.30)

W/C = 0.30	***Avg Load (KN)***	***F.S. (MPa)***	***F.S.(kg/cm^2)***
Controlled (FR00)	35.3475	6.284	62.84
10% FA (FR10)	34.0575	6.055	60.55
20% FA (FR20)	32.0775	5.703	57.03
30% FA (FR30)	29.67	5.275	52.75
10%RHA (FR01)	27.6525	4.916	49.16
20%RHA (FR02)	22.2275	3.952	39.52
30%RHA (FR03)	19.6075	3.486	34.86
10%FA,10%RHA (FR11)	22.5025	4.001	40.01
20%FA, 10%RHA (FR21)	21.0125	3.735	37.35
10%FA, 20%RHA (FR12)	19.70275	3.502	35.02

Table 4.5 Flexural Strength(F.S.) of 5.0 MPa flexure design. (w/c = 0.35)

W/C = 0.35	***Avg Load (KN)***	***F. S. (MPa)***	***F.S.(kg/cm²)***
Controlled (FR00)	32.8575	5.842	58.42
10% FA (FR10)	32.0825	5.704	57.04
20% FA (FR20)	30.7375	5.465	54.65
30% FA (FR30)	27.9325	4.966	49.66
10%RHA (FR01)	26.2825	4.673	46.73
20%RHA (FR02)	21.4175	3.808	38.08
30%RHA (FR03)	17.99	3.199	31.99
10%FA,10%RHA (FR11)	21.7625	3.868	38.68
20%FA, 10%RHA (FR21)	19.6675	3.496	34.96
10%FA, 20%RHA (FR12)	18.0875	3.215	32.15

Table 4.6 Flexural Strength(F.S.) of 4.5 MPa flexure design.(w/c = 0.40)

W/C = 0.40	**Avg Load (KN)**	**F.S. (Mpa)**	**F.S.(kg/cm²)**
Controlled (FR00)	29.4825	5.242	52.42
10% FA (FR10)	28.4525	5.059	50.59
20% FA (FR20)	27.955	4.977	49.77
30% FA (FR30)	25.045	4.452	44.52
10%RHA (FR01)	24.535	4.361	43.61
20%RHA (FR02)	18.9375	3.367	33.67
30%RHA (FR03)	16.29	2.896	28.96
10%FA,10%RHA (FR11)	19.3575	3.442	34.42
20%FA, 10%RHA (FR21)	18.8075	3.343	33.43
10%FA, 20%RHA (FR12)	16.38	2.912	29.12

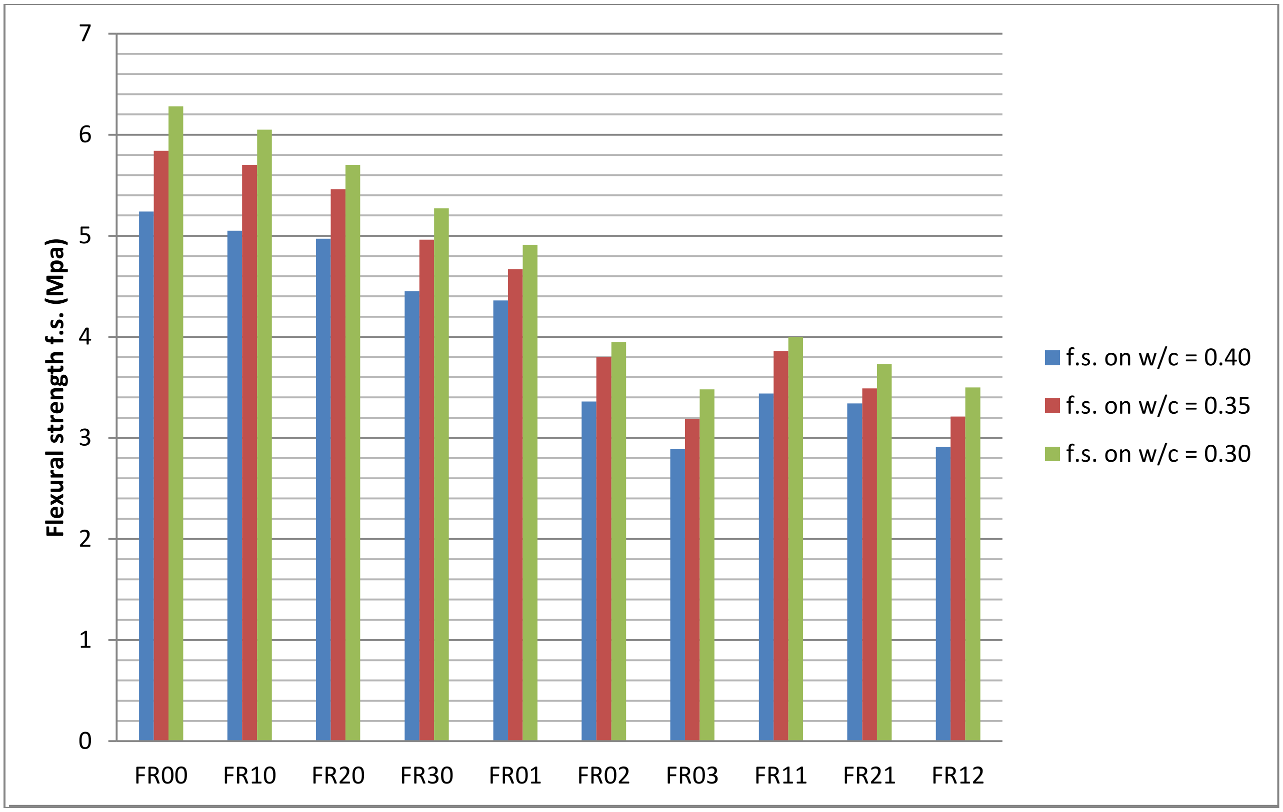

Fig 4.8 28-day flexural strength

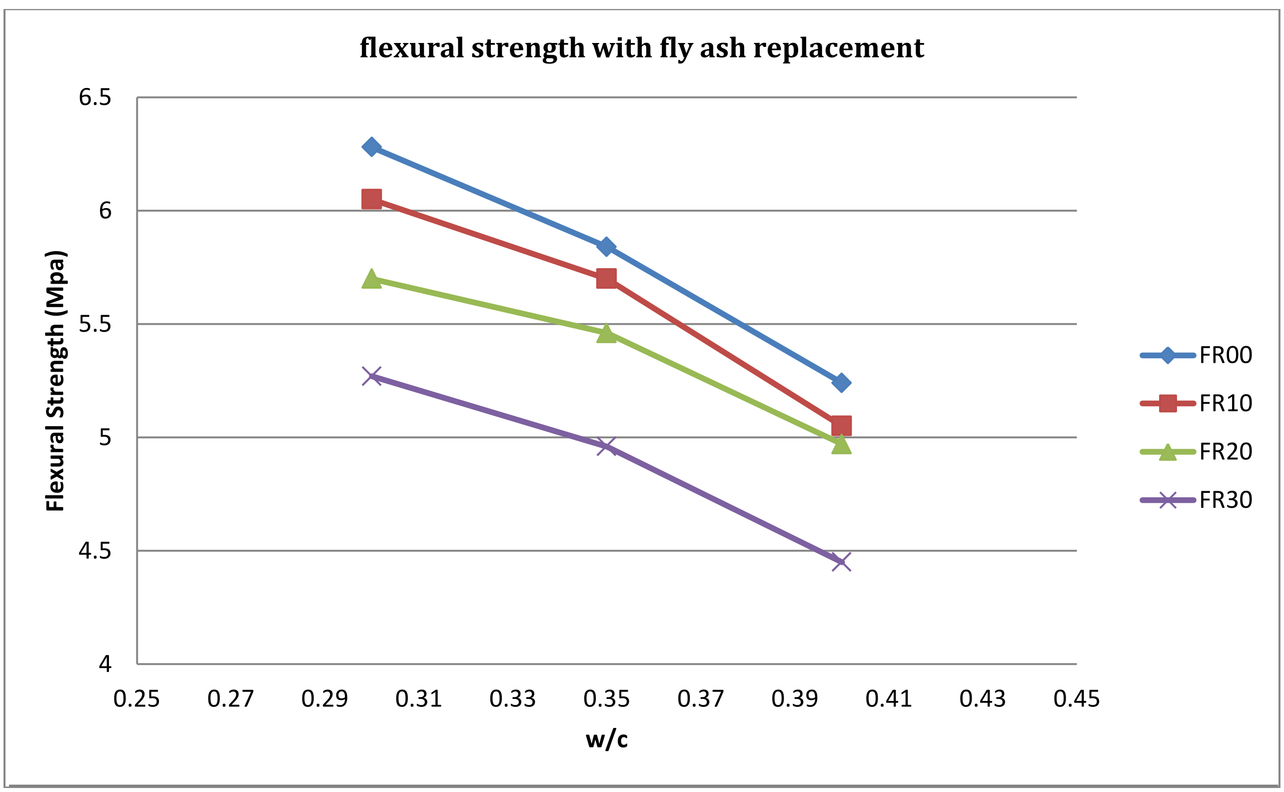

Fig 4.9 Flexural strength with only fly ash replacing cement.

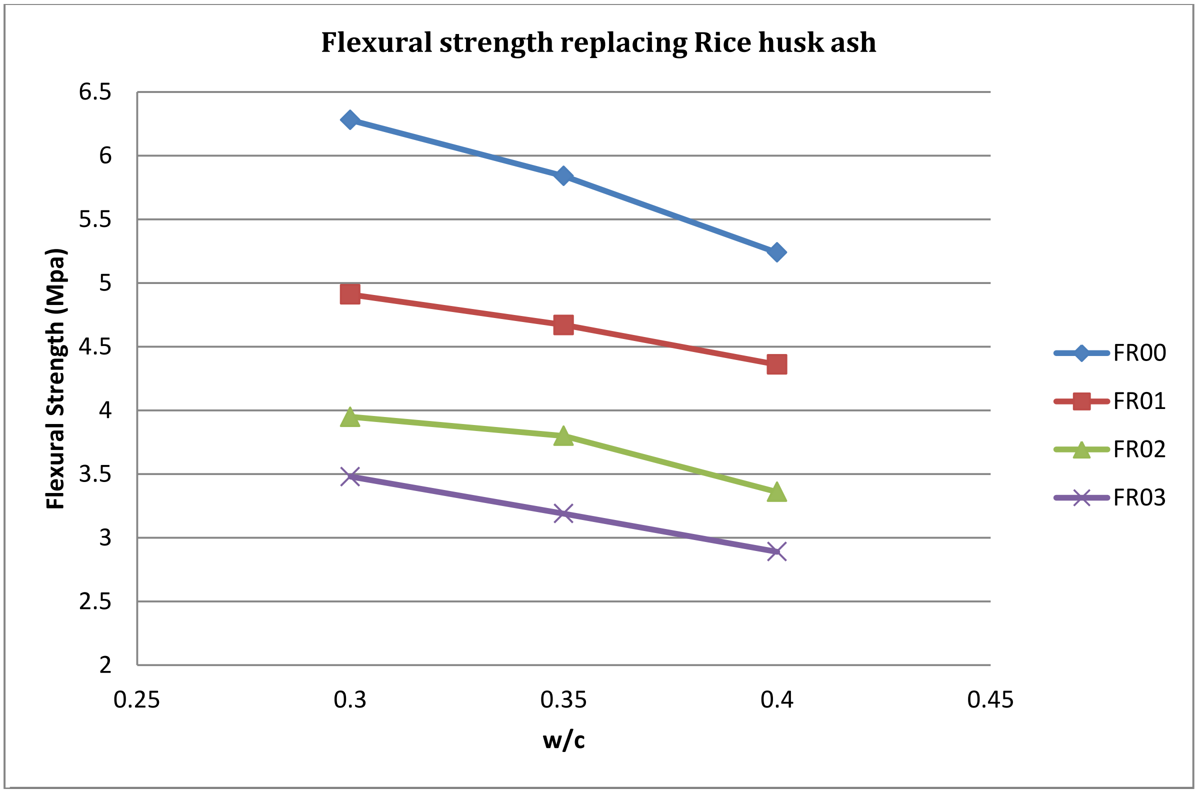

Fig 4.10 Flexural strength with rice husk ash replacing cement

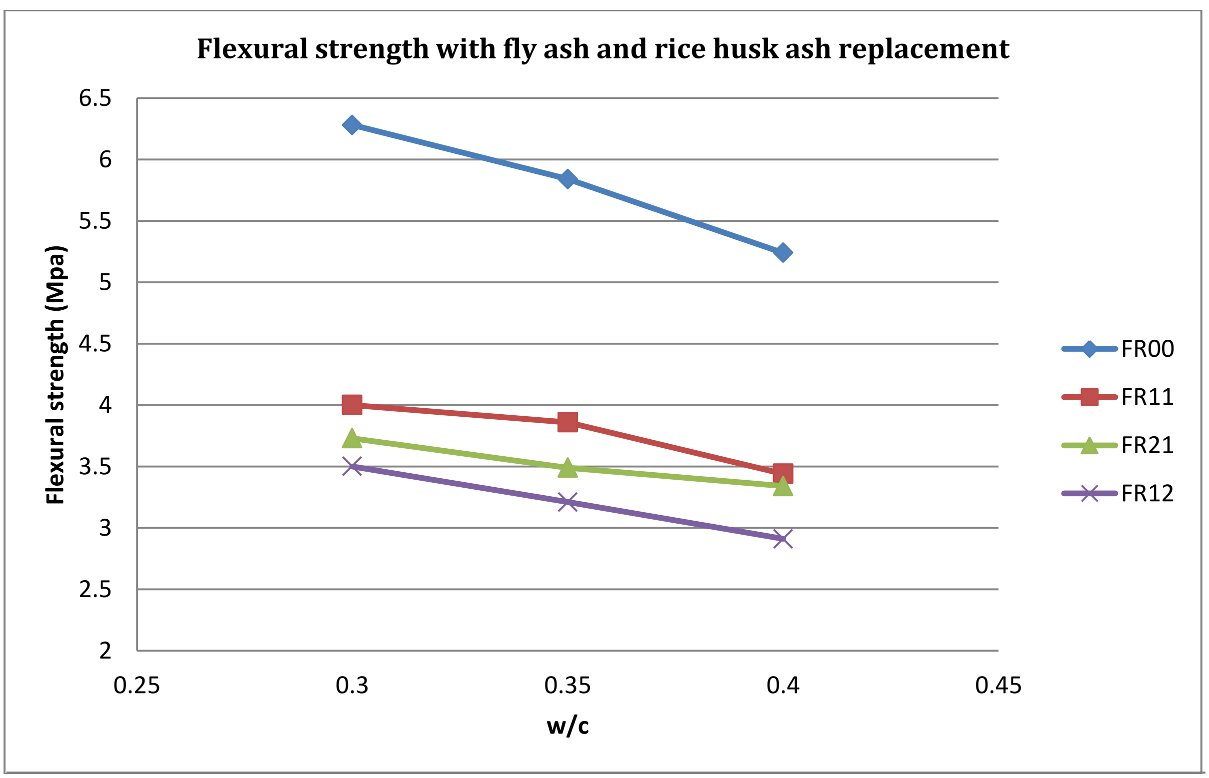

Fig 4.11 Flexural strength with cement replaced by fly ash and rice husk ash both

4.4 DISCUSSION OF RESULTS

4.4.1 Effect of Fly ash and Rice husk Ash Replacement on Compressive Strength of PQC

a) ***Effect of age on compressive strength***

Figs. 4.1 to 4.7 and Tables 4.1 to 4.3 show the variation of compressive strength of Pavement Quality Concrete (PQC) due to variation in the replacement levels of fly ash and rice husk ash, individually as well as in combinations, at the curing ages of 7 and 28 days.

The Tables 4.7 to 4.9 and Figures 4.12 to 4.14 show the percentage increase in values of compressive strengths with age (from 7 to 28 days) for all the replacement combination concrete mixes with w/c ratios of 0.30, 0.35 & 0.40 respectively. From the data as presented, it can be seen that the mixes with only fly ash replacement has a lesser rate of increase in strength from 7days to 28 days though they have high initial strength, than the mixes with rice husk ash replacement only and mixes with both fly ash and rice husk ash as replacement of cement. The mixes with the inclusion of both rice husk ash and fly ash as replacement material show the highest rate of increase of strength for all water to cement ratios indicating that pozzolanic activity initiates early for such mixes.

Table 4.7 - Effect of Age on Compressive Strength of PQC w/c = 0.3

W/C = 0.3	*7-DAY*	*28-DAY*	*% INCREASE*
FR00	46.56	61.68	32.47
FR10	42.41	53.3	26.27
FR20	44.43	53.05	19.40
FR30	32.81	46.31	41.14
FR01	26.74	44.25	65.4
FR02	23.41	32.25	37.76
FR03	21.24	27.1	27.58
FR11	30.49	44.69	46.57
FR21	25.98	39.65	52.61
FR12	22.24	35.98	61.78

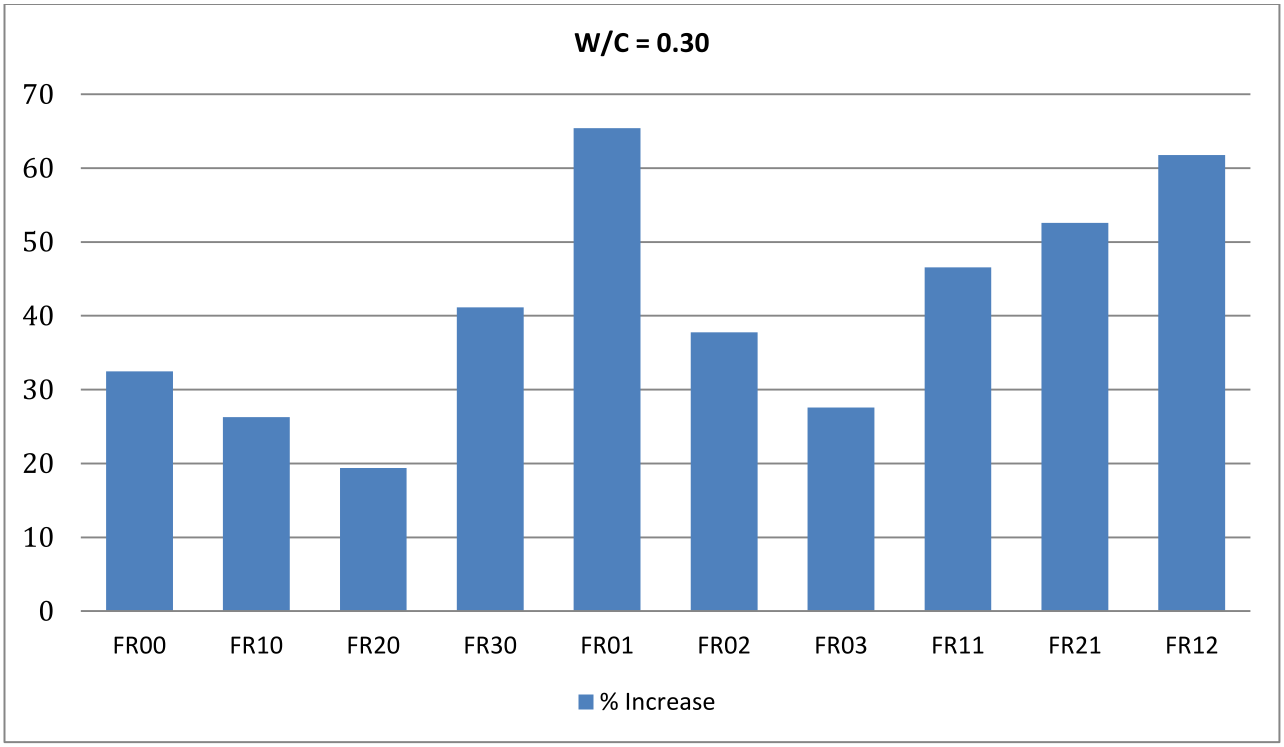

Fig 4.12 Percentage increase in compressive strengths of 7days to 28 days w/c = 0.30

Table 4.8 - Effect of Age on Compressive Strength of Paving Concrete w/c = 0.35

W/C = 0.35	7-DAY	28-DAY	% INCREASE
FR00	40.91	56.87	39.01
FR10	37.81	49.78	31.65
FR20	35.18	48.14	36.83
FR30	32.64	42.81	31.15
FR01	24.17	33.71	39.47
FR02	21.63	31.67	46.41
FR03	19.35	26.85	38.75
FR11	23.9	34.97	46.31
FR21	20.89	32.21	54.18
FR12	18.27	30.46	66.72

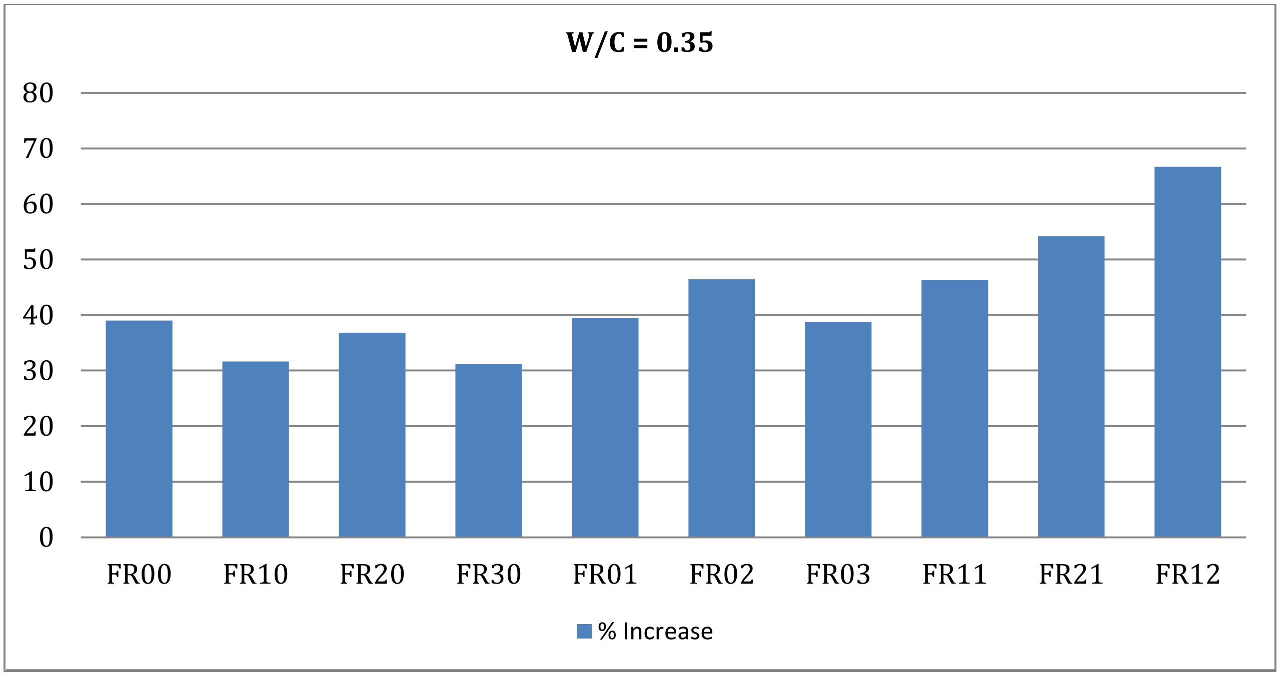

Fig 4.13 Percentage increase in compressive strengths of 7days to 28 days w/c 0.35

Table 4.9 - Effect of Age on Compressive Strength of Paving Concrete w/c = 0.40

W/C = 0.40	7-DAY	28-DAY	% INCREASE
FR00	38.31	51.01	33.15
FR10	37.7	48.94	29.81
FR20	34.37	45.16	31.39
FR30	29.2	41.6	42.46
FR01	22.6	32.63	44.38
FR02	19.58	28.71	46.62
FR03	16.35	25.59	56.51
FR11	21.01	32.35	53.97
FR21	16.43	27.56	67.74
FR12	15.97	25.89	62.11

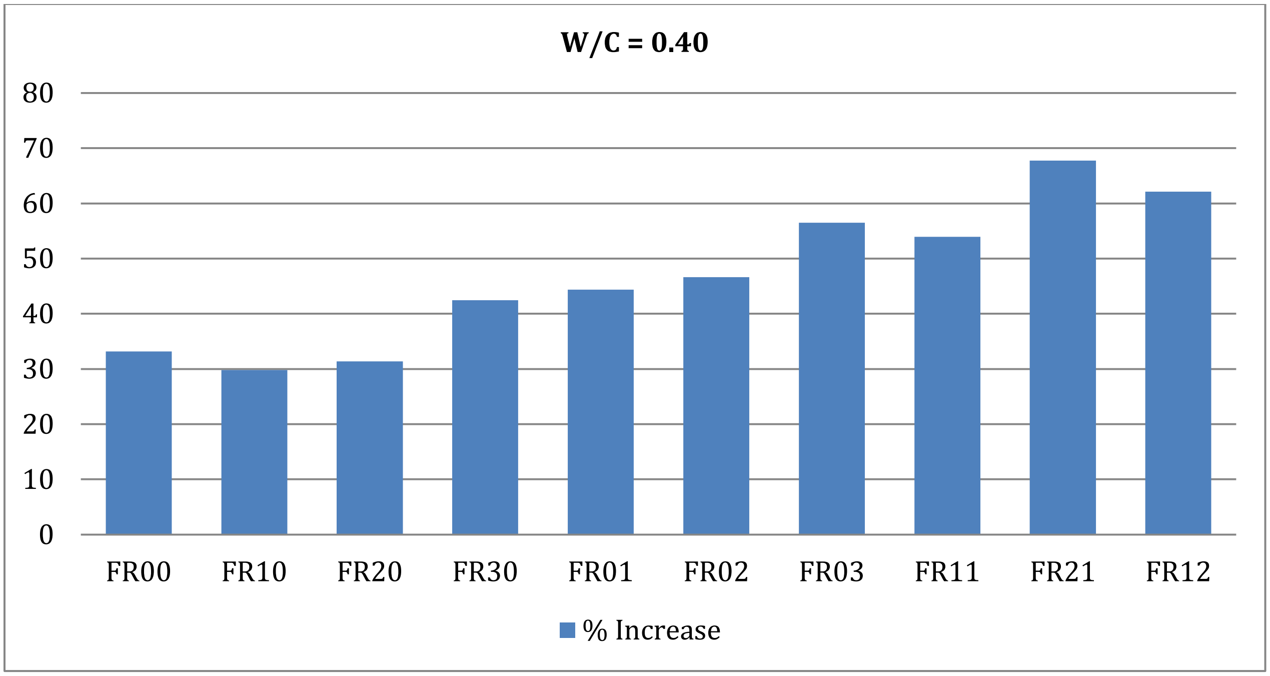

Fig 4.14 Percentage increase in compressive strengths of 7days to 28 days w/c = 0.40

b) ***Effect of replacement levels of mineral admixtures on compressive strength of PQC***

The Tables 4.12 to 4.14 and Figures 4.15 to 4.20 show the percentage variation in compressive strengths of the mixes with fly ash and rice husk ash as replacement materials, individually as well as in combination, as compared to the strength of the control mix specimen and also relative to the minimum required design compressive strength for PQC mixes (as per MORTH standards the value is 40MPa). The variations are shown for all the three water to cement ratios of 0.30, 0.35 and 0.40. It is observed that for all the water-cement ratios none of the concrete mixes, with partial cement replacement with fly ash and rice husk ash , could achieve the compressive strength value of the control mix in 28 day curing period. The mixes containing only 10% fly ash could achieve 85% of the control strength, whereas, the mixes containing only 30% rice husk as replacement achieved only 45% of the target controlled strength.

When compared with the minimum required design compressive strength for PQC mixes (as per MORTH standards the value is 40MPa), it is observed that all the mixes with fly ash replacement showed higher compressive strengths than required for PQC for all replacement levels and for all water to cement ratios. The concrete mixes with replacement of cement by rice husk ash only, could not achieve the desired PQC strength for water to cement ratios 0.35 and 0.40, but with 10% replacement of rice husk ash and combined replacement of 10% each of fly ash and rice husk ash with a water-cement ratio of 0.3 higher compressive strengths were observed as compared to the minimum required for PQC.

Table 4.10 - Comp. strength relative to controlled mix and min. reqd. strength w/c = 0.30

W/C = 0.30	*% variation in compressive strength relative to that of control specimen*	*% variation in compressive strength relative to the minimum design strength (as per MORTH standard for PQC) of 40MPa*
FR00		+54.2
FR10	-13.58	+33.25
FR20	-13.99	+32.625
FR30	-25.08	+15.525
FR01	-28.25	+10.625
FR02	-47.71	-19.375
FR03	-56.06	-32.25
FR11	-27.54	+11.725
FR21	-35.71	-0.875
FR12	-41.67	-10.05

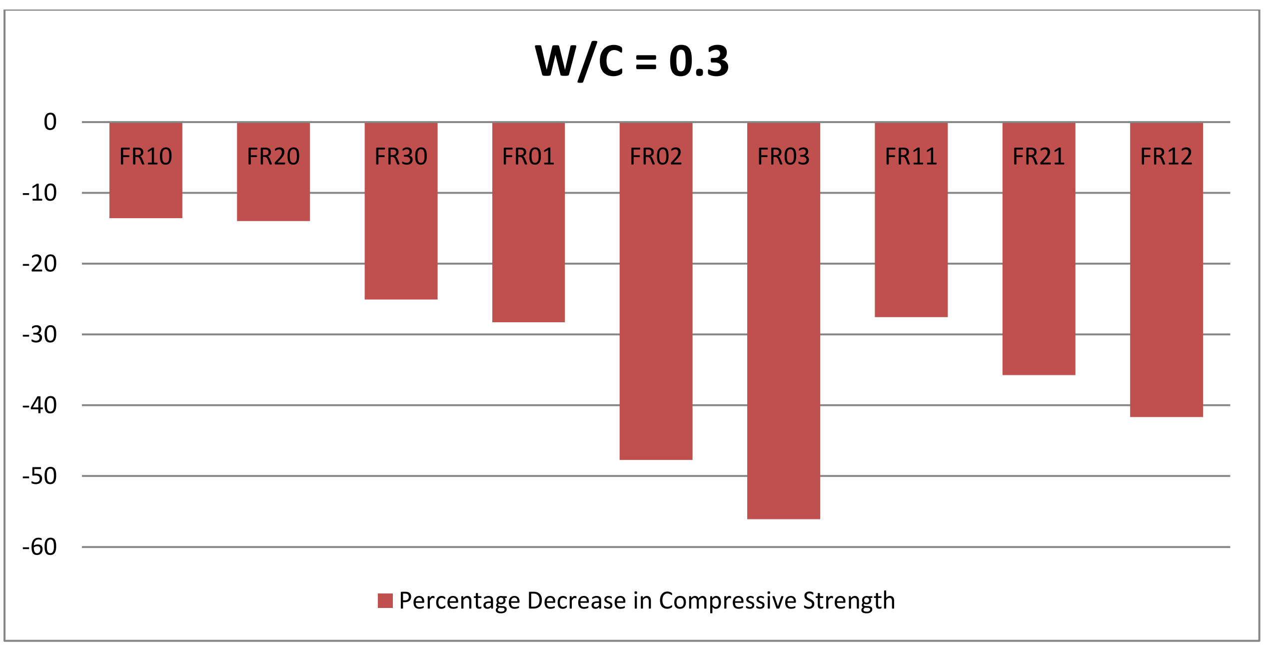

Fig 4.15 Percentage decrease in comp strength relative to controlled mix strength w/c = 0.3

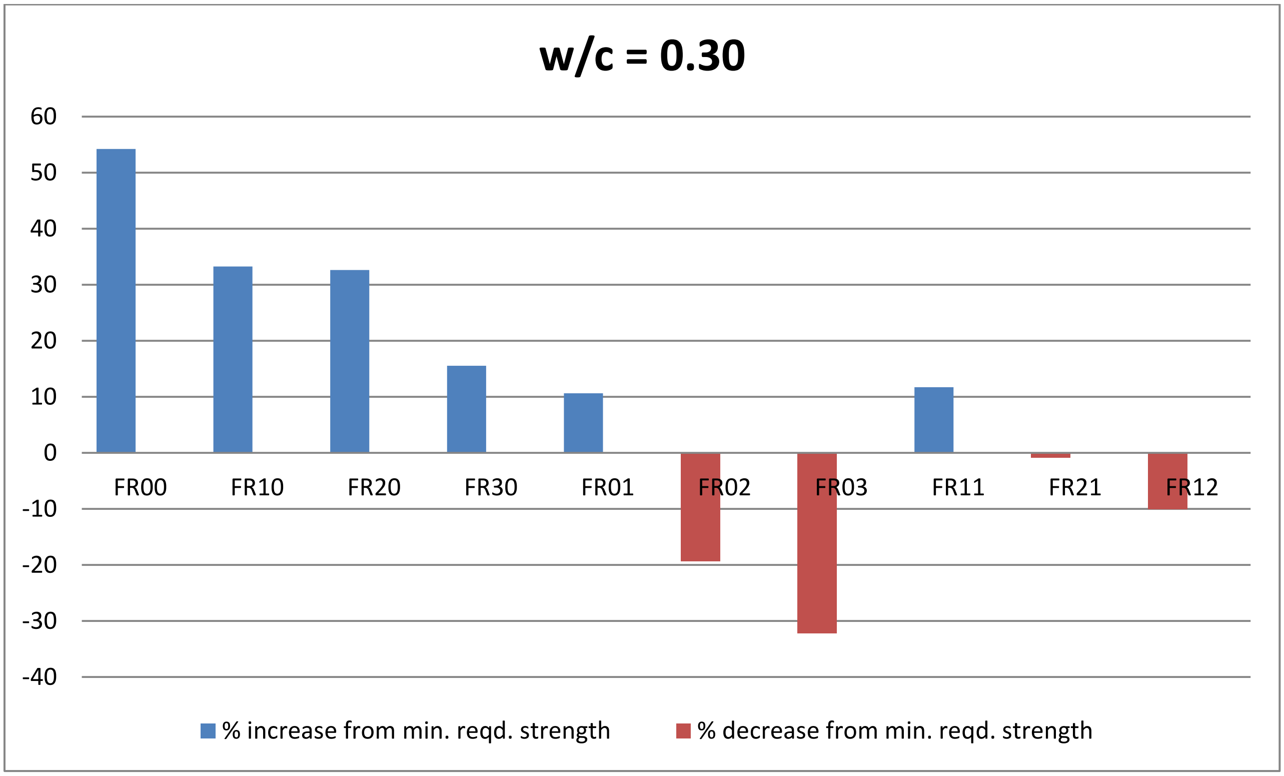

Fig 4.16 Percentage of compressive strengths relative to reqd. min. strength at w/c =0.30

Table 4.11 Comp. strength relative to controlled mix and reqd. min. strengths at w/c = 0.35

W/C = 0.35	% variation in compressive strength relative to that of control specimen	% variation in compressive strength relative to the minimum design strength (as per MORTH standard for PQC) of 40MPa
FR10	-12.46	+24.45
FR20	-15.35	+20.35
FR30	-24.72	+7.025
FR01	-40.72	-15.725
FR02	-44.31	-20.825
FR03	-52.78	-32.875
FR11	-38.50	-12.575
FR21	-43.36	-19.475
FR12	-46.43	-23.85

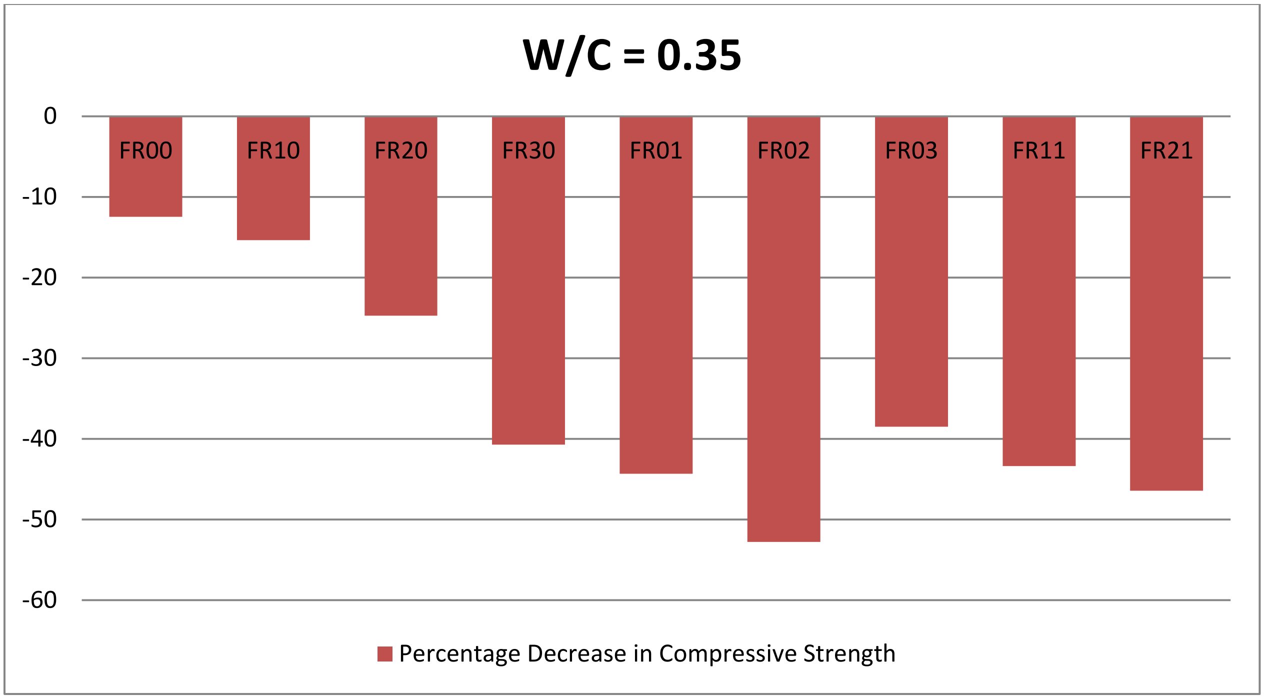

Fig4.17 Percentage decrease in comp strength relative to controlled mix strength at w/c = 0.35

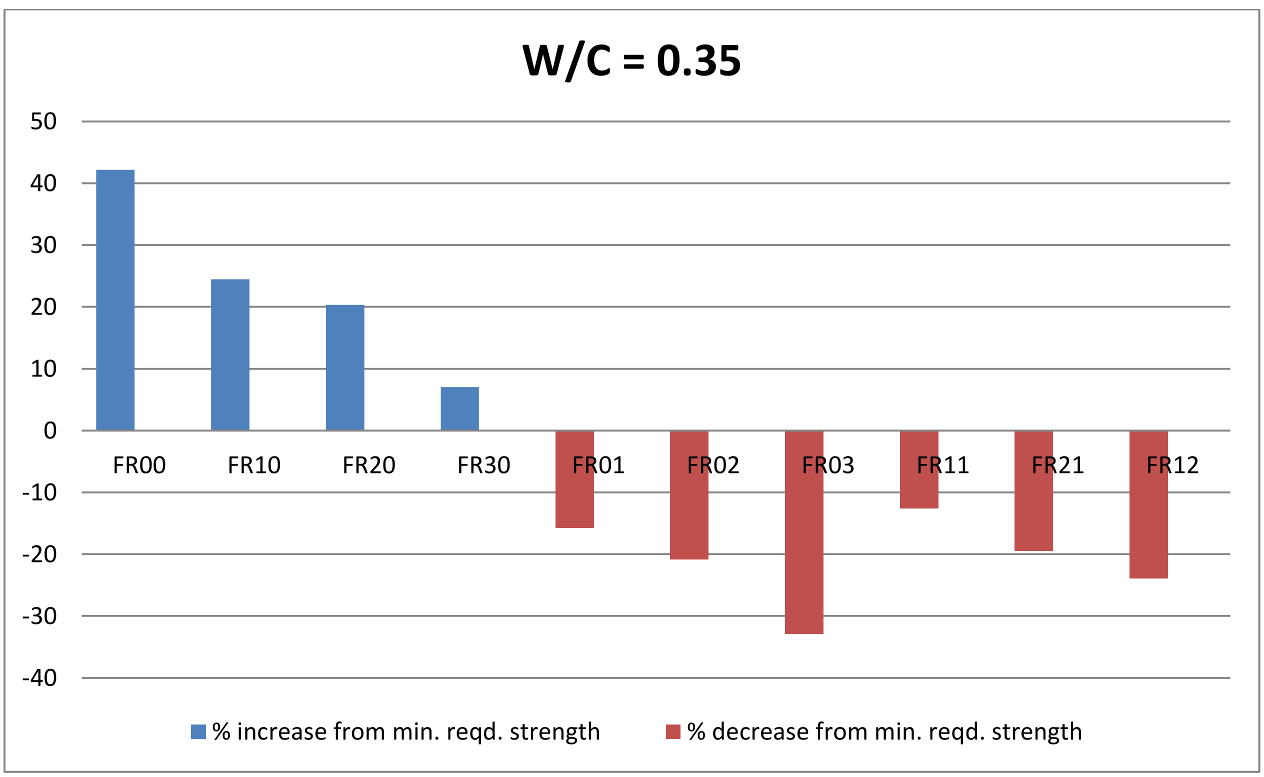

Fig 4.18 Percentage of compressive strengths relative to min. reqd. strength at w/c 0.35

Table 4.12 Comp. strength relative to controlled mix and min. reqd. strength at w/c = 0.40

W/C = 0.40	*% variation in compressive strength relative to that of control specimen*	*% variation in compressive strength relative to the minimum design strength (as per MORTH standard for PQC) of 40MPa*
FR10	-4.05	+22.35
FR20	-11.46	+12.9
FR30	-18.44	+4
FR01	-36.03	-18.425
FR02	-43.71	-28.225
FR03	-49.83	-36.025
FR11	-36.58	-19.125
FR21	-45.97	-31.1
FR12	-49.24	-35.275

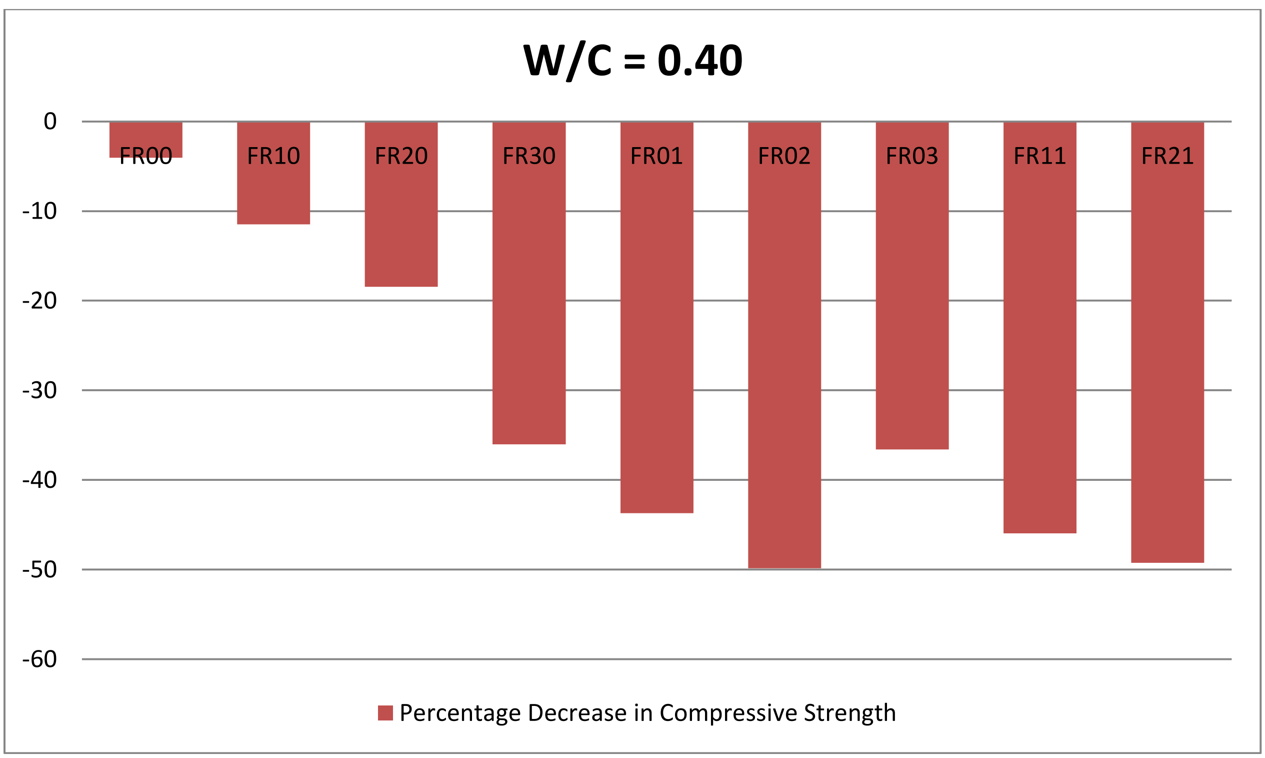

Fig 4.19 Percent decrease in comp strength relative to controlled mix strength at w/c=0.40

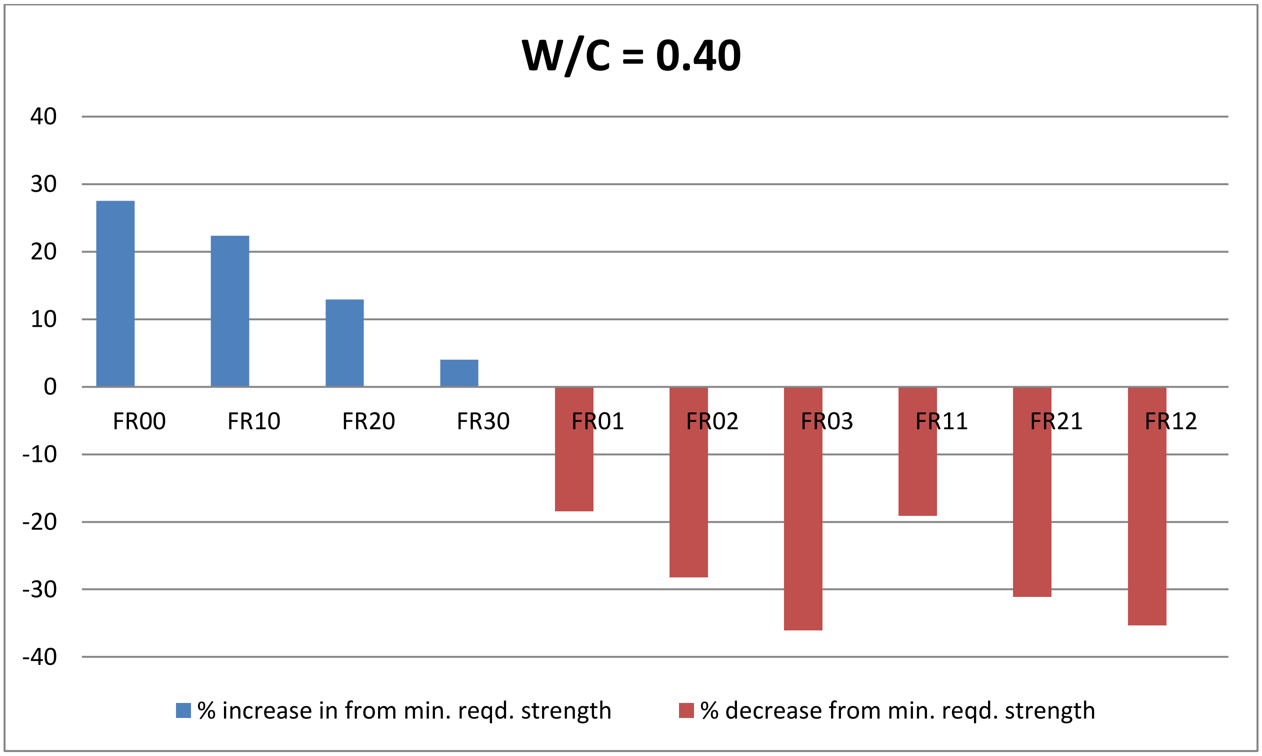

Fig 4.20 Percentage of compressive strengths relative to min. reqd. strength at w/c =0.40

4.4.2 Effect of Fly ash and Rice husk Ash Replacement on Flexural Strength of PQC

a) ***Effect of replacement levels of mineral admixtures on flexural strength of PQC***

The Tables 4.15 to 4.17 and Figures 4.21 to 4.26 show the percentage variation in flexural strengths of the mixes with fly ash and rice husk ash as replacement materials, individually as well as in combination, as compared to the strength of the control mix specimen and also relative to the different design flexural strengths for PQC mixes taken in the study. The variations are shown for all the three water to cement ratios of 0.30, 0.35 and 0.40. It is observed that for all the water-cement ratios none of the concrete mixes, with partial cement replacement with fly ash and rice husk ash, could achieve the flexural strength value of the control mix in 28 day curing period. The mixes containing only fly ash could achieve 85 to 95% of the control strength, whereas, the mixes containing only 30% rice husk as replacement achieved only 55% of the target controlled strength.

When compared with the required design flexural strength for different PQC mixes (flexural strength of 4.5, 5.0 and 5.5MPa) it is observed that the mixes with only 10 and 20% fly ash replacement showed higher flexural strengths than the design values for PQC and for all water to cement ratios. The concrete mixes with replacement of cement by rice husk ash only, or those containing fly ash and rice husk ash in combination, could not achieve the desired design PQC strength for all water to cement ratios 0.35 and 0.40.

The figures 4.27 to 4.29 also show the comparison of the achieved flexural strength relative to the design PQC value of 4.5, 5.0 and 5.5 MPa. From the figure also, it is observed that fly ash up to 20% replacement for all the water-cement ratios shows higher flexural strengths than minimum required flexural strengths. Thus, it can be concluded that up to 20% cement replacement by fly ash can be used in designing pavement quality concrete mixes.

The economic effect of these replacement levels and the effect on the slab thickness for PQC slabs is discussed in Chapter 6.

Table 4.13 Flexural strength relative to controlled mix and min. reqd. strengths w/c = 0.30

W/C = 0.30	***% variation in flexural strength relative to that of control specimen***	***% variation in flexural strength relative to the minimum design strength (as per MORTH standard for PQC) of 5.5 MPa***
FR00	0	+14.18182
FR10	3.66	+10
FR20	9.23	+3.636364
FR30	16.08	-4.18182
FR01	21.81	-10.7273
FR02	37.10	-28.1818
FR03	44.58	-36.7273
FR11	36.30	-27.2727
FR21	40.60	-32.1818
FR12	44.26	-36.3636

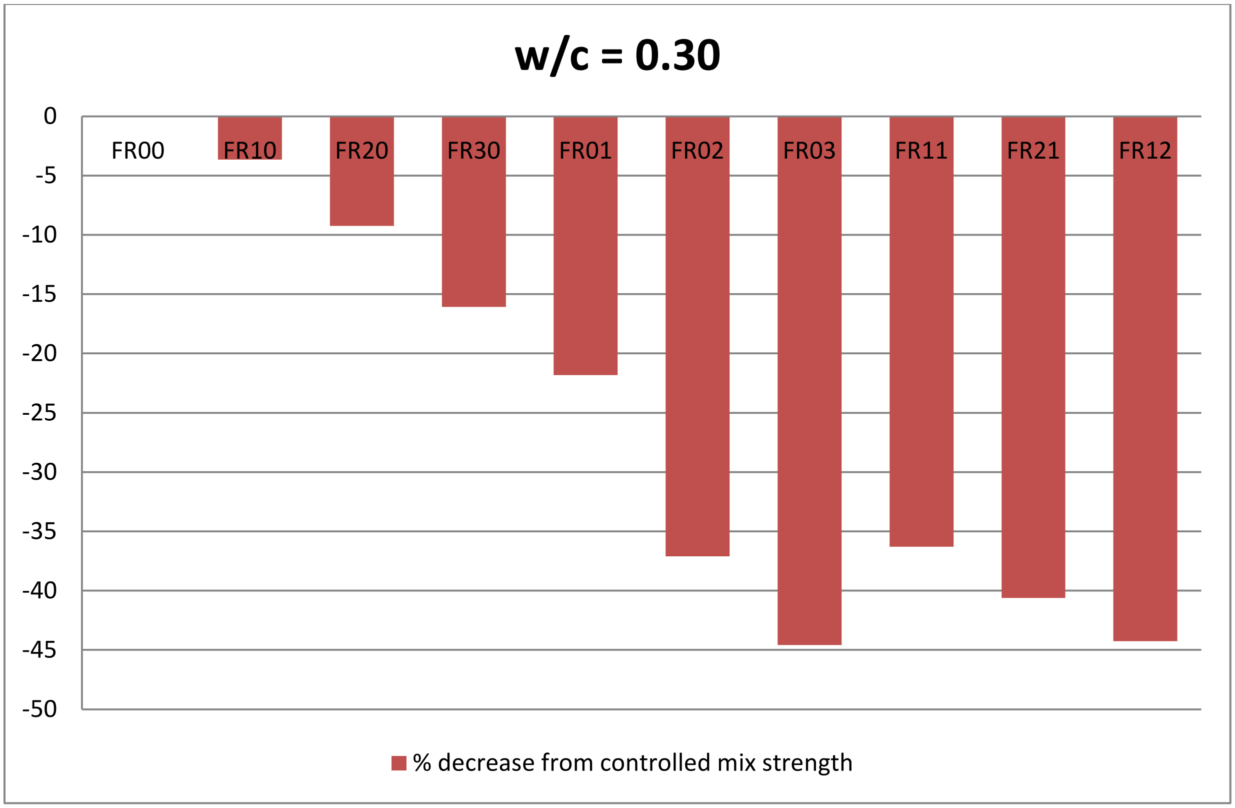

Fig 4.21 Percentage decrease in flexural strength relative to controlled mix strength w/c = 0.3

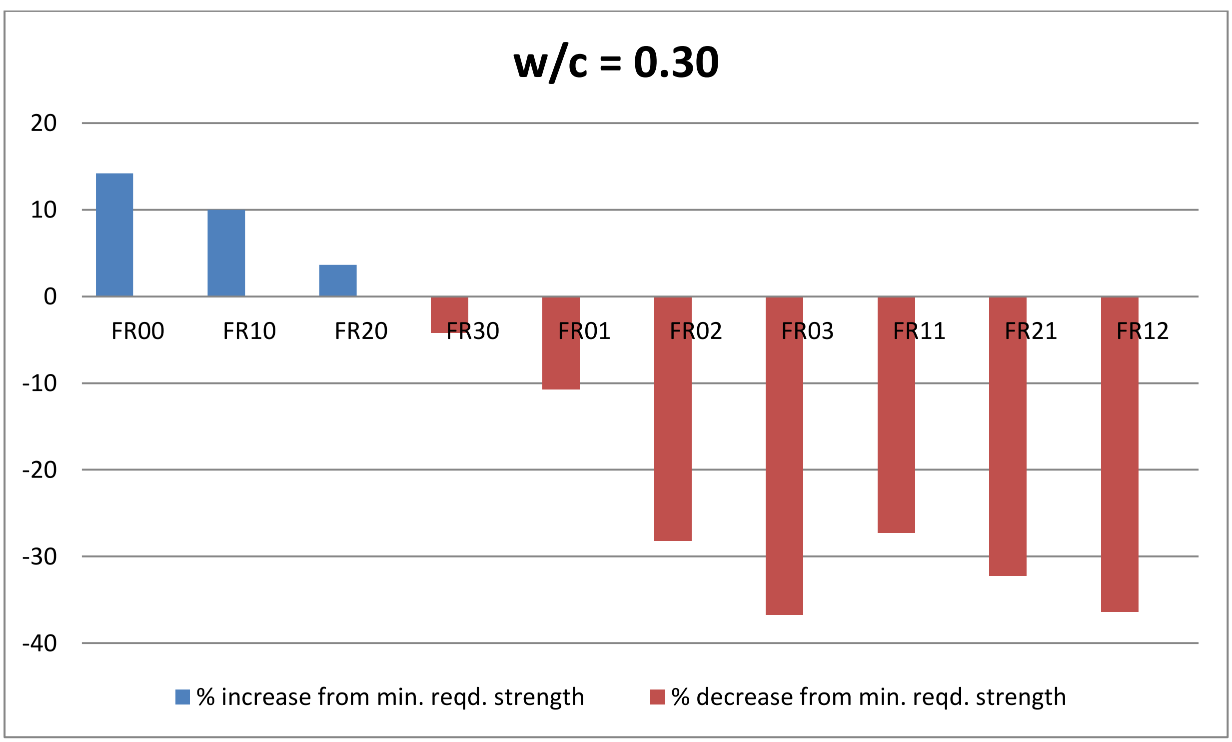

Fig 4.22 Percentage change of flexural strengths relative to target strength w/c 0.30

Table 4.14 Flexural strength relative to controlled mix and min. reqd. strengths w/c = 0.35

W/C = 0.35	*% variation in flexural strength relative to that of control specimen*	*% variation in flexural strength relative to the minimum design strength (as per MORTH standard for PQC) of 5.0MPa*
FR00	0	+16.8
FR10	-2.39726	+14
FR20	-6.50685	+9.2
FR30	-15.0685	-0.8
FR01	-20.0342	-6.6
FR02	-34.9315	-24
FR03	-45.3767	-36.2
FR11	-33.9041	-22.8
FR21	-40.2397	-30.2
FR12	-45.0342	-35.8

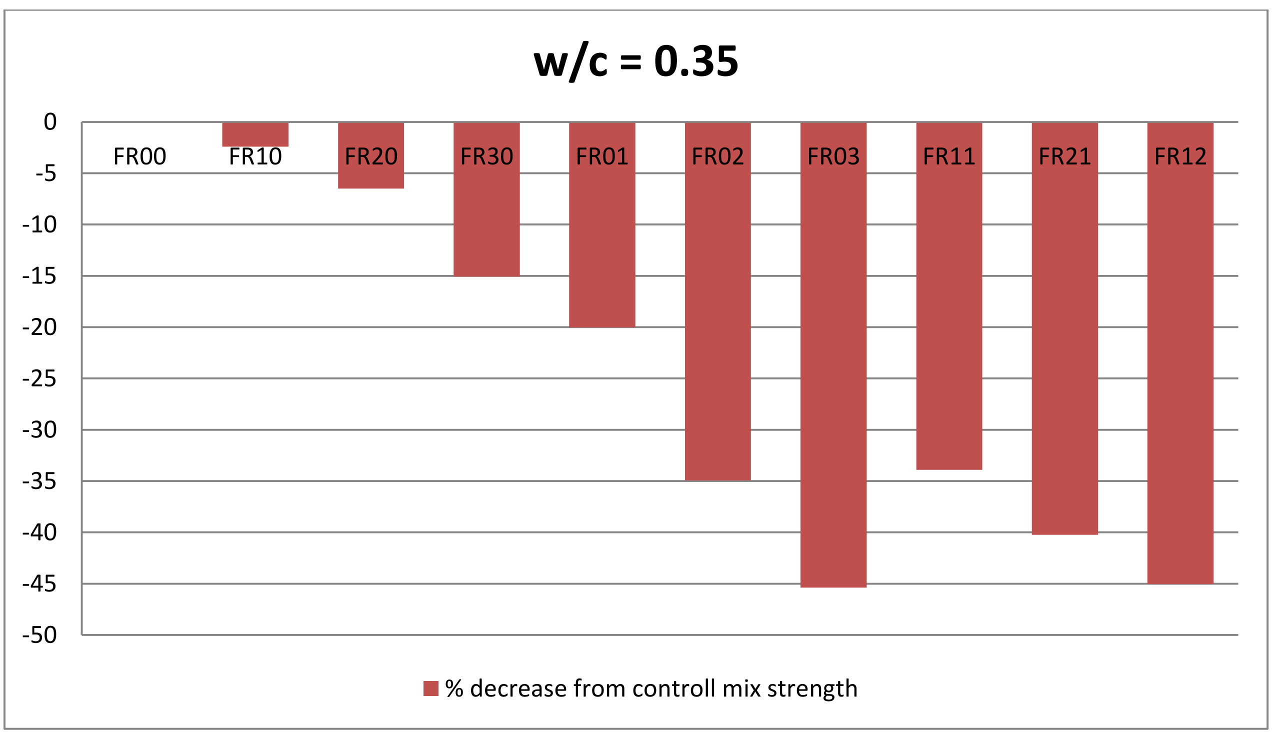

Fig 4.23 Percentage decrease in flexural strength relative to controlled mix strength w/c = 0.35

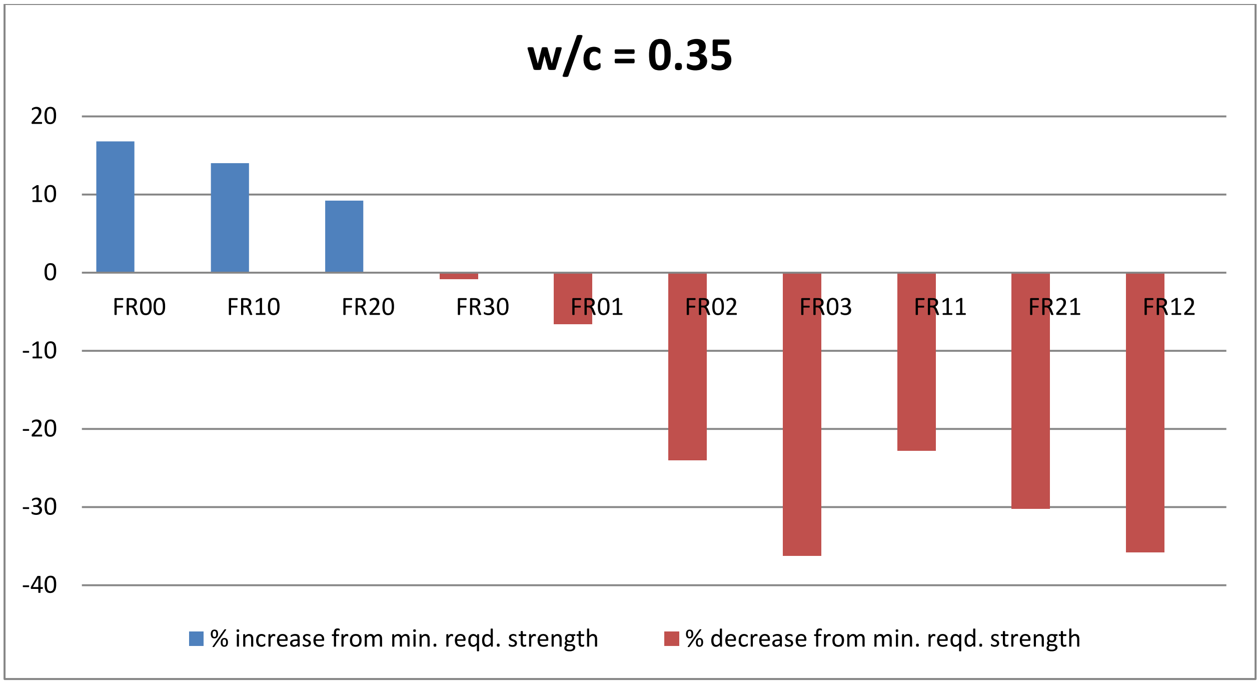

Fig 4.24 Percentage change of flexural strengths relative to target strength w/c 0.35

Table 4.13 Flexural strength relative to controlled mix and min. reqd. strengths w/c = 0.40

W/C = 0.40	**% variation in flexural strength relative to that of control specimen**	**% variation in flexural strength relative to the minimum design strength (as per MORTH standard for PQC) of 4.0MPa**
FR00	0	+16.44444
FR10	-3.62595	+12.22222
FR20	-5.15267	+10.44444
FR30	-15.0763	-1.11111
FR01	-16.7939	-3.11111
FR02	-35.8779	-25.3333
FR03	-44.8473	-35.7778
FR11	-34.3511	-23.5556
FR21	-36.2595	-25.7778
FR12	-44.4656	-35.3333

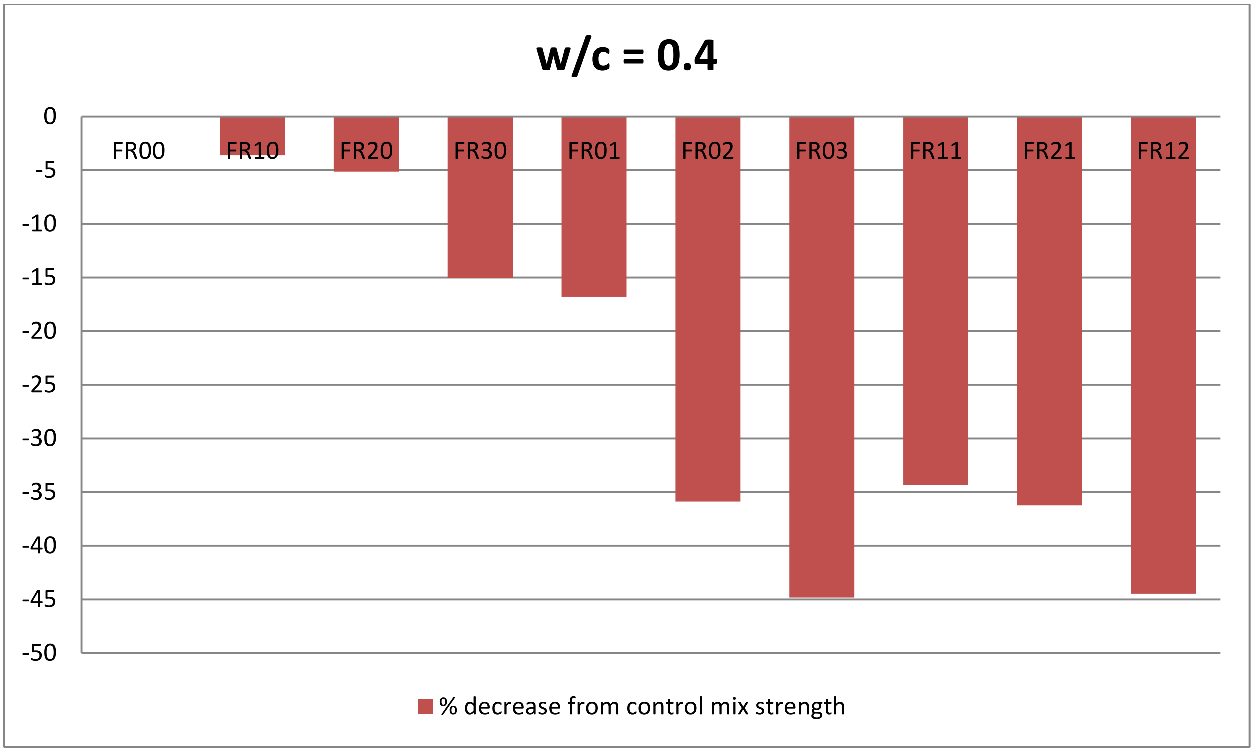

Fig 4.25 percentage decrease in Flexural strength relative to controlled mix strength w/c = 0.40

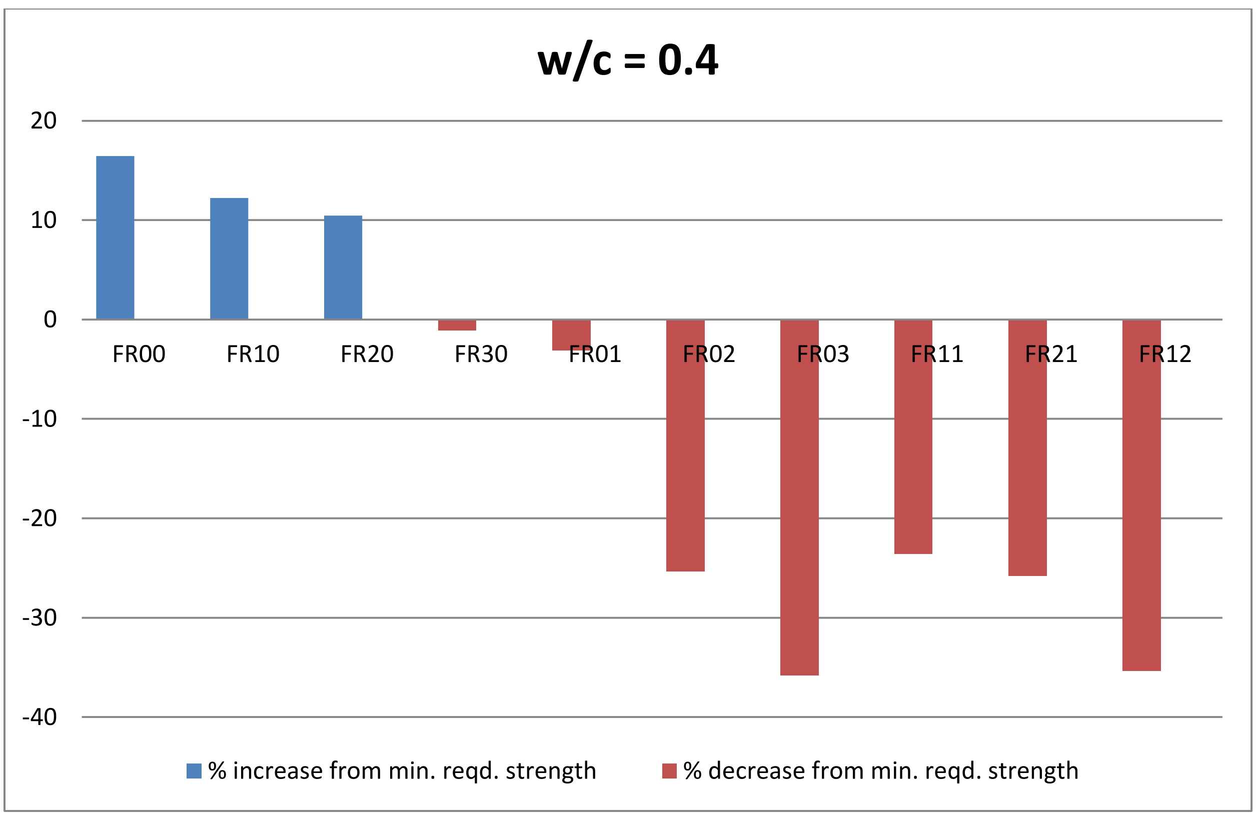

Fig 4.26 percentage change of flexural strengths relative to target strength at w/c 0.40

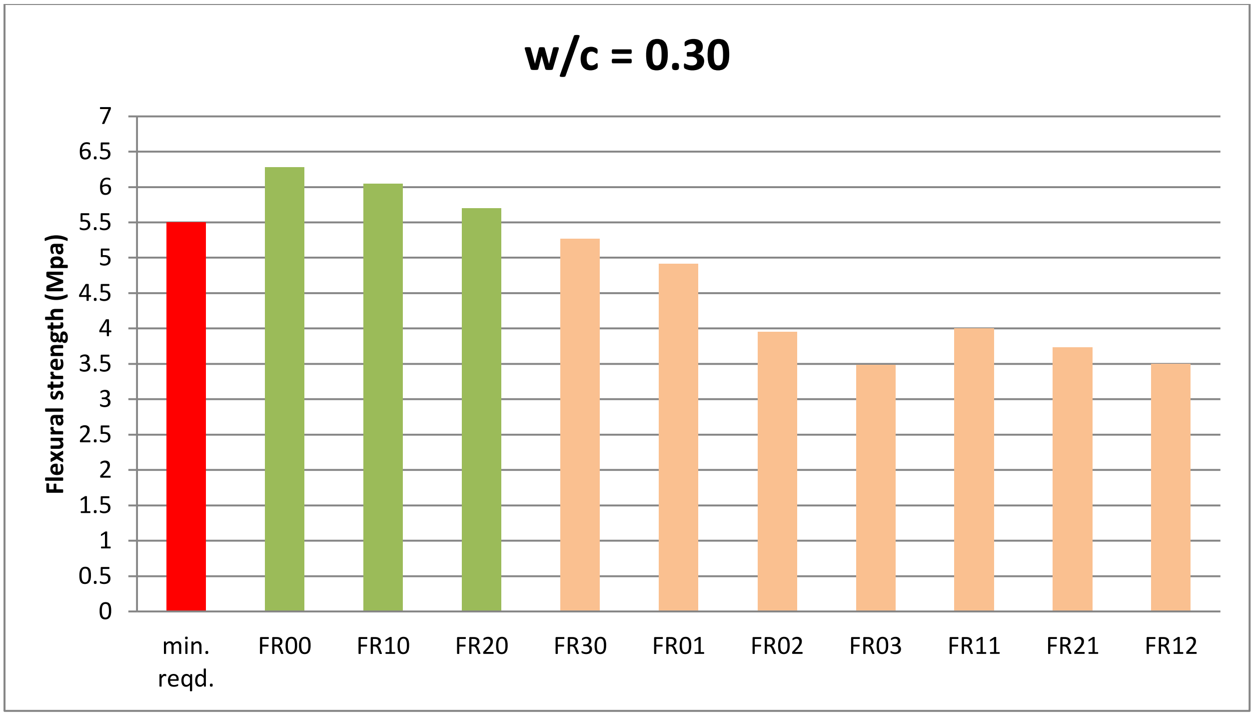

Fig 4.27 flexural strengths relative to required minimum design strength of 5.5 MPa.

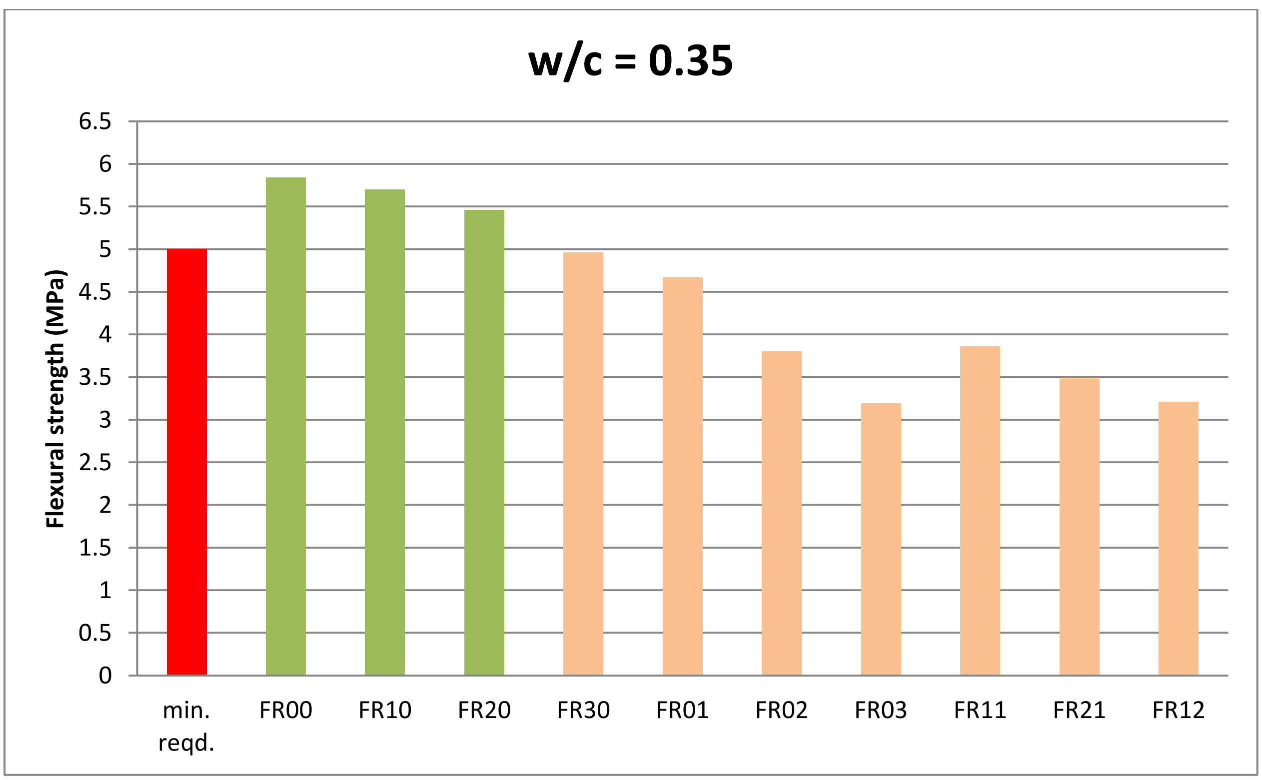

Fig 4.28 flexural strengths reletive to required minimum design strength of 5.0 MPa.

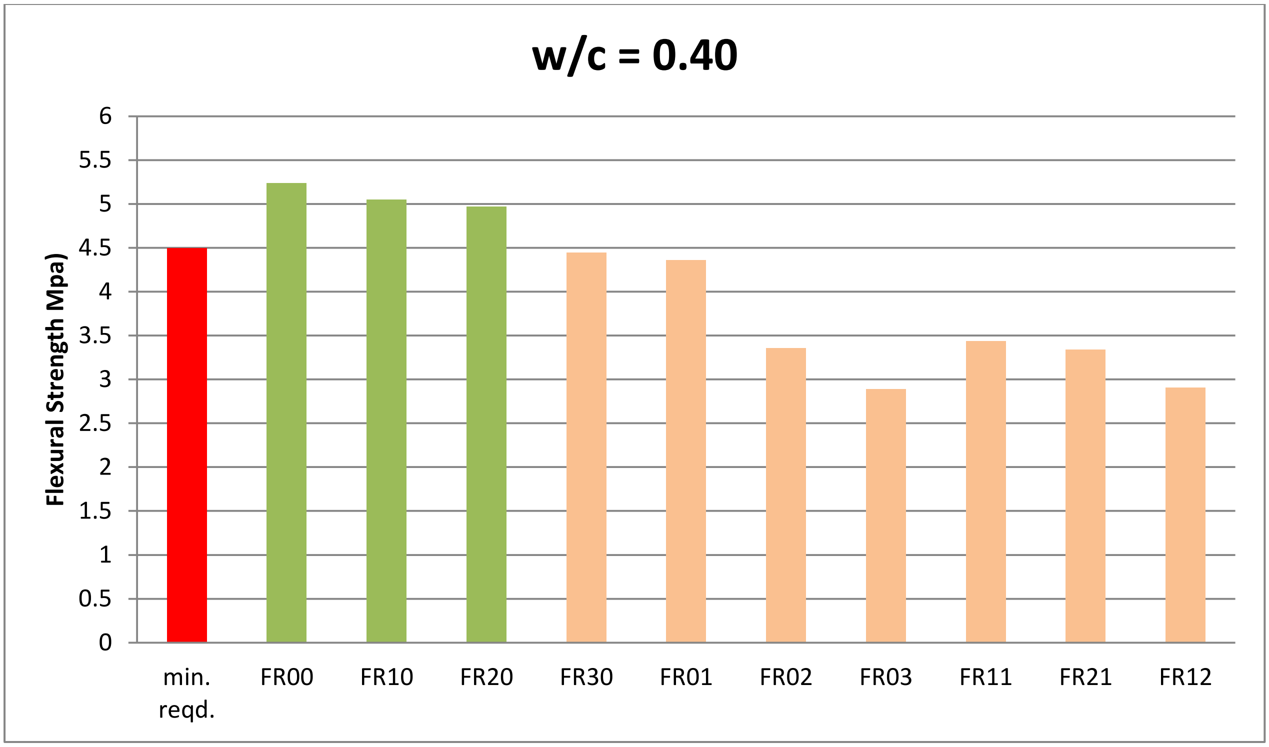

Fig 4.29 flexural strengths relative to required minimum design strength of 4.5 MPa.

CHAPTER 5
DESIGN AND ANALYSIS OF PAVEMENT QUALITY CONCRETE SLAB

5.1 DESIGN OF PAVEMENT QUALITY CONCRETE (PQC) FOR INDIAN HIGHWAYS

5.1.1 Introduction

Guidelines for the design of rigid pavements for highways were first approved by the Cement Concrete Road Surfacing Committee in March 1973 and published subsequently in the IRC: 58-2002 editions. In view of the recent upward revision of the legal limit on the maximum laden axle loads of commercial vehicle from 8160 kg to 10200 kg, appropriate modifications have become necessary in some sections of the Guidelines of IRC:58-1988, which resulted in the publication of IRC:58-2002, "Guidelines for the Design of Plain Jointed Rigid Pavements for Highways". The early approach to the Design of Rigid Pavement was based on Westergaard`s analysis. Recent advance in knowledge have led to vast changes in the design methodology.

5.1.2 Factor Governing Design

a) Factor Governing Design: It considers single and tandem axle loads, their repetition, tyre pressure and lateral placement characteristics of commercial vehicles.

b) Wheel Load: The legal axle load limits in India have been fixed as 10.2, 19 and 24 tonnes for single axles, tandem axles and tridem axles respectively, a large number of axles operating on National Highways carry much higher loads than the legal limits. Data on axle load distribution of the commercial vehicles is required to compute the number of repetitions of single and tandem axles of different weights expected during the design period. For most of the commercial highways vehicles, the tyre pressure ranges from 0.7 to 1.0 MPa but it is found that stresses in concrete pavements having thickness of 20 cm or more are not affected significantly by the variation of tyre pressure in the range mentioned earlier. A tyre pressure of 0.8 MPa may be adopted for design.

For computation of stresses in the pavements, the magnitude of axle loads should be multiplied by *Load Safety Factor (LSF).* For important roads, like, Expressways, National Highways and other roads where there will be uninterrupted traffic flow and high volume of truck traffic, the value of LSF is taken as 1.2. For roads of lesser importance having lower proportion of truck traffic, LSF may be taken as 1.1. For residential and other streets that carry small number of commercial traffic, the LSF may be taken as 1.0.

c) Design Period: Normally, cement concrete pavements have a life span of 30 years and should be designed for this period. When the traffic intensity cannot be predicted accurately for a long period of time, and for low volume roads, a design period of twenty years may be considered. However, the design engineer should use his judgement about the design life taking into consideration the factors, like, traffic volume, the traffic growth rate, the capacity of the road.

d) Design Traffic: Assessment of average traffic should normally be based on 7×24 hours count made in accordance with IRC: 9 "Traffic Census on Non-Urban Roads". The actual value of growth rate 'r' of heavy commercial vehicles should be determined. However, if actual data is not available, an average annual growth rate of 7.5 percent may be adopted.

The cumulative number of repetitions of axles during the design period may be adopted from the following formula:

$$C = \frac{365 \times A\{(1+r)^n - 1\}}{r}$$

Where

C = Cumulative number of axles during the design period.

A = Initial number of axles per day in the year when the road is operational.

r =Annual rate of growth of commercial traffic (expressed in decimals).

n = Design period in years.

In most of design problems, it is expected that the weights and number of trucks travelling in each direction are fairly equal. This may not be true for roads, like, Haul roads in mine areas where many of the trucks haul full loads in one direction and return empty in the other direction. In such cases, a suitable adjustment should be made.

e) Temperature Differential: Temperature differential between the top and the bottom of concrete pavements causes the concrete slab to wrap, giving rise to stresses. The temperature differential is a function of solar radiation received by the pavement surface at the location, losses due to wind velocity and thermal diffusivity of concrete and is thus affected by geographical features of the pavement location. As far as possible, values of actually anticipated temperature differentials at the location of the pavement should be adopted for pavement design.

5.2 CHARACTERISTICS OF SUBGRADE AND SUB BASE

5.2.1 Strength

The strength of subgrade is expressed in terms of modulus of subgrade reaction *k*, which is defined as pressure per unit deflection of the foundation as determined by plate bearing tests. As the limiting design deflection for cement concrete pavements is taken as 1.25 mm, the *k*-value is determined from the pressure sustained at this deflection. As *k*-value is carried out by test plate diameter, the standard test is to be carried out with a 75 cm diameter plate. IS: 9214-1974, "Method of Determination of

Modulus of Subgrade Reaction of Soil in the field" may be referred. A frequency of one test per km per lane is recommended for assessment of *k*-value, unless the foundation changes with respect of subgrade soil, type of sub-base or the nature of formation i.e. cut or fill when additional tests may be conducted. An approximate idea of *k*-value of a homogenous soil subgrade may be obtained from its soaked CBR value using Table 5.1. It is advisable to have a filter layer above the subgrade for drainage of water to prevent (i) excessive softening of subgrade and (ii) erosion of the subgrade particularly under adverse moisture condition.

The approximate increases in *k*-values of subgrade due to different thickness of sub-bases made up of untreated granular, cement treated granular and dry lean concrete (DLC) layers may be taken from Table 5.2. 7-day unconfined compressive strength of cement treated granular soil should be a minimum of 2.1 MPa. Dry Lean Concrete should have a minimum compressive strength of 7 MPa at 7 days.

Table 5.1 Approximate *K*-Value Corresponding to CBR Values For Homogenous Soil Subgrade

Soaked CBR value %	2	3	4	5	7	10	15	20	50	100
k-value (kg/cm²/cm)	2.1	2.8	3.5	4.2	4.8	5.5	6.2	6.9	14.0	22.2

Table 5.2 *K*-Values with Dry Lean Concrete Sub-Base

k-value of subgrade kg/cm²/cm	2.1	2.8	4.2	4.8	5.5	6.2
Effective k over 100 mm DLC, kg/cm²/cm	5.6	9.7	16.6	20.8	27.8	38.9
Effective k over 150 mm DLC, kg/cm²/cm	9.7	13.8	20.8	27.7	41.7	----

The maximum value of effective k will be 38.9 kg/cm^2/cm for 100 mm of DLC and 41.7 kg/cm^2/cm for 150 mm of DLC.

5.2.2 Separation layer between sub-base and pavement:

Foundation layer below concrete slabs should be smooth to reduce the inter layer friction. A separation membrane of minimum thickness of 125 micron polythene is recommended to reduce the friction between concrete slabs and dry lean concrete slab-base (DLC).

5.3 CHARACTERISTICS OF CONCRETE:

5.3.1 Design Strength:

The concrete pavements fail due to bending stresses, it is necessary that their design is based on the flexural strength of concrete. The relationship b/w the flexural strength and compressive strength may be worked out. The mix should be so designed that the minimum structural strength requirement in the field is met at the desired level. Thus,

S^1 = Characteristics flexural strength at 28 days.

S = Target average flexural strength at 28 days.

Z_a = Tolerance factor for the desired confidence level, known as the standard normal variate

σ=Expected standard deviation of field test samples; if it is not known, it may be initially
Assumed as per IS: 456-2000

Then the target average flexural strength is given as:

$$S = S^1 + Z_a\sigma$$

For pavement construction, the concrete mix should preferably be designed and controlled on the basis of flexural strength. Flexural strength should be determined by modulus of rupture tests under third point loading. The preferred size of the beam should be 15 cm × 15 cm × 70 cm when the size of the aggregate is more than 19 mm. When the maximum size of aggregate is less than 19 mm, 10 cm × 10 cm × 50 cm beams may be used. IS: 516 should be referred for the test procedure.

5.3.2 Fatigue behaviour of cement concrete:

Due to repeated application of flexural stresses by the traffic loads, progressive fatigue damage takes place in the cement concrete slab in the form of gradual development of micro-cracks especially when the applied stress in terms of flexural strength of concrete is high. The ratio b/w the flexural stress due to the load and the flexural strength of the concrete is known as stress ratio (SR). If the stress ratio is less than 0.45, the concrete is expected to sustain infinite number of repetitions. As the stress ratio increases, the number of load repetitions required to cause cracking decreases. The relation b/w fatigue life (N) and stress ratio is given as:

N = unlimited for SR < 0.45

$$N = \left(\frac{4.2577}{SR-0.4325}\right)^{3.268} \qquad \text{When } 0.45 \leq SR \leq 0.55$$

$$\log_{10} N = \frac{0.9716-SR}{0.0828} \text{ for SR > 0.55}$$

The values of fatigue life for different values of stress ratio are given in Table 5.3.

5.4 DESIGN OF SLAB THICKNESS

Step by step procedure of Design of Slab Thickness Pavement as per IRC 58-2002.

Step 1. As Per IRC 58:2002, Axle load (AL) in Single Axle and Tandem Axle (Tonnes) is given.

Step 2. From the given data, cumulative repetition in 20 yrs. can be calculated from the formula which is given below:

$$C = \frac{365 \times A\{(1+r)^{n}-1\}}{r}$$

Where,

C = Cumulative number of axles during the design period.

A = Initial number of axles per day in the year when the road is operational.

r =Annual rate of growth of commercial traffic (expressed in decimals).

n = Design period in years.

Step 3. After calculating cumulative repetition, Design traffic can be calculated by 25% of cumulative repetition.

Step 4. The Single and Tandem Axle load is multiplied with a factor of 1.2 respectively.

Step 5. Stress (kg/cm^2) is calculated from the given graphs as mentioned in IRC 58: 2002 (Graph between slab thickness and flexure strength).

Step 6. Stress Ratio is calculated from $\frac{Stress}{Flexure\ Strngth}$.

Step 7. Expected Repetition is calculated from the Design Traffic which is multiplied with a percentage of respective Axle loads.

Step 8. Allowable Repetition is calculated from the Table which is mentioned in IRC 58:2002 which is also shown below Table 5.3.

Step 9. Fatigue life consumed is calculated which is the ratio of fatigue life (N) and expected repetition (n). The design is *safe* if the cumulative fatigue life consumed is *less* than 1.0.

Table 5.3 Stress Ratio and Allowable Repetitions in Cement Concrete

Stress Ratio	Allowable Repetitions	Stress Ratio	Allowable Repetitions
0.45	6.279×10^7	0.66	5.83×10^3
0.46	1.4335×10^7	0.67	4.41×10^3
0.47	5.2×10^6	0.68	3.34×10^3
0.48	2.4×10^6	0.69	2531
0.49	1.287×10^6	0.70	1970
0.50	7.62×10^5	0.71	1451
0.51	4.85×10^5	0.72	1099
0.52	3.26×10^5	0.73	832
0.53	2.29×10^5	0.74	630
0.54	1.66×10^5	0.75	477
0.55	1.24×10^5	0.76	361
0.56	9.41×10^4	0.77	274
0.57	7.12×10^4	0.78	207
0.58	5.4×10^4	0.79	157
0.59	4.08×10^4	0.80	119
0.60	3.09×10^4	0.81	90
0.61	2.34×10^4	0.82	68
0.62	1.77×10^4	0.83	52
0.63	1.34×10^4	0.84	39
0.64	1.02×10^4	0.85	30
0.65	7.7×10^3		

The slab design process as per IRC 58-2002 for the pavement quality concrete tested in the laboratory is presented in Tables 5.5. The Table 5.4 contains the input traffic data in terms of the expected repetitions for the single and tandem axles, for which the slab thicknesses have been calculated. The other input parameters are as below:

Road category: Two-lane Two way National Highway Cement Concrete Pavement

Modulus of sub-grade reaction with 150mm DLC: 8kg/cm^3

Elastic Modulus of Concrete: 3 x 10^5 kg/cm^2

Tyre Pressure: 8kg/cm^2

Rate of increase of traffic: 7.5%

The Tables 5.5 to 5.13 present the design of slab thickness by calculating the fatigue life consumed for the given axle load traffic (Table 5.4) & flexural strength of PQC achieved in the laboratory. Table 5.5 presents the calculations for the minimum flexural strength, as per IRC specifications, of 4.5MPa. Subsequent tables present the design for flexural strengths achieved by varying the percentages of Marble dust and steel fibres in the concrete mix.

Table 5.4 Input Traffic Data for Slab Design

Flexure Strength= 45 kg/cm^2 Assumed Slab thickness = 33 cm			
Single Axle Load		**Tandem Axles**	
Load in Tonnes	*Expected Repetitions*	*Load in Tonnes*	*Expected repetitions*
20	71127	36	35564
18	177820	32	35564
16	569023	28	71128
14	1280303	24	213384
12	2608024	20	177820
10	27622135	16	59274
< 10	3556397	< 16	237093

Table 5.5 Slab Design for Minimum Flexural Strength of 45 kg/cm^2 as per IRC Specifications

Axle load(AL), tonnes	**A.L × 1.2**	**Stress kg/cm^2**	**Stress Ratio**	**Expected repetition, n**	**Allowable Repetitions, N**	**Fatigue life consumed**
Single Axle						
20	24	24.10	0.53	71127	2.16×10^5	0.33
18	21.6	21.98	0.49	177820	1.29×10^5	0.14
16	19.2	19.98	0.44	569023	∞	0.00
14	16.8	17.64	0.39	128030	∞	0.00
Tandem Axle						
36	43.2	19.38	0.43	35560	∞	0.00
						= 0.47

Table 5.6 Slab Design for Flexural Strength of 50.59 kg/cm² (FR10, W/C=0.4)

Flexure Strength = 50.59 kg/cm² Slab thickness = 30 cm						
Axle load(AL), tonnes	**A.L × 1.2**	**Stress kg/cm²**	**Stress Ratio**	**Expected repetition, n**	**Fatigue life, N**	**Fatigue life consumed**
Single Axle						
20	24	28	0.55	71127	124000	0.573
18	21.6	25.5	0.50	177820	762000	0.233
16	19.2	23.2	0.45	569023	62790000	0.009
14	16.8	19.8	0.39	128030	∞	0.00
Tandem Axle						
36	43.2	19.38	0.39	35560	∞	0.00
						= 0.81

Table 5.7 Slab Design for Flexural Strength of 49.77 kg/cm² (FR20, W/C=0.4)

Flexure Strength = 49.77 kg/cm² Slab thickness = 31 cm						
Axle load(AL), tonnes	**A.L × 1.2**	**Stress kg/cm²**	**Stress Ratio**	**Expected repetition, n**	**Fatigue life, N**	**Fatigue life consumed**
Single Axle						
20	24	26	0.52	71127	326000	0.218
18	21.6	24.8	0.49	177820	1287000	0.138
16	19.2	22	0.44	569023	∞	0.00
14	16.8	20	0.40	128030	∞	0.00
Tandem Axle						
36	43.2	20.7	0.41	35560	∞	0.00
						= 0.35

Table 5.8 Analysis of saving of materials for designed flexure of 45 kg/cm²

w/c = 0.40	*Pavement Thickness*	*Saving in Pavement Thickness*	*Saving of concrete per k.m. length (2-lanes)*	*Saving in Cement Content*	*Saving in Fine Aggregate*	*Saving in Coarse Aggregate*
For flexure 45 kg/cm²	330 mm					
10% F.A. [FR10]	300 mm	30mm	225 m^3	87750 kg +87750 kg =175500 kg	160425 kg	242775 kg
20% F.A [FR20]	310 mm	20mm	150 m^3	58500 kg +181350 kg =239850 kg	106950 kg	161850 kg

Table 5.9 Slab Design for Minimum Flexural Strength of 50 kg/cm² (W/C = 0.35)

Flexure Strength = 50 kg/cm² Slab thickness = 30 cm						
Axle load(AL), tonnes	**A.L × 1.2**	**Stress kg/cm²**	**Stress Ratio**	**Expected repetition, n**	**Fatigue life, N**	**Fatigue life consumed**
Single Axle						
20	24	28	0.56	71127	941000	0.075
18	21.6	25.5	0.51	177820	485000	0.366
16	19.2	23.2	0.46	569023	14335000	0.039
14	16.8	19.8	0.39	128030	∞	0.000
Tandem Axle						
36	43.2	19.38	0.43	35560	∞	0.000
						= 0.48

Table 5.10 Slab Design for Flexural Strength of 57.64 MPa (FR10, W/C=0.35)

Flexure Strength = 57.64 kg/cm² Slab thickness = 28cm						
Axle load(AL), tonnes	**A.L × 1.2**	**Stress kg/cm²**	**Stress Ratio**	**Expected repetition, n**	**Fatigue life, N**	**Fatigue life consumed**
Single Axle						
20	24	30.5	0.53	71127	229000	0.310
18	21.6	28.4	0.49	177820	1287000	0.138
16	19.2	25.6	0.44	569023	∞	0.000
14	16.8	22.2	0.39	128030	∞	0.000
Tandem Axle						
36	43.2	23	0.43	35560	∞	0.000
						= 0.44

Table 5.11 Slab Design for Flexural Strength of 54.65 MPa(FR20, W/C=0.35)

Flexure Strength = 54.65 kg/cm² Slab thickness = 29 cm						
Axle load(AL), tonnes	**A.L × 1.2**	**Stress kg/cm²**	**Stress Ratio**	**Expected repetition, n**	**Fatigue life, N**	**Fatigue life consumed**
Single Axle						
20	24	29	0.53	71127	229000	0.310
18	21.6	26.7	0.48	177820	2400000	0.074
16	19.2	24.4	0.44	569023	∞	0.000
14	16.8	20.8	0.38	128030	∞	0.000
Tandem Axle						
36	43.2	22.3	0.40	35564	∞	0.000
						= 0.38

Table 5.12 Analysis of saving of materials for designed flexure of 50 kg/cm^2

w/c = 0.35	*Pavement Thickness*	*Saving in Pavement Thickness*	*Saving of concrete per k.m. length (2-lanes)*	*Saving in Cement Content*	*Saving in Fine Aggregate*	*Saving in Coarse Aggregate*
For flexure 50 kg/cm^2	300 mm					
10% F.A. [FR10]	280 mm	20mm	150 m^3	60000 kg +84000 kg = 144000 kg	108900 kg	164700 kg
20% F.A [FR20]	290 mm	10mm	75 m^3	30000 kg +174000kg =204000kg	54450kg	82350 kg

Table 5.13 Slab Design for Minimum Flexural Strength of 55 kg/cm^2 as per IRC Specifications

Flexure Strength = 55.00 kg/cm^2 Slab thickness = 29 cm						
Axle load(AL), tonnes	**A.L × 1.2**	**Stress kg/cm^2**	**Stress Ratio**	**Expected repetition, n**	**Fatigue life, N**	**Fatigue life consumed**
Single Axle						
20	24	29	0.52	71127	326000	0.218
18	21.6	26.7	0.48	177820	240000	0.740
16	19.2	24.4	0.44	569023	∞	0.000
14	16.8	20.8	0.37	128030	∞	0.000
Tandem Axle						
36	43.2	22.3	0.40	35564	∞	0.000
						= 0.95

Table 5.14 Slab Design for Minimum Flexural Strength of 60.55 MPa (FR10, W/C=0.3)

Flexure Strength = 60.55 kg/cm² Slab thickness = 27 cm						
Axle load(AL), tonnes	**A.L × 1.2**	**Stress kg/cm²**	**Stress Ratio**	**Expected repetition, n**	**Fatigue life, N**	**Fatigue life consumed**
Single Axle						
20	24	32.5	0.53	71127	229000	0.310
18	21.6	29.7	0.49	177820	1287000	0.138
16	19.2	27	0.44	569023	∞	0.000
14	16.8	22.5	0.37	128030	∞	0.000
Tandem Axle						
36	43.2	25	0.41	35564	∞	0.000
						= 0.44

Table 5.15 Slab Design for Minimum Flexural Strength of 57.03 MPa (FR20, W/C=0.3)

Flexure Strength = 57.03 kg/cm² Slab thickness = 28cm						
Axle load(AL), tonnes	**A.L × 1.2**	**Stress kg/cm²**	**Stress Ratio**	**Expected repetition, n**	**Fatigue life, N**	**Fatigue life consumed**
Single Axle						
20	24	30.5	0.53	71127	229000	0.310
18	21.6	28.4	0.50	177820	762000	0.233
16	19.2	25.6	0.44	569023	∞	0.000
14	16.8	22.2	0.38	128030	∞	0.000
Tandem Axle						
36	43.2	23	0.43	35560	∞	0.000
						= 0.54

Table 5.16 Analysis of saving of materials for designed flexure of 55 kg/cm^2

w/c = 0.30	***Pavement Thickness***	***Saving in Pavement Thickness***	***Saving of concrete per k.m. length (2-lanes)***	***Saving in Cement Content***	***Saving in Fine Aggregate***	***Saving in Coarse Aggregate***
For flexure 55 kg/cm^2	290 mm					
10% F.A. [FR10]	270 mm	20mm	150 m^3	61500 kg +83025kg =144525kg	111000 kg	167700 kg
20% F.A [FR20]	280 mm	10mm	75 m^3	30700 kg +172200kg =202900kg	55500kg	83850 kg

CHAPTER-6

CONCLUSIONS

6.1 GENERAL

The present study was undertaken to investigate the effect of partial replacement of cement with fly ash and rice husk ash on compressive strength and flexural strength of concrete mix. Cement was partially replaced by fly ash at three different levels of replacement i.e. 10%, 20% and 30% and same with rice husk ash as well as with combined replacements of fly ash and rice husk ash. Tests were performed after 28 days of curing of concrete. Cubes and beams were prepared for determining compressive strength and flexural strength of concrete with different water-cement ratio as 0.30, 0.35 and 0.40 for min required flexural strengths 5.5 N/mm^2 5 N/mm^2 4.5N/mm^2 respectively. Super-plasticizer was used in all the mixes at 1% level by weight of cementitious material.

6.2 CONCLUSIONS

From the experimental results, the conclusions of compressive strength, flexural strength and the pavement quality concrete slab thickness are concluded as under:

6.2.1 COMPRESSIVE STRENGTH

- The mixes with only fly ash replacement has a lesser rate of increase in strength from 7days to 28 days despite the fact that they have high initial strength, than the mixes with rice husk ash replacement only. The mixes with the inclusion of both rice husk ash and fly ash as replacement material show the highest rate of increase of compressive strength for all water to cement ratios which indicates indicates that pozzolanic activity initiates early for such mixes.
- Concrete mix with up to 30% percent replacement of cement with fly ash for all water-cement ratios have higher compressive strengths than minimum required as per MoRT&H specifications.
- Concrete mixes with 10% replacement of rice husk ash in w/c= 0.3 have higher compressive strengths than minimum required as per MoRT&H specifications.
- Concrete mixes with combined replacement of 10% each of fly ash and rice husk ash in w/c= 0.3 showed higher compressive strengths than minimum required as per MoRT&H specifications.

6.2.2 FLEXURAL STRENGTH

- The mixes containing only fly ash could achieve 85 to 95% of the control strength, whereas, the mixes containing only 30% rice husk as replacement achieved only 55% of the target controlled strength.
- Fly ash up to 20% replacement for all the water-cement ratios showed higher flexural strengths than minimum required flexural strengths as per PQC design standards. Thus, 20% cement replacement by fly ash can be used in designing pavement quality concrete mixes with significant saving in cost.
- Partial replacement of cement along with rice husk ash does not significantly contribute to gain in flexural strengths for all the replacement levels and for all water to cement ratios.
- Mixes with combination of fly ash and rice husk ash were unable to achieve desired flexural strengths.

6.3 SAVING OF MATERIALS IN DESIGN OF SLAB THICKNESS PAVEMENT

According to the results of the study, compiled for different mixes incorporating fly ash and rice husk ash in the figures 6.1 and 6.2, other than nominal mix, there is a noticeable change (i.e. decrease in material usage or saving of materials on economical basis) of following materials:

- Concrete Pavement Thickness
- Cement content (kg)
- Fine Aggregate (kg)
- Coarse Aggregates (kg)

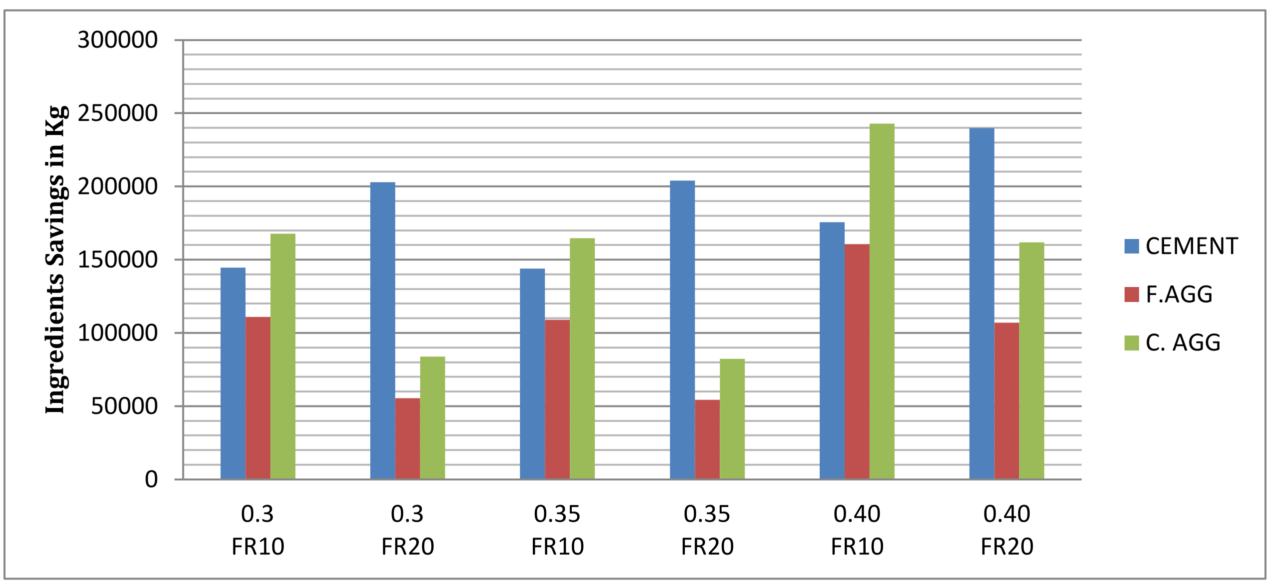

Fig 6.1 Saving of ingredients in Kg.

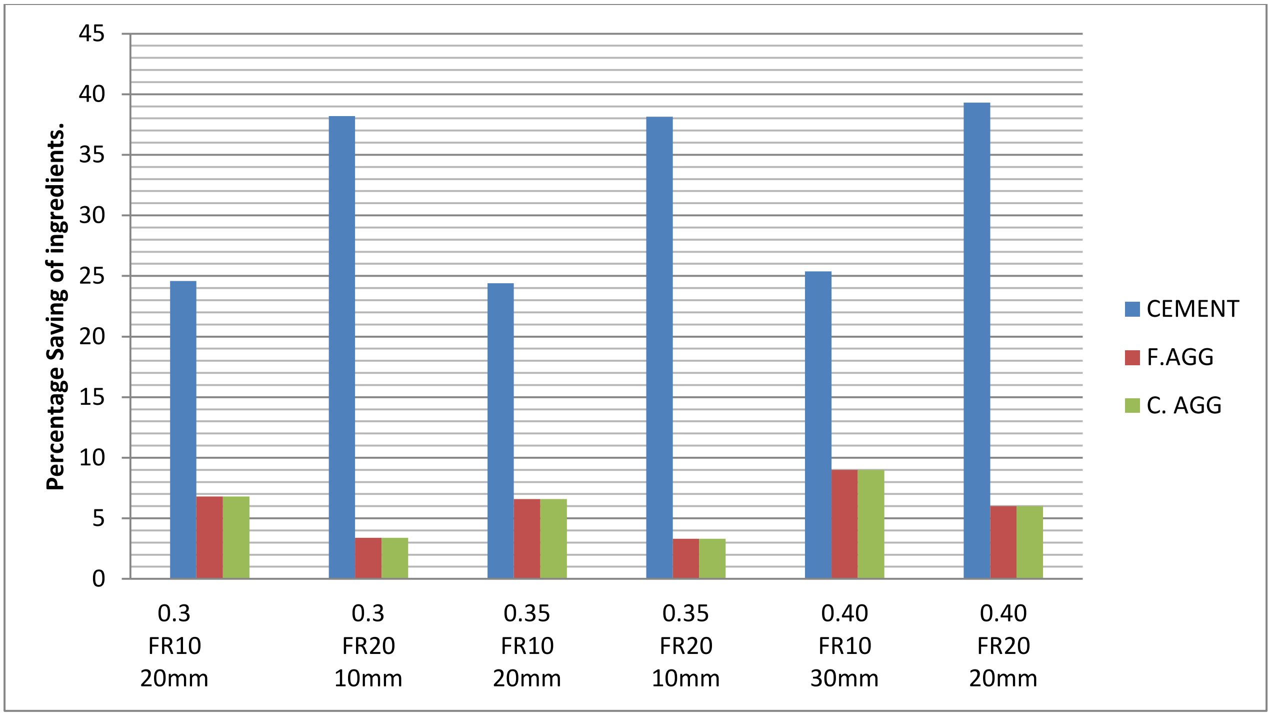

Fig 6.2 Percentage savings of ingredients

A 10 to 30mm saving in the thickness of the quality concrete pavement containing 10 to 20% fly ash as cement replacement is achieved. The most economic pavement design was achieved by replacing 20% fly ash in minimum required 4.5MPa flexural strength design using w/c of 0.40.

CHAPTER- 7
REFERENCES

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