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Late Sri Gokaraju Rangaraju Garu

Great Visionary and our Guiding Spirit

Director, GRIET

I congratulate the Department of Civil Engineering for conducting National Conference on Recent Innovations in Civil Engineering (RICE 2017) on 15-16 December 2017.

I wish them all the best.

Principal, GRIET

We feel privileged to conduct a two day National Conference on "Recent Innovations in Civil Engineering (Rice 2017)" on 15-16 December 2017 at GRIET. It is a moment of pride for our Civil Engineering Department, youngest department at GRIET, to take initiative to cover every aspect, potential for sustainable environment, as part of various themes under the Conference. We expect the participants to derive quality content as well as contentment from these interactions, thus enriching the academic circles for the growth and sustenance of society for generations to come. I wish the conference a grand success.

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The Department of Civil Engineering is established in the year 2008, with an intake of sixty students which is further increased to 120 students from the academic year 2009. It is a fast growing discipline in tune with the infrastructure growth. The department has Master's programme in Structural Engineering, established in the year 2014 with an intake of eighteen students. The department has well equipped laboratories with an emphasis on practical skills and fundamentals. The Department has experienced and well talented faculty which includes six doctorates.

About RICE 2017

National Conference on Recent Innovations in Civil Engineering

RICE 2017 held during December 15 – 16, 2017 in Department of Civil Engineering, Gokaraju Rangaraju Institute of Engineering and Technology, Hyderabad, Telangana, India, helps to bring together researchers, academicians and Industrial experts in the field of Civil Engineering to a common forum.

The primary goal of the conference is to promote research and development activities in Engineering and Technology. Secondly, it provides a platform to exchange the knowledge and scientific information amongst researchers, academicians, engineers, students and practitioners.

This conference will be held every year to make it an ideal workstation for researchers, academicians, engineers and students to share views and research and field experiences on recent innovations in relevant areas of Civil Engineering.

I sincerely thank the participants of the conference for contributing their research findings. Department of Civil Engineering is grateful to the Management of GRIET for motivating us to conduct this conference.

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Professor of Civil Engineering & OSD to Vice-Chancellor JNTUH

Valedictory Ceremony:

Dr. Pradeep Kumar Ramancharla

Professor of Civil Engineering & Registrar IIIT Hyderabad

KEYNOTE LECTURES

1. Dr. G K Viswanadh, Professor of Civil Engineering, JNTUH CEH
2. Dr. M V Seshagiri Rao, Professor of Civil Engineering, CVR College of Engineering, Hyderabad
3. Dr. N Darga Kumar, Asst. Professor & HOD of Civil Engineering, JNTUH Manthani
4. Ar. Er. S P Anchuri, Chief Consultant, Anchuri and Anchuri, Vice President-S, IAStructE, Hyderabad
5. Dr. R Pradeep Kumar, Professor of Civil Engineering & Registrar IIIT Hyderabad

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Chair of Session 1: Dr. N Sanjeev, Professor of Civil Engineering, GRIET

Chair of Session 2: Dr. Md. Hussain, Professor of Civil Engineering, GRIET

Chair of Session 3: Dr. M V S Rao, Professor of Civil Engineering, CVR COE

Chair of Session 4: Dr. G V V Satyanarayana, Professor of Civil Engineering, GRIET

PROGRAM

Day 1 (15-12-2107) (Friday)

9.30 am -10.00 am	Inauguration Ceremony
10.00 am – 11.00 am	Keynote Lecture by
	Dr. G K Viswanadh, Professor of Civil Engineering, JNTUH CEH
11.00 am – 11.15 am	Tea Break
11.15 am – 12.45 pm	Keynote Lecture by
	Dr. M V Seshagiri Rao, Professor & Dean, Department of Civil Engineering, CVR College of Engineering, Hyderabad
12.45 pm – 2.00 pm	Lunch
2.00 pm – 3.30 pm	Keynote Lecture by
	Dr. N Darga Kumar, Asst. Professor & HOD of Civil Engineering, JNTUH Manthani
3.00 pm – 3.15 pm	Tea Break
3.15 pm – 5.00 pm	Paper Presentation (Parallel Session -1&2)
	Chair of Session 1: Dr. N Sanjeev, Professor of Civil Engineering, GRIET
	Chair of Session 2: Dr. Md. Hussain, Professor of Civil Engineering, GRIET

Day 2 (16-12-2107) (Saturday)

9.30 am -11.00 am	Keynote Lecture by
	Ar. Er. S. P. Anchuri, Chief Consultant, Anchuri and Anchuri, Vice President-S, IAStructE, Hyderabad
11.00 am – 11.15 am	Tea Break
11.15 am – 1.00 pm	Paper Presentation (Parallel Session -3&4)
	Chair of Session 3: Dr. M V S Rao, Professor of Civil Engineering, CVR COE
	Chair of Session 4: Dr. G V V Satyanarayana, Professor of Civil Engineering, GRIET
1.00 pm – 2.00 pm	Lunch
2.00 pm – 3.30 pm	Keynote Lecture by
	Dr. R Pradeep Kumar, Professor of Civil Engineering & Registrar IIIT Hyderabad
3.00 pm – 3.15 pm	
3.15 pm onwards	

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FUZZY MODEL DEVELOPMENT IN GREEN BUILDING MATERIAL SELECTION

Akula Prakash[1], Naveen Reddy K[2], Tejeshwar Reddy P[3] and Gunashekar M[4]

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[2,3,4] Students, Department of Civil Engineering, GRIET, Hyderabad – 500090.

Abstract

Fuzzy logic is a moderately new method for tackling building control issues. This method is regularly used to execute frameworks beginning from simple, small or even inserted up to monster. Fuzzy set hypothesis has been usual model frameworks that are challenging to diagram precisely. As a system, fuzzy set hypothesis joins imprecision and subjectivity into the model definition and arrangement prepare. Probabilistic techniques are being utilized progressively in development building. However, when a parameter is communicated in phonetic as opposed to numerical terms, traditional likelihood hypothesis neglects to consolidate the data. The etymological factors can be converted into scientific measures utilizing fuzzy set and framework hypothesis. A development administration issue i.e., estimation of term of an action, is unraveled utilizing this hypothesis. The proposed procedure is not touchy to little varieties in the participation values. This is an extremely alluring property. In any case, the strategy is delicate to the decision of the fuzzy relations. The instability in the fuzzy relations can be demonstrated alongside different wellsprings of vulnerability. The mean and difference of the parameters required in the issue under thought are evaluated here utilizing another technique. The strategy boosts the result of the whole of the enrollment relationship for a specific recurrence of event and the comparing recurrence of event. One of the primary favorable circumstances of the proposed method is that it can be effortlessly actualized in existing PC programs for venture planning. In Green building planning viewpoints, material choice assumes a significant part in accomplishing the building guidelines; it is possible that it might be of green or non-green materials. Keeping in mind the end goal to accomplish perspective consummately, essential Fuzzy logic approach is to be actualized in order to choose the best option among different accessible materials. This should be possible by dispensing the essential enrollment values based upon the past encounters and positioning them in a request in view of our development necessities all the while. By utilizing this approach, we can likely choose the best material in view of both subjective and quantitative perspective.

***Keywords**: Green Building material; Fuzzy set theory; Fuzzy Model Development*

1.0 Introduction

Nowadays, individuals give cautious thought to biological security; in this way develop another example called Green Buildings. It's not about the shading green, but instead has something to do with another basic thought. The "Green Building" is an interdisciplinary subject, where the green building thought joins countless, fragments and philosophy which veer to a couple subtopics that weaved to outline the green building thought. Generally, the green building is thought to be a characteristic fragment, as the green building materials are created from neighborhood eco-sources, i.e. biologically cordial materials, which are then used to make an eco-advancement subject to an eco-arrange for that give a strong situation in view of the social and compositional legacy being developed while ensuring security of trademark resources.

Green building implies both a structure and the using of systems that are earth competent and resource capable all through a building's life-cycle: from alluding to blueprint, improvement, operation, upkeep, overhaul, and annihilation. Toward the day's end, green building plot incorporates finding the amicability among homebuilding and the supportable environment.

This requires close coordinated effort of the setup gather; the artists, the authorities, and the client at all suspect stages.

2.0 Green Building Material

Green Building material is a sort of building material which would not make harm human body. As it were, Green Building material is of low-contamination, low-stench building material. The toxic substance in the building material would spread through inside design and connect to the indoor environment. To the individuals who stay inside for quite a while, because of long introduction to this sort of lethal environment, there is a greatly negative effect on human body. To recognize the advantageous building material that shields individuals from toxin and risk, evaluation of building material is for the most part in light of indoor development material and beautification material. Those building material which is met all requirements for the assessing standard would be given the marker called "Green Building Material."

There are three noteworthy points of interest:

- Diminishes the biological load and vitality utilization of the substance blend material.
- Diminishes the generation of vitality and asset utilization by reusing.
- Utilizing common material and low unpredictable natural building material may lessen the peril of combination material.

Need of Green Building Material

The inside adornment material and floor surface material ought to be Green Building material. As indicated by research examines, the decide that rate of Green Building material ought to take up no less than 30 percent of the aggregate inside adornment material in addition to floor surface material. In this way, the utilization of Green Building material is by all methods important to Green Buildings.

Categories of Green Building material:

Green Building material is isolated into four sorts: the natural building material, the sound building material, the elite building material, and the reusing building material.

Biological Green Building material:

In correlation with other building material, the Green Building material is the slightest prepared, subsequently the most normal, biological material; it expends the minimum vitality and asset

Solid Building material:

Solid building material is of low contamination, low request, and low physiological peril. It points predominantly at low unpredictable natural mixes, for example, water ecological neighborly paint, water-wood paint, and epoxy gum paint.

Superior Building material:

Superior building material can vanquish the inadequacy of customary building material, enhancing quality execution. Elite soundproof Green Building material can adequately counteract clamor impacts on the personal satisfaction

Reusing Building material:

Reusing building material is low handled, low vitality expending, low carbon dioxide release, low contamination release, actually disintegrated, and reusable. Blended material reusing building material alludes to wood or stone building material blended with waste plastics, glass, and so forth, which creates new building material, for example, impersonation wood and water porous blocks.

3.0 Fuzzy Set

In traditional set hypothesis, a set is characterized as a gathering of articles having a general property, e.g., a class or an arrangement of solid blends, or a gathering of worker. A representative works either as an agreement premise or for perpetual. On the off chance that he chips away at contract premise, he is said to have a place with the gathering of agreement individuals, or to have a participation of 1 in the class of worker. On the off chance that he is not chipping away at an agreement premise, he then has no participation in the class or his enrollment review is zero. On the off chance that this idea is reached out to grasp another sort of set, say a subset of "extremely experienced" contract worker. A tasteful response to this question is troublesome one as the class of "extremely experienced" representative is not a set in the established sense, but rather has a place witha fuzzy, not freshly characterized sort. The meaning of "extremely experienced" may include a range of human observations and the class of "exceptionally experienced" contractual worker representative is in this manner said to speak to a fuzzy set.

In science, fuzzy sets will be sets whose components have degrees of participation. In established hypothesis, fuzzy set allows the continuous evaluation of enrollment of components in a set, this is portrayed with the guide of a participation work esteemed in the genuine unit interim of [0, 1]

4.0 Related Literature

The first fuzzy set hypothesis by Zadeh and Goguen demonstrate the expectation of the creators to sum up the established idea of a set and a suggestion to oblige fuzziness as in it is contained in human dialect i.e., in human judgment, assessment, and choices. Presenting the fuzzy set hypothesis by Zadeh in 1965 opened promising new skylines to various logical zones such as project scheduling. fuzzy hypothesis, with assuming imprecision in choice parameters and using mental models of specialists is a way to deal with adjust planning models into reality

Zadeh composes: 'The thought of a fuzzy set gives a helpful purpose of flight for the development of a theoretical structure which parallels in many regards the system utilized as a part of the instance of standard sets, yet is more broad than the last mentioned and conceivably may demonstrate to have a much more extensive extent of materialness, especially in the fields of example grouping and data handling. Basically, such a system gives a characteristic method for managing issues in which the wellspring of imprecision is the nonappearance of forcefully characterized criteria of class enrollment as opposed to the nearness of irregular factors.

To this end, a few strategies have been created amid the last three decades, the acknowledgment of this hypothesis developed gradually in the 1970s of the most recent century. In the second 50% of the 1970s, in any case, the principal effective useful applications in the control of innovative procedures by means of fuzzy run based frameworks, called fuzzy control (warming frameworks, concrete industrial facilities, and so forth.), and supported the enthusiasm for this region extensively. Effective applications, especially in Japan, in clothes washers, camcorders, cranes, metro trains, thus on activated further intrigue and research in the 1980s so that in 1984 as of now roughly 4000 distributions existed

The first strategy called FPERT (Fuzzy PERT), was proposed by Chanas and Kamburowski (1981). They introduced the venture fulfillment time as a fuzzy set in the time space. Gazdik (1983) built up a fuzzy system of an a priori unknown venture to evaluate the action term, and utilized fuzzy arithmetical administrators to compute the span of the venture and its basic way. This work is called FNET

The lists of sources by Zimmerman (1983) and Lai and Hwang (1994) survey the writing on fuzzy sets in operations inquire about and fuzzy different target basic leadership separately. Maiers and Sherif (1985) audit the writing on fuzzy mechanical controllers and give a list of utilizations of fuzzy set hypothesis to twelve branches of knowledge including basic leadership, financial aspects, designing and operations examine.

Karwowski and Evans (1986) distinguish the potential utilizations of fuzzy set hypothesis to the accompanying regions of generation administration: new item improvement, offices area and design, creation planning and control, stock administration, quality and money saving advantage investigation. Karwowski and Evans distinguish three key reasons why fuzzy set hypothesis is important to creation administration explore. To start with, imprecision and ambiguity are natural to the chief's mental model of the issue under review. In this manner, the chief's involvement and judgment might be utilized to supplement built up speculations to cultivate a superior comprehension of the issue. Second, in the generation administration environment, the data required to figure a model's target, choice factors, imperatives and parameters might be obscure or not absolutely quantifiable. Third, imprecision and unclearness thus of individual predisposition and subjective assessment may additionally hose the quality and amount of accessible data.

Fuzzy enhancement and operations research was given by Negoita (1981), Zimmerman (1983) and Kaufmann (1986). A thorough survey of fuzzy master frameworks in modern designing, operations research, and administration science was given by Turksen (1992).

Han et al. (1994) consider the n work, single machine most extreme delay planning issue with fuzzy due dates and controllable machine speeds. The goal is to locate an ideal calendar and employment astute machine speeds which limit the aggregate total of costs related with disappointment of all occupation culmination times and occupation insightful machine speeds. A direct participation capacity is utilized to depict the level of fulfillment concerning work culmination times. Incremental machine speed expenses are characterized as the cost related with electrical power or potentially work. A polynomial time calculation is utilized to acquire arrangements.

Ishii and Tada (1995) display an effective calculation for deciding non commanded plans for the n work single machine booking issue when a fuzzy priority relationship exists between occupations. The bi-criteria target of the calculation is to limit normal occupation delay while boosting the insignificant fulfillment level as for the fuzzy priority connection. The unpredictability of the calculation is contemplated and bearings for future research on employment shop planning with fuzzy priority relations are distinguished.

Roy and Zhang (1996) build up a fuzzy element booking calculation (FDSA) for the n work m machine work shop planning issue. Fuzzy rationale is utilized to consolidate customary occupation shop planning tenets to shape total heuristic guidelines. Participation capacities for occupations, measuring plans for need rules utilized in FDSA, and the fuzzy administrators required in playing out the fuzzy changes are characterized. Recreation tests including 20 occupations and up to 15 machines are directed. Routine need rules (FCFS, SPT, EDD, and CR) are contrasted with three fuzzy heuristic guidelines under FDSA for the accompanying execution measures: greatest and mean stream time, most extreme and mean occupation delay, and the quantity of late employments. Comes about show that the fuzzy heuristic guidelines perform well in the employment shop issues examined

Other scientists, for example, Kuchta (2001), Yao and Lin (2000), Chanas and Zielinski (2001), and, Oliveros and Robinson (2005), utilizing fuzzy numbers, introduced different strategies to acquire fuzzy basic ways and basic exercises and action delay. Past work on system booking utilizing fuzzy hypothesis gives techniques to planning ventures.

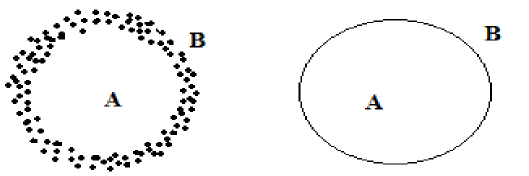
5.0 Fuzzy Model Development

Give X a chance to be a universe, or an arrangement of components x's, and let A be a subset of X. Every component "x" is related with an enrollment incentive to the subset A. In the event that A is an standard, non-fuzzy or fresh set, then the membership function is given by

$$\mu A(x) = \begin{cases} 1 & \text{; if x belongs to A} \\ 0 & \text{; if x does not belong to A} \end{cases}$$

In the above condition, there are just two conceivable outcomes for a component x, either being an individual from A. For this situation, A has sharp limits. Then again, if the membership function is permitted to take values in the interim (0, 1), A is known as a fuzzy set. In fuzzy set, A does not have sharp limits and the enrollment of x to A is fuzzy.

The documentation of fuzzy sets, concede fractional participation. A fuzzy set is subsequently a net with individuals having a continuum of evaluations of participation, from 0 to 1. Equation 1 represents the fundamental idea of fuzzy sets by their membership definition.



FUZZY SET (M) CRISP SET (N)
$\mu M (A) = 0 \text{ to } 1$ $\mu N (A) = 1$

$A = \{ x, \mu A(x) \}, x \in A \text{ and } A \subset U;\ 0 \le \mu A(x) \le 1$ -- (1)
U = Union Matrix

For example, let x be the level of experience of labor which may range from excellent experience i.e., x =1.0, to “never been to a construction site,” i.e., x= 0. By dividing the range of labor experience into increments of 0.1, “short experience,” A, as a linguistic variable and can be expressed as Equation 2.

A= x_1=0.5 / $\mu_A(x_1)$ = 0.1, x_2=0.4 / $\mu_A(x2)$ = 0.2, x_3=0.3 / $\mu_A(x_3)$ = 0.5, x_4 =0.2 / $\mu_A(x_4)$ = 0.6, x_5=0.1 / $\mu_A(x_5)$ = 0.9, x_6=0 / $\mu_A(x_6)$ = 1.0 -- (2)

Or in short, it can be expressed as

A = [0.5/0.1, 0.4/0.2, 0.3/0.5, 0.2/0.6, 0.1/0.9, 0.0/1]

6.0 Research Methodology:

The accompanying strategy is created to choose the best material among the different accessible materials utilizing fuzzy logic approach:

- Identify the applicable qualitative and quantitative components.
- Allocate membership values for such components from specialists.
- Develop a position network with the assistance of apportioned membership values for every material
- Calculate the predominance lattice qualities to speak to the dominances of materials.
- Sum up the rows and columns.
- Select the materials with greatest column sum and low row sum.
- Rank the materials which come about the best option among the available materials based upon the best qualitative and quantitative elements.

Example for Position Matrix:

	M_1	M_2	•	•	•	M_n
N_1	D_{11}	D_{12}	•	•	•	D_{1n}
N_2	D_{21}	D_{22}	•	•	•	D_{2n}
N_3	D_{31}	D_{32}	•	•	•	D_{3n}
•	•	•	•	•	•	•
•	•	•	•	•	•	•
•	•	•	•	•	•	•
•	•	•	•	•	•	•
N_m	D_{m1}	D_{m2}	•	•	•	D_{mn}

Where,
$M_1, M_2, M_3, M_4 \ldots M_n$ are various available materials
$N_1, N_2, N_3, N_4 \ldots N_m$ are various quantitative and qualitative parameters for that material

$D_{11}, D_{12}, D_{13}, D_{14} \ldots D_{1n}$ and $D_{11}, D_{21}, D_{31}, D_{41} \ldots D_{n1}$
Membership values allocated by experts for such parameter to that particular material

Conversion of Position matrix to Dominance Matrix:

	M_1	M_2	•	•	•	M_n
M_1	d_{11}	d_{12}	•	•	•	d_{1n}
M_2	d_{21}	d_{22}	•	•	•	d_{2n}
M_3	d_{31}	d_{32}	•	•	•	d_{3n}
•	•	•	•	•	•	•

$$
\begin{array}{c|cccccc}
\cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot \\
\cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot \\
\cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot \\
M_m & d_{m1} & d_{m2} & \cdot & \cdot & \cdot & d_{mn}
\end{array}
$$

Where,

d_{11}, d_{12}, d_{13}, d_{14}...... d_{1n} and d_{11}, d_{21}, d_{31}, d_{41},... d_{n1} are dominance values calculated for that particular material based upon the qualitative and the quantitative aspects.

d_{12} indicates how many times column 2 is dominating 1
d_{23} indicates how many times column 3 is dominating 2
d_{21} indicates how many times column 1 is dominating 2
d_{32} indicates how many times column 2 is dominating 3

Whereas,

$d_{11} = d_{22} = d_{33} = \ldots\ldots = d_{(n-2)(n-2)} = d_{(n-1)(n-1)} = d_{nn} = 0$ Because there is no self dominance among themselves

Note:

- Obtained dominance matrix should be always a square matrix
- If row sum is not minimum while developing a dominance matrix, consider only maximum column sum
- If both of the column sums or row sums are equal in number, then consider any material arbitrarily.

Material determination criteria:

Material determination handle assumes a vital part in the building development rehearses. Determination criteria rely on upon the cost and amount, as well as rely on upon the subjective perspectives like usefulness, material accessibility, client appearance, development rehearses and their troubles. The way toward selecting materials is in this manner a basic leadership prepare including an extensive variety of criteria for which the data can be uncertain and objective. The material determination prepares including various criteria have been performed methodically by utilizing essential fuzzy set operations in this proposition.

In this process, I have taken different building materials, for example, Green building materials, Non Green building materials and in addition recycled materials. Utilizing fundamental fuzzy set operations, best option can be picked based upon qualitative and the quantitative viewpoints regardless of the cost of that material.

In this procedure, I have considered the choice procedure of Concrete which gathers the real bit in the development business and accepted that Fly Ash Concrete (15% Fly Ash) and Ground Granulated Blast Furnace Slag Cement Concrete (20% GGBS) as Green building materials, Temperature Controlled Concrete and Regular Concrete as Non Green building materials, Recycled Concrete (Waste Concrete gathered from another Constructed site and afterward it is reused and utilized for readiness of new cement for new Construction endless supply of appropriate admixtures) is expected as Recycled material as appeared in Figure 1.

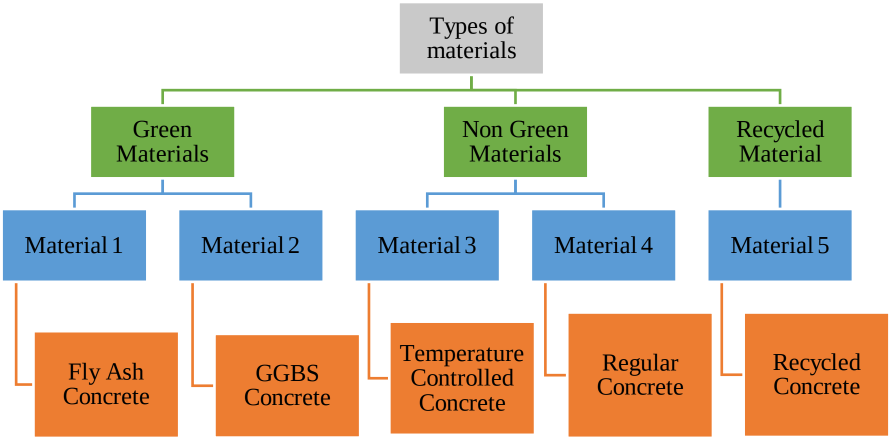


Figure 1: Categorization of available materials

Subsequent to selecting the materials in view of the prerequisite, consider the qualitative and also the quantitative angles for every material, which may change the choice criteria. In this theory, I have accepted different parameters, for example nature of material, usefulness, material accessibility, client bid, development hones, development challenges and cost of materials.

Membership values are allocated based upon the accessible parameters for every individual component, and they are to be positioned among themselves based upon the prerequisite for the development hones as shown on Table 1. For instance, a material can be picking either by high caliber or else with low quality material, in some cases either with high cost or else with ease material. In any case, utilizing fundamental fuzzy set operations we can interlink the different parameters. For example, material of high caliber with ease or high caliber with high cost individually. To grow such relationship amongst qualitative and the quantitative perspectives, classify the participation values for every last material considering each of the individual viewpoints separately.

Based upon the enrollment values allotted to different materials regarding their qualitative and quantitative parameters, material determination will be finished. These determination criteria is not in light of a solitary parameter, it incorporates every single individual parameter that can be assessed by building up the relationship between them, which should be possible by utilizing essential fuzzy approach where we can classify the participation estimations of the considerable number of materials as for their qualitative classifications with a specific end goal to build up the position framework, and from that point changing over it to the strength network by ascertaining the predominance values which speaks to that predominance of such material when contrasted with different materials separately.

Table 1: Membership values allocated for various materials with their rankings

Membership Value / Ranking Allocated					
Criteria	Green Material		Non Green Material		Re-cycled
	M 1	M 2	M 3	M4	M5
Quality	$\frac{0.65}{0.7}$	$\frac{0.74}{1.0}$	$\frac{0.69}{0.9}$	$\frac{0.57}{0.5}$	$\frac{0.60}{0.6}$
Functionality	$\frac{0.79}{0.9}$	$\frac{0.74}{0.7}$	$\frac{0.53}{0.6}$	$\frac{0.46}{0.4}$	$\frac{0.83}{1.0}$
Material Availability	$\frac{0.61}{0.6}$	$\frac{0.55}{0.5}$	$\frac{0.76}{0.8}$	$\frac{0.79}{1.0}$	$\frac{0.65}{0.7}$
Appearance	$\frac{0.58}{1.0}$	$\frac{0.62}{0.8}$	$\frac{0.82}{0.5}$	$\frac{0.76}{0.6}$	$\frac{0.74}{0.7}$
Construction Practices	$\frac{0.78}{0.6}$	$\frac{0.74}{0.7}$	$\frac{0.55}{1.0}$	$\frac{0.43}{0.8}$	$\frac{0.88}{0.5}$
Construction Difficulties	$\frac{0.68}{0.5}$	$\frac{0.57}{0.6}$	$\frac{0.38}{0.8}$	$\frac{0.36}{1.0}$	$\frac{0.43}{0.7}$
Cost Of Material	$\frac{0.68}{0.5}$	$\frac{0.67}{0.6}$	$\frac{0.58}{0.9}$	$\frac{0.62}{0.7}$	$\frac{0.54}{1.0}$

Position matrix:

Tabulate the membership values of each and every element with respect to their corresponding qualitative parameter.

	M1	**M2**	**M3**	**M4**	**M5**
Quality	0.65	0.74	0.69	0.57	0.60
Functionality	0.79	0.74	0.53	0.46	0.83
Material Availability	0.61	0.55	0.76	0.79	0.65
Appearance	0.58	0.62	0.82	0.76	0.74
Construction Practices	0.78	0.74	0.55	0.43	0.88
Construction Difficulties	0.68	0.57	0.38	0.36	0.43
Cost of Material	0.68	0.67	0.58	0.62	0.54

Dominance matrix:

	M1	M 2	M3	M4	M5
M1	0	2	3	2	4
M2	5	0	2	2	4
M3	4	5	0	2	3
M4	5	5	5	0	4
M5	3	3	4	3	0

After developing the dominance matrix using membership values, sum up all the rows and columns of dominance values and choose column with maximum sum and row with minimum sum respectively.

Step 1:

	M1	M2	M3	M4	M5	Sum
M1	0	2	3	2	4	11
M2	5	0	2	2	4	13
M3	4	5	0	2	3	14
M4	5	5	5	0	4	19
M5	3	3	4	3	0	13
Sum	17	15	14	9	15	

Choose the material with maximum column sum and minimum row sum. In the above matrix, Material 1 satisfies such condition, as a result Material 1 will be the best alternative among available materials

Step 2:

	M2	M3	M4	M5	Sum
M2	0	2	2	4	8
M3	5	0	2	3	10
M4	5	5	0	4	14
M5	3	4	3	0	10
Sum	13	11	7	11	

In the above matrix, Material 2 satisfies such condition, as a result Material 2 will be the 2nd best alternative among available materials.

Step 3:

	M3	M4	M5	Sum
M3	0	2	3	5
M4	5	0	4	9
M5	4	3	0	7
Sum	9	5	7	

In the above matrix, Material 3 satisfies such condition, as a result Material 3 will be the 3rd best alternative among available materials.

Step 4:

	M4	M5	Sum
M4	0	4	4
M5	3	0	3
Sum	3	4	

In the above matrix, Material 5 satisfies such condition, as a result Material 5 will be the 4th best alternative among available materials.

Step 5:

	M4	Sum
M4	0	0
Sum	0	

In the above matrix, Material 4 will be the last alternative among available materials

Results and Discussions:

Table 2: Material selection order based on Fuzzy logic approach

S.No	Preference order	Description	Type of material
1	Material 1	Fly Ash Concrete	Green material
2	Material 2	GGBS Concrete	Green material
3	Material 3	Temperature Controlled Concrete	Non Green material
4	Material 5	Recycled Concrete	Recycled material
5	Material 4	Regular Concrete	Non Green material

Despite the fact that we are dispensing higher cost to Green material than alternate materials, best chose elective utilizing essential Fuzzy Logic approach technique is likewise a Green material as appeared in Table 2. It essentially says that material choice won't depend just upon the cost parameter; it likewise incorporates subjective and quantitative viewpoints as talked about before. The designation of enrollment qualities is simply a supposition based upon our prerequisite. As information parameter changes, naturally yield determination criteria will likewise be modified. Material determination request of two people may not correspond with each other; it is absolutely based upon the independently assigned participation values for such materials. So keeping in mind the end goal to assess Fuzzy Logic approach in choice process, ensure that each individual achieves same enrollment values for the accessible materials, generally the examination of the last report may not harmonize with others. Comparative system is connected in determination of different materials.

Conclusions:

- Material choice assumes a vital part in the building outline and possesses a noteworthy bit in the development rehearse. Every material ought to be tried whether that material meets the Green building measures or else; those materials are to be supplanted by some different materials.
- Material determination by Fuzzy Logic approach is simply done on a creative energy premise. Designated enrollment values for those materials are given based upon the individual enthusiasm for such material, and correspondingly rankings are likewise distributed based upon the past encounters, as appeared in Table 1
- Material determination request of two people may not match with each other; it is absolutely based upon the exclusively dispensed participation values for such materials. So as to assess Fuzzy Logic approach in determination handle, ensure that each individual achieves same participation values for the accessible materials; generally the examination

of the last report may not match with others, as appeared in Table 2

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A FEASIBILITY STUDY ON BENEFICIAL USE OF CRUMB RUBBER FOR THE IMPROVEMENT OF CBR VALUE

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Abstract

Use of waste material to stabilize the weak soil is one of the best solution for waste disposal. Tires which are not properly disposed is significantly contribute to fire hazards. Many countries have banned the disposal of the waste tires in sanitary landfills. The use of waste tires as fuel is now prohibited by the Indian Government due to its environmental impact. In this regard, to develop newer application a study has been conducted for the application of crumb rubber for possible use in highway construction. .This paper presents the feasibility study conducted on weak soil to study the effect of crumb rubber on CBR Value with varying percentages. The study revealed that there was a considerable decrease in pavement thickness and improvement in subgrade with addition of shredded tire chips up to 10%.

Keywords*: Shredded tire chips (Crumb rubber powder), CBR Value.*

1.0 Introduction

Disposal of wastes is one of the major problem which has been found a great deal of concern since the sprout of industrialization. This attention is mainly because of scarcity in usable land and increase in industrial activities on the environment. Immense usage of industrial waste in various applications is one of the best alternative to the land disposal. For many years different types of non conventional materials like flyash fibers bio-enzymes etc (Laya N Nayar 2016) have been immensely used to improve the problematic soils. This has given chance to many researches to find an alternative non conventional materials in improving the subgrade property and also to reduce the thickness of pavement. How ever crumb rubber powder have proven to be a best non conventional material which is of major concern in waste disposal (Binod Tiwari,2014). . It was used particularly to improve the shear strength of a locally available soil where friction angle was founded to improve with addition of crumb rubber powder to portage sand increases its shear strength and intial fiction angle increases as shred content was increased (J.Bosscher1996) (Joon C Lee 2007) used

2.0 Material Properties and Methodology

Soil used in the study was locally available soil which has low strength and was dried naturally to remove all the moisture content present in it and was passed through 425 micron sieve before conducting the tests. Crumb rubber was procured from the local market (workshop) for the purposed work which is a recycled rubber from automotive and truck scrap tires. The specific gravity of crumb rubber powder was found to be 0.85. The index and Engineering properties are tabulated in table 1

Locally available soil which was taken to know the variation of CBR Value with addition of crumb rubber powder .By Atterberg limits the soil was classified as MH or OH and the plasticity index is greater than 10% hence in its natural state it cannot be used as subgrade for pavement. Since crumb rubber was locally available to find a suitable use for them it was added to the soil in different percentages and was mixed with varying percentages of crumb rubber powder viz 2%, 6%, 10% & 14% respectively and the behavior was studied by conducting standard proctor test and CBR Test.

Property	value
Specific Gravity	2.58
Coefficient of Uniformity (Cu)	5.65
Coefficient of Curvature (Cc)	1.01
Liquid limit	79%
Plastic limit	20%
Plasticity Index	59%
Soil Classification	MH or OH
Maximum Dry Density	16.5g/cc
Optimum Moisture Content	16%

Fig 2.1 Crumb rubber from cracker mill

3.0 Results and Discussions

Fig 3.1 and 3.2 shows the Dry density of soil with variation in moisture content and penitration of soil against load respectively . From the graph it was observed that for soil without addition of crumb rubber powder the optimum moisture content was found to be 1.65g/cc and 16% respectively. The corresponding CBR value was found to be
4.23%

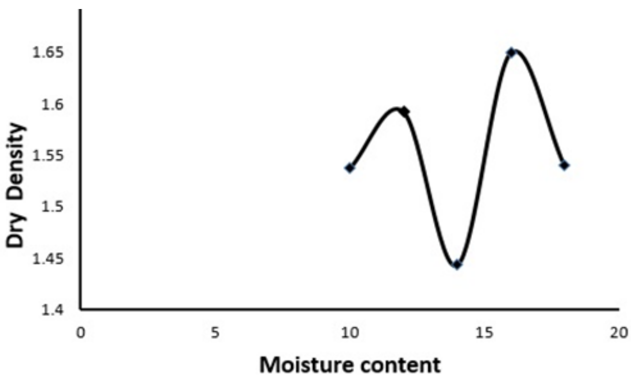

Fig 3.1 :Dry density vs W% for soil without addition of crumb rubber powder

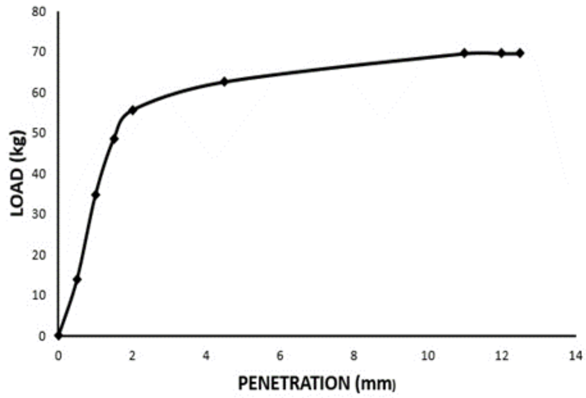

Fig 3.2 : load-penetration curve for soil without addition of Crumb rubber powder

Similarly it can be observed from fig 3.3 and 3.4 that the drydensity of soil after addition of 2% crumb rubber was found to be 1.63g/cc with the same optimum moisture content. The corresponding CBR value for the addition of 2% crumb rubber was found to be 8.62% which is slightly more when compared to the CBR value of soil which was not stabilised with crumb rubber.From this it can be inferred that with addition of shredded tyre chips the CBR value can increased so that thickness of pavement can be decreased considerably .

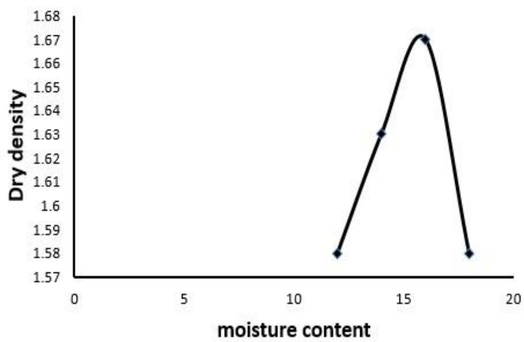

Fig 3.3 :Dry density vs water content for soil without addition of crumb rubber powder

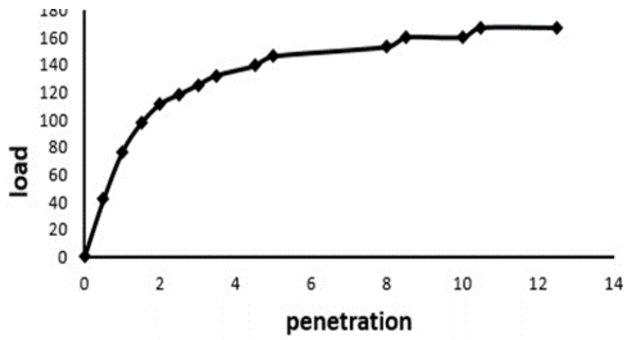

Fig 3.4 : load-penetration curve for soil with 2% Crumb rubber powder

The drydensity -moisture content plot for 6% 10% and 14% is shown in figure 3.5 and 3.7 respectively. Also load penetration plot for 6% 10% and 14% is shown in fig 3.6 ,3.8 and 3.9 respectively . It can be observed from these plots that for soil stabilised with 6% the dry density and OMC was found to be 1.61g/cc and the corresponding OMC was found to be 14%. The corresponding CBR value for 6% , 10% and and 14% was found to be 9.13%

,9.63% and 6.08% respectively . It can be infered from the above analysis that there is a considerable decrease in CBR value for percentage of crumb rubber powder beyond 10% and this may be attributed to the fact that the soil cotaining 10% crumb rubber powder gets maximum stabilisation beyond which even after addition of crumb rubber powder leads to reduction in CBR value and and also maximum dry density respectively

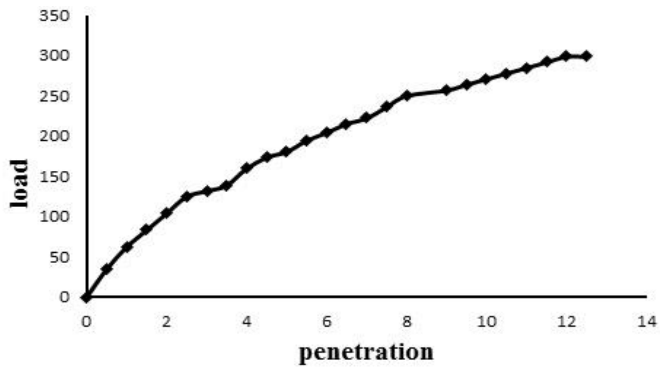

Fig 3.5 : load-penetration curve for soil with 6% Crumb rubber powder

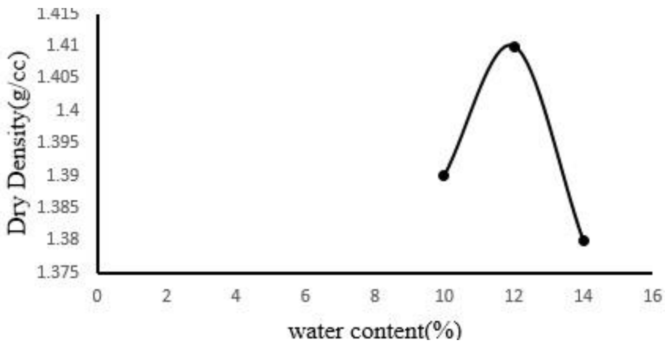

Fig 3.6 :Dry density vs water content for soil With 10% of crumb rubber powder

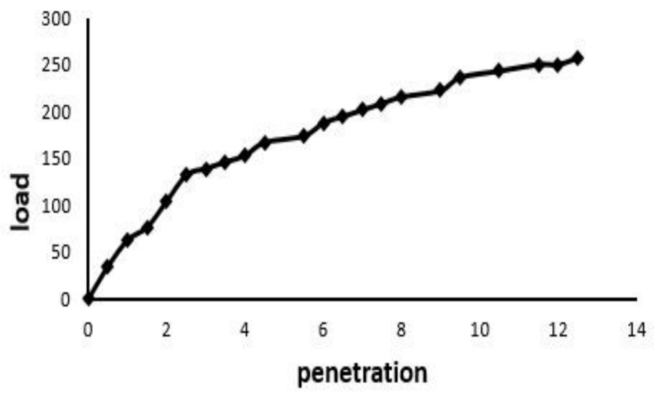

Fig 3.7 load-penetration curve for soil with 10% Crumb rubber powder

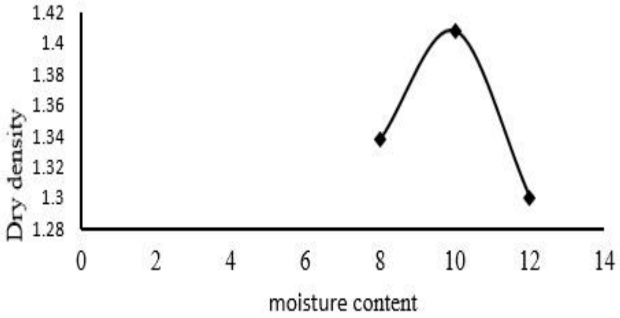

Fig 3.8 :Dry density vs water content for soil With addition of 14% crumb rubber powder

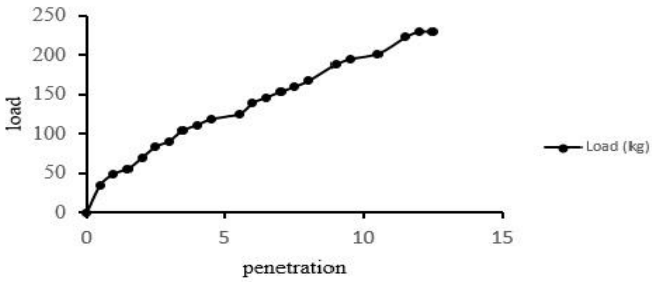

Fig 3.9 : load-penetration curve for soil with 14% Crumb rubber powder

4.0 Conclusions

Based on the analysis on the crumb rubber stabilized soils the following observations and conclusions are drawn:

- Crumb rubber powder mixed with the soil showed improvement in CBR value with its addition up to 10% and there onwards decreased with further increase in crumb rubber powder.
- Percentage improvement in CBR value of soil is 10%. Increase in CBR value reduces the overall thickness of the pavement and the total cost involved in construction of roads.
- There was a considerable increase in CBR value with addition of crumb rubber powder which intern leads to reduction in thickness of pavement
- The maximum load carrying capacity followed by less value of rebound deflection is obtained for waste tyre rubber reinforces pavement as compared to unreinforced flexible pavement system.
- Based on results the CBR value for 10% shredded tire chips was found to be greater and feasible when compared to other varying percentages of tire chips added to soil. Hence it is concluded that for light ,medium and heavy traffic the thickness of pavement was found to be 390mm 450mm 600mm respectively .
- Based on the above study ,it can be inferred that the use of crumb rubber as a stabilizer introduces a low cost method for stabilization and it significantly reduces the waste tire disposal problem that currently exists.

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PERFORMANCE OF HYBRID FIBRE REINFORCED SELF-COMPACTING CONCRETE MADE WITH STEEL AND GLASS FIBERS

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Abstract

The development of Self Compacting Concrete (SCC) is described by the Concrete Society and Building Research Establishment (BRE) as a quiet revolution in the construction industry resulting in massive usage of SCC worldwide. Incorporation of fibres has further enhanced the strength and durability of the properties of SCC. In the present work, SCC mix of M30 grade was developed without fibres and with Glass, Steel and Hybrid Fibres. The mechanical properties like compressive strength and tensile strength were studied. Detailed studies have revealed that the Hybrid Fibre Self Compacting Concrete made with a combination of high dispersion Glass Fibres and Steel Fibres displays better performance. For this purpose, M30 grade plain SCC mix design suggested by Nan-Su(2001) is adopted. For developing GFRSCC, high dispersion glass fibres with 857 aspect ratio and 12mm filament length (0.025% percentage per 1 cu.m. of concrete) were used and For SFRSCC, steel fibres of 30 aspect ratio and 0.4mm diameter (1.3% percentage per 1 cu.m. of concrete recommended was 1.3%) were used. The HFRSCC was obtained by combining 0.024% of glass fibres and 1.3% steel fibres to the plain SCC mix satisfying the fresh concrete properties. All the mixes developed satisfied the fresh and hardened properties of SCC.It was observed that the 28days compressive strength of plain SCC mix is 35.31 MPa. When glass fibers are added, the strength observed is 37.51 MPa, that is, an increase of 6.23%. When steel fibers were added, the compressive strength observed is 38.20 MPa, that is, an increase of 8.18%. This was found to be 39.49 MPa when hybrid fibres are used with a percentage increase of 11.84.The percentage enhancement of split tensile strength for GFRSCC over plain SCC is 2.38 %, for SFRSCC 12.90%, and for HFRSCC 14.43%. The increase is due to the presence of fibers.

Keywords: *SFRSCC, GFRSCC, HFRSCC, steel fibre, glass fibre, hybrid fibre, SCC*

1.0 Introduction

The development of Self Compacting Concrete (SCC) by Professor Hajme Okamura in 1986 has made a remarkable impact on the construction industry by overcoming some of the problems associated with fresh concrete. Although SCC has proved to be an efficient material, there is a need to conduct more research on the standardization of self-compacting characteristics and its behaviour when used in different structural elements, paving way for the acceptance of its usage in all hazardous and inaccessible project zones for greater quality control. Development of latest generation 'concretes' in the recent past needs to update knowledge on the behaviour of SCC with the addition of fibres to make them more efficient and effective. Incorporation of fibres enhances the benefits of this special concrete at fresh stage and in the hardened state. Hybridization of different types of fibres in concrete is another concept which is proved to offer more attractive and enhanced properties to concrete. Hybrid fibre concept consists of using a combination of minimum two types of fibres which use the potential properties of fibres more efficiently. Many studies were reported by different researchers on the characteristics of conventional concrete with hybrid fibre reinforcement. In the present study, the effect of Hybrid Fibre Reinforcement consisting of

Glass and Steel Fibres in SCC is sought to be investigated. For this purpose, investigations were made to explore the behaviour of Steel Fibre Reinforced SCC (SFRSCC), Glass Fibre Reinforced SCC (GFRSCC) and Hybrid Fibre Reinforced SCC (HFRSCC) under axial compression. As the fibers used in the present investigations are of steel and glass, a brief description of these fibers is given below.

2.0 Experimental Programme

The present investigations are aimed at producing standard grade (M30) FRSCC with Glass Fibers, Steel Fibers and Hybrid Fibers.

The different phases of the present research work are as follows:

Phase I: Development of M30 grade SCC and obtaining its fresh and hardened properties.

Phase II: Development of

1. Glass Fibre Reinforced SCC (GFRSCC),
2. Steel Fibre Reinforced SCC (SFRSCC),
3. Hybrid Fibre Reinforced SCC (HFRSCC), and study of fresh and hardened properties.

3.0 Experimental Investigations

In the present studies, the mechanical behaviour of Fibre Reinforced Self-Compacting Concrete of M30 grade made with Steel Fibres, Glass Fibres and Hybrid Fibres (with a mixture of glass and steel fibres) is examined. The experimental investigations was taken up in various steps to achieve the following objectives:

1. To develop plain SCC of M30 grade and obtain its fresh and hardened properties.
2. To develop Steel and Glass Fibre Reinforced SCC of M30 grade separately and study their fresh and hardened properties.
3. To develop Hybrid Fibre Reinforced SCC with a combination of glass and steel fibres and study its fresh and hardened properties.

4.0 Materials Used

Cement

Ordinary Portland cement of 53 grade available in the local market was used in the present investigations.

Coarse Aggregate

Crushed angular granite available from a local market was used in the investigations.

Fine Aggregate

River sand available in the local market was used as fine aggregate.

Fly Ash

Fly ash from a thermal power station in Andhra Pradesh was used in the investigations.

Super Plasticizer

Super plasticizer with Sulphonated Naphthalene based Formaldehyde (SNF) conforming to IS: 9103–1999 was used in the present investigations.

Viscosity Modifying Admixture (VMA)

Viscosity modifying agent from a standard agency conforming to standard specifications was also used.

Water

Potable water conforming to IS: 3025–1986 part 22 & 23 and IS:456–2000 was used in the investigations.

Glass Fibres

Anti-Crack High Dispersion Fibres having a Modulus of Elasticity (E) of 72 GPa and 857 Aspect Ratio were used.

Steel Fibres

Plain steel fibres of 0.4 mm dia. and Aspect Ratio of 30, cut from steel wire were used in the investigations.

Fig. 1 Glass Fibres

Fig. 2 Steel Fibres

5.0 Development of Plain SCC

In this phase of investigations, M30 grade SCC mix is developed using mineral and chemical admixtures to study its fresh and hardened properties. For developing SCC of M30 grade, the mix proportions were designed based on the method suggested by Nan-Su et al(2001) using fly ash as mineral admixture and chemical admixtures like Super Plasticizers (SP) and Viscosity Modifying Agents (VMA). Finally, SCC mixes which have given required compressive strength with satisfactory fresh properties were taken for the next phase of investigations. This is explained in detail as given below.

6.0 Mix Design and Mix Proportions of Self Compacting Concrete

An SCC mix of M30 standard grade was aimed and the initial mix proportion was obtained using the mix design methods as mentioned above. The mix proportion thus obtained was fine-tuned by incorporating different guidelines available and making various trial mixes to obtain the mix which satisfies the required fresh and hardened properties. The final mass of ingredients for 1m^3 of SCC are as follows:

Mass of Cement=330.0 kg

Mass of filler (Fly Ash)=150.0 kg

Mass of water=186.0 kg

Mass of Coarse Aggregate (CA)=794.4 kg

Mass of Fine Aggregate(FA)=860.6 kg

Super plasticizer dosage = 1.5% by weight of cement (bwc)

VMA dosage= 0.6% by weight of cement (bwc)

6.1 Mixing of Ingredients

The mixing of ingredients was carried out in a power operated pan type concrete mixer. Initially coarse aggregate, fine aggregate, cement and fly ash were put in the pan mixer and mixed in the dry state for a few seconds. Then Superplasticizer (SP) was added to the water, thoroughly mixed and added to the material in the concrete mixer. The required amount of VMA was added and further mixed till a mix of required uniform consistency was achieved.

To obtain the Glass Fibre Reinforced SCC, anti-crack high dispersion glass fibre dosage was added to the already developed mix, maintaining the fresh SCC requirements. Similarly, Steel Fibre Reinforced SCC was made by adding suitable dosage of steel fibres maintaining the

fresh SCC requirements. The Hybrid Fibre Reinforced SCC was prepared by adding glass and steel fibres in different proportions till fresh SCC properties were satisfied.

6.2 Testing of SCC in Fresh State

The SCC mix in fresh state was tested to get fresh properties like filling ability, passing ability and segregation resistance by performing Slump cone, V-funnel, and L-box tests as explained above.

7.0 Development of FRSCC

7.1 Addition of Anti Crack High Dispersion Glass Fibres to SCC Mixes

Anti-crack high dispersion glass fibres were added in different dosages to the selected SCC mixes in the first batch of investigation and Glass Fibre Reinforced Self-Compacting Concrete (GFRSCC) was developed. After adding glass fibres to SCC mixes, its influence on fresh and hardened states was observed by conducting tests on fresh and hardened GFRSCC. The tests on fresh and hardened GFRSCC were conducted in the same way as they were conducted for SCC.

Fig. 3: Addition of Fibres to SCC mix

7.2 Development of Steel Fibre Reinforced Self-Compacting Concrete and Hybrid Fibre Reinforced Self-Compacting Concrete

The GFRSCC mix with an optimum dosage of glass fibres, satisfying the fresh and the hardened properties, was considered for the next phase of investigation. Similarly, the dosages of steel fibres in Steel Fibre Reinforced Self-Compacting Concrete (SFRSCC) with a fixed aspect ratio and Hybrid Fibre Reinforced Self-Compacting Concrete (HFRSCC) consisting of a mixture of glass and steel fibres, were developed by trial mixes. The ratio of steel fibres to glass fibres was determined in the laboratory by trial mixes satisfying the fresh and the hardened properties.

8.0 Test Results

The test results of experimental investigations carried out in different phases are presented as follows:
The first phase of investigations was carried out to develop SCC mix of a minimum strength M30 grade using fly ash and chemical admixtures, and to study its fresh and hardened properties. For developing SCC of strength M30 grade, the mix was designed based on Nan-Su method of SCC mix design using fly ash as the mineral admixture. Finally, SCC mixes which yielded satisfactory fresh properties and required compressive strengths, were selected and taken for further investigations.

8.1 Determination of Optimum Dosage of Glass Fibres and Steel Fibres

Based on the literature and trial mixes, the minimum optimum dosage of glass fibre and steel fibre in SCC was selected as 0.6 Kilograms per cubic meter and 31.42 kilograms per cubic meter of SCC, respectively. The GFRSCC, SFRSCC and HFRSCC mixes with optimum dosage of glass fibres and steel fibres satisfying fresh and hardened properties were considered for the next phase investigations. The mix proportions are shown in Tables 1 and 2.

Table 1. Mix Proportions of SCC and FRSCC

S. No	Cement kg	Fly Ash kg	CA kg	FA kg	Water kg	SP % bwp	VMA % bwp	Glass Fibres % of 1 m^3 concrete	Steel Fibres % of 1 m^3 concrete	Design-ation
1	330	150	794.4	860.6	186	1.2	0.06	-	-	SCCP
2	330	150	794.4	860.6	186	1.2	0.06	0.025	-	GFRSCC
3	330	150	794.4	860.6	186	1.4	0.06	-	1.3	SFRSCC
4	330	150	794.4	860.6	186	1.5	0.06	0.025	1.3	HFRSCC

bwp -by weight of cement and fly ash

Table 2 Fibre Content of FRSCC

Designation	**Mix proportion**
GFRSCC	SCCP + 0.6 kg/m^3 of HD Glass Fibre
SFRSCC	SCCP + 31.42 kg/m^3 of Steel fibre
HFRSCC	SCCP + 0.6 kg/m^3 of HD Glass Fibre + 31.42 Kg/m^3 of Steel Fibre

Table 3. Fresh and hardened properties

Fresh concrete properties					**Remarks**	**Designation**
Slump Test		**V Funnel Test**		**L Box Test**		
Slump mm	**T50 time Sec.**	**Time for Discharge Sec.**	**T5 min. Sec.**	**H2/H1**		
670	2.68	3.93	8.31	0.947	RS	SCCP
660	2.80	5.55	9.60	0.726	RS	GFRSCC
655	4.27	6.17	9.82	0.88	RS	SFRSCC
655	4.69	6.85	8.68	0.90	RS	HFRSCC

RS – Results satisfactory in fresh state

Table 4 Hardened concrete properties of SCC and FRSCC at28 days

S.No	Designation	Cube Compressive Strength MPa	% Increase	Cylinder Compressive Strength MPa	% Increase	Split Tensile Strength MPa	% Increase
1	SCCP	35.31	-	26.02	-	5.89	-
2	GFRSCC	37.51	(+) 6.23	27.15	(+) 4.34	6.03	(+) 2.38
3	SFRSCC	38.20	(+) 8.18	28.52	(+) 9.61	6.65	(+)12.90
4	HFRSCC	39.49	(+)11.84	31.67	(+)21.71	6.74	(+)14.43

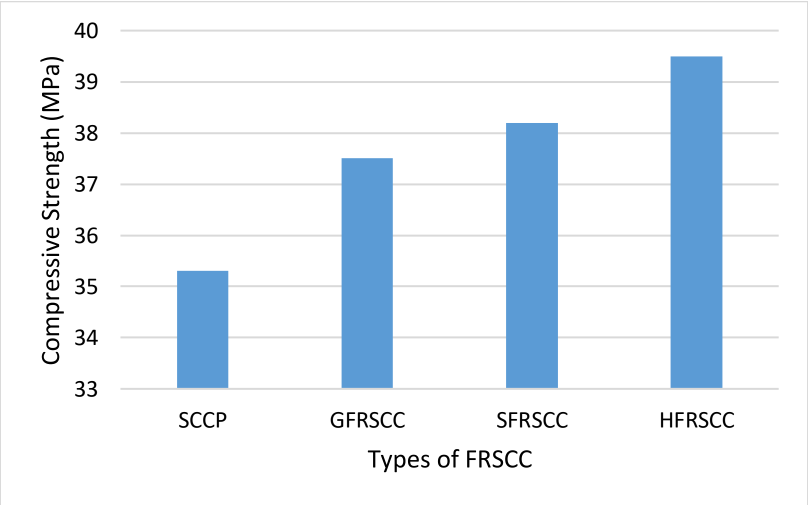

Fig 4. Variation of Cube compressive strength in SCC made with different fibres

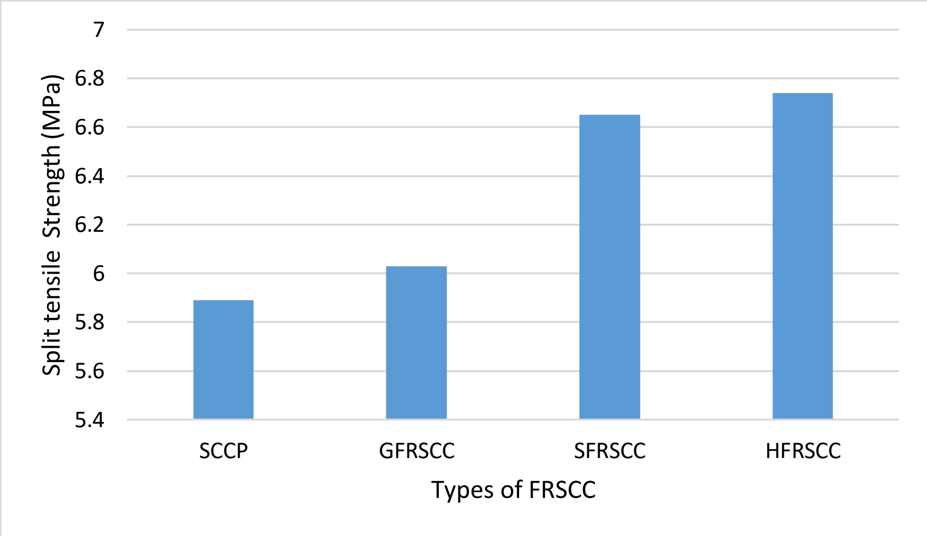

Fig. 4 Variation of split tensile strength in SCC made with different fibres

Table 5. Compressive strength of SCC Mixes at Different Ages

Description	**Cube Compressive Strength in MPa**			
	7 days	**14 days**	**28 days**	**56 days**
SCCP	20.63	25.63	35.31	38.58
GFRSCC	25.66	28.74	37.51	40.25
SFRSCC	29.21	30.21	38.20	42.7
HFRSCC	29.94	31.75	39.49	44.7

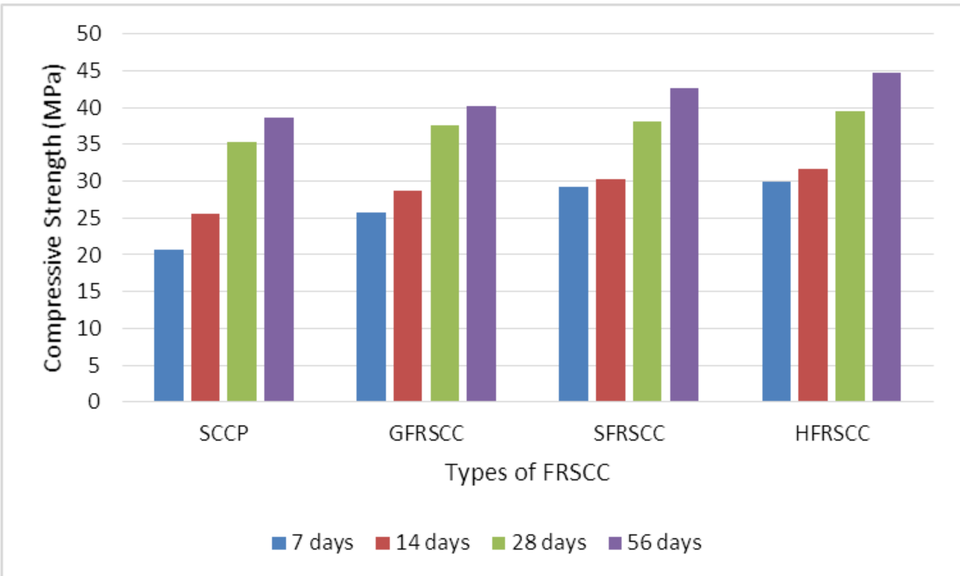

Fig. 5. Cube Compressive Strength of SCC mixesat 7, 14, 28 and 56days

8.0 Discussion of Test Results

The test results obtained from the experimental investigations on plain SCC, GFRSCC, SFRSCC and HFRSCC in fresh and hardened states, are discussed in the following sections:

1. Fresh and hardened properties of Plain SCC.
2. Fresh and hardened properties of GFRSCC
3. Fresh and hardened properties of SFRSCC
4. Fresh and hardened properties of HFRSCC

8.1 Water / Powder Ratio of Self Compacting Concrete Mixes

Water-powder ratios, by weight, for the SCC plain mix and GFRSCC, SFRSCC and HFRSCC mixes, are found to be 0.3875. For this value, it is observed that the required fresh properties of SCC and strength of the mixes are satisfied. The main thrust of the present investigations is to develop M30 grade SCC and to study its behaviour when different types of fibres are introduced into it. For this purpose, M30 grade plain SCC was developed using fly ash as an ingredient with super plasticizer and viscosity modifying agent as a chemical admixture. The method of mix design suggested by Nan-Su(2001) is adopted. The number of trial mixes were developed in the laboratory and a mix satisfying the guidelines given by EFNARC in fresh state and compressive strength in hardened state, was finalized. The cement content used is 330 kg/m^3 with 150kg of fly ash and 182 litres of water giving a water/binder ratio of 0.3875.

In this trial, SCC with three different types of fibers, namely glass fibres, steel fibres and hybrid fibres, consisting a mixture of glass and steel fibres satisfying strength and fresh properties, were developed. Here, the basic proportion of mix is not altered but the volume and aspect ratio of fibres were so chosen as to satisfy the fresh properties. For developing GFRSCC, high dispersion glass fibres with 857 aspect ratio and 12mm filament length (0.025% percentage per 1 cu.m. of concrete) were used and For SFRSCC, steel fibres of 30 aspect ratio and 0.4mm diameter (1.3% percentage per 1 cu.m. of concrete recommended was 1.3%) were used. The HFRSCC was obtained by combining 0.024% of glass fibres and 1.3% steel fibres to the plain SCC mix satisfying the fresh concrete properties.

8.2 Fresh Properties of Self Compacting Concrete Mixes

From Table 3, fresh properties of Self-Compacting Concrete like the filling ability, passing ability and segregation resistance values, which are the basic requirements for SCC in fresh state, were found to be satisfying. These parameters are in accordance with EFNARC specifications (2005).

8.2.1 Compressive strength

From the compressive cube strength results shown in Table 5.2.1.4, it can be seen that the 28days compressive strength of plain SCC mix is 35.31 MPa. When glass fibers are added, the strength observed is 37.51 MPa, that is, an increase of 6.23%. When steel fibers were added, the compressive strength observed is 38.20 MPa, that is, an increase of 8.18%. This was found to be 39.49 MPa when hybrid fibres are used with a percentage increase of 11.84.

The above results clearly show that the addition of fibres has enhanced the compressive strength which is due to the holding of the concrete that is, confining the concrete. However, the effect is different in different types of fibers, and hybridization of fibers enhanced the confining effect partly due to the presence of high dispersion glass fibers holding the concrete at micro-crack level and steel fibers at a later stage.

8.2.2 Split Tensile Strength

From the Table 5.2.1.4, it can be seen that the split tensile strength of plain SCC is 5.89 MPa, that is, 16.68% of compressive strength, and it was enhanced with the addition of different fibers. The percentage enhancement of split tensile strength for GFRSCC over plain SCC is 2.38 %, for SFRSCC 12.90%, and for HFRSCC 14.43%.

9.0 Conclusions

Fibre Reinforced Self-Compacting Concrete (FRSCC) of M30 grade using three kinds of fibres was developed. It has satisfied all the guidelines prescribed by EFNARC. Based on the investigations carried out on Fibre Reinforced Self-Compacting Concrete Mixes the following conclusions are presented in this chapter.

1. Fibre Reinforced Self-Compacting Concrete can be produced byincorporating different types of fibres. However, the use of appropriate dosage of superplasticizer and viscosity modifying agent is essential to maintain the fresh properties of self-compacting concrete.
2. In the case of high dispersion of glass fibres, a dosage of 6 kg of fibres/m^3 of concrete (0.025%) is used as optimum dosage by suitably adjusting the dosage of admixtures.
3. The aspect ratio and volume of steel fibres are selectedsatisfying the fresh and hardened properties of self-compacting concrete by suitably adjusting the dosage of admixtures. In the case of steel fibres, a dosage of 31.42 kg of fibres/m^3 of concrete (1.3%) is used as optimum dosage by suitably adjusting the dosage of admixtures.
4. The compressive strengths of the FRSCC design mixes are foundto be increased by the addition of fibres.
5. The addition of glass fibres and steel fibres has shown improvedcompressive strengths. The increase in compressive strength in SFRSCC was found to be higher than that of GFRSCC.
6. In the case of HFRSCC, the compressive strengths were found tobe further enhanced due to the combined action of glass and steel fibres, and the increase in compressive strength is 11.84% over plain SCC.
7. The addition of fibres improved the split tensile strength whichis found to be maximum in HFRSCC. Hence, it is concluded that the hybridization of glass and steel fibres is useful in improving the strength properties of FRSCC.

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MECHANICAL AND SORPTIVITY COEFFICIENT STUDIES OF SYNTHESIZED HIGH STRENGTH SELF COMPACTING CONCRETE WITH METAKAOLIN AND MICRO SILICA AS ADMIXTURES

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Abstract

Self-compacting concrete (SCC) can flow into the frame work of reinforcement under its own weight without the need for vibrating compaction.The mineral admixtures in concrete increase the strength, reduction in water demand, and cost savings. In Nan Su's type mix design, Fly Ash and GGBS together are mineral admixtures. Here reported the SCC mixes contains different proportions of Metakaolin(MK)and Microsilica(MS) mineral admixture design for M70 Grade of SCC with water-cement W/C Ratio=0.27,Packing Factor =1.10 with aggregate 10mm coarse size. Destructive test assess the quality of concrete in the hardened state on cement replacing with MK for 10%,20%,30%,40% mix of micro silica 8% samples. The Slump flow, V-funnel, L-box and U-box tests revealed the filling ability, segregation resistance and passing ability of self-compacting concrete. A 30 total, 100 mm cubes, prisms (100 mm x 100 mm x 500 mm) and cylinders (150 mm dia, 300 mm height) were cast for determining various mechanical properties. The compressive strength is highest in M70 SCC of MIX3 (10%) at 7 (56.715 N/mm^2) and 28 days (79.80N/mm^2).The split tensile strength (4.2N/mm^2) and flexural strength (6.291 N/mm^2) found to be high in the same ration at 28 days. The sorptivity test of cube specimens immersed in water 5-10 mm depth showed the HSSCC with MK is less sorptivity compared to HSVC, revealing the more homogeneous micro structures and denser interfacial zones of HSSCC. The best fit curve resulted sorptivity coefficient of 0.0114x10^{-3} m/min$^{0.5}$ for MK and 0.0142x10^{-3} m/min$^{0.5}$for HSVC. The right ration SCC and mineral mix improves the strength, capillary absorption coefficient (k), reduce cost and new constituents to meet many varied requirements of the construction industry.

1.0 INTRODUCTION

Normal concrete usually needs vibration. The vibrations cause noise that not only leads to stress on construction site but also affect the surrounding neighbourhood. The definitions of SCC given in the literature is that 'a concrete that is able to flow under its own weight and completely fill the formwork, while maintaining homogeneity even in the presence of congested reinforcement, and then consolidating without the need for vibrating compaction'. The use of SCC offers a more industrialized production. It can also reduce the technical costs of in situ cast concrete constructions, due to improved casting cycle, quality, durability, surface finish. However, SCC is a sensitive mix, strongly dependent on the composition and the characteristics of its constituents. It has to possess the incompatible properties of high flow ability together with high segregation resistance. This balance is made possible by the dispersing effect of high-range water-reducing admixture (superplasticizer) combined with cohesiveness produced by a high concentration of fine particles in additional filler material.

The main mechanisms controlling this fine balance are related to surface physics and chemistry hence, SCC is strongly dependent on the activity of the admixtures, as well as on the large surface area generated by the high content of fines. The particles are affected by a complex balance of inter-particle forces (i.e. interlocking, frictional, colloidal, and electrostatic forces), generating a time dependence and visco-plastic non Newtonian behavior. The development of SCC has thus been strongly dependent on surface active admixtures as well as on the increased specific surface area obtained through the used fillers.

Hajime Okamuraet al.,[2003] and Ozawa K et al.,[1989] have employed the following methods to achieve self-compactability: (a) Limited aggregate content ,(b) Low water-powder ratio and (c) Use of Super Plasticizer (SP). The frequency of collision and contact between aggregate particles increases as the relative distance between the particles decreases and the internal stress increases when concrete is deformed, particularly near obstacles. It has been revealed that the energy required for flowing is consumed by the increased internal stresses, resulting in blockage of aggregate particles. Limiting the coarse aggregate content, whose energy consumption is particularly intense, to a level lower than the normal proportions is effective in avoiding this kind of blockage. The scope of present work is to compare the Sorptivity Coefficient of Self Compacting Concrete with metakaolin & microsilica as mineral admixture (M70 Grade) and Sorptivity Coefficient of High Strength Vibrated Concrete. The mix proportions are obtained on the basis of NAN-SU mix design. The Concrete mixes contains different proportions of Metakaolin & cement and constant proportions of water binder , microsilica, Coarse aggregate and Fine aggregate for constant water-cement ratio 0.27 with a Packing factor of 1.10. The work focused on replacement of Cement with metakaolin for 10%,20%,30%,40% mix of micro silica 8%, Destructive Test is also conducted to assess the quality of concrete in the hardened state.

2.0 Experimental

The experimental program consisted of casting and testing specimens for testing the fresh and hardened properties on M70 grade of concrete with metakaolin as filler material. Nan Su method of mix design [2001] was adopted to arrive at the suitable mix proportions. A total of 30 cubes of standard size 100 mm x 100 mm x 100 mm, 30 prisms of standard size 100 mm x 100 mm x 500 mm and 30 cylinders of 150 mm diameter and 300 mm height were cast for determining the compressive strength, flexural strength and split tensile strength respectively. The materials used in the experimental investigation of SCC were

1.Ordinary Portland cement-53 grade
2.River sand
3.Coarse Aggregate of size 10mm
4.Water
5. Admixture
 a. Mineral Admixtures (Metakaloin)
 b. Mineral Admixtures (Micro Silica)
 c. Chemical Admixtures (B233)

The physical properties of ordinary Portland cement and fine aggregates & Coarse Aggregates (10 mm) materials are given in table 1 and 2. Water used for mixing and curing was potable water, confirming to IS : 3025 – 1964 part22, part 23 and IS : 456 – 2000 [Code of practice for plain and reinforced concrete]. The pH value should not be less than 6. Metakaolin differs from other supplementary cementitious materials (SCMs). Metakaolin is produced by heating of the most abundant natural clay minerals, to temperatures of 650-900 °C. Metakaolin also decreases concrete permeability, which in turn increases its resistance to sulfate attack and chloride attack, Metakaolin may reduce autogenous and shrinkage and avoids cracking. Silica fume is a very fine amorphous (non crystalline) silica produced in electric arc furnaces as a byproduct of the production of elemental silicon or alloys containing silicon; also known as condensed silica fume or micro silica. The properties are given in Table 1-6.

3. RESULTS AND DISCUSSION OF TEST

Testing of hardened concrete plays an important role in controlling and confirming the quality of self compacting concrete.

3.1 Compressive Strength

Compressive strength of a material is defined as the value of uniaxial compressive stress reached when the material fails completely. In this investigation, the cube specimens of size 100 mm x 100 mm x 100 mm are tested in accordance with IS: 516 – 1969 [Method of test for strength of concrete]. The testing was done on a compression testing machine of 200 tons capacity.

Table: 1 Physical properties of Portland cement

S. No	Property	Test Resul
1.	Normal Consistency	30%
2.	Specific gravity	2.9
3.	Initial setting time Final setting time	99 min 207 min
4.	Fineness	1.3%
5.	Soundness	2 mm

Table 2 Physical Properties of fine aggregates & Coarse Aggregates (10 mm)

S.No.	Characteristics	F. A Value	C.A Valu
1	Specific gravity	2.61	2.630
2	Bulk density	1450 kg/m^3	-
3	Fineness modulus	2.64	6
4	Grading Zone	Zone – II	-

Table: 3 Physical properties of Metakaolin

S.No	Description of physical properties	Result
1	Bulk Density	356 Gm/liter
2	Moisture (EX-Work)	0.22%
3	RESIDUE on 325 Mesh	0.13%
4	PSD –D(50)- 50% particles	1.68µ
5	Specific gravity	2.63 kg/m^3

Table: 4 Physical properties of silica fume

S.No	Property	Result
1	State	Amorphous Sub micron Powder
2	Specific Gravity	2.1 to 2.4
3	Specific Surface	18m^2/g
4	Bulk density (a) Densified (b)Un Densified	608 to 720 g/m^3 192 to 320 kg/m

Table 5 Details of Viscosity Modifying Admixture

Property	Result
Aspect	Colourless free flowing l
Relative density	1.01
pH	≥6
Chloride ion content	< 0.2%
Dosage	50 to 500 ml/100 kg of cementitious material.

Table 6 Typical Properties of Super plasticizer

Property	Result
Aspect	Yellowish free flowi liquid
Relative Density	1.09 ($\pm$) 0.01 at 25^{0}c
PH	7 (+ or -) 1
Chloirde ion content	<0.2%
Specific Gravity	1.09
Dosage	500ml to 1500ml pe 100kg

3.2 Flexural Strength

Standard beam test (Modulus of rupture) was carried out on the beams of size 100 mm x 100 mm x 500 mm as per IS: 516 [Method of test for strength of concrete], by considering that material is homogeneous.

3.3. Split Tensile Test

Tensile strength tests are used to assess the cracking resistance of concrete and bond strength to reinforcing bars. A concrete cylinder of size 150 mm diameter and 300 mm height is subjected to the action of a compressive force along two opposite edges, by applying the force. The cylinder is subjected to uniform tensile stress. Horizontal tensile stress (f_s) =2P/ΠDL, Where (f_s) is the splitting strength (MPa), P is the failure load (kN), L is the length of cylinder (mm) and D is the diameter of cylinder (mm). The Tests on fresh Concrete are given in Table 7-10.

Table.7 List of methods for testing workable properties of SCC

S.No	METHOD	PROPERTY
1	Slump flow by Abrams cone	Filling ability
2	T_{50} Slump flow	Filling ability
3	V-Funnel	Filling ability
4	V-Funnel at T5 minutes	Segregation resistance
5	L-Box test	Passing ability
6	U-Box test	Passing ability

Table.8 Acceptance criteria for SCC as per EFNARC guide lines

Method & units	Typical values	
	Min	Max
Slump flow by Abrams cone	650	800
T_{50} slump-flow	2	5
V-funnel (Sec)	6	12
V-funnel at T5	6	15
L-Box (H_2/H_1)	0.8	1.0
U-Box((h_2-h_1)mm)	0	30

Table: 9 Fresh properties of M70 SCC (Metakaolin Variable)

Workability Tests	Concrete Mixes				
	MIX 1(0%)	MIX 2(10%)	MIX (20%)	MIX (30%)	MIX (40%)
Slump flow (mm)	705X705	700X700	695X695	685X685	670X670
T 500(sec)	4.10	4.12	4.2	4.45	4.75
V-funnel (sec)	9.10	9.12	10.65	11.35	11.5
V-funnel T5 min	12.13	12.15	13.20	14.35	14.65
L-box (h2/h1)	0.96	0.98	0.95	0.92	0.85
U-box (mm)	2	3	4	5	6

Table: 10 Mechanical Properties of M70 SCC

Type of mix	Compressive Strength(N/mm^2)		Split Tensile Strength (N/mm^2)		Flexural Strength (N/mm^2)	
	7days	28days	7days	28days	7days	28days
MIX1(0%)	52.16	75.8	3.58	4.00	5.11	6.119
MIX2 (10%)	54.95	78.45	3.65	4.16	5.28	6.242
MIX3 (20%)	56.715	79.80	3.84	4.2	5.30	6.291
MIX4 (30%)	53.83	77.18	3.716	4.021	5.16	6.18
MIX5(40%)	53.544	76.82	3.24	3.721	5.12	6.162

3.4 Non Destructive Tests

The main objective of the present experimental investigations is to assess the quality, structural integrity and estimated compressive strength of metakaolin incorporated self compacting concrete of grade M70 using Rebound hammer and Ultrasonic pulse velocity measurements. The average rebound hammer gives the quality of concrete based on average rebound number of > 40 for very good hard layer, 30-40 for good and 20 to 30 for fair and < 20 for poor quality of concrete.

3.5. SORPTIVITY TEST

The aim of this study is to determine the total water absorption capacity a measure the volume of voids present in two mixes of concrete i.e HSSCC with Metakaolin and HSVC as per ASTM C1585(Table 11-12). Sorptivity measures the rate of penetration of water into the pores in concrete by capillary suction.

Table.11 Mix Proportion

	CEMENT	MK	FA	CA	SP	WATER	M.SILICA
QUANTITY (kg/m^3)	574.02	52.4	829.4	790.94	11.28	173.26	40.18
PROPORTION	1	0.09	1.44	1.38	0.02	0.26	0.08

Table.12 Quantities of mix for different percentage of Metakaolin as mineral admixture with I

Mix Components	Concrete Mixes				
	MIX 1(0%) Metakaolin	MIX 2(10%) Metakaolin	MIX 3 (20%) Metakaolin	MIX 4 (30%) Metakaolin	MIX 5 (40%) Metakaolin
	Qty.	Qty.	Qty.	Qty.	Qty.
CEMENT	574.02	517.00	459.216	401.81	344.41
C.A	790.94	790.94	790.94	790.94	790.94
F.A	829.4	829.4	829.4	829.4	829.4
WATER	173.26	173.26	173.26	173.26	173.26
MK	52.400	109.802	167.2	224.608	282.08
S.P	11.28	11.28	11.28	11.28	11.28
MS	40.18	40.18	40.18	40.18	40.18
V.M.A	1.722	1.722	1.722	1.722	1.722

Measuring Intervals t_i (min)	Type of Concrete				Type of Concrete			
	HSSCC with MK		HSVC		$=(W/(Axd))x10^{-3}$(m)		$=(I/\sqrt{t})x10^{-3}$(m/min $^{0.5}$)	
	m_o= 2545g		m_o= 2410g		HSSCCMK	HSVC	HSSCCMK	HSVC
	m_i(gms)	W(gms)	m_i(gms)	W(gms)				
0	2545	0	2410	0	0	0	0	0
15	2550.6	5.6	2416	6	0.56	0.6	0.144	0.16
30	2551.5	6.5	2417	7	0.65	0.7	0.118	0.13
60	2552.1	7.1	2417.5	7.5	0.71	0.75	0.091	0.1
120	2552.3	7.3	2417.8	7.8	0.73	0.78	0.066	0.07
240	2552.4	7.4	2418	8	0.74	0.8	0.047	0.052
360	2552.6	7.6	2418.2	8.2	0.76	0.82	0.040	0.043
720	2552.8	7.8	2418.3	8.3	0.78	0.83	0.029	0.031
1440	2553.1	8.1	2418.5	8.5	0.81	0.85	0.021	0.022
2880	2553.2	8.2	2418.7	8.7	0.82	0.87	0.015	0.016
4320	2553.3	8.3	2418.8	8.8	0.83	0.88	0.012	0.0134

Table 13. Water Absorption and Sorptivity Coefficient at different time intervals for Sorptivity test of HSSCC and HSVC

3.6 Sorptivity studies of HSSCC and HSVC at Room temperature

Due of small initial surface tension and buoyancy effects, the relationship between cumulative water absorption (kg/m^2) and square root of exposure time ($t^{0.5}$) shows deviation from linearity during first few minutes. Thus, for the calculation of sorptivity coefficient, only the section of the curves for exposure period from 15 min to 72 hrs, where the curves were consistently linear, was used for the calculation of sorptivity. The values are given in Table13.

Fig. 1 shows plot between gain in mass and duration and Fig.2 shows plot between the gain in mass per unit area over the density of water (I) and the square root of the elapsed time (√t). The slope of the line of best fit of these points is reported as the sorptivity coefficient (k). From the test results HSSCC with Metakaolin shows less sorptivity, when compared to HSVC (Fig 3). This is due to the more homogeneous micro structures and denser interfacial zones of HSSCC. Based on best fit curve Sorptivity coefficient for Metakaolin is 0.0114x10^{-3} m/min 0.5 and for HSVC is 0.0142x10^{-3} m/min 0.5. The capillary absorption coefficient (k) is greatly influenced by the addition of mineral admixtures to the concrete. The water absorption, capillary and porosity characteristics indirectly reflect the durability performance of the high strength self compact concrete.

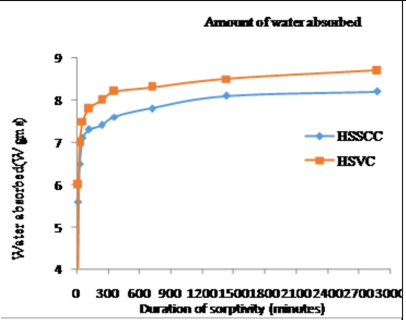

Fig.1 Water absorbed vs duration for Sorptivity

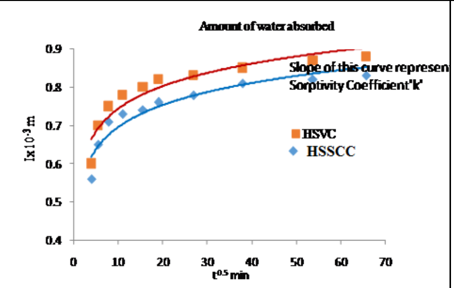

Fig.2 I vs duration for Sorptivity

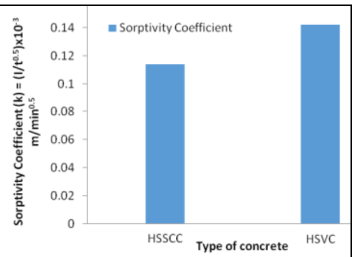

Fig.3 Sorptivity Coefficient vs Type of Concrete

Conclusions

1. The water absorption is found to be less in High Strength Self Compacting Concrete when compared to High Strength Vibrated Concrete at all periods.
2. The water absorption found to increase with time. It is found to be more in first 15 minutes in both concretes.
3. The Sorpitivity is found to be decreased from 75 min to 300 min in both cases .
4. From 60 min onwards the sorptivity is found to be further reduced to 2.81% from 60 to 120 min and later on it reached a constant of 2.81% for every 120 min interval after 120 min.
5. The same trend is observed in both High Strength Self Compacting Concrete and High Strength Vibrated Concrete.
6. The Sorptivity Coefficient is found to be less by.

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SETTLEMENT ANALYSIS OF PILED RAFT SYSTEM IN OFFSHORE LAYERED SOILS

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Abstract

To compute and compare settlement by the effect of length of piles, number of piles and thickness of raft on the behavior of piled raft system. They are modeled in Plaxis 3D Software and compared Load-Settlement curves for different pile raft configurations. The settlement was measured at the centre of the models of pile raft with nine and sixteen piles. The Settlement of piled raft computed with constant load in case of 16 piles of short length and 9 piles of enduring length. The settlement is less in case of 9 high length piles as compared with the 16 short length piles of the same loading cases.

***Keywords**: group piles, load-settlement, piled raft, and stratified.*

1.0 Introduction

In a conservative method of design for pile foundations, once the decision to introduce piles to a raft foundation has been made because the differential settlement of the raft alone was large, the applied load is considered to be supported by piles and the capacity of the raft is ignored. A contribution of the raft resting on a soil surface to the capacity of the pile group or against the group settlement could be taken into account in design of piled raft foundations.

Valuable measurements of load distribution for piles and rafts were made by analytical methods. Settlement reducing piled, means to reduce the differential settlement of shallow foundations using friction piles, whose shaft capacity may be fully mobilized under working conditions. However, both the manner of the load transfer from the foundation to the soil and the settlement characteristics of piled raft systems are not well understood due to the complex system consisting of the piles, the raft and the surrounding soil.

2.0 Literature Review

A piled raft system is the composite structure which consists of the 3 elements: raft, pile and the ground. Applied uniform loads are transferred to the soil both through the toped raft and the beneath piles. This load transfer mechanism shown in figure 1 can be simply Load sharing between raft and piles are the main distinctive feature that diversifies this type of foundation from other type of piled foundations design.

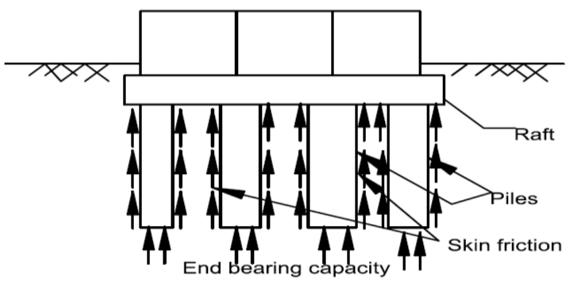

Fig. 1: Simple piled raft system with load transfer mechanism

2.1. Methods for the Analysis of Piled Raft Foundations

Poulos H.G [5] categorized the methods of analysis of piled raft foundations into 3 categories;

I. Simplified count strategies

According to Poulos H.G [5] overall stiffness equation is operative up to the fully mobilization of pile capacity.

II. Estimated PC based techniques

The approximate computer-based strategies are based on elastic hypothesis and mainly have two approaches as; strip on springs and plate on springs. In these methodologies, the raft is treated as a strip and as a thin plate respectively. Also, piles are dealt with as springs and the soil as an elastic continuum, which are additionally rearranged into springs, for the foundation-structure interaction.

III. More thorough PC based techniques

More thorough strategies mainly include boundary element techniques and finite element strategies. In addition, for the different members of the foundation, combination of these strategies has been applied.

3.0 Methodology

3.1 Modeling of Pile Raft

Modeling of piled raft system with 16 piles are analysed and the settlements are considered and comparison made by 9 Piles piled raft system, and the results plotted as load-settlement curves are shown in figures 5 and 6.

Table 1: Properties of Pile & Raft

S. No	Material	Properties	Value
1	Pile and Raft (concrete)	Elastic Modulus, E (kN/m^2)	$2.9x10^5$
		Poisson's Ratio, υ *	0.2

Table 2: Material properties, pile and raft model used for the numerical model.

	Type of Layer	Unsaturated & saturated Unit, γ (kN/m^3)	Elastic Modulus, E (kN/m^2)	Poisson's Ratio, υ	Undrained cohesion, C_u	Angle of Shearing Resistance, Ø (^{0})
Material Properties	Very soft clay	16	4500	0.3	4	17
	Soft Clay	18	7000	0.3	5	22
	Medium Clay	18.9	14000	0.3	8	23
Pile model	Pile Area = 43.56 m^2 (16 Piles)			Pile length = 9 m (16 Piles)		
	Pile Area = 36.36 m^2 (9 Piles)			Pile length = 12 m (9 Piles)		
Raft Model	Raft Area = 100 m^2			Raft Thickness = 0.3 m		

The Properties of the Pile and Material (Table 1 & 2) were taken from the Poulos, H.G, (2001) [3].

In piled raft system in normally consolidated clay, we can determine the settlement in the following manner

$$S = \left[\frac{CcxH}{1+e_0}\right]\log\left[\frac{\sigma_0+\Delta\sigma}{\sigma_0}\right] \quad \_\_\_\_\_\_\_\_ \text{ Equation (1)}$$

Where S= Total Settlement (mm).

H= total thickness of the soil media (m).

Cc = coefficient of consolidation.

e_0 =initial void ratio.

σ_0 = Initial over burden pressure (kN/m^2)

$\Delta\sigma$ = Incremental Pressure (kN/m^2)

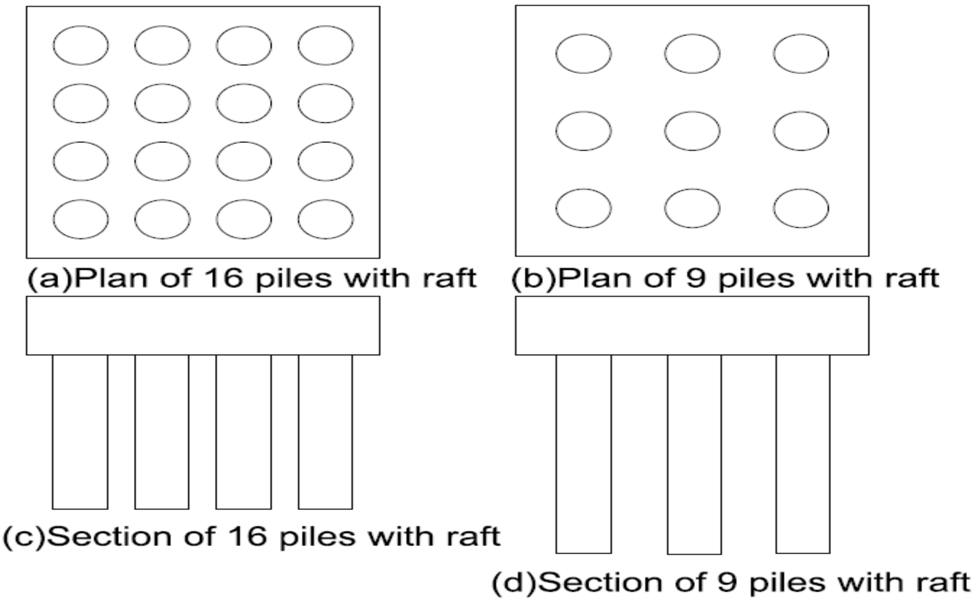

Fig.2: Modeling of Piled group for Numerical Analysis in layered soils

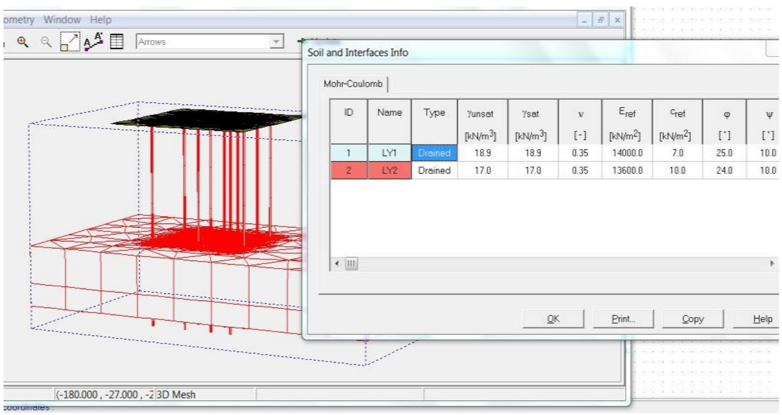

Fig.3: The massive Pile group Settlement in Layered clay with Soil Properties

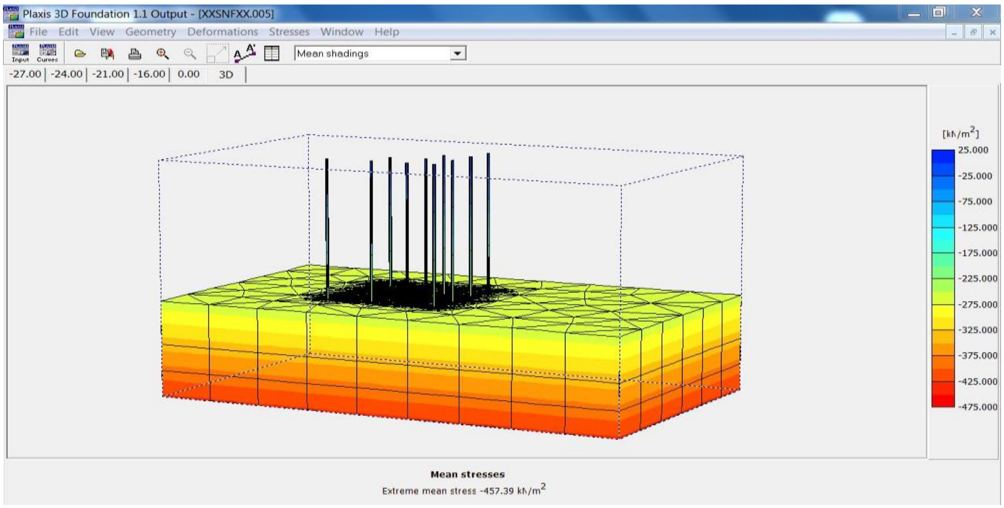

Fig.4: The massive Pile Group Stress Intensities

4.0 Results and Discussion

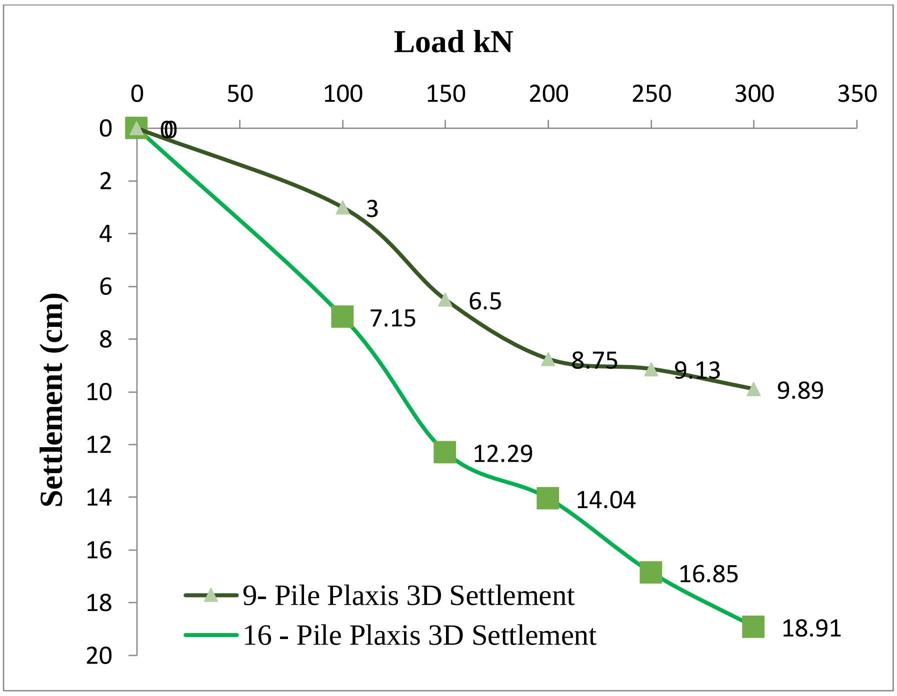

Fig.5: Load – Settlement Curves 16 and 9 Piled Raft Plaxis model.

From the above figure, it has been observed that the settlement of the Piled raft system decreases in case of more length of less number of piles compared to the short length of more number of piles.

5.0 Conclusion

The numerical modeling of the piled raft system considering the load effect, length of piles, and number of piles using the finite element method through the program PLAXIS reveals the following conclusions:

1. Piled raft system is beneficial to be used as settlement reducer in soft soils.

2. The length of the piles increases the load carrying capacity increases; settlement is also reduced as compared to the short length of huge number piles.
3. This study seems, the Plaxis 3D Numerical model can be a time effective tool to get fairly reliable results, hence the FEM based Software's can be used to analyze and execute real time Projects.

6.0 References

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WATERSHED ANALYSIS OF ANEGUNTA

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Abstract

Watershed describes an area of land that contains a common set of streams and rivers that all drain into a single larger body of water, such as a larger river, a lake or an ocean. For example, the Mississippi River watershed is an enormous watershed.
The study area is Anegunta village which has more water scarcity.It is located in Medak district of Telangana state and 6Kms from the Zahirabad watershed. Now a days Remote sensing is widely used in many water resource management programs. The report was done by using Q-GIS software. ***Quantum GIS*** *(**QGIS**) is an open-source desktop GIS tool that helps you visualize, manage, edit, analyze, and compose maps with geographic data.*

Watershed Approach/Concept

Theoretically 'watershed' is a hydrological unit of an area draining to a common outlet point. 'Basin' and 'catchment 'are synonymous as far as definition is concerned. However, whereas watershed is used for agricultural development purpose, 'basin' is used for drainage area of drainage area of a river and 'catchment' is generally referred to contributing area of a reservoir/dam. Therefore, whereas an agricultural operational watershed may be around 500 ha area, the catchment and basins are much larger in size.

Scope of the present study:

Geo-scientific studies of the terrain, socio-economic appraisal of the stake holders and knowledge about local practices aimed at providing well coordinated and synthesized information on the overall watershed area are the need of the hour for the watershed development. These include soils, hydro-geomorphology, land use/land cover pattern, surface & ground water potential surveys in addition to gradient of the terrain particularly, slope and aspect which plays a vital role in suggesting/implementing various soil and moisture conservation measures.

The use of Remote Sensing Data for faster assessment of natural resources such as soil, geology, drainage etc. as well as assessment of economic activities through land use and infrastructure of the watershed area is well known. This is also used for monitoring of watershed development at later years. GIS is a very powerful tool for development of the watershed area with all natural and socio-economic facets for better planning, execution and monitoring of the project. A GIS based model of the terrain with all relevant spatial data related to natural resources, infrastructure and administrative boundaries attached with relevant attribute data will enable the planners, stake holders and funding agencies to develop the watershed keeping in mind to fulfill the stake holders need in the backdrop of natural resources potential as well as limitation

Watershed Development:

Watershed development is carried out to: rehabilitate the watershed though proper land use and conservation measures in order to minimize erosion; reduce the damage caused by sedimentation to the multipurpose reservoir; develop the watershed's crop, livestock, forestry, fish culture and recreational activities; ensure that the watershed provides water of the highest quality for municipal uses; and manage the watershed in order to minimize natural disasters such as floods, drought and landslides, etc.

The watershed approach has been universally accepted as the most scientific way of land resource management for several seasons. Few of them described below:

Location of the Study Area:

The study area comprises, Anegunta Micro-watershed of Zahirabad watershed of Medak District, Telangana State. Which spreads over an area of 39.84 Sq.kms. These area lies geographically between the 77^0 33' to 77^0 37' E Longitude and 17^0 37' to 17^0 33' N Latitude. And falls in Survey of India Toposheet Nos. 58 1/9 and 58 1/13 East scale 1:50000 published in 2009. The total area of Anegunta watershed is 39.84 Sq.kms.

Methodology

To achieve the above objectives, the following methodology and procedure is adopted in the present study.

1.Procurement of high resolution satellite data, Survey of India Topographical maps, Collection of rainfall and temperature data and other collateral data covering the study area.

2.Preparation of base map on 1:50000 scale using Survey of India Topographical Maps.

3.Preparation of Settlement and Transport network map using SOI 1: 50,000 topographical maps.

4.Preparation of Drainage Map using SOI topographical maps and updating with the satellite data

5.Preparation of contour map of 5mts contour interval using SOI 1: 50,000 topographical maps.

6.Preparation of DEM (Digital Elevation Model) from contour map (Using 3D Analyst Module of Q-GIS 9.0 software).

7.Preparation of slope map using contours from SOI toposheet 1:50,000 scale.

8.Preliminary pre-field interpretation of Hydro geomorphology, soils and land use and land cover maps using Satellite data on 1: 10,000 Scale.

9.Ground truth data collection, verification of doubtful areas and Collection of representative water samples for chemical analysis from different lit units.

10.Correction, modification and transfer of post field details of Hydro geomorphology,soil and Land use / Land cover on to original maps.

11.Generation of land & water resources development plan.

Results and Analysis

Base Map

A base map is the frame to which all your ancillary data will be registered. Base maps can be river networks, shorelines or parcel maps to name a few. The base map allows all users to generate numerous data layers, such as schools, floodplains or drainage areas, at different times and places which will deop into an evolving spatial database.

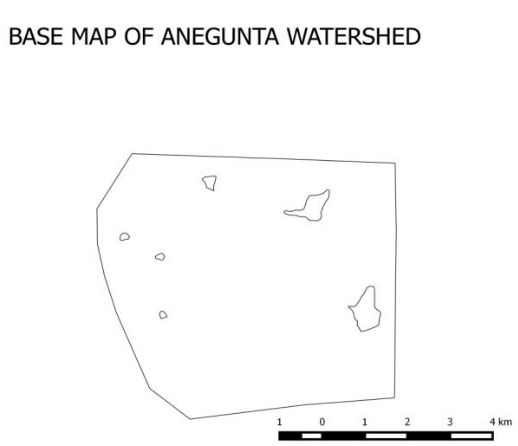


Fig 5.1 Base map of study area

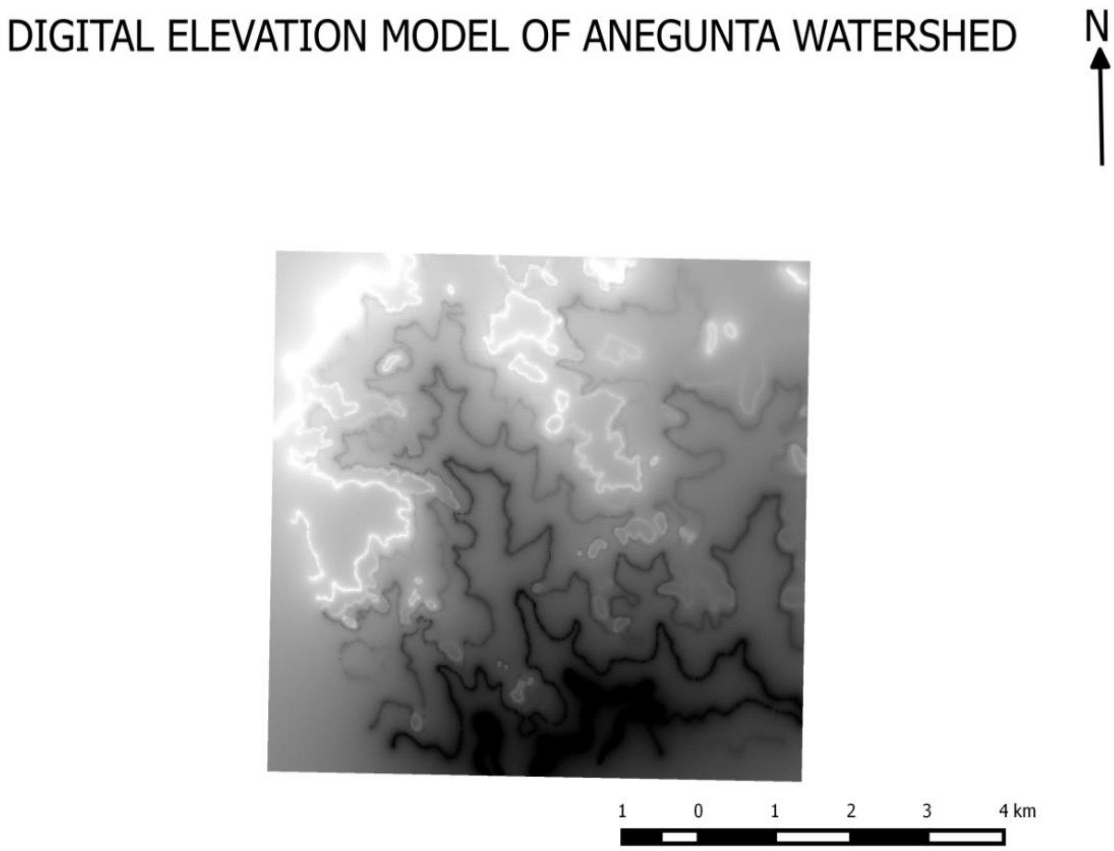
Drainage work:

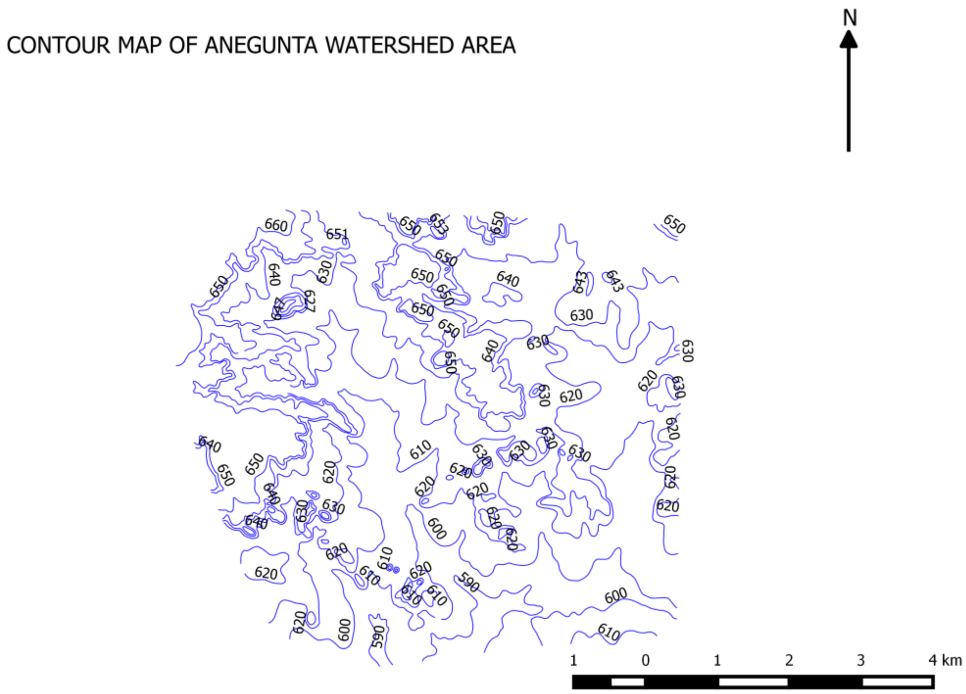
Generation of TIN & DEM:

In order to establish flow accumulation points and possible stream network an elevation raster has been created. For this contour data thus generated in the vector format has been used to generate TIN (Triangulated Irregular Network) using Q-GIS analyst functions Later this TIN has been converted to DEM (Digital Elevation Model) raster form by spatial analyst functions.

Fig 5.2 Drainage Network Map of Study Area

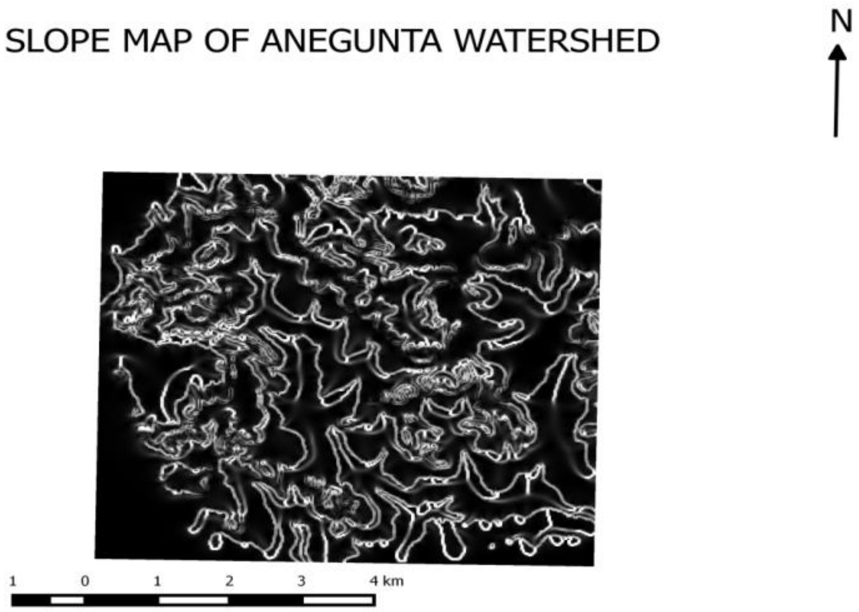
Contour map of study area

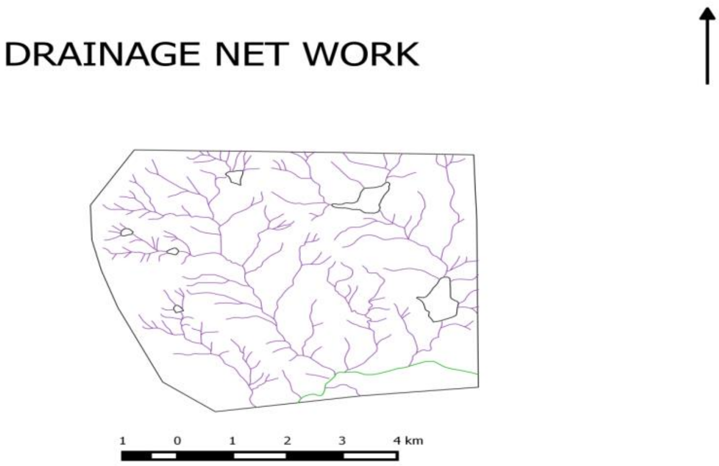


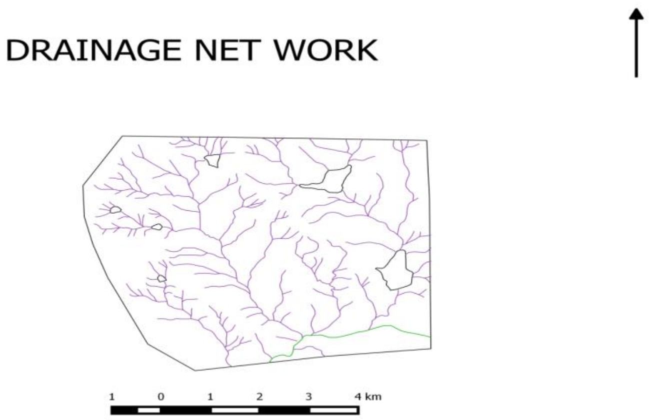


DEM of Study area

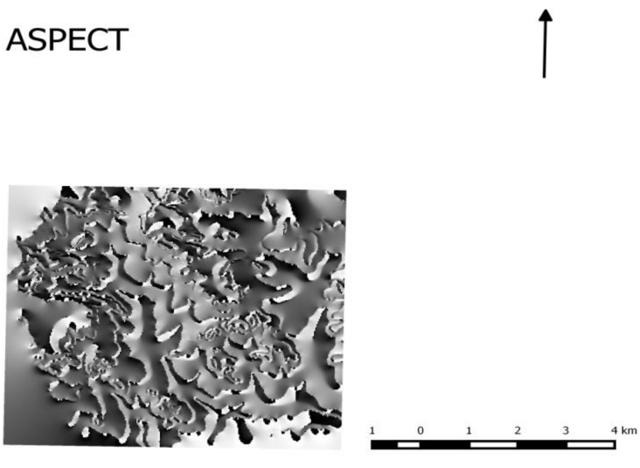
Slope and Aspect:







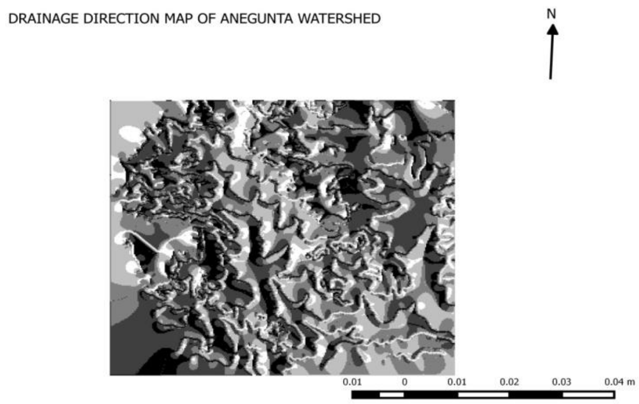
Slope map of study area



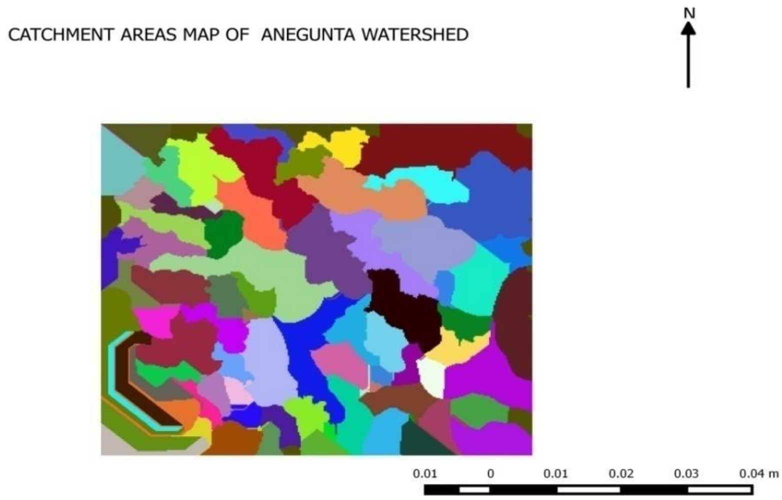
Aspect map of study area

GIS Analysis:

The hydrological analysis process in GIS is one of the effective methods in terms of cost and time in proposing various water harvesting structures. This process deals with assessing various hydrological characteristics of a surface. The basic parameter that controls the surface water flow (run-off) is the shape of the surface (terrain). Slope and aspect play a vital role in determining the shape of a surface. The basic inputs required to generate a hydrological model for a region are slope, aspect, sinks, flow direction, flow accumulation, pour points and a possible stream network. The whole hydrological process can be broadly divided into 2 phases i.e. (1) Surface analysis and (2) Hydrological analysis



Flow Direction map of Study Area



Catchment Area Map of Study Area

EFFECT OF GGBS CONTENT ON AGE OF FLY ASH AND GGBS BASED GEOPOLYMER MORTAR

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Abstract

Geopolymer is an innovative alternative material for cement concrete thus decreasing greenhouse emissions and leading to better construction practices and is produced by complete replacement of cement and water with fly ash and GGBS & alkaline activators. To examine the use of Geopolymer as replacement to cement, it is essential to study the properties of binders and their combinations in preparing geopolymer mix. Sodium silicate and sodium hydroxide (NaOH) solution were used as alkaline activator in this study. The present research aims at studying the effect of fly ash and GGBS combination on mechanical properties of Geopolymer mortars. The concentration of NaOH was maintained as 8M. The geopolymer samples were prepared with different combinations of Fly ash and GGBS (100GGBS-0FA, 75GGBS-25FA, 50GGBS-50FA, 25GGBS-75FA and 0GGBS-100FA) and Na_2SiO_3/NaOH ratio is taken as 2.5.The main evaluation technique in this study were compressive strength. The geopolymer samples were tested at different ages to study the effect of GGBS under outdoor condition. XRD analysis was done to study the mineralogical variations and the main minerals which are responsible for strength contribution at different ages for the combinations of fly ash and GGBS.The obtained results concluded that increase in GGBS content increases the compressive strength of geopolymer mortar increases and which is also shown by XRD analysis.

***Keywords**: Alkaline activator, Compressive Strength, outdoor curing, Geopolymerisation, Geopolymer Mortar, XRD analysis*

1. Introduction

Concrete is the most commonly construction material in the world. Ordinary Portland cement is used as the binding material to produce the concrete. In present days there is a rapid growth in infrastructure development which leads to a boom in the housing sector which leads to demand for the construction material cement. Cement production is highly energy intensive and which is a source for production of lot carbon dioxide (CO_2) into atmosphere. Due to the environmental concern there arises a strong realization which leads to sustainable development and make an alternative for the energy intensive and also the carbon reduction materials. There develops an interest over the alternative materials which can be used as binders for the production of the concrete. To overcome these problems, geopolymer system has been introduced that can completely eliminate cement with industrial by-products (Fly ash and GGBS) and water with alkaline solution. These alternative binders can be activated through the alkaline solution for the production of geopolymers. **Davidovits [1999]** recommended that binders could be formed by a polymeric chain reaction of alkaline liquids with alumina and silica present in the source materials of geological origin or by-products such as Fly ash, GGBS and rice husk ash. He termed these binders as geopolymers. **[Wang SD et al., 1995]** reported that NaOH and Na_2SiO_3 are commonly used in combination to form alkaline solution. This solution activates silica and alumina in fly ash to form alumino silicate hydrate and forms C-S-H by reacting with calcium in GGBS. **[Van Jaarsveld et al., 2003]** found that fly ash with higher amount of Cao produced higher compressive strength, due to the formation of calcium-aluminate hydrate and other calcium compounds, especially in the early ages. Alternate binders using fly ash are produced through an alkaline activation process which results in the formation of geopolymers.Geopolymerization is an exothermic process involving formation of complex geopolymers through a chemical reaction between alumina-silicates which are present in the raw materials and alkaline activators. The commonly used activators are combination of Sodium Hydroxide NaOH or Potassium Hydroxide KOH and; Sodium Silicate Na_2SiO_3 or Potassium Silicate K_2SiO_3. In general, activation can be done by only Sodium Hydroxide or only Potassium Hydroxide. However, the reaction process takes place at a slower rate in case of only hydroxides. Thus, the combinations of hydroxides along with silicates (sodium hydroxide or potassium hydroxide and; sodium silicate or potassium silicate) are more useful which accelerates the reaction rate and polymerization process. Concentration of Sodium Hydroxide plays a crucial role in the dissolution of elements present in the source materials. An increase in concentration leads to more dissolution of aluminosilicate which results in faster geopolymerization process. First step in polymerization process is the dissolution of solid reactants. This dissolution starts with the leaching of Al^{3+} and Si^{4+} ions from the source materials. Sodium based activators helps in higher rate leaching of aluminosilicate when compared to potassium based activators. Sodium silicate solution along with high concentration of sodium hydroxide solution produces a high alkaline environment which involves a better dissolution of solid particles (or) reactants which leads to leaching the Si and Al atoms subsequently leads to the reorientation, solidification reactions which involves some specific structures partially/totally amorphous to compacted cemented frame work. Dissolution of aluminosilicate leads to release of the monomeric aluminates and silicates from source materials. These monomeric elements interact with each other forms dimers, in turn these elements react with other monomeric elements to form trimmers, tetramers and so on.An experimental evaluation, combination fly ash and GGBS in the alkaline environment, the role of the activating solution and the type of product formed and the influence of the GGBS content on the compressive strength achieved are presented.In this study, an attempt has been made to prepare geopolymer mortars with fly ash and GGBS and hardened properties such as compressive strength were determined and Phase transformations were studied through XRD analysis.

2. Research Significance

The test data provide a measure of the geopolymer mortar specimens prepared by using different combinations of fly ash and GGBS as the source materials has the ability to gain the strength with short period of time so that engineers can feel self-assured in considering these new geo polymer mortars in structural constructions.

3. Experimental Program

3.1 Materials:

3.1.1. Fly ash and GGBS were used as source materials in the present study. GGBS was obtained from Toshali Cements Pvt ltd, Bayyavaram, India and fly ash was collected from National thermal power plant, Ramagundam, India. Specific gravity of fly ash and GGBS were 2.17 and 2.90 respectively. Chemical Composition details are shown in Table 1. The morphology of fly ash and GGBS were examined using Scanning electron Microscope (SEM) and are shown in figures 1-2. Fly ash particles were spherical in shape and are mainly composed of large percentages of silica and alumina. The shape of the GGBS grains is crystalline and angular form. From the EDAX, it can be observed that GGBS is predominated with calcium and silica compared to other elements. The calcium content of the slag results in raised basicity and increases the compressive strength. The mineralogical characterization of fly ash and GGBS sample were carried out by X-Ray diffraction analysis which is presented in figure 3. The XRD image of GGBS depicts glass content as 99%. The higher amounts of glass content helps in increasing the hydraulic activity thus accelerates polymerisation process.

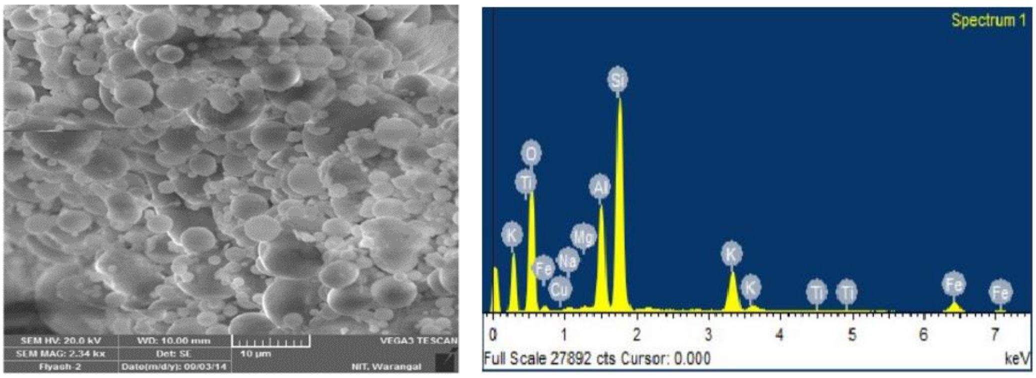


Fig 1: SEMand EDAX of fly Ash

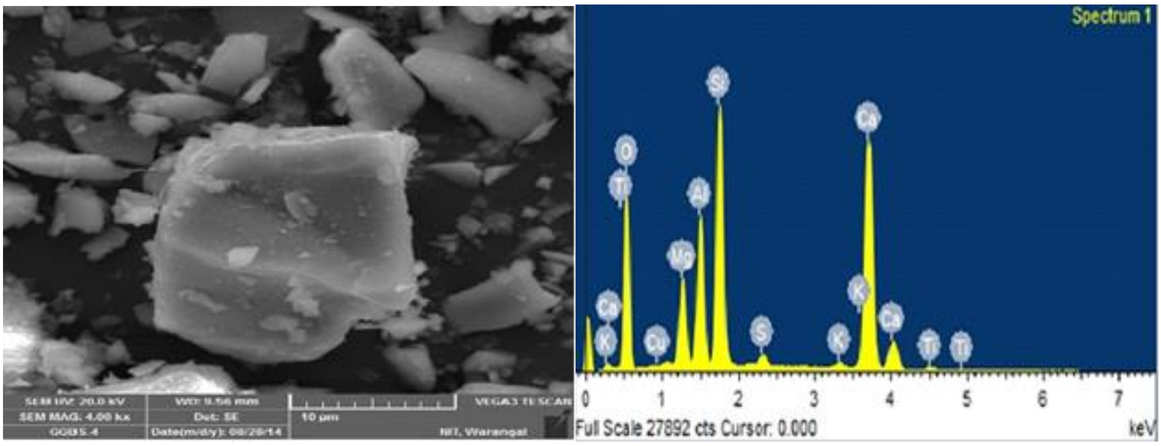


Fig 2: SEM and EDAX of GGBS

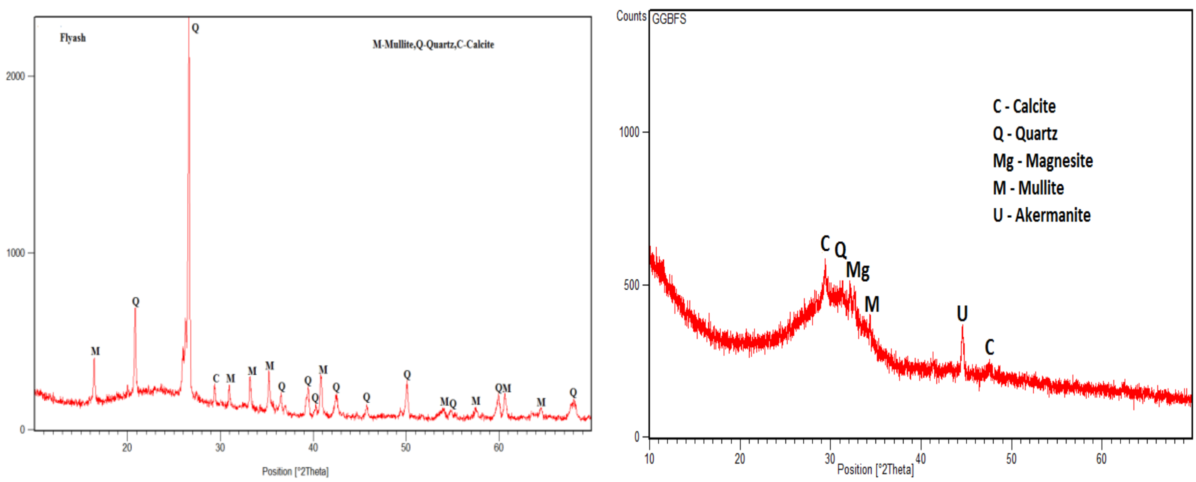


Fig 3: XRD analysis for fly ash and GGBS

Table 1. Chemical composition of fly ash and GGBS (% by mass)

Chemical Composition	Fly ash (% by mass)	GGBS (% by mass)
SiO_2	60.11	34.06
Al_2O_3	26.53	20
Fe_2O_3	4.25	0.8
SO_3	0.35	0.9
CaO	4.00	32.6
MgO	1.25	7.89
Na_2O	0.22	NIL
LOI	0.88	NIL

3.1.2. Fine Aggregate: The nearby river sand conforming to Zone-2 according to IS 383 (BIS, 1970) is preferred as fine aggregate. The specific gravity and bulk density of fine aggregate are 2.65 and 1.45 g/cm^3 respectively.

3.1.3. Water: Potable water was used in the experimental work for preparation of alkaline Solution.

3.2 Preparation of Alkaline Solution

In the present experimental work, the properties of geopolymer concrete were examined with Sodium Hydroxide 8M. The ratio of Na_2SiO_3/NaOH Solution is taken as 2.5.The mixed solution is stored for 24 hours at room temperature because the dissolution of NaOH in water is an exothermic reaction and a substantial amount of heat is generated. So in order to use the solution in concrete, it is to be left at room temperature until it cools down.

3.3 Casting and Curing of Geopolymer Mortar

For the preparation of the geopolymer mortars the individual dry materials (fine aggregate, fly ash and GGBS) were weighed and mixed using electrically operated mortar mixer. Fine aggregate, fly ash and GGBS are dry mixed before adding to the alkaline solution for 2 minutes in an electrically operated mortar mixer. The calcined source materials, fine aggregate and alkaline solution are mixed for another 10 minutes in mortar mixer to ensure homogeneity. Then, additional water was added to the mix to increase the workability and homogeneity of the mortar. Conventional table vibrator is used for compaction of the mortar. Steel moulds of dimensions 100mm × 100mm ×100mm are used for casting cube mortar specimens. The specimens are demoulded after 24 hours of casting and cured in outdoor. For outdoor curing, specimens are left out in outdoor (Temperature- 35±2^{0}C and relative humidity - 75%) up to specified age of testing. Temperature and humidity control is not necessary for outdoor cured specimens. Outdoor temperature curing is effective if the fly ash content is partially replaced by GGBS. The specimens were tested for 1, 3, 7 and 28 days.

3.4 Mix Proportions of Fly ash-GGBS Geopolymer Mortars

Five mixes were proposed according to the ratio of Fly ash -GGBS. The percentage ratio of Fly ash: GGBS were taken as 100GGBS:0FA, 75GGBS:25FA, 50GGBS:50FA, 25GGBS:75FA and 0GGBS:100FA for the source material. The ratio of Na_2SiO_3/NaOH was kept constant as 2.5. By assuming the alkaline liquid to binder ratios as 0.5. The molarity of alkaline activator is chosen as 8M. The density of geopolymer samples are varied is 2000-2200 kg /m^3. By knowing the density of mortars the amount of binder and quantity of alkaline liquids were determined. The mix proportions were shown in table 2.

Table 2: Mix Proportions of geopolymer Mortar of Sodium Silicate to Sodium Hydroxide ratio 2.5

Mix ID/ Proportion of binders	Fly ash (kg/m^3)	GGBS (kg/m^3)	Fine Aggregate (kg/m^3)	NaOH (kg/m^3)	Na_2SiO_3 (kg/m^3)	Alkaline liquid (kg/m^3)
	Na_2SiO_3/NaOH = 2.5					
$M_1F_{100}G_0$	880	0	880	125.7	314.28	440
$M_2F_{75}G_{25}$	660	220	880	125.7	314.28	440
$M_3F_{50}G_{50}$	440	440	880	125.7	314.28	440
$M_4F_{25}G_{75}$	220	660	880	125.7	314.28	440
$M_5F_0G_{100}$	0	880	880	125.7	314.28	440

3.5 Testing Procedure for Compressive Strength

Standard Cube specimens of size 100mm × 100mm × 100mm were cast and tested under compression using 2000 KN compression testing machine at standard rate of loading suggested by **IS 516 (BIS, 1956)**. The strength results reported at 1, 3, 7 and 28 days were the average values of three cubes.

4.0 Results and Discussion

The results of the different tests are discussed and analyzed to study the strength properties of geopolymer Mortars. Five different mixes are prepared varying Fly ash and GGBS 100GGBS:0FA, 75GGBS:25FA, 50GGBS:50FA, 25GGBS:75FA and 0GGBS:100FA with Alkaline/Binder Ratio is maintained constant for all the mixes as 0.5 for different Na_2SiO_3/NaOH ratio is taken as 2.5 and curing regime is selected as outdoor in order to eliminate the oven curing. Three cubes of each geopolymer mortar set withdimensions 100mm × 100mm × 100mm were cast and tested in compression for 1,3 7 and 28days compressive strength.

4.1 Compressive Strength

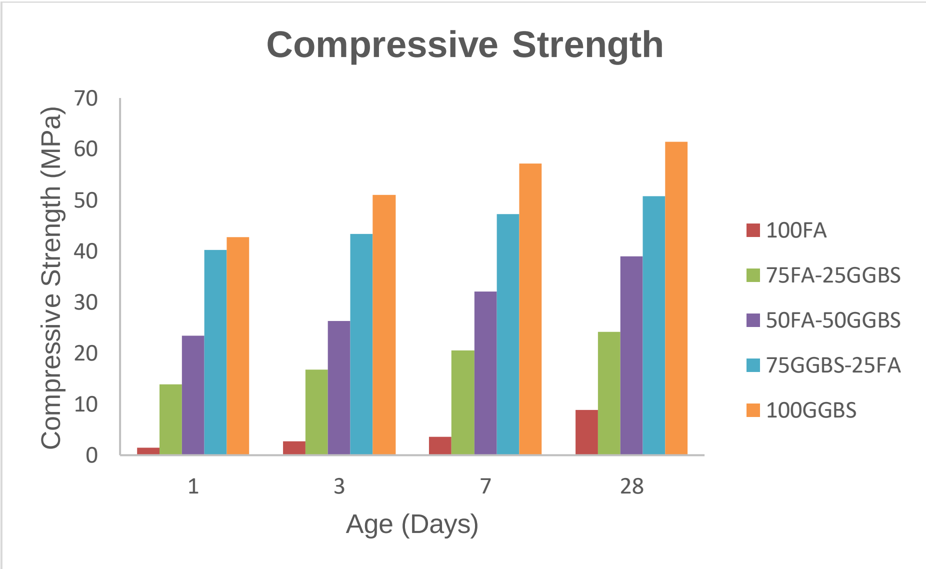


Figure 4: Compressive strength results of fly ash – GGBS based geopolymer mortars

The 28 days compressive strength of geopolymer mortar prepared with 8M NaOH solutions ranged from 3.55MPa to 61.4MPa respectively. The specimens prepared with 100% fly ash has very low strength compared to specimens with 100% GGBS. The 1 day strength is very minimum and increases with increase in GGBS content. High early strength can be achieved with higher proportion of GGBS. Only 16.16% of 28 day compressive strength is obtained for 1 days for a cube prepared with 100% fly ash. Whereas for cube with 100% GGBS, 90% of 28 day compressive strength is obtained within 7 days. The reason for increase in compressive strength due to GGBS can be attributed to higher calcium content present in GGBS. To utilize both fly ash and GGBS, a mix with 50% fly ash and 50% GGBS is desirable. GGBS plays an important role for compressive strength development. A higher dosage of GGBS results in a higher compressive strength of geopolymer mortar. The compressive strength of geopolymer mortar ranges from 3.55 MPa to 61.4 MPa. Mix with 100% fly ash shows the less compressive strength among all the mixes. The mixes with 75FA-25GGBS, 50FA-50GGBS, 25FA-75GGBS and 0FA-100GGBS shows the higher compressive strength with inclusion of GGBS content in the mix produced the highest strength, while a further decrease in the GGBS content reduced the compressive strength. Another reason is that the quantity of soluble Calcium content depends on the volume of GGBS present in the mixture, which has a direct effect on the compressive strength. The attainment of the strength in GPC mainly depends on the GGBS content in the mix. The calcium present in the GGBS content dissolution will be more compared with silica and alumina present in the fly ash. Due to the faster dissolution the formation of the dissolution reactants like N-A-S-H gel and C-A-S-H gel will be formed and which contribute the strength for Geopolymer mortar. The differences in the content and ratios of the main elements as a function of the partial replacement (Fly and GGBS) composition indicated the formation of the reaction products with different compositions and structures (mineralogical variation) studied through XRD analysis. In alkali-activated 100% Fly ash the formation of alkali aluminosilicate geopolymer gel (N–A–S–H gel) albite and microcline took place, while the main reaction product of alkali-activated GGBS was calcium silicate hydrate (C-A-S-H) gel (Anorthite) was found. These minerals are responsible for the strength. Mix with 100% fly ash shows the less compressive strength among all the mixes. The mixes with 75FA-25GGBS, 50FA-50GGBS, 25FA-75GGBS and 0FA-100GGBS shows the higher compressive strength with inclusion of GGBS content in the mix produced the highest strength, while a further decrease in the GGBS content reduced the compressive strength. Another reason is that the quantity of soluble Calcium content depends on the volume of GGBS present in the mixture, which has a direct effect on the compressive strength. The calcium content present in the mix plays an important role in attaining the strength of GPC.As the GGBS content in the mix increases the strength of the geopolymer is also increases. The attainment of the strength in GPC mainly depends on the GGBS content in the mix. The calcium present in the GGBS content dissolution will be more compared with silica and alumina present in the fly ash. Due to the faster dissolution the formation of the dissolution reactants like N-A-S-H gel and C-A-S-H gel will be formed and which contribute the strength for Geopolymer mortar.

4.1 Effect of age on strength of geopolymer Mortars

The work has attempted to estimate the strength of geopolymer mortar at 1, 3, 7 and 28 days strength. The strength of geopolymer depends mainly on curing regime, type of binder content and molarity of alkaline activator and the ratio of sodium silicate and sodium Hydroxide Solution. The gain of strength for the geopolymer mortars was faster at early age compared to that later age. This was observed in all the mixes of fly ash and GGBS combinations. The mortar cubes were cast and cured at the age of 1, 3, 7 and 28 days were tested for evaluating its compressive strength. The initial curing temperature influences the polymerization process but the specimens were left under outdoor condition. At the initial ages the gain of the strength was more for all the mixes of fly ash and GGBS combination where as for 100FA-0GGBS the gain of strength was lessat the initial ages compared with the other mixes. For the fly ash mixes in outdoor curing the gain in the strength is less due to slow polymerization process. Compressive strength of all the combinations increased over the time and higher at 28 days. The gels which are formed during polymerization is responsible of the mechanical development of fly ash and GGBS based materials. With the age of curing, a higher amount of gels was formed. The mixes containing the more slag gives the better strength than other mixes due to their chemical composition and their slag content had much effect on strength development of the geopolymer samples at any age.

Table 3: Percentage gain in Strength of Geopolymer Mortar

Mix ID	1 Day	3 Days	7 Days	28 Days
$M_1F_{100}G_0$	16.16	30.26	40.15	100
$M_2F_{75}G_{25}$	57	69	84	100
$M_3F_{50}G_{50}$	60	67	82	100
$M_4F_{25}G_{75}$	79.23	85.45	81.33	100
$M_5F_0G_{100}$	69.54	83	93	100

4.2 Ultrasonic Pulse Velocity test for Fly ash and GGBS based Geopolymer Mortars

The UPV test is a measure to presence of voids and the consistency of mortar. The 28 days UPV for 100FA:0GGBS mix was found 2.58 km/s for outdoor cured specimens where as for the 100GGBS:0FA specimens it is in the range of 3.78 km/s where as for 100GGBS:0FA there is little rise in the values compared to that of 100FA:0GGBS. **Whitehurst** classified fly ash based geopolymer concrete as excellent, good, doubtful, poor and very poor for UPV values of 4.5 km/s and above, 3.5–4.5, 3.0–3.5, 2.0–3.0 km/s and below 2.0 km/s, respectively. Generally, high pulse velocity reading in mortar is indicative of mortar of good quality. The presence of voids has been recognized to have an influence on the UPV transmission. The measured 7 and 28 days UPV values for all geopolymer mortar specimens are presented in **table 4.** For the samples, velocities at Outdoor curing are found within 3.0 to 3.5 km/sec and these can be treated as of good quality. There is no proper guidelines for the geopolymer specimens. Here the microstructure is quite different compared with the conventional mortar. In geopolymer specimens for the attainment of strength polymerization process plays an important role. The quality of the concrete mainly depends on the internal microstructure formed. In microstructure of geopolymer mortar be further investigated and reason for the less UPV value should be find out and more investigation should be done.

Table 4 UPV results of fly ash – GGBS based geopolymer mortars (Km/s)

Percentage Variation of Fly ash and GGBS	**Age (Days)**	
100GGBS-0FA	3.69	3.78
75GGBS-25FA	3.56	3.67
50GGBS-50FA	3.4	3.45
25GGBS-75FA	3.19	3.24
0GGBS-100FA	1.55	2.58

4.3 XRD analysis Fly ash – GGBS based Geopolymer mortars

XRD analysis was conducted on geopolymer powdered samples obtained after crushing the cube. XRD analysis is done on all the samples at respective ages of 1, 3, 7 and 28 days and is shown in Figures 5 to 9**.** From XRD patterns, it is evident that some dissolvable minerals such as quartz and Mullite have remained in all the products. The broad band **(between 26^0 and 30^0)** observed in the XRD patterns of the fly ash and GGBS based geopolymer shows that the geopolymer material has both semi-crystalline and amorphous structure, and that of the reaction products between 26^0 and 30^0 with relatively smaller intensities indicates that the geopolymer material has almost complete amorphous structure. By comparing the patterns of geopolymer mortar by varying the sodium silicate to sodium hydroxide, significant change was observed in geopolymer mortar with replacement of fly ash with GGBS (25%, 50%, 75% and 100%). The major components of raw fly ash are quartz (SiO_2) and Alumina (Al), sodium-aluminum-silica complex such as albite and microcline. The other trace materials present in this curve were quartz (SiO_2), mullite (Al_2O_3 SiO_2) and calcite ($CaCO_3$). The presence of a broad elevation, i.e., hump from 20° to 36°, indicates the presence of amorphous silicates. Amorphous compounds dissolve easier than crystalline compounds during the first step of geopolymerization (dissolution of species), yielding higher amounts of reactive SiO_2 and Al_2O_3 to combine during the precipitation of the geopolymeric reaction product which is responsible for the mechanical strength. **(Fernandez-Jimenez and Palomo**

2003). For 100% fly ash based samples the main mineral identified as analcime along with quartz, mullite and calcite which is responsible for the strength contribution. The geopolymer consisted mainly of amorphous aluminosilicate products with similar or very slightly increased in amount of crystal of predominantly quartz and mullite from fly ash. Increase in crystalline products increased compressive strength of geopolymer.

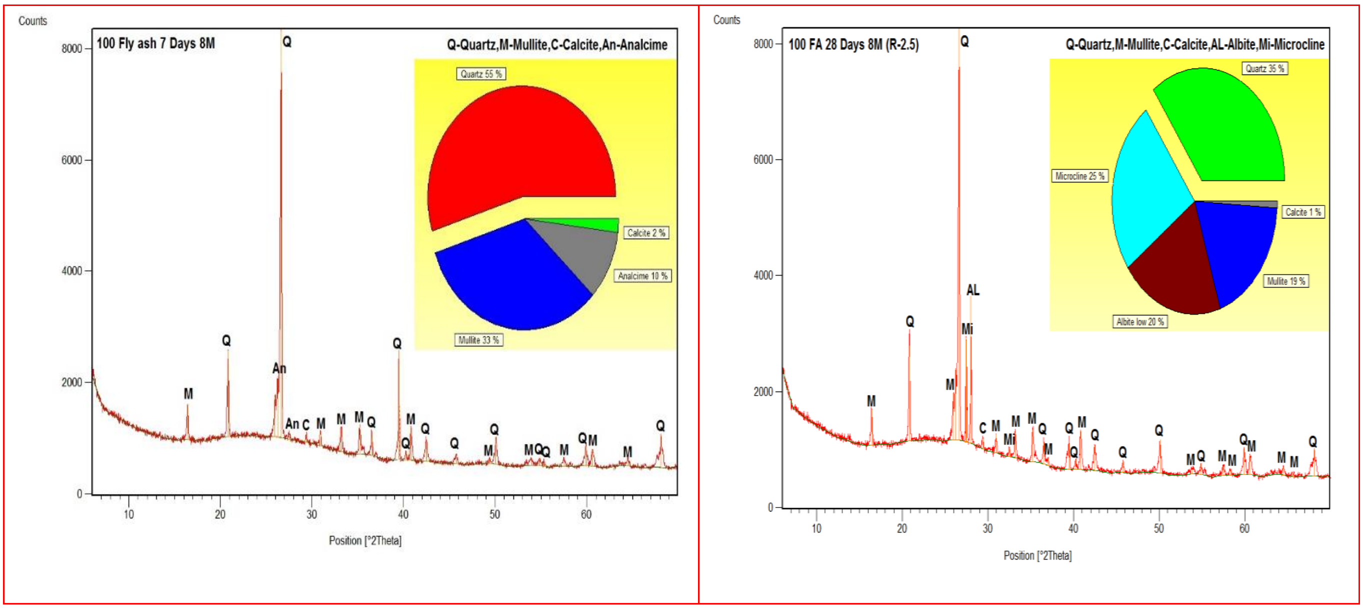
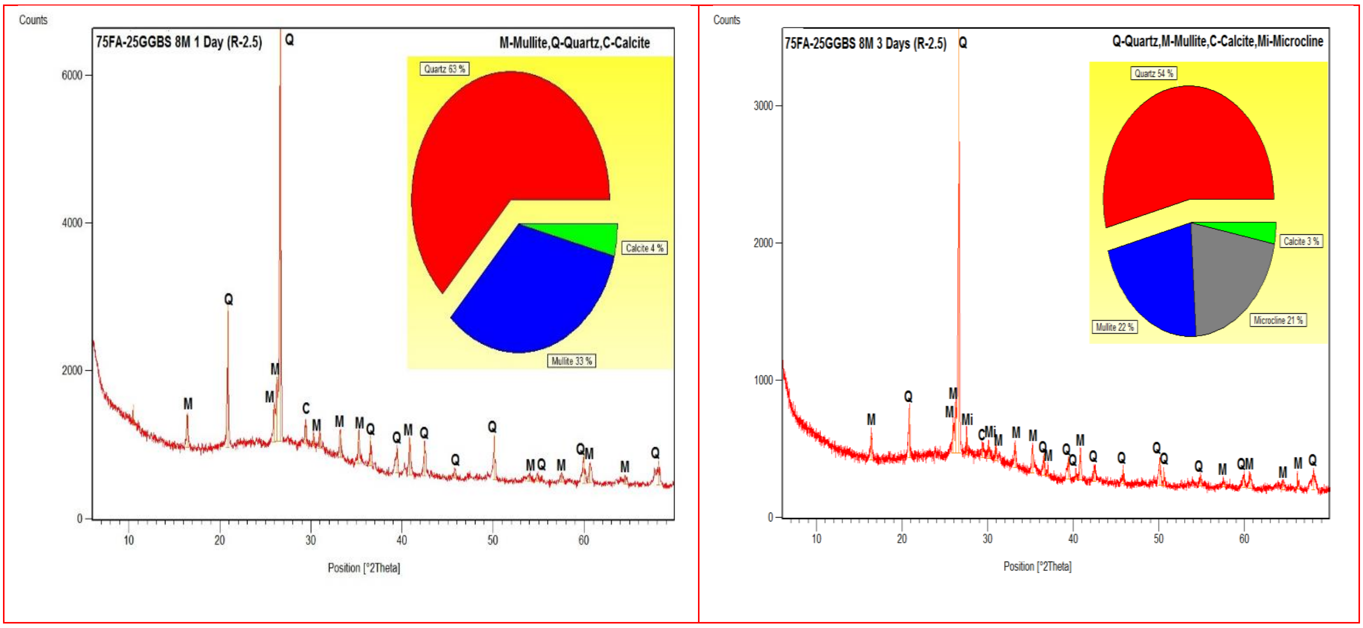


Fig 5: XRD analysis for 100 FA-0GGBS with sodium silicate to sodium hydroxide ratio 2.5 for different ages



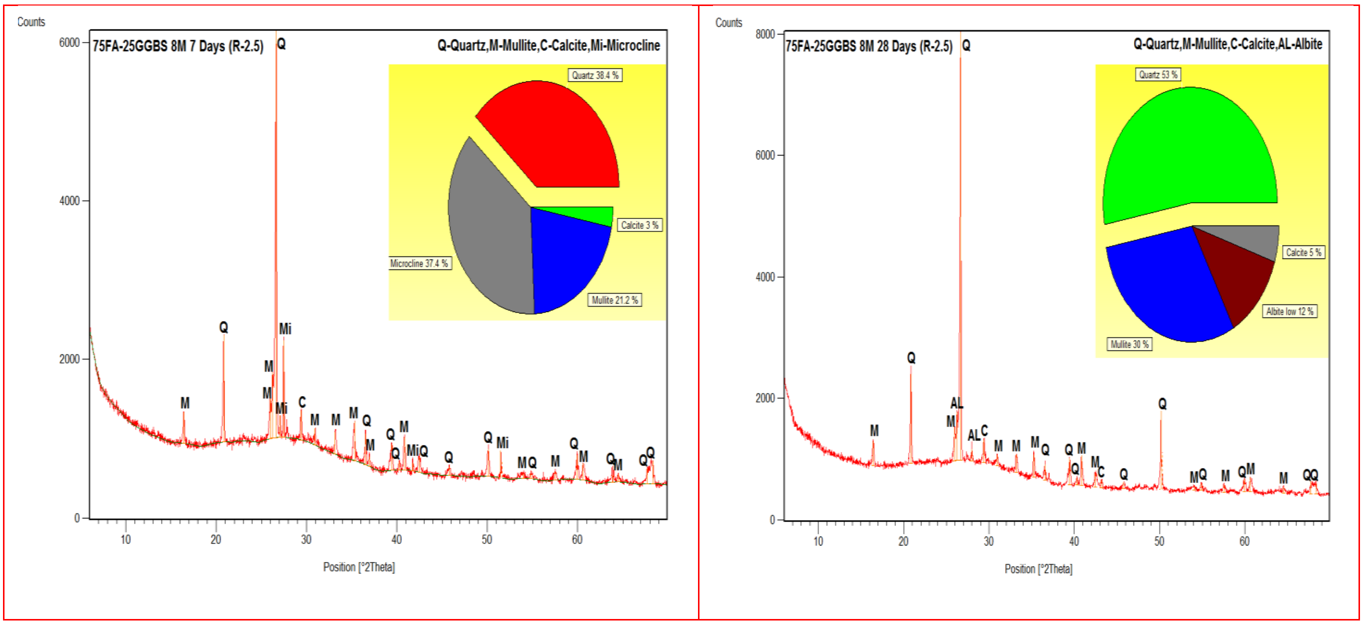
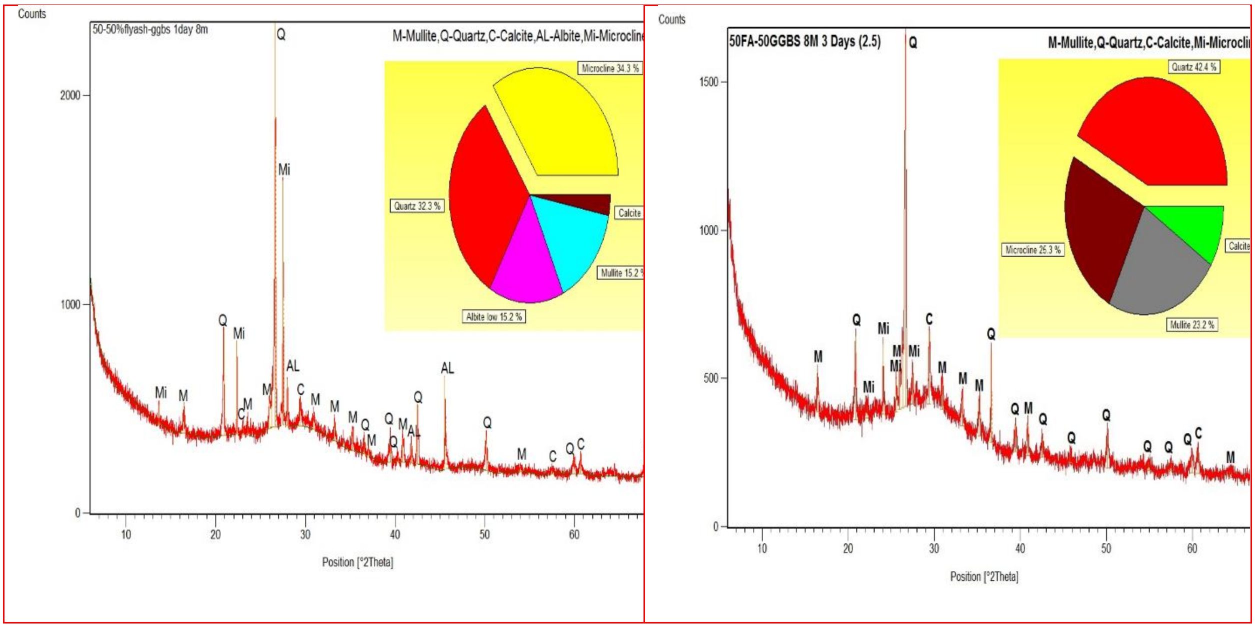


Fig 6: XRD analysis for 75FA-25GGBS with sodium silicate to sodium hydroxide ratio 2.5 for different ages



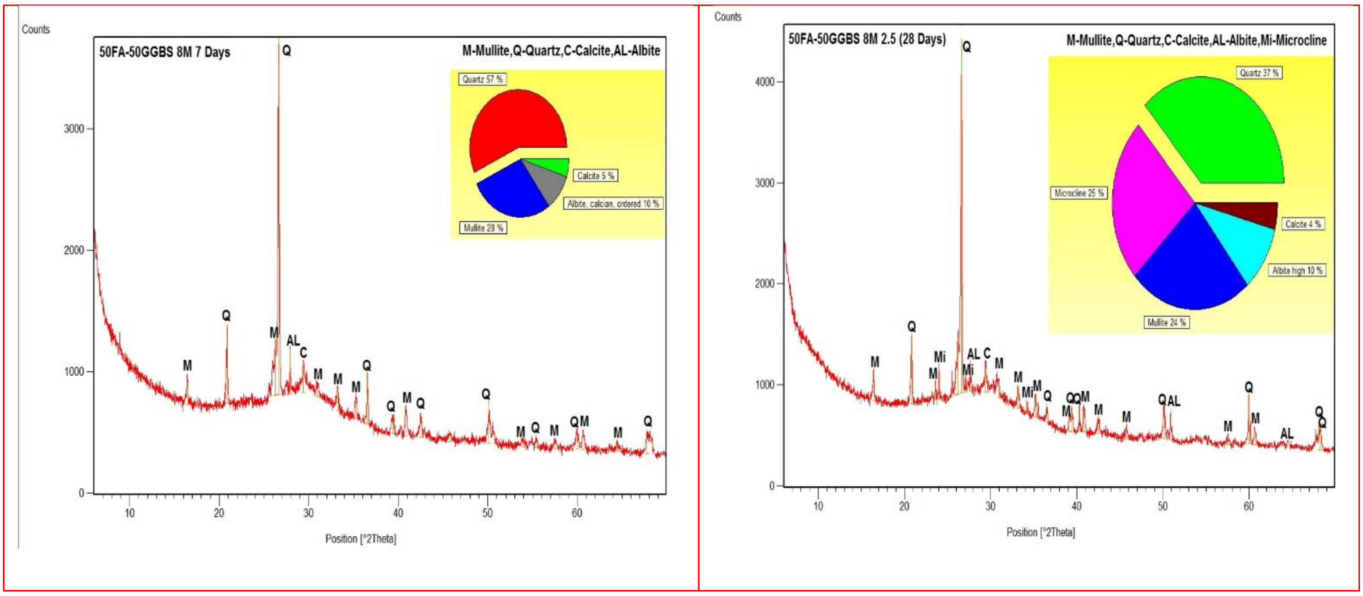
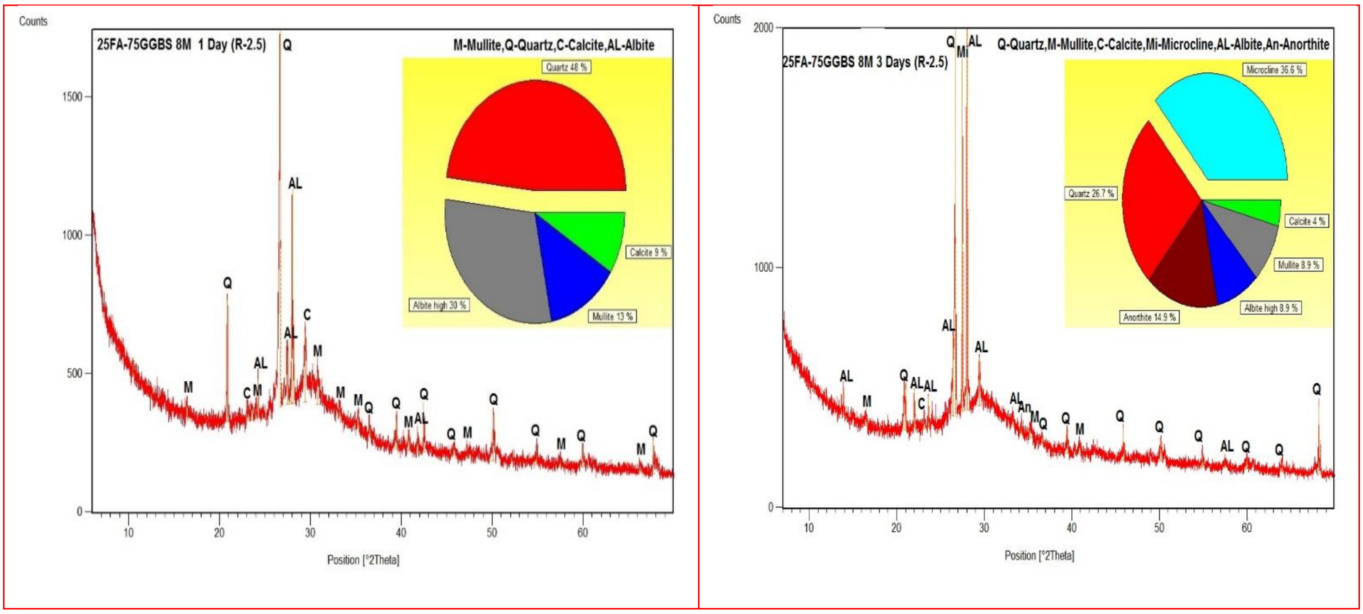


Fig 7: XRD analysis for 50FA-50GGBS with sodium silicate to sodium hydroxide ratio 2.5 for different ages



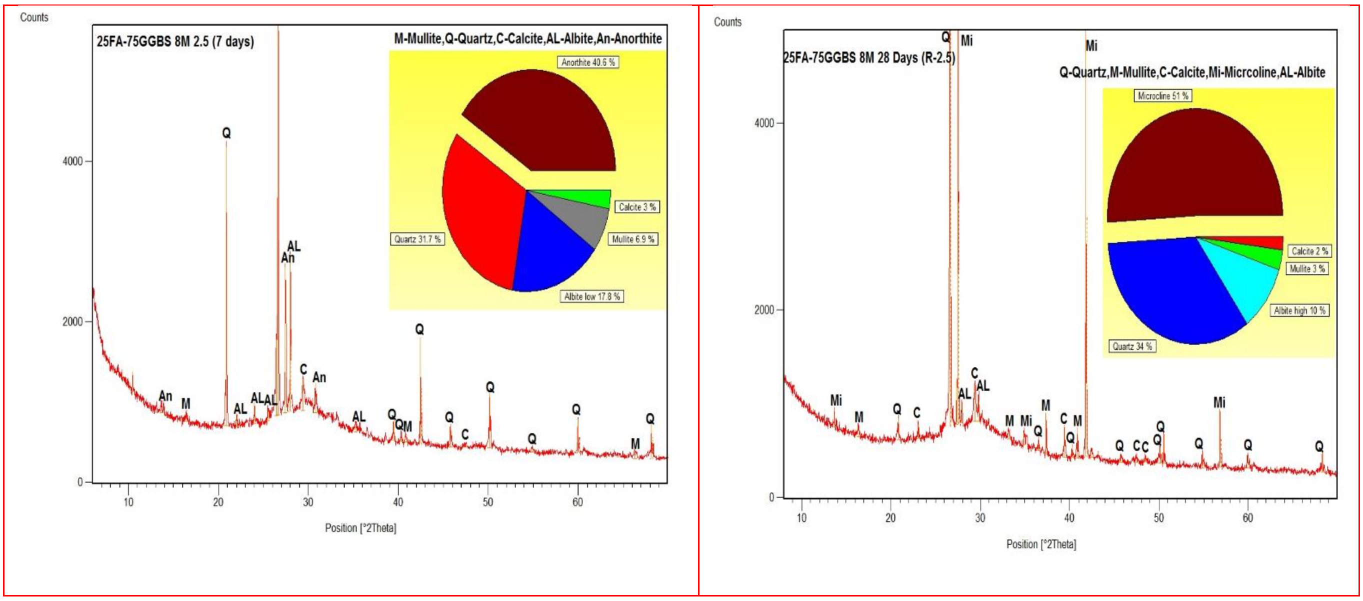
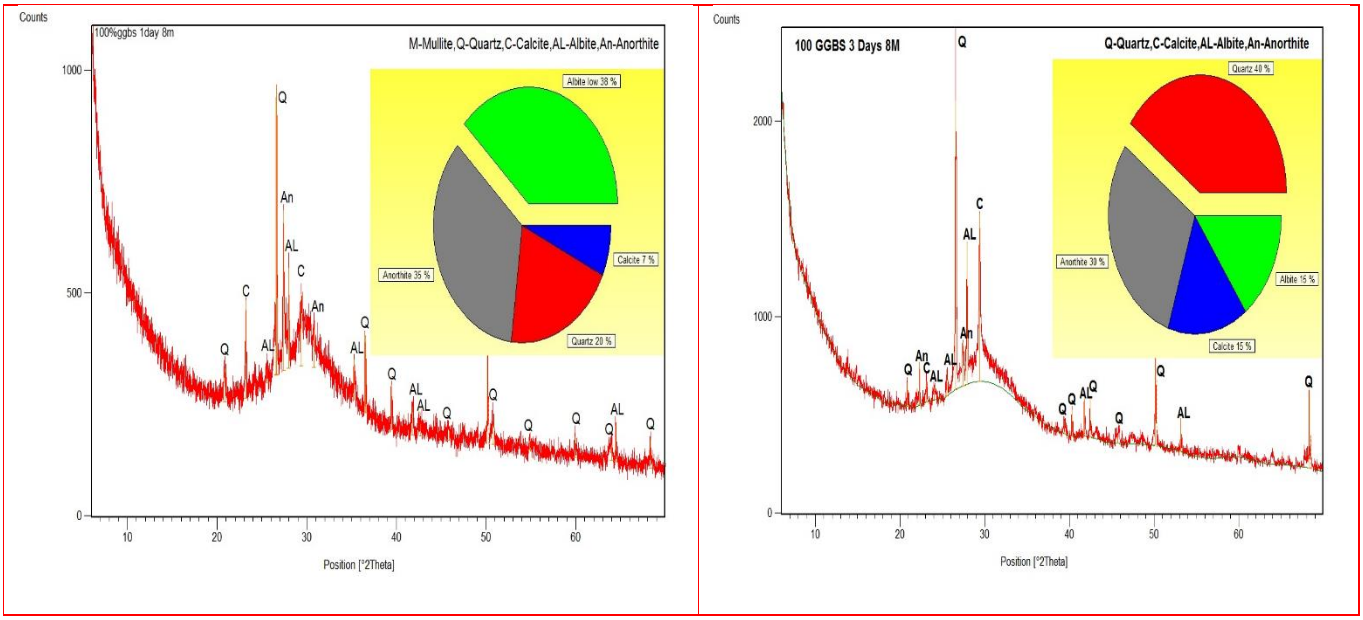


Fig 8: XRD analysis for 25FA-75GGBS with sodium silicate to sodium hydroxide ratio 2.5 for different ages



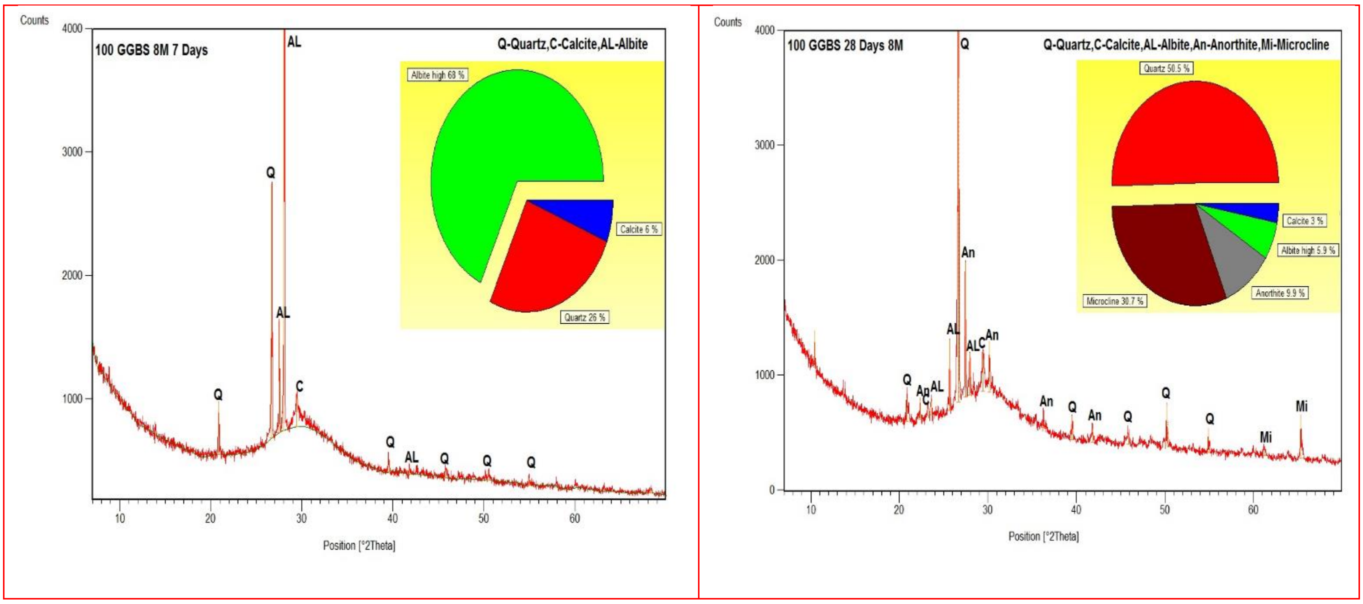


Fig 9: XRD analysis for 0FA-100GGBS with sodium silicate to sodium hydroxide ratio 2.5 for different ages

5 Conclusions

From the experimental investigation carried out on GPM,the following conclusions were arrived

1. Compressive strength of geopolymer mortar increases with increase in percentage replacement of fly ash with GGBS.
2. To develop geopolymer concrete under outdoor curing condition, combination of fly ash with GGBS is a possible solution.
3. For 100% fly ash based samples the main mineral identified as analcime along with quartz, mullite and calcite which is responsible for the strength contribution.
4. The minerals identified at 100% GGBS were albite, microcline and anorthite. These minerals are responsible for the strength gain and the maximum strength was observed for 100% GGBS mix and the N-A-S-H and C-A-S-H gel is responsible for the strength.
5. For the combinations of fly ash and GGBS, the minerals albite and microcline were responsible for improving the compressive strength of fly ash and GGBS based geopolymer mortars.

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PERFORMANCE OF BASALT ROCK FIBRE IN STRENGTH CHARACTERSTICS OF CONCRETE

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Abstract

In this paper, the new material Basalt rock fibre can improve the properties of concrete. The aim of this research was to evaluate the performance of high strength concrete (HSC) containing supplementary cementations materials. Concrete had a good future and is unlikely to get replaced by any other material on account of its ease to produce , infinite variability , uniformity, durability and economy with using of basalt fibre in high strength concrete. The point of choosing the basalt rock fibre, it has a good proportions of silica which can improve the strength of concrete. In this project basalt fibre is replaced partially with 0.5%,1%,1.5%,2%,2.5% to the weight of cement and examined over the properties by performing slump cone test, compressive strength test and tensile strength test.Finally, compared the basalt fibre concrete with high performance concrete (M30) with good domination over ordinary concrete. I can conclude basalt rock fibre is advantageous than ordinary concrete.

Keywords*: Basalt rock* fibre, high performance concrete.

1.0 OBJECTIVE OF WORK

The main objective of this experimental work is to investigate the behaviour of basalt fibre in concrete in which the cement is replaced partially with basalt fibre by 0.5%,1%,1.5%,2%,2.5%. The different characteristics studied such as workability, compressive strength, tensile strength. A design mix of M30 is prepared by using IS 10262:2009. 28 days water curing was adopted for all the testing specimens and repeated for the basalt fibre concrete specimens. Strength characteristics of basalt fibre concrete are compared with the reference mix (M30 concrete without fibre).

Fig.1.basalt fibre

2.0 Preliminary Experimental Programs

Materials used in the experiment are namely cement, coarse aggregate, fine aggregate and other admixtures have tested in the laboratory. According to Indian Standards the study was presented in this chapter.

2.1 Test results of cement:

Table1. Test results of ement

Sl.no	Test	Result	Requirement as per IS code	IS code number
1	Normal consistency	32%	26-33%	IS 4031 (part-4)
2	Specific gravity	3.11	3-3.2	IS 2720 (part-3)
3	Fineness	1%	10%	IS 4031 (part-2)
4	Initial setting time	3mm(30 minutes)	30-60 minutes	IS 4031 (part-5)
5	Final setting time	1 mm(600 minutes)	600 minutes	IS 4031 (part-5)
6	Soundness test	5 mm elongation	7mm	IS 4031(part-3)
7	Compressive strength	25.8N/mm^2(3 days) 38N/mm^2(7 days) 54N/mm^2(28 days)	27.8 N/mm^2 37.8N/mm^2 53.8 N/mm^2	IS 4031 (part-7)

2.2 Fine Aggregates:

Fine aggregate is natural sand which has been washed and sieved to remove particles larger than 5 mm and coarse aggregate is gravel which has been crushed, washed and sieved so that the particles vary from 5 up to 50 mm in size**.** . The reason for using a mixture of fine aggregate is that by combining them in the correct proportions, a concrete with very few voids or spaces in it can be made and this reduces the quantity of comparatively expensive cement required to produce a strong concrete.

Table 2. Test results of fine aggregates

Sl no.	Test	Result	Is code
1	Specific gravity	2.605	IS 2386 (Part-3) -1963
2.	Water absorption	1%	IS: 383–1970
3	Fineness modulus	3.52	IS 2386 (Part-1) -1963

2.3 coarse aggregates:

Properties of aggregates which influence the properties of both the fresh and the hardened concretes are mainly the particle size distribution, the maximum size of particles and the shape and the surface texture of the particles. Furthermore, the density and porosity together with water absorption and moisture content have to be considered when the concrete is proportioned.Locally available crushed aggregates confirming to IS: 383–1970 were used in this project work

Table 3 Test results of Coarse Aggregates.

Sl.no:	Test	Results	IS code
1	Specific gravity test	2.84(for 20mm aggregates)	IS 2386(part 3)
2	Water absorption	0.5%	IS 383-1970
3	Impact value(%)	17.14	IS 4031 part 4
4	Crushing value(%)	26.13%	IS 4031 part 4
5	Flakiness index(%)	29	IS 2386 part 1
6	Elongation index(%)	27	IS 2386 part 1
7	Fineness modulus(%)	3.875	IS 2386 (Part I) – 1963

2.4 Basalt Fibre:

Table 4.Properties of basalt fibre

Property	Value
Tensile strength	4.84 Gpa
Elastic modulus	89 Gpa
Elongation at break	3.15%
Density	2.7g/cm^3
Aspect ratio	1500
Filament diameter	9-12 microns

3. Mix Design and Experimental Program

Water/cement: Cement: Fine Aggregate: Coarse Aggregate.

0.4	1	1.296	2.509

Table 5.Mix design for different grades of concrete

Sl.no	Design strength	Cement Kg/m^3	Fine aggregates	Coarse aggregates	water	w/c	Basalt fibre %	Basalt fibre in weight (kgs)
1	M 30	478.620	620.753	1201.820	190.6	0.4		-
2	0.5% cement replacement	476.227	620.753	1201.820	190.4	0.4	0.5	2.3931
3	1% cement replacement	473.833	620.753	1201.820	189.3	0.4	1	4.787
4	1.5% cement replacement	471.440	620.753	1201.820	188.6	0.4	1.5	7.18
5	2% cement replacement	469.047	620.753	1201.820	187.6	0.4	2	9.573
4	2.5% cement replacement	466.654	620.753	1201.820	186.7	0.4	2.5	11.966

4. Results and Discussion

Table 6.WORKABILITY TEST RESULTS BY SLUMP CONE TEST

Percentage of basalt fibre by weight fraction	Slump observed (mm)
0.00%	30
0.50%	15
1%	13
1.50%	7
2.00%	0
2.50%	0

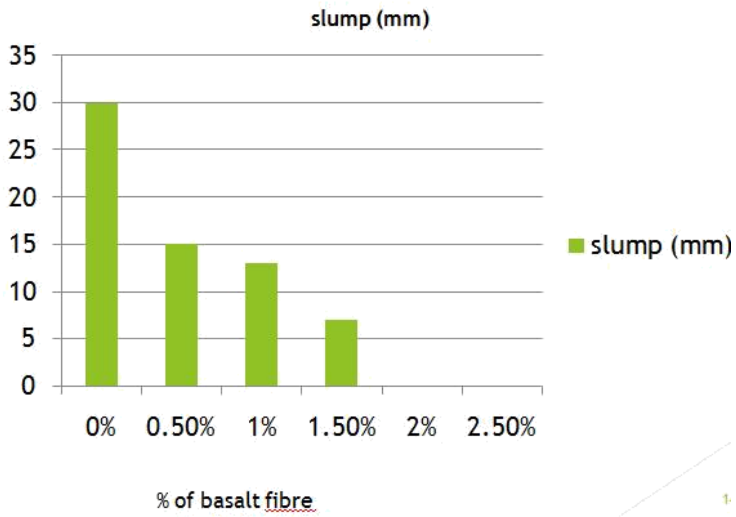

Fig 2 Bar chart between % of basalt fibre and variation of slump

4.1 Compressive Strength Test Results:

The casted cubes of M30 grade concretes are cured, exposed to temperatures and tested for its compressive strength at 7, 15 and 28 days after cooling to room temperature. The compressive strength increases gradually upto 1.00% and decreases gradually. Hence the optimum percentage is 1.00%

Table 7 compressive strength at 7 days

Percentage of basalt fibre by weight fraction	Compressive strength (Mpa)	Percentage increase or decrease in compressive strength w.r.t reference mix.
0.00%(reference mix)	25.21	-
0.50%	40	58.66%
1.00%	57.7	128%
1.50%	17.7	29.7%(-)
2.00%	16.88	33.4%(-)
2.50%	15.55	38.319%(-)

Table 8 compressive strength at 15 days

Percentage of basalt fibre by weight fraction	Compressive strength (Mpa)	Percentage increase or decrease in compressive strength w.r.t reference mix.
0.00% reference mix	30	-
0.50%	61.3	104.4%
1.00%	61.3	104.4%
1.50%	31.5	18.3%
2.00%	26.66	11.3%(-)
2.50%	22.22	25.93%(-)

Table 9 compressive strength at 28 days

Percentage of basalt fibre by weight fraction	Compressive strength (Mpa)	Percentage increase or decrease in compressive strength w.r.t reference mix.
0.00% (reference mix)	37.21	-
0.50%	61.33	64.82%
1.00%	64.4	73.17%
1.50%	40	7.49%
2.00%	31.1	16.4%(-)
2.50%	24	34.42%(-)

4.2 Tensile Strength Test Results:

The casted cubes of M30 grade concretes are cured, exposed to temperatures and tested for its tensile strength at 15 and 28 days after cooling to room temperature. The tensile strength of concrete increases upto 1% and decreases gradually. Hence, the optimum percentage is 1.00%

Table 10 Tensile strength result at 15 and 28days

Percentage of basalt fibre by weight fraction	Tensile strength (Mpa) 15 days	Percentage increase or decrease in tensile strength w.r.t reference mix	Tensile strength (Mpa) 28 days	Percentage increase or decrease in tensile strength w.r.t reference mix
0.00%(reference mix)	2.26	-	2.56	-
0.50%	2.88	27.43%	3.53	37.89%
1.00%	2.97	31.41%	3.61	14.01%
1.50%	2.78	23.00%	33	30.07%
2.00%	2.68	18.58%	3.24	26.58%
2.50%	2.57	13.57%	3.11	21.48%

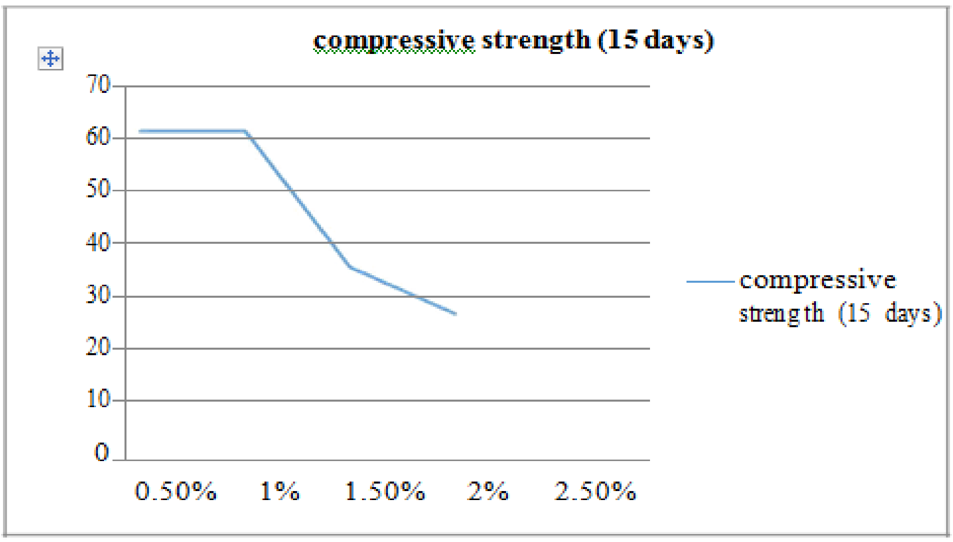

Fig.3.Graph between % of basalt fibre and variation of compressive strength at 15 days

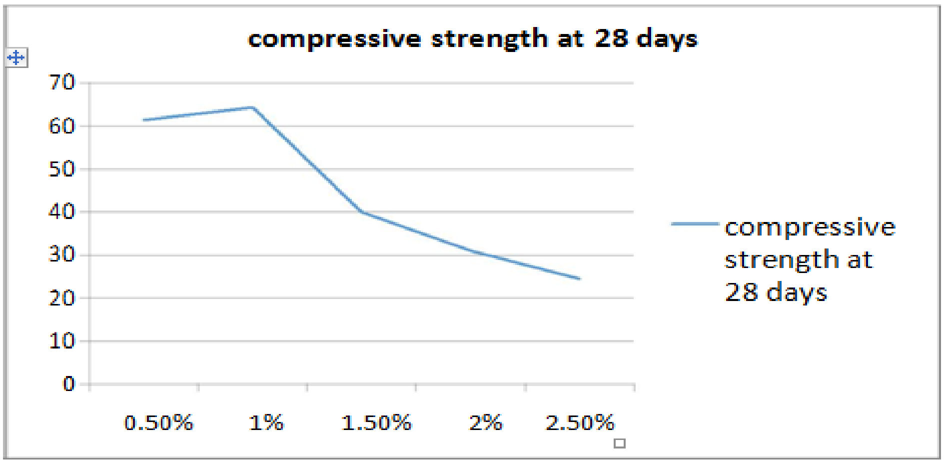

Fig.4 Graph between % of basalt fibre and variation of compressive strength at 28 days

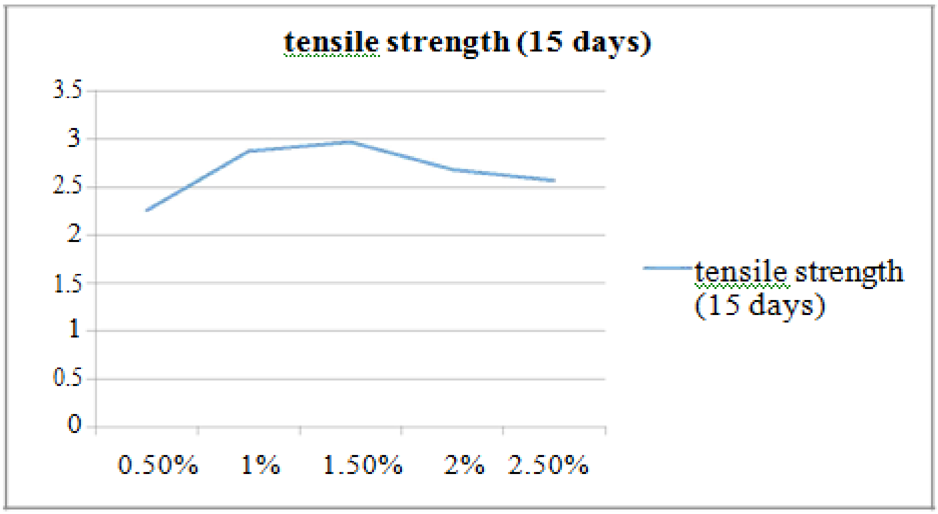

Fig.5.Graph between % of basalt fibre and tensile strength at 15 days

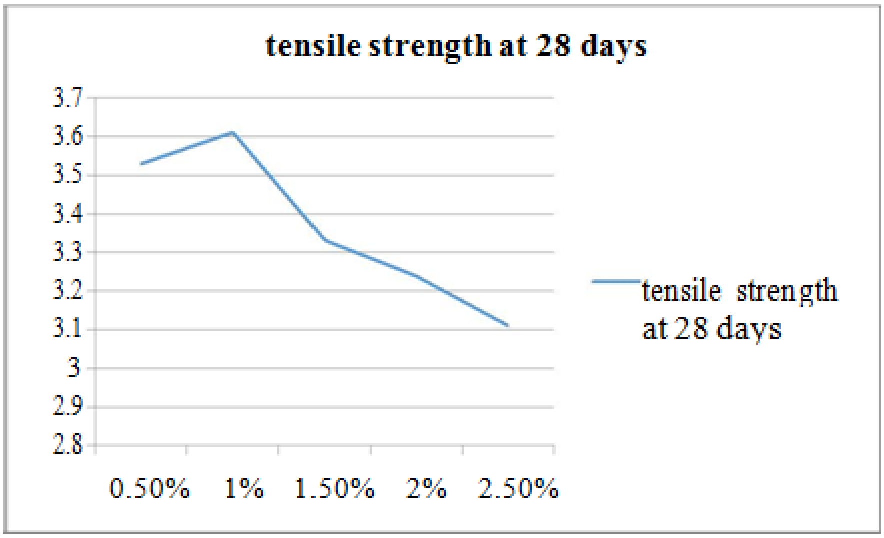

Fig.6. Graph between % of basalt fibre and tensile strength at 28 days

5. Conclusions

1. Compressive strength for the concretes increases from 0 to 1.00% and then decreased gradually.
2. Tensile strength for the concretes increases from 0 to 1.00% and then decreased.
3. Hence 1% of basalt fibre with replacement of cement is optimum percentage.
4. As the slump value decreased from 0.5% to 2.5% of fibre, admixtures can be added to improve workability.
5. There is 31.41% increase in tensile strength at 1% replacement of basalt fibre and 30% increase in tensile strength at 1.50% replacement of basalt fibre at 28days.
6. There is 64.82% increase in compressive strength at 0.5% replacement of basalt fibre and 73.17% increase in compressive strength at 1.00% replacement of basalt fibre at 28days.
7. Therefore, the use of basalt fibre in high strength concrete is good advantage than in medium strength concrete.

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STUDY ON EFFECT OF HIGH TEMPERATURE ON BOND OF HYSD STEEL BARS WITH FLY-ASH

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Abstract

Concrete is a heterogeneous material with a wide variety of usage in structural design. The design of fire resistant structural elements requires realistic knowledge on the behavior of concrete at high temperatures.

Concrete's thermal properties are more complex than for most materials because not only is the concrete a composite material whose constituents have different properties, but its properties also depend on moisture and porosity. Replacement of cement with fly ash, an industrial waste product, offers a sustainable alternative. The goal of this research was to explore the feasibility of using fly ash concrete for structural applications by testing the material's reinforcement bond properties.

Bond behavior between concrete and reinforcing bars was observed under elevated temperatures. Four different concrete compositions were used. Hundred and twenty pull-out specimens (Ø100 mm, 100 mm) were prepared. Maximum temperatures of specimen (33 °C, 100 °C, 150 °C, 200 °C and 250 °C). Specimens were then cooled down in laboratory conditions. Finally the specimens were tested at room temperature. In order to check the compressive strength standard cubes were cast, cured, and then tested to compressive strength. The results showed considerable changes in steel-concrete bond under high temperatures. This experiment presents a study about the bond of concrete with high strength steel. The concrete strength was about 30 MPa and the steel was a 500 MPa grade. Bar diameters used were 12 and 16 mm. In order to investigate the effect of fly ash with 30%, 40% and 50% replacement of cement by weight 100 mm diameter cylinder were used.

It can be stated that the pullout specimen with the smaller bar size has greater bond strength than the specimen with the larger diameter bar.

Keywords*: concrete, reinforcement, bond strength, elevated temperature, fly ash.*

1.0 Introduction

Concrete is a heterogeneous material with a wide variety of usage in structural design. The design of fire resistant structural elements requires realistic knowledge on the behavior of concrete at high temperatures.

Concrete's thermal properties are more complex than for most materials because not only is the concrete a composite material whose constituents have different properties, but its properties also depend on moisture and porosity. Because thermally induced dimensional changes, loss of structural integrity, and release of moisture and gases resulting from the migration of free water could adversely affect plant operations and safety, a complete understanding of the behavior of concrete under long-term elevated-temperature exposure as well as both during and after a thermal excursion resulting from a postulated design-basis accident condition is essential for reliable design evaluations and assessments. Because the properties of concrete change with respect to time and the environment to which it is exposed, an assessment of the effects of concrete aging is also important in performing safety evaluations. In some areas of bond behavior (like fatigue and especially high temperatures) limited information is available. The main reasons are complexity of the experiments and the high cost. During the exposure to high temperatures, concrete undergoes changes in its chemical composition, physical structure and water content. These changes primarily occur in the hardened cement paste. The resulting physical changes and chemical decomposition of major concrete constituents are demonstrated by e. g. cracks, explosive spalling or both. Investigations on the bond strength between concrete and reinforcing steel at room temperature have been carried out over many years however, only few experiments have been carried out to study the effects of high temperature on the bond characteristics. The percentage reduction of bond strength for ribbed bars at elevated temperatures is less than that for plain round steel bars.

1.1 BEHAVIOR OF BOND

The transfer of axial force from a reinforcing bar to the surrounding concrete results in the development of tangential stress components along the contact surface. The stress acting parallel to the bar along the interface is called bond stress. For reinforced concrete to function effectively as a composite material it is necessary for the reinforcing steel to be bonded to the surrounding concrete. Bond ensures that there is little or no slip of the steel relative to the concrete and the means by which stress is transferred across the steel-concrete. Bond resistance is made up of chemical adhesion, friction and mechanical interlock between the bar and surrounding concrete. In the plain bars, only the first two of these components contribute to the bond strength. In the deformed bars, the surface protrusions or ribs interlocking with and bearing against the concrete key formed between the ribs contribute more positively to bond strength, and is the major reason for their superior bond effectiveness.

1.2 FLY-ASH

Fly ash is finely divided residue that results from the combustion of coal and transported by flue gas. India is a resourceful country for fly ash generation with an annual output of over 110 million tones, but utilization is still below 20% in spite of quantum jump in last three to four years. Availability of consistent quality fly ash across the country and awareness of positive effects of using fly ash in concrete are pre- requisite for change of perception of fly ash from 'A waste material' to 'A resource material'. Nowadays due to strict control on quality of coal and adopting electrostatic precipitators, fly ash of consistent quality is separated and stocked, and it is gaining popularity as a good pozzolanic material for partial replacement.

Fly ash is classified into three classes depending on its calcium content, in recognition of the difference in behavior between low and high lime fly ashes. These classes are as follow:

a) Type F, low calcium, 8% CaO
b) Type CI, intermediate calcium, 8–20% CaO
c) Type CH, high calcium, 20% CaO

Low CaO fly ashes generally provide good resistance to alkali-silica reaction (ASR) and sulphate attack. The spherical shape of fly ash particles causes an improvement in the workability, and the particles alter the flocculation of cement, with a resulting lowering of the quantity of water required. The addition of fly ash causes a reduction in the water required for a given slump, typically in the order of 5-15 % when compared with a Portland cement only mix. The improvement in ultimate strength obtained with the use of fly ash is due to its pozzolanic action and the ability of small fly ash particles to fit in between cement particles.

Fly ash causes an increase of strength because of the “packing” of the fly ash particles at the aggregate-cement interface, but the beneficial impact of fly ash on both strength and workability is not extended beyond 30 % of the cementitious material.

TABLE I.1
PHYSICAL PROPERTIES OF FLY-ASH

S. No	**Physical Properties**	**Test Results**
1	Color	Grey (Blackish)
2	Specific gravity	2.13
3	Lime reactivity- Average compressive strength After 28 days of mixture	4.90 MPa

TABLE I.2
CHEMICAL PROPERTIES OF FLY-ASH

S.No	**Constituents**	**% By Weight**
1	Loss of ignition	4.17
2	Silica (SiO_2)	58.55
3	Iron oxide (Fe_2O_3)	3.44
4	Alumina (Al_2O_3)	28.20
5	Calcium oxide (CaO)	2.23
6	Magnesium oxide (MgO)	0.32
7	Total Sulphur (SO_3)	0.07
8	Insoluble residue	-
9	Alkalies a) Sodium oxide (Na_2O) b) Potassium oxide (k_2O)	 0.58 1.26

2. PROCEDURE

2.1 FIGURES AND TABLES

- By observing the existing literatures, this work is carried out with different percentages of fly ash as the replacement of cement; the percentages of fly ash used were 30%, 40% and 50%. In the previous research papers no work is done to study the effect of elevated temperatures on bond strength of concrete with partial replacement of cement with fly ash.
- The percentage replacement of Fly-ash is 70% in one literature. The drawback for the HVFA concrete was that once the concrete began to crush around the reinforcing bar, bond strength reduces at a higher rate.
- The percentage reduction of bond strength for ribbed bars at elevated temperatures is generally less than for plain round steel bars.
- Bond strength is reduced as temperature increases and the reduction rate is greater compared to concrete strengths.
- The difference in the diameter of bar have significant effect on bond strength.

- The type of aggregate in the concrete affects the bond strength at elevated temperatures.

3.OBSERVATION AND DISCUSSION OF TEST RESULTS

To study the effect of high temperature on concrete one hundred and twenty cylinders were tested and compare with specimens at room temperature. The specimens were heated at varying temperature from 100°C to 250°C.The target temperature was maintain for one hour and the specimens then cool down to room temperature before testing. Results obtained from experimental investigation to study the bond loss between the concrete and HYSD steel at elevated temperature with and without fly ash are discussed in detail as given below

3.1 TEST RESULTS

3.1.1 PULL OUT TEST

The steel rebar was pulled out using a universal testing machine with a maximum capacity of 100kn.the BS was calculated from external load (p) on the bar and total surface area of the embedded portion of the reinforcing bar, assuming a constant stress along the bonded length of the bar (i.e., BS =P/ (∏DL), where d and l are the diameter of the bar and embedded length, respectively). the pair of specimens tested for bond behavior showed similar results. the effect of replacement of fly ash in different proportions with cement on bond strength for m30 grade was investigated by preparing a cylindrical specimen of (100mm dia and 100mm height). the replacement level of fly ash was 30%, 40%, and 50%. specimens were cured in water and tested at age of 28 days. after removing the specimens from curing pond they are heated at elevated temperatures.pull out test results of 12mm dia bar are given below in table 3.1 and are represented graphically in fig 3.1.pull out test results using 16mm dia bar., are given below in table 3.2 and are represented graphically in fig 3.2.

Table 3.1 Bond Strength with % Replacement Of Fly Ash At Different Temperatures

PERCENTAGE REPLACEMENT OF FLY ASH	BOND STRENGTH IN MPA				
	TEMPERATURES				
	33° C	100°C	150°C	200°C	250°C
0%	9.01	8.48	8.04	7.29	6.23
30%	7.4	6.5	6.2	5.6	5.4
40%	7.6	7.1	6.7	6.3	5.8
50%	7.3	6.4	5.9	5.33	5.03

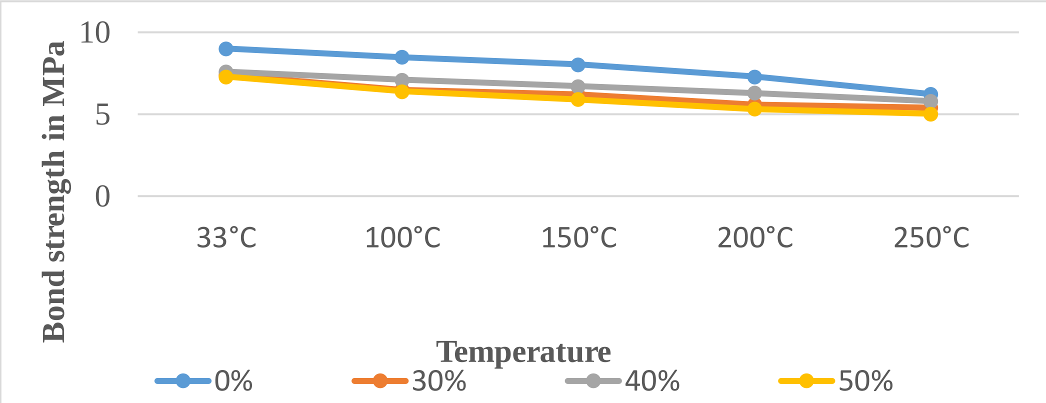


FIG. 3.1 BOND STRENGTH VS TEMPERATURE USING 12MM DIA BAR

TABLE 3.2 BOND STRENGTH WITH % REPLACEMENT OF FLY ASH AT DIFFERENT TEMPERATURES

PERCENTAGE REPLACEMENT OF FLY ASH	**BOND STRENGTH IN MPA**				
	TEMPERATURES				
	33°C	**100°C**	**150°C**	**200°C**	**250°C**
0%	8.02	7.5	7	6.43	5.96
30%	6.7	6.2	5.8	5.6	5.2
40%	7	6.5	6.2	5.8	5.5
50%	6	5.8	5.6	5.2	5.13

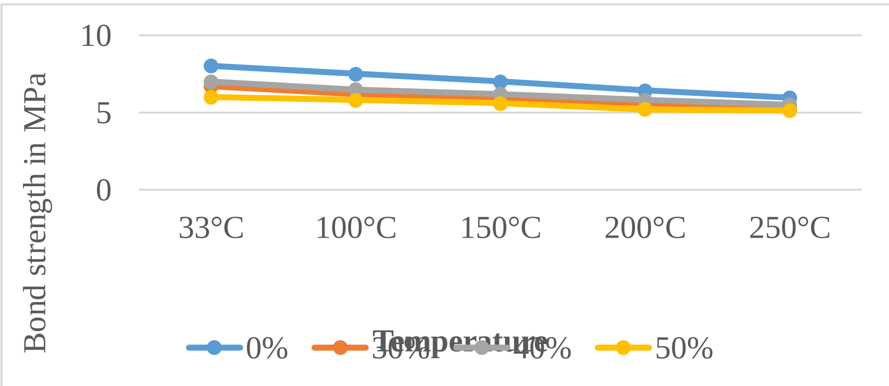
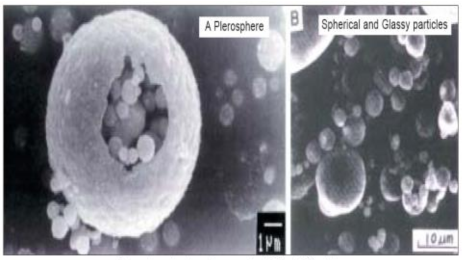


FIG. 3.2 BOND STRENGTH VS TEMPERATURE USING 16MM DIA BAR

INFERENCES ON TEST RESULTS:

Based on the test results of one hundred and twenty pullout specimens, it can be stated that the pullout specimen with the smaller bar size has greater bond strength than the specimen with the larger diameter bar. The maximum bond strength value occurred in specimen with

12mm diameter bar at room temperature whereas the minimum bond stress value was in specimen with 16mm diameter bar with 50% replacement of fly ash subjected to temperature of 250°c. Most considerable reduction of bond strength took place between 150°c and 250°c. At temperature up to 100 °c the concrete specimens resulted in smaller reduction in bond strength. Smaller the diameter of the bar more is the bond strength, larger the diameter of the bar less is the bond strength.40 percent replacement level of the fly ash shows better bond strength when compare to 30 and 50 percent replacement level.



Scanning electron micrograph of Class F, Fly Ash.

4. RESULTS AND DISCUSSIONS

From the experimental study on the loss of bond strength with and without fly ash, with the fly ash replacement as 30, 40 and 50 percent. Bond behavior between concrete and reinforcing bars was observed under elevated temperatures. Four different concrete compositions were used. A Hundred and twenty pull-out specimens (Ø100 mm, 100 mm) were prepared. The temperature range used in study is (33 °C, 100 °C, 150 °C, 200 °C and 250 °C). Specimens were then cooled down in laboratory conditions. Finally the specimens were tested at room temperature. The results showed considerable changes in steel-concrete bond under high temperatures.

By conducting the pull out tests, the bond strength between concrete and reinforcements can be obtained from the pull out load. In general, the bond stress corresponding to the pull out can be regarded as the bond strength. Smaller the diameter of the bar more is the bond strength, larger the diameter of the bar less is the bond strength. From the results it can be stated that 12mm diameter bar shows better bond when compare with 16mm diameter of the bar. as the exposure temperature increases the bond strength decreases.40 percent replacement level of the fly ash shows better bond strength when compare to 30 and 50 percent replacement level. Optimum replacement level was found to be 40 percent.

Conclusions

- Based on the test results it can be stated the pullout specimens without fly ash shows better bond strength compare to specimens with fly ash.
- Based on the test results of one hundred and twenty pullout specimens, it can be stated that the pullout specimen with the smaller bar size has greater bond strength than the specimen with the larger diameter bar.
- The influence of high temperatures on bond strength. Pull-out specimens tested at cold state after heated up to (100 °C, 150 °C, 200 °C and 250 °C). The types of mixes were Convention, 30, 40 and 50 percent replacement of fly ash. Type of steel reinforcement was deformed rebar. Most considerable reduction of bond strength took place between 150°C and 250°C. As the temperature was increases to 250°C the bond strength of concrete decreased progressively to 30 percent of the strength at room temperature.

- At temperature up to 100 °C the concrete specimens resulted in smaller reduction in bond strength nearly 7 to 10 percent of the strength at room temperature.
- Optimum percentage of mix containing Fly ash as partial replacement of cement was found to be 40%.

Based on the test results it can be stated that, as the exposure temperature increases the bond strength decreases.

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FRP RETROFITTING OF RC BUILDING ON JOINTS USING STAAD PRO

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Abstract

Fibre reinforced polymer (FRP) composites consist of high strength fibres. Fibres typically used in FRP are glass, carbon and aramid. These fibres are all linear elastic up to failure, with no significant yielding compared to steel. The repair and upgrading of reinforced concrete structures damaged by seismic actions are challenging fields of study in earthquake engineering. Seismic retrofitting of contructions are vulnerable to earthquake.Most of the indian building stock is vulnerable to seismic action even if located in areas that have long been consideredof high sesmic hazard. This is four storey reinforced concrete structure has been considered, which lies in Zone II according to IS 1893:2000 classification of seismic zones in India.The structure is designed in STAAD.Pro v8i, considering M20 concrete and Fe250 steel reinforcement for with and without earthquake loading conditions.Aim is to focus on beam-column joints safe for occupation.The limitations of this project in existing building which may require extensive repair before it is generally useful or considered are that not much is known about the behavior of FRP materials and thus, no standardization has been achieved in it commercially. Also the code does not give a specific method of jacketing columns

***Keywords**: Equivalent Static Method, Demand Capacity Ratio, Flexural Capacity, Shear Capacity, Reinforced Concrete Structure, FRP Strengthening.*

1.0 General

Fibre reinforced polymer (FRP) composites consist of high strength fibres. Fibres typically used in FRP are glass, carbon and aramid. These fibres are all linear elastic up to failure, with no significant yielding compared to steel. The repair and upgrading of reinforced concrete structures damaged by seismic actions are challenging fields of study in earthquake engineering, which have been developed during the last two decades guidance. On the other hand, the failure of joints may, in many cases, lead to general failure of the whole construction. Research in this area is essential since engineers in seismic-prone regions often face the problem of designing repair or strengthening works for damaged buildings without quantitative.The primary functions of the matrix in a composite are to transfer stress between the fibres, to provide a barrier against the environment and to protect the surface of the fibres from mechanical abrasion. Adhesives are used to attach the composites to other surfaces such as concrete. The most common adhesives are acrylics, epoxies and urethanes. Epoxies provide high bond strength with high temperature resistance, whereas acrylics provide moderate temperature resistance with good strength and rapid curing. Several considerations are involved in applying adhesives effectively. Careful surface preparation such as removing the cement paste, grinding the surface by using a disc sander, removing the dust generated by surface grinding using an air blower and carful curing are critical to bond.

- Ductile components are to be designed with sufficient deformation capacity at least to satisfy displacement-based demand-capacity ratio to have good energy dissipation.
- Brittle components should be designed to achieve sufficient strength levels so as to satisfy strength-based demand-capacity ratio

- Jacketing is a member-level retrofit technique. It can be used to increase concrete confinement, shear or flexural strength of the members. Popular practices involve jacketing with concrete, steel or fiber reinforced polymers (FRP).

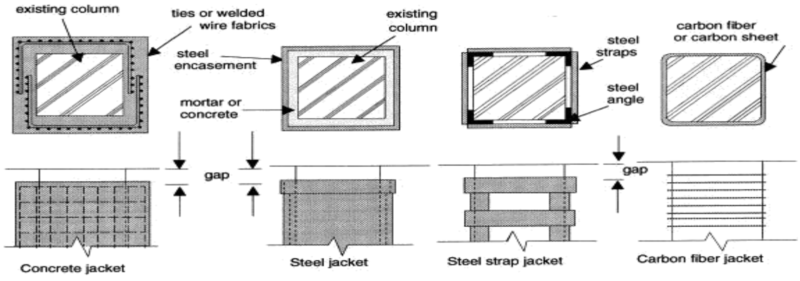


Fig. 1.1 Types of jacketing in seismic retrofitting

2 .THEORY

2.1 Demand Capacity Ratio

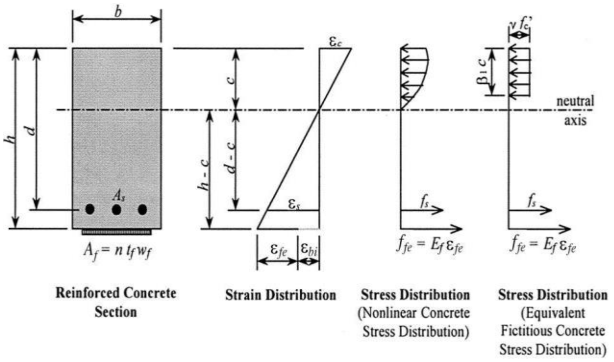
The calculation of Demand Capacity Ratio to identify the failing members, is the part of Equivalent Static Analysis.

DCR= Demand/Capacity

If DCR is lesser than 1, the member passes, else it passes. It is an important tool used to determine whether a certain member of the structure is passing or failing due to moment and/or shear. The check for DCR exceeding 1 was performed for both flexural and shear capacities of the beams as well as columns of the structure.

2.2 FRP Strengthening of Concrete Members

The design philosophy for such sections is coherence with limit state principles. This approach sets acceptable levels of safety against the occurrence of both serviceability limit states (excessive deflections, cracking) and ultimate-limit states (failure, stress rupture, fatigue). By applying limit state analysis, the internal strain and stress distribution for a rectangular section of concrete can be found out at the ultimate stage. Thereafter, the strain level in the FRP reinforcements can be determined.. The maximum strain for the FRP will be developed at the point at which concrete crushes, FRP ruptures or FRP debonds from the substrate. ACI-440.2R-02 (Clause 11.3.2) mentions that confining rectangular sections with FRP is effective in improving the ductility of compression members but not in increasing their axial strength. Hence, due to lack of any suggested method, the design of FRP jacketing was performed only for the failing beams.



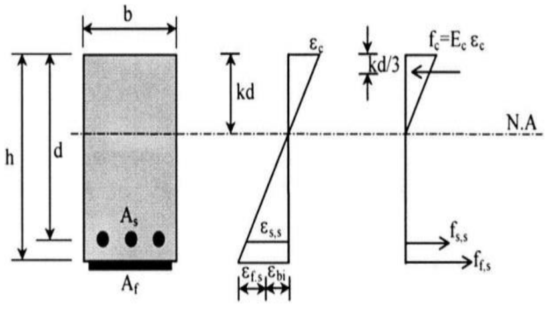


Fig 2.1 Internal strain and stress distribution for a rectangular concrete section (FRP strengthened) under flexure at ultimate stage

Fig 2.2 Elastic strain and stress distribution

3. CALCULATION OF DCR

The structural engineering group that tried to use the design spectra for the analysis and retrofit of the bridge found that a large number of members in the structure required retrofit. After a careful study of the maximum peak values of the member forces (especially the large peak axial forces), it was decided to run new time-history analyses using the basic time-history records that were used to create the design spectra. After running all the time-history records, the maximum Demand/Capacity Ratios were reduced by approximately a factor of three compared to the design spectra results.

The detailed evaluation of the building involves equivalent static lateral force procedure, load with response reduction factors and Demand Capacity Ratio (DCR) for ductility as in IS 13920:1993. Since the building dates back to a period 50 years early, the grade of concrete is assumed to be M20 and for steel Fe250.

Checks done by these types :

1. DCR for moments of resistance in sagging and hogging for beams
2. DCR for shear capacity in beams
3. DCR for moment of resistance in columns
4. DCR for shear capacity in columns.

For finding DCR for moments of resistance in sagging and hogging:

- In hogging, the capacity moment of resistance is calcuated by the formula-
  Mr (H) = 0.36fckbXu (d- 0.44Xu) + fscAsc (d-d')
- In sagging, the capacity moment of resistance is calcuated by the formula-
  Mr (S) = 0.36fckbXu (d- 0.44Xu) + fscAst (d-d')

d' = effective cover= 33 mm

For finding DCR for moments of resistance in columns:

- The Interaction diagram in SP-16 has been used to find the value of $Mu/fckbD^2$ for the corresponding values of P/fck and Pu/fckbD

For finding DCR for shear capacity of beams:

- Calculate the percentage of steel 100As/bd.
- For the corresponding percentage, find the value of τc(design shear strength of concrete) from table 19 of IS 456: 2000. The following are calculated-

Vus = 0.87
fyAsv d / Sv
Vu1 = Vus+
τcbd

Vu2= 1.4 [MR(H) + MR(S)]/
Lc whereLc= clear span of
the member

Shear resisted (capacity) is given by the maximum of Vu1 and Vu2

For finding DCR for shear capacity of columns:

- From table 19 of IS 456: 2000. The calculation are follows

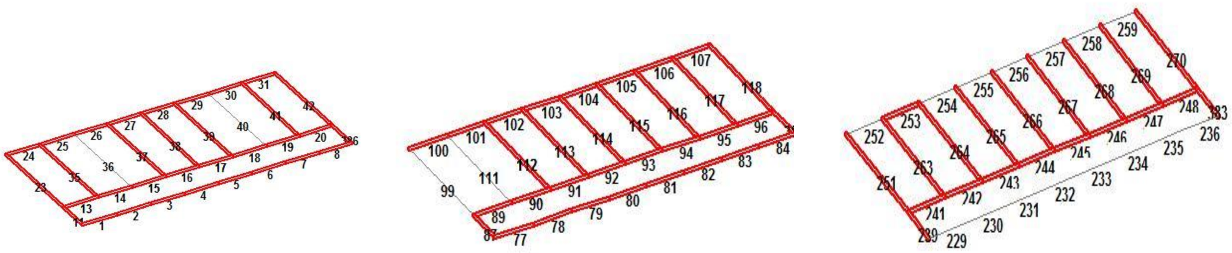


Fig. 3.1 Beams Failing due to Flexural Capacity

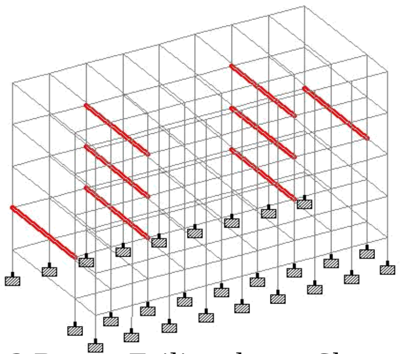
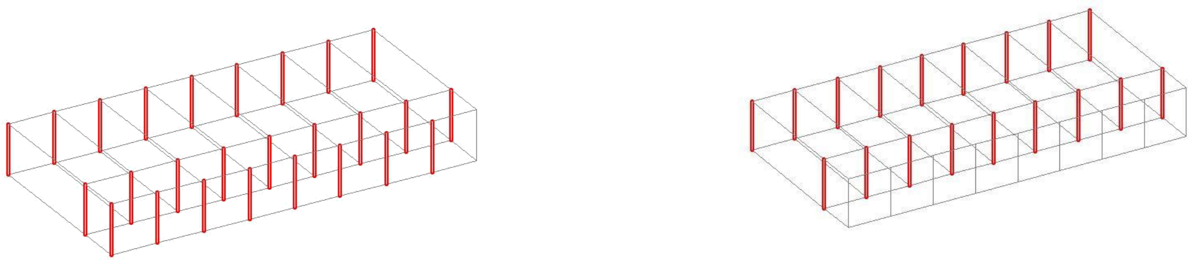


Fig. 3.2 Beams Failing due to Shear Capacity



1st Level — 2nd, 3rd and 4th Levels

Fig. 3.3 Columns Failing due to Flexural Capacity

3.1 DESIGN OF FRP JACKETING:

3.1.1 PROVIDING FLEXURAL STRENGTHENING TO BEAMS USING FRP:

Stress level in the reinforcing steel and FRP-

$f_s = E_s \varepsilon_s <= f_y$

ffe= Efεfe

Design flexural strength of the section-

$$\Phi Mn = \Phi[Asfs\ (d- b1c/2) + yAfffe(h-b1c/2))]$$

This should be greater than the required moment strength Mu

Calculation of service stresses in the reinforcing steel and the FRP by-

$$k= [(psEs/Ec + pfEf/Ec)^2 + 2\ (psEs/Ec + pfEfh/Ecd)-(psEs/Ec + pfEf/Ec)]^{0.5}$$

$$f_{s,s} = \frac{[M_s + \varepsilon_{bi} A_f E_f (h - kd/3)](d - kd)\,E_s}{A_s E_s\,(d-kd/3)\,(d-kd) + A_f E_f\,(h - kd/3)(h-kd)}$$

$f_{s,s} <= 0.80\ f_y$ (Serviceability conditions)

$f_{f,s} = f_{s,s}\ [E_f(h-kd)/E_s(d-kd)] - \varepsilon_{bi} E_f$

$f_{f,s} <= 0.55\ f_u$ (Creep-rupture stress limit)

For the calculations values assumed are as follows-

Environmental Reduction Factor Ce for carbon fibers = 0.89

Rupture Strain of FRP system efu* = 0.020

Ultimate Tensile strength of FRP system ffu * = 0.70kN/mm2

Modulus of Elasticity of FRP system Ef = 35kN/mm2

Dimensions of FRP strips

Thickness tf = 0.04" = 1.016 mm

Width wf = 12 in = 304.8 mm

4 . RESULTS

4.1 FRP Design Calculations

Table3.1.1 **1st STOREY**

Beam No.	**Demand Moment (kNm)**	**ΦMn (kNm)**	**fs,s (N/mm^2)**	**ff,s (N/mm2)**	**No. of plies**
20	100.034	98.259	68.401	15.106	4
25	99.5.261	106.137	52.397	10.973	5
30	115.292	102.12	55.977	11.696	5

Table3.1.2 **2nd STOREY**

Beam No.	**Demand Moment (kNm)**	**ΦMn (kNm)**	**fs,s (N/mm^2)**	**ff,s (N/mm2)**	**No. of plies**
65	50.485	75.61	40.716	9.749	3

90	40.57	44.369	51.64	10.299	4
92	100.49	96.42	70.259	12.691	5

Table3.1.3 **3rd STOREY**

Beam No.	**Demand Moment (kNm)**	**ΦMn (kNm)**	**fs,s (N/mm^2)**	**ff,s (N/mm2)**	**No. of plies**
183	70.836	99.196	50.963	10.678	3
184	482.371	530.496	90.604	15.872	3
190	280.009	348.326	92.495	19.212	4

Table3.1.4 **TERRACE**

Beam No.	**Demand Moment (kNm)**	**ΦMn (kNm)**	**fs,s (N/mm^2)**	**ff,s (N/mm2)**	**No. of plies**
250	192.458	245.06	100.08	20.627	2
264	180.208	210.564	110.47	22.239	2
282	170.496	118.565	108.391	22.517	2

4. CONCLUSIONS

The analysis of beams by Equivalent Static Method revealed that most of the beams failed in flexural capacity. The number of failing beams decreased with increasing storeys. Based on the above observations, the immediate need to counter deficiency in flexural capacity was identified and the FRP jacketing scheme was suggested only for beams, failing in flexure. Due to the high tensile strength and stiffness, stability under high temperatures and resistance to acidic/alkali/organic environments, carbon fiber was chosen as the FRP material to be used. The FRP design method used in this project is essentially trial and error where the value of the depth of neutral axis has to be assumed and compared with the value obtained. This would ensure feasibility of application of the FRP system to the beams. From the results of the experimental program, effective methods for rehabilitating existing deficient beamcolumn joints are developed. A comparison between the performance of original specimens and rehabilitated ones shows that the GFRP jacket was capable of increasing the shear resistance of the joint and enhancing the performance of the connection from a ductility point of view. The proposed rehabilitation techniques for beam-column joints.

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EXPERIMENTAL STUDY ON BACTERIAL-BASED SELF-HEALING CONCRETE

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Abstract

In recent times rapid growth in infrastructure development can be seen in concrete structures, due to its durability. The ingress of deleterious substances into the concrete due to the formation of micro-cracks may cause deterioration or structural failure. Further, high maintenance and repair costs were required. Self-healing concepts were adopted for the enhancement of durability of concrete and extending the service life of the concrete structures. In which, Bio-mineralization is an eco-friendly bio-process, shows promising results in sealing the micro-cracks by Microbially Induced $CaCO_3$ Precipitation (MICP). Introduction of specific bacteria into concrete embedding with self-healing agent helps to mediate MICP. Adopting greener alternatives and utilization of bio-minerals increases the ability to seal the cracks in concrete. The mechanical and durability aspects such as compressive strength, split tensile strength were done. The visual examination is done to observe the sealing of cracks in concrete. SEM and EDS analysis were done to investigate the calcite precipitation in concrete specimens and chemical depositions were seen.
***Keywords**: Microbially Induced $CaCO_3$ Precipitation (MICP), Bacteria, SEM, EDS, self-healing.*

1 Introduction

Concrete is the major construction material which is mostly used in construction and infrastructure development due to its high compressive strength. Concrete is weak in tension, tensile stresses leads to deterioration of concrete or a structural failure. This reduces the service life of the structure. Formation of tensile cracks leads to ingression of chemicals or water into the concrete. The renovation of buildings costs much; the expensive renovations could be prevented by approaching self-healing techniques. To overcome these situations adopting a novel based self-healing technique that can be used to prevent cracks in concrete. A realistic self-healing technique which was rapidly developing by mineral deposition can be achieved by incorporating bacteria as a healing agent into the concrete. Bacterial-based self healing technique is an eco friendly bio-mineralization process which can able to heal the cracks formed in concrete without human interaction. When cracks appear, the water finds its way into the concrete and activates the bacteria, therefore precipitation takes place, and this closes the cracks in the concrete. Bacillus pasteurii, which is a common soil bacterium, rich in urease production is adopted for the bio-mineralization process. The bacteria present in the concrete remains dormant and activates when water ingresses into these formed cracks [1, 3]. The presence of micro-organisms in concrete helps in remediation of cracks and improvement of compressive strength, in high pH environment conditions [13]. The main concept of autogenous self-healing approach is adopted for non-hydrated cement particles in traditional concrete. The Ca Co_3 precipitation helps to seal and heal the micro-cracks in concrete by immobilizing the urea producing bacteria in concrete by healing agent [2, 4]. The impact of vegetative cells on hydration of cement in concrete matrix, compressive strength and the influence of urea producing bacterial cells shows precipitation of Ca Co_3 which helps in self healing of cracks in the concrete [17]. This research is mainly focused on the increment of life time of the structures by self-healing technique. H.M. Jonkers et.al investigated the immobilization of bacteria and calcium lactate in expanded clay particles [3]. The survival of bacteria in the expanded clay aggregates was found to be sustainable. The EDS analysis had shown the presence of calcite deposition in the form of $CaCO_3$ precipitation in the concrete specimens. Bacterial cell walls acts as a promising admixture in enhancing mechanical performance of concrete. Increased carbonation of $Ca(OH)_2$ and formation of calcium carbonate on the surface of the concrete and inside the cracks can be seen [2,12]. The visual inspection of healing the cracks proved that the bacteria suspended in water had survived and helped in the self-healing process. Split tensile strength of the concrete is considered in the design criteria, so, split tensile strength of concrete is more concentrated on this research. Healing of cracks and percentage of load taken after healing is done in order to remediate the tensile cracks formed. By comparing the conventional bacterial concrete techniques with the bio-mineralization techniques of concrete can achieve an eco-friendly and effective biological sustainable concrete [2] .

2 Project Significance

Cement: OPC 53 grade of cement with specific gravity 3.14 conforming IS: 12269-1987.
Coarse aggregates: locally available graded aggregates of size 20 mm were used confirming IS: 383-1970.
Fine aggregates: Natural river sand of zone **III** confirming of IS: 383-1970 was used.
Calcium lactate: $C_6H_{10}O_6$ $Ca.5H_2O$ white powder thing which is bought from local chemical merchant is added 6% by weight of cement into concrete mix.
Bacteria: Bacillus pasteurii which is formerly known as sporosarcina pasteurii is a common bacterium found in soil, produce urease. The bacterial pure culture strain was bought from National collection of micro-organisms (NCIM) Pune.

3. Experimental study:

The concept of self-healing in both normal and light weight bacterial concrete is to form $CaCO_3$ precipitation on the surface of the cracks. The ability to seal the cracks can be achieved by Microbial Induced Calcium carbonate Precipitation (MICP). Calcium carbonate can be formed on the surface of the cracks by reacting with CO_2 present in the calcium hydroxide by following reactions

$$CO_2 + Ca\,(OH)_2 \rightarrow Ca\,CO_3 + H_2O \qquad (1)$$
$$Ca\,(C_3H_5O_2)_2 + 7\,O_2 \rightarrow CaCO_3 + 5\,CO_2 + 5\,H_2O \qquad (2)$$

This process is more efficient because active metabolic conversions of calcium nutrients and bacteria present in the concrete [6]. The bacteria incorporated in the specimens for self-healing gives rise to pH increase, resulting in formation of carbonate ions.

$$Ca^{2+} + cell \rightarrow cell\text{-}\,Ca^{2+} \qquad (3)$$
$$Cell\text{-}Ca^{2+}\,CO_3^{2-} \rightarrow cell\text{-}\,Ca\,CO_3 \downarrow \qquad (4)$$

In the above equations, the cell wall of bacteria is negatively charged; cations from the environment were drawn by bacteria. Including the Ca^{2+} ions to deposit on the cell surface, Subsequent reaction with $CO_3{}^{2-}$ -ions can be seen by Ca^{2+} -ions leads to $CaCo_3$ precipitation at the cell surface.

3.1. Preparation of Bacterial solution:

The bacteria should be added may be of liquid (broth) or solid (agar). For the purpose of adding in the concrete we choose liquid form i.e., Nutrient broth. The bacteria to be added should be grown in specific media. The broth solution can be prepared by suspending 13.0 grams of nutrient broth in 1000 ml of distilled water which consists of ingredients such as sodium chloride, beef extract, yeast extract and peptic digest of animal tissue. These ingredients act as food for bacteria in the concrete. The media was autoclaved at 121^o C for 20 minutes. Then the bacterial pure culture was inoculated in broth media. The inoculated broth media was kept in incubation chamber at 37^o C for 24-48 hrs.

3.2. Adding bacteria into Concrete:

The addition of bacteria into the concrete for Microbilly induced calcium carbonate precipitation (MICP) can be done by direct addition process. The bacteria is added to the concrete directly at the time of mixing the concrete. The healing agent was mixed with cement and added to the concrete to achieve better pore structure in concrete. This direct addition process can be adopted for preparing the normal bacterial-based self healing concrete.

4. Results and Discussions:

4.1. Compressive strength test:

Concrete specimens of 100x100x100 mm specimens were casted and allowed to harden for different ages like 7,14,28,56 and 90 days. The specimens were loaded in compressive testing machine for maximum load. Average compressive strength for conventional and bacterial concrete specimens were shown in fig.1 gradual increase in compressive strength can be seen in bacteria induced concrete. Bacterial concrete specimens have attained a strong bonding between the cracks formed. This helps in developing a bridging between the internal cracks formed. Similar compressive strength, increased internal bonding strength and better workability can be observed in bacterial concrete produced when compared to conventional bacterial concrete.

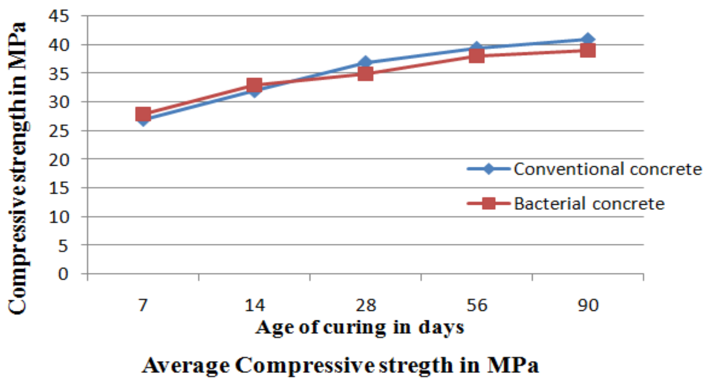

Fig.1: *Average Compressive strength for conventional and bacterial concrete specimens*

4.2. Ultrasonic pulse velocity test:

Ultrasonic pulse velocity test was conducted on the concrete specimens of 100x100x100 mm size at different ages of curing to determine the quality of the concrete. For conventional

concrete specimens it can be seen that the quality of concrete is achieved good at 14 days of curing. As for bacterial concrete the quality of the concrete is good at 28 days of curing.
The average ultrasonic pulse velocity values for conventional and bacterial concrete specimens at different ages can be seen in fig.2.

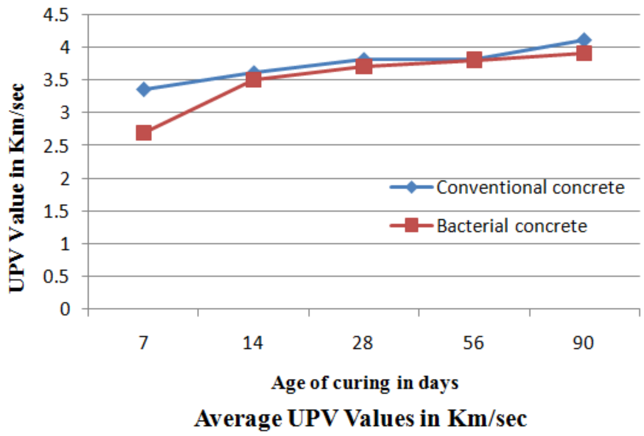

Fig.2: *Average UPV values for Normal and Light weight concrete specimens at different ages*

4.3. Split tensile strength:

The split tensile strength test was conducted on concrete cylinder specimens of size 100 mm diameter and 200 mm depth at different ages of curing. The specimens were kept horizontally in order to create tensile cracks. The load was applied maximum and at first failure the specimen was taken out and allowed for curing in water for self-healing observation. The average split tensile strength for conventional and bacterial concrete specimens were shown in fig.3.

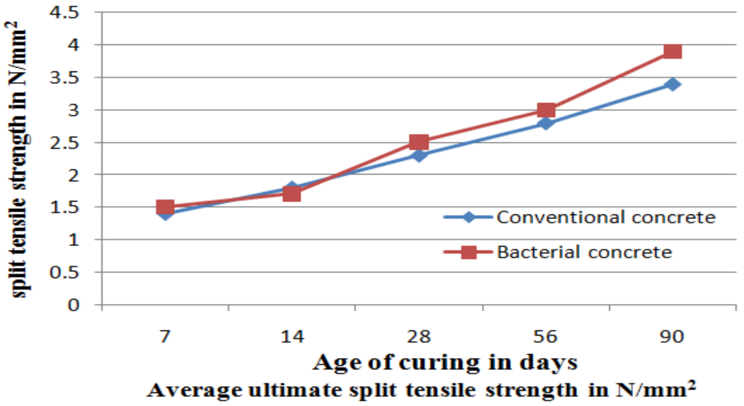

Fig.3: Average Ultimate split tensile strength for Normal and Light weight concrete specimens

After 56 days of curing of the cracked specimens, the healed specimens were again tested for split tensile strength as shown in table.1- an average of 53% of the load was taken by bacterial concrete specimens' respectively after 56 days healing of cracked specimens as shown in table 1. The light weight bacterial concrete shows better results in tensile strength by encapsulation of bacteria inside the light weight aggregates. The internal healing of the light weight concrete specimens can be seen by SEM investigation of light weight aggregates present in the light weight concrete. Comparing both the types of bacterial specimens light weight concrete shows better healing ability to seal the internal and external cracks formed in the concrete.

Table 1: split tensile strength of cracked specimens after 56 days of healing

S.NO	Split tensile strength when cracked (N/mm^2)	Split tensile strength after 56 days of healing (N/mm^2)
1	2.4	1.27
2	2.7	1.52
3	2.8	1.40
4	3.2	1.46

4.4. Crack width analysis:

The approximate crack widths for bacterial concrete specimens have done by using digimizer software. By visual inspection of the specimens the healing can be observed. By using digimizer software the approximate crack widths and mean values of cracks can be observed. The cracks before and after healing of bacterial concrete specimens were shown in fig.4 and fig.5.

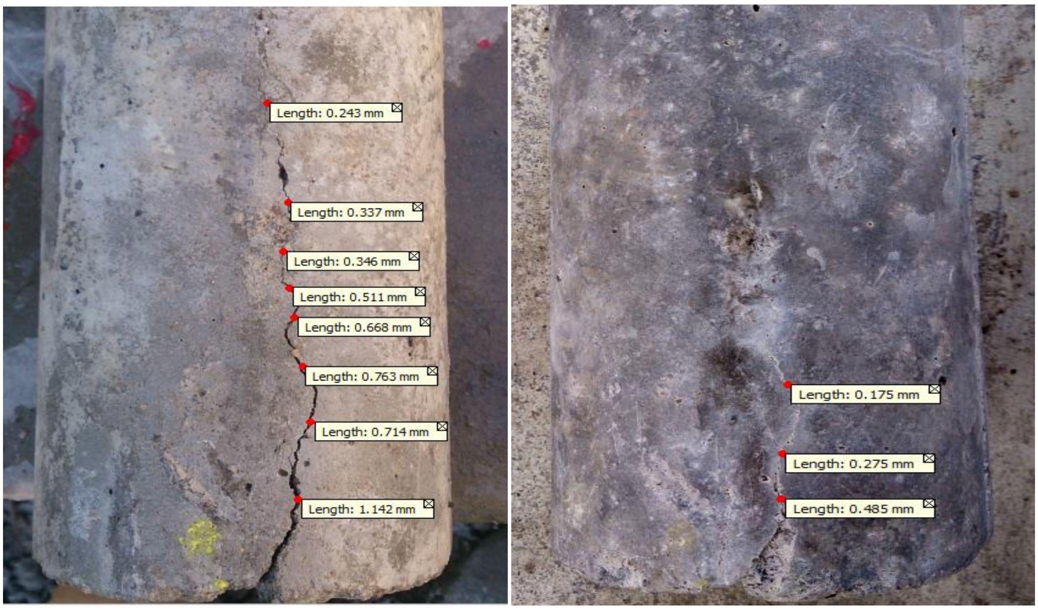

Fig.4: *Crack widths of bacterial concrete before and after 56 days of healing*

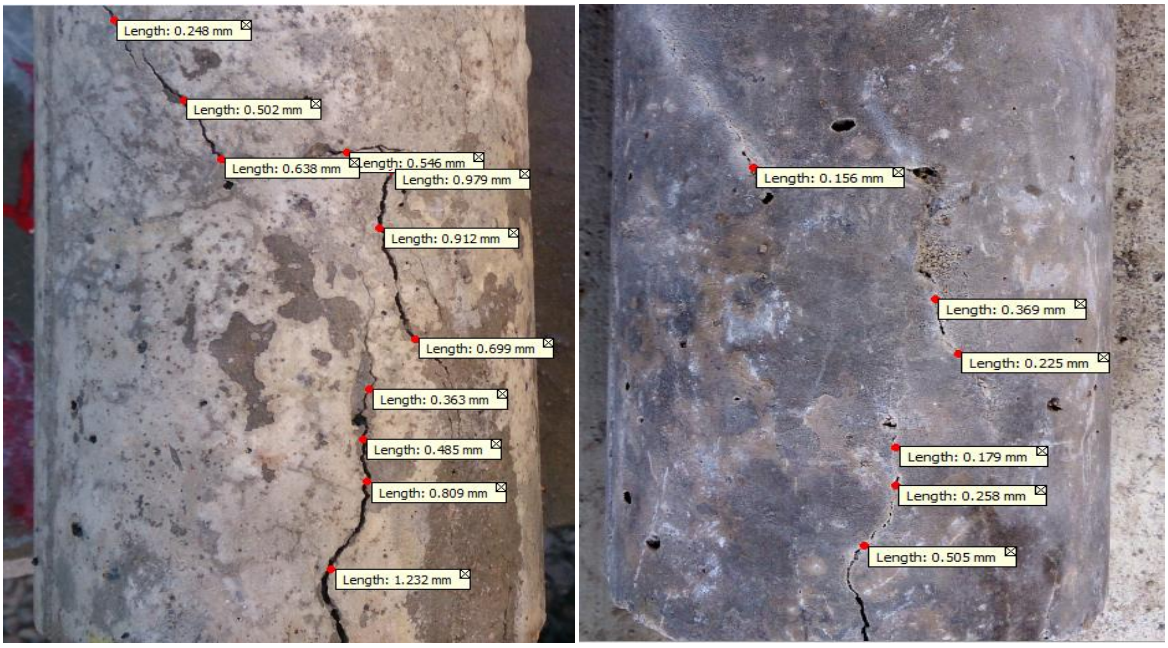

Fig.5: Crack widths of bacterial concrete before and after 56 days of healing

Cracks up to 0.5-0.6 mm approximately were healed for bacterial concrete at the age of 56 days of curing. The visual inspection of healing ability of the bacterial specimens shows the healing capacity of the cracks. The bacterial concrete specimens show adequate formation of $CaCO_3$ on the cracks. The healing of cracks was achieved in bacterial concretes shows better healing in sealing the cracks.

4.5. Scanning electronic microscope (SEM):

The Scanning electronic microscopic (SEM) images were taken for bacterial specimens to check the $CaCO_3$ precipitation on the surface of the cracks formed. SEM images revealed that the $CaCO_3$ precipitation is rich in bacterial concrete as shown in fig.6.

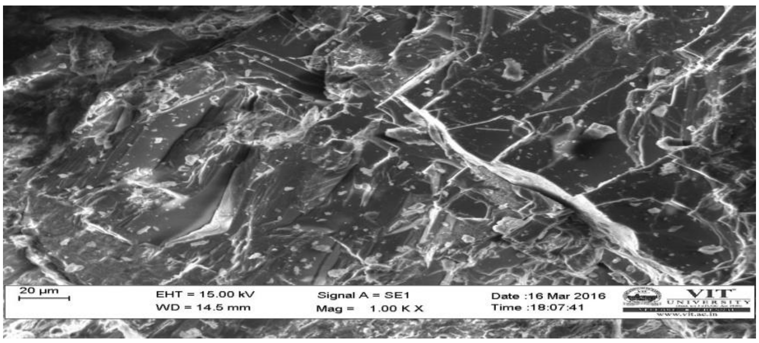

Fig.6: *Formation of $CaCO_3$ on bacterial concrete specimen viewed in SEM*

4.6. EDS analysis:

Energy-dispersive X-ray sectroscopic (EDS) analysis have done for the bacterial concrete specimens to determine the chemical characteristics of the minerals formed in the cracked specimens. EDS anlysis confimed the deposition of calcium compunds in the concrete specimens. The chemical compositions present in the cracked specimens have shown in fig.7 for bacterial concrete specimens.

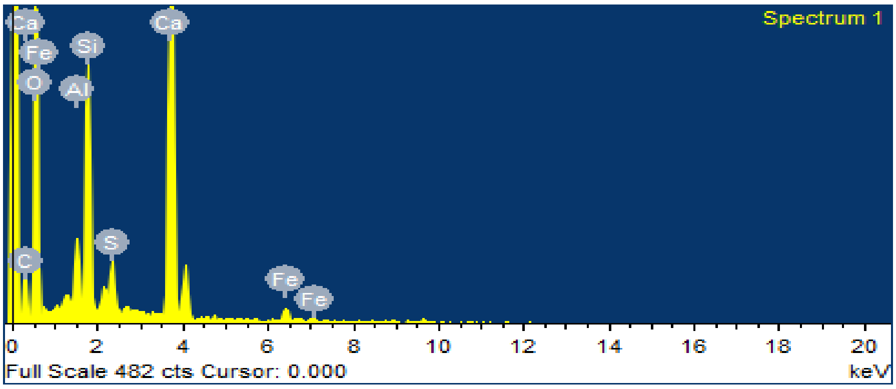

Fig.7: *EDS spectrum of minerals deposited in bacterial concrete specimens*

The EDS analysis revealed that the calcium deposit present in the bacterial concrete specimens is due to formation of $CaCO_3$ on the surface of the cracked specimens. This analysis was carried out to check the capacity of aggregates in forming the $CaCO_3$ on the aggregates which helps in internal healing and bonding of the concrete.

5. Conclusions:

From this study it can be seen that the ability to heal the micro-cracks with the help of bacteria and healing agent was observed in SEM analysis and confirmed by EDS, that Ca Co_3 precipitation helps in sealing the micro-cracks and the following conclusions were drawn:

- A tensile strength of around 50% after the curing of specimens subjected to healing process can be seen.
- Increased tensile strength percentage is observed in the bacterial concrete proven to be more effective than conventional bacterial concrete.
- The visual examination and UPV transmission tests have proved the efficiency of biological treatment of concrete by bonding between the cracks. Complete sealing and better healing of cracks was observed.
- Cracks of width 0.24 mm, 0.34 mm, 0.5 mm were completely sealed in bacterial concrete specimens.
- The enriched calcium source helps in remediating the cracks by spore forming bacteria embedded in the bacterial concrete specimens confirmed by EDS analysis.SEM investigation revealed the healing of cracks by $CaCO_3$ precipitation incorporating with bacteria and healing agent.
- Compared to conventional techniques, microbial self-healing agents have shown better ability to seal the cracks and enhancement of durability in concrete structures. Adopting smart materials helps in the development of sustainable concrete for long lasting structures.

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OPTIMIZATION OF ELECTROLYSIS CELL USING CFD

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Abstract

The effect of bubbles on the current density distribution over the electrodes of analkalineelectrolyzer cell is studied using a two-dimensional computational fluid dynamics model. Model includes Eulerian-Eulerian two-phase flow methodology to model the multiphase flow of Hydrogen and Oxygen with water and the behavior of each phase is accounted for using first principle. Hydrogen/Oxygen evolution, flow field and current density distribution are incorporated in the model to account for the complicated physics involved in the process. Model is validated with mesh refinement study and by comparison with experimental measurements. Model is found to replicate the effect of cell voltage and inter-electrode gap (distance between the electrodes) on current density accurately. Further, model is found to capture the existence of optimum cell height. The validated model is expected to be a very useful tool in the design and optimization of alkaline electrolyzer cells.

Keywords: *Alkaline electrolysis, Hydrogen production, Two phase flow Electrode distance.*

1 Introduction

Alkaline electrolysis is an attractive path toward renewable Hydrogen production. However, cost is a big impediment to using hydrogen more widely as a fuel. Low cost alkaline electrolysis requires innovation in cell and system design. Computational fluid dynamics (CFD) model presented in this paper is developed to aid in the design and optimization of such electrolyzer cells. The presence of gas phase in the liquid electrolyte of the alkaline electrolyzer makes it a two phase flow problem. Bubbles nucleate and grow on electrode surfaces due to electrochemical reaction. During their growth, bubbles make part of electrode inactive for reaction. Once bubblereach bulk electrolyte flow, they decrease the effective electric conductivity of the electrolyte by reducing the cross-sectional area of pure electrolyte available for current transport. Hence, bubble/gasrelease and motion significantly affect the performance of the alkalineelectrolyzer. Dahlkild[1] applied the mixture two-phase flow model for electrolysis process and implemented a boundary layer analysis. The numerical results showed that ionic species concentrations are essentially homogeneous as the mixing effect of the bubble suspension usually is much larger than dispersion by molecular diffusion even at laminar flow conditions. He also showed that the non-uniformity of the bubble distribution along the electrode results in a non-uniform current density distribution. Mat et al.[2,3] developed a two-phase mathematical model for hydrogen production ina forced flow electrochemical system and found that model successfully captures the main characteristics of electrolysis process. Nagai et al.[4] experimentally investigated effects of current density and space between electrodes on efficiency of water electrolysis. They also studied the341effects of electrode heights, system temperature and electrode inclination in the presence as well as absence of separator.

2.METHODOLOGY

2.1 TWO PHASE FLOW

The **two phase** flow of **gas** and **liquid** is interpreted using mathematical equation continua . As following is the two phase flow and volume occupied by one phase cannot be occupied by another phase the concept of phase volume fraction is introduced. These volume fractions are assumed to be continuous functions and their sum is equal to one. $\alpha l + \alpha g = 1$................(1).
Where l and g are represented to liquid and gas phase respectively. Conservation equations were developed to set of equations (2), (3). Which has similar structure for all phases. Standard electrolysis with 2 phase flow is shown in Fig.1.

Continuity:

$$\frac{\partial}{\partial t}(\alpha i \rho i) + \nabla.(\alpha i \rho i \bar{\vartheta}) \ldots\ldots\ldots\ldots\ldots\ldots(2)$$

Momentum:

$$\frac{\partial}{\partial t}(\alpha i \rho i \bar{\vartheta}) + \nabla.(\alpha i \rho i \bar{\vartheta}\bar{\vartheta})$$
$$= -\alpha i \nabla \rho + \quad \nabla.(\alpha i \mu eff \nabla \bar{\vartheta}) \quad + \alpha i \rho i \bar{g} + kji\overline{\vartheta r} + \quad kji(\frac{Dj}{\alpha j \sigma ji}\nabla \alpha j$$
$$- \frac{Di}{\alpha i \sigma ji}\nabla \alpha i) \ldots\ldots\ldots\ldots\ldots\ldots..(3)$$

Where i& j represent gas and liquid phases, μeff is effective viscosity which is the combination of turbulent and laminar flows ,$\alpha i \rho i \bar{g}$ is buoyancy force, $\bar{g}$ is gravity vector, $\alpha i \rho i \bar{g}$ is buoyancy force .drag function. Where

$$f = \frac{CdRe}{24} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\mathbf{(4)}$$

$$\vartheta r = k_{ji}(\frac{Dj}{\alpha j \sigma ij}\nabla \frac{Di}{\alpha i \sigma ji}\nabla \alpha i) \ldots\ldots\ldots\ldots\ldots (5)$$

$$kji = \frac{\alpha i \alpha j \rho j f}{\tau j} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

$$C_D = \begin{cases} 24(1+0.15Re^{0.687})/Re, Re \leq 1000 \ldots\ldots (7) \\ 0.44,\ Re > 1000 \end{cases}$$

$$\nabla.(\sigma eff \nabla \emptyset i) = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots(8)$$

$$\sigma eff = \sigma 0(1-\alpha) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

2.1 Experimental setup

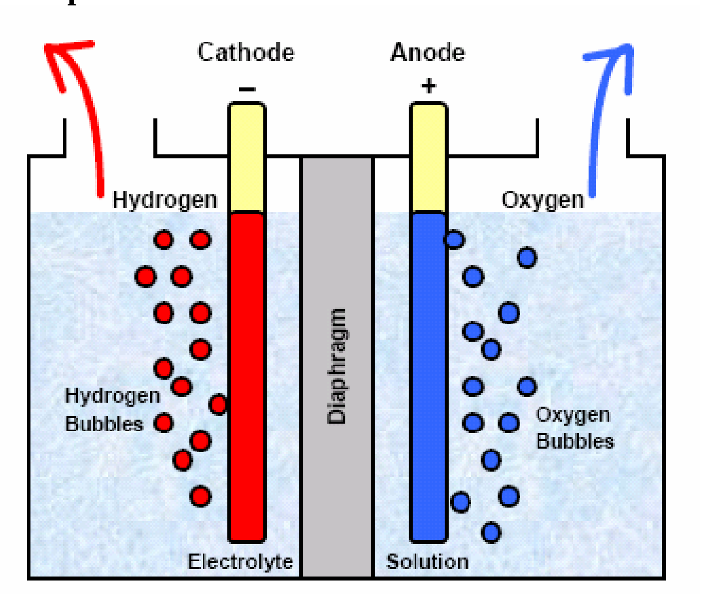

Fig.1 Schematic diagram of Standard Electrolysis

Experimental setup is made as per the requirement. We have done the experiment for two cases. In the first case, the electrode plates are placed parallel to each other and a spacing of 1mm maintained between the electrode and the membrane shown in Fig.2.In the second case the electrodes are kept in an inclined position as in Fig.3. and is tested for efficiency. KOH solution is placed in between the electrodes plates, anode 24 and cathode 14, asbestos membrane 30.

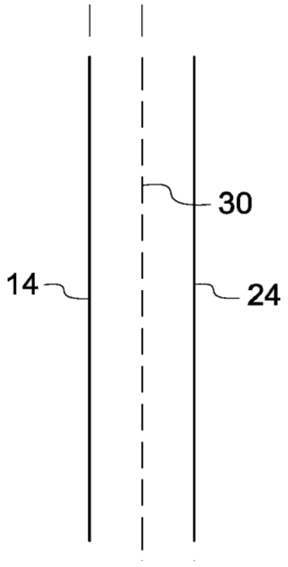

Fig.2 Electrode plates at equal distance parallel to each

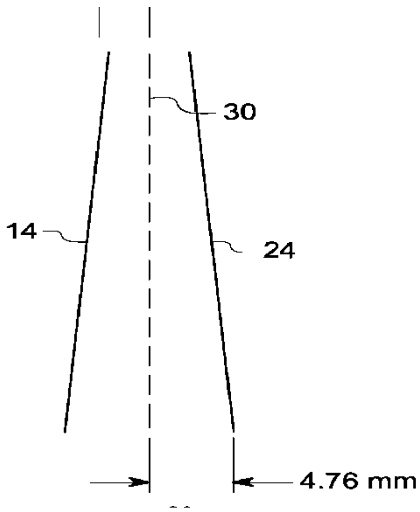

Fig.3 Electrode plates inclined at top

2.3 EXPERIMENTAL DETAILS

Current-cell voltage curves were recorded in a 5”x5” square parallel plate divided or undivided flow cell equipped with a Tin cathode and 204 stainless steel anode. The data was obtained at temperatures ranging from 20 to 60˚C, using a Xantrex XHR20-50 power supply as a DC source. The electrocatalytic cathodes were prepared by co-depositing nickel and zinc or nickel and aluminum onto a stainless steel 204 substrate by a wire-arc deposition method. Most of the zinc oraluminum was then removed from the surface of the electrode by leaching in circulating 30% KOH solution at 60˚C for periods of 6 to 24 hours. The anode material was 204 Stainless Steel. The active electrode surface area was 161cm2. 30% w/w KOH solution was prepared by dilution of Fisher Certified 45% w/w solution with de-ionized water. In a typical experiment, a 30% KOH solution at temperatures of 20˚to 60˚C was circulated through the cell at flow rates of up to 2L/minute. The current was increased in 5A increments, and the cellvoltage at each current was recorded manually.

2.4 THEORITICAL CONSIDERATIONS

The hydrodynamic and electrochemical components of the model are coupled through the source terms in the continuity equation and through the voidage of the electrolyte solutiondue to gas bubbles. Mass of Hydrogen produced at the cathode is obtained from the local current density value using the first law of electrolysis. This value of Hydrogen mass is given as the mass ‘source’ value in the continuity equation for Hydrogen. Similarly, mass of Oxygen produced at the anode is obtained from the local current density and given as mass ‘source’ value in the corresponding continuity equation. Mass ‘sink’ term for water is also obtained using the first law of electrolysis and added in it’s continuity equation. Voidage computed with these source/sink terms is used to find the new local current density.

$$V_{theoritical} = (R\ I\ T\ t) / (F\ p\ z)$$

Where:
R=8.314 Joule/(mol Kelvin)
I = current in amps
T = the temperature in Kelvins (273 + Celsius temperature)
t = time in seconds
F = Faraday's constant = 96485 Coulombs per mol
p = ambient pressure = about 1 x 105 pascals (one pascal = 1 Joule/meter3)
z = number of "excess" electrons (2 for Hydrogen)

Table 1: Standard values of different parameters

Symbol	Value
ρ_l	1050 kg/m^3
D	100e-6m
σ_0	50-100S/m
i_0	0.2 A/m^2
T	20 – 60 ˚C
F	96487 C/mol
R	8.314 J/mol

2.5 LOSSES IN THE COLLECTION OF H_2

1. Failure to capture all the hydrogen
2.Energy lost due to heat
3. Various measurement error

3. RESULTS AND DISCUSSION

Experimental setup was designed and the results that was obtained are as follows.

3.1 Amount of H_2 Evolved :

Trial -1:

- Total volume of test tube = 65 ml
- Voltage supplied = 6V
- Time interval = 5 mins
- Reduction in Height of water = 1 cm
- Volume of H_2 produced = 1 cm^3

Trial 2:

- Total volume of test tube = 65 ml
- Voltage supplied = 8V
- Time interval = 5 mins
- Reduction in height of electrolyte = 1.3 cms
- Volume of H2 produced = 1.3 cm^3

Trial 3:

- Total vol of test tube = 65 ml
- Voltage supplied = 12V
- Time interval = 5 mins
- Reduction in height of electrolyte = 2.7 cms
- Volume of H_2 produced = 2.7 cm^3

Thereotical value
Trial 1:
Temperature = 273 + 37 C
Time = 5 mins
Current = 11 amp
$V_{theoritical}$ = 0.719 cm^3
Efficiency (in %) = 100 x $V_{produced}$ / $V_{theoretical}$

3.2.Best Result Validation

Experimental setup was simulated in the software by following the above conditions and constrains. The results that will be obtained are as follow. Different values or constants that are used in simulation

MODEL VALIDATION WITH EXPERIMENTAL OBSERVATION

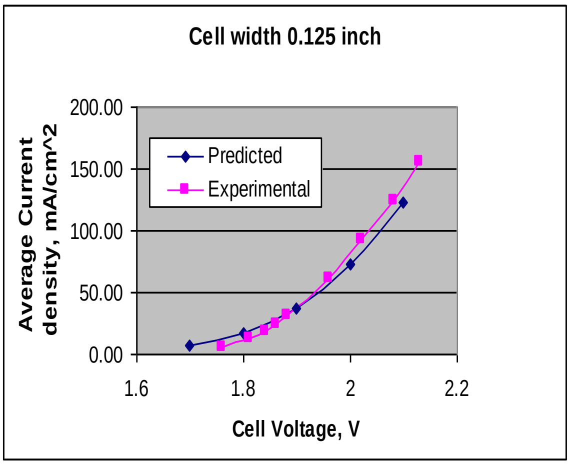

Fig.4: (ModellingBubble Flow and Current Density Distribution in an Alkaline Electrolysis Cell)

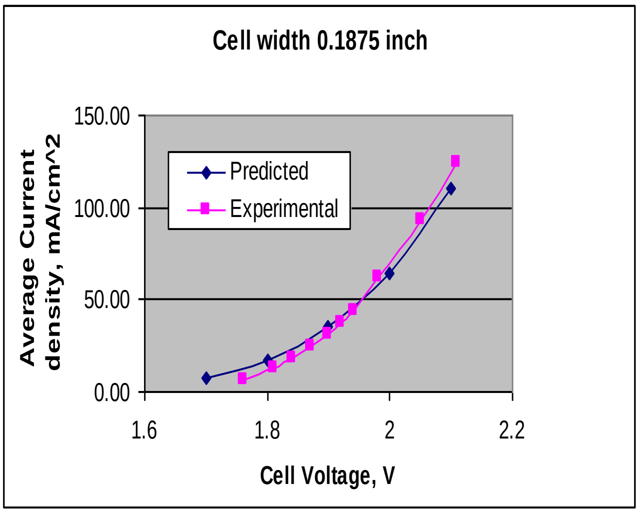

Fig.5(Modelling Bubble Flow and Current Density Distribution in an Alkaline Electrolysis Cell)

3.3 Advantages of hydrogen over other fuels:

- Highest energy weight ratio of all fuels
- Diffuses in air faster than petrol & CH_4
- Burns in air over a wide range.

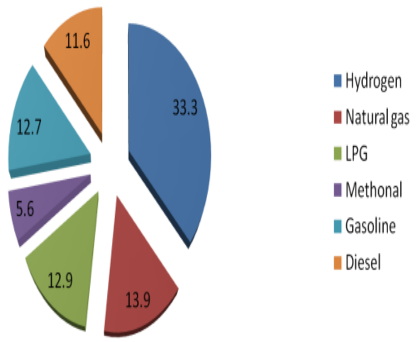

Fig.6 weight and volume related energy densities for hydrogen in comparison with Other energy carriers

5. Conclusions

Eulerian-Eulerian two-phase flow model available in Fluent 6.2 is used to develop a computational fluid dynamics model for multiphase flow involved in alkaline electrolysis.Electrochemistry isincorporated using the UDF (User Defined Function) feature of Fluent. Electrolyzer physicsincorporated in the model is validated by mesh refinement study and by comparison with in-houseexperimental data. Model predicts the existence of an optimum inter-electrode gap at any particular electrolyte flowrate. Further, optimum inter-electrode gap is found todecrease with increase in flow rate. Modelwould serve as a very useful tool in the design andoptimization of alkaline electrolyzer cell and stack.

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A MATERIAL BY ITSELF CAN BE ECO-FRIENDLY BAMBOO: AS A CONSTRUCTION MATERIAL

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Abstract

In this world of constantly increasing population and depleting resources there is urge to adopt cost effective and ecofriendly structures. These papers discuss the potential of bamboo and project the possibilities of usage of bamboo in the construction field. Bamboo is an ancient solution for the present day problem. Bamboo is an appropriate substitute for the present convention building material such as steel and wood. The main characteristic of the bamboo which makes it a suitable building material is it's high tensile strength which is equivalent to mild steel at the yield point and very good weight strength ratio making it high resilient against the forces created by the earth quakes and hurricanes. Bamboo can replace 70% of steel and wood used in the construction and reduce the cost by 40%. Bamboo can be used from scaffolding to every stage of construction like in footings, beams, columns, slabs, stair cases, doors, windows etc. Bamboo is the renewable resource with amazing growth rate, rejuvenates the soil and grows in varied climatic conditions. Bamboo absorbs carbon dioxide and releases 35% more oxygen into the atmosphere than other hardwood trees. There are few building codes also available for the usage of bamboo in the construction such as ISO 22156: 2004 Bamboo structural design, ISO 22157: 2004 Bamboo physical and mechanical properties, IS 9096: 1979 Code of practice for preservation of bamboo for structural purposes. Thus bamboo is environmental friendly, energy efficient and cost effective material.

Keywords*: Bamboo, ISO 22156:2004, ISO 22157: 2004, IS 9096: 1979*

1 Introduction

Bamboo is primarily a type of giant grass with woody stems. The stems are called "shoots" when the plant is young and "culms" when the plant is mature. Each bamboo plant consists of two parts – the "Culm"/stem that grows above the ground and the underground "rhizome" that bears the roots of the plant. "A single bamboo clump can produce up to 15 kilometers of usable pole (up to 30 cm in diameter) in its lifetime."

Fig1: Various species of Bamboo

1.1 POTENTIAL OF BAMBOO

It is Fastest growing plant. Bamboo has highest carbon dioxide absorption. It has Continuous absorption of carbon dioxide and release of oxygen .Quick harvest is possible which can be also continuous harvest .Sustains green cover of world.

1.2 GROW BAMBOO AT YOUR OWN PLACE

Just a 2ft length, 2 ft width and 2 ft depth pit is required. Mixture of 1/3rd of soil sand and manure each .The bamboo is to be properly watered it properly until it is nourished. If you don't have proper place it can also be planted in a 100 liters plastic drum. To protect it once in every 6 months remove weak shoots affected by insects.

1.3 BAMBOO STRUCTURES IN THE WORLD

Bird-Like Amphitheater, Hanoi, Vietnam: This amphitheater in Vitenam used for the plays, auditorium is constructed only with bamboo and ropes.

Fig2: Bird-Like Amphitheater

Green School, Bali: The Green School in Bali is the school with no walls. It's one of the green schools in the world where education is taught in the laps of environment.

Fig3: Green School, Bali

1.4 BAMBOO FOOTINGS

For use as foundation, the bamboo poles are directly driven into the ground. They have to, however, be pre-treated for protection from rot and fungi.

1.5 BAMBOO TRUSSES

For the spanning larger distances in public utility buildings like schools, storage areas, commercial buildings, bamboo is utilized as a truss member. Bamboo has a high strength /weight ratio and hence is a good alternative for roof framing.

Fig4: Bamboo Truss

1.6 BAMBOO WALLS

Bamboo walls are constructed by nailing a thin bamboo mat to either sides of a braced timber frame.

Fig4: Bamboo Walls

1.7 BAMBOO SCAFFOLDING

Since ancient times, bamboo poles have been tied together and used as scaffolding. The properties of bamboo such as resilience, shape and strength make it an ideal material for the purpose. The working platforms for masons can also be built of bamboo.

Fig5: Bamboo Scaffolding

1.8 BAMBOO TILE ROOFING

- This is the simplest form of bamboo roofing. The culms are split into halves, the diaphragms scooped out and these run full length from eave to ridge.
- The first layer of bamboo splits are layed concave side up and the second layer interlock over the first with convex side up. Though a very simple method, it can be completely watertight. The minimum pitch of the roof should be 30°.

Fig6: Bamboo Tile Roofing

1.9 BAMBOO REINFORCEMENT

Besides the use of bamboo as a building material, there have been proposals to use bamboo as reinforcement in RC columns, beams and slabs. One of the examples is a silo made of bamboo-reinforced concrete. This is the avenue for further research in the process of combining the ancient of bamboo building with modern materials like concrete**.**

Fig7: Bamboo Reinforcement

2. PROPERTIES OF BAMBOO

Tensile strength: The fibers of the bamboo run axial. In the outer zone are highly elastic vascular bundles that have a high tensile strength. The tensile strength of these fibers is higher than that of steel.

Shrinking: Bamboo shrinks more than wood when it loses water. The canes can tear apart at the nodes. Bamboo shrinks in the cross section ca. 10-16 %, in the wall thickness ca. 15-17 %.

Fire resistance: The fire resistance is very good because of the high content of silicate acid. Filled up with water, it can stand a temperature of 400° C while the water cooks inside

Strength Compressive: The portion of lignin affects the compressive strength. Whereas the high portion of cellulose influences the buckling and the tension strength, because it represents the building substance of the bamboo fiber.

Elastical modulus: In connection with the elastic modulus you can see an advantage in the use of slim tubes in relation to their cross section, too. The accumulation of highly strong fibers in the outer parts of the tube wall also work positive in connection with the elastically modulus like it does for the tension shear and bending strength. There exist an perfect relation of the cross section of the tube, if you fall below or above it the elastically modulus decreases (the higher the elastically modulus of the bamboo, the higher is the quality). Like the elastically modulus of solid wood the one of bamboo also decreases 5 to10% with growing stress. The enormous elasticity makes bamboo to be a very useful building material in areas with high risk of earthquakes. In Asia they still construct scaffolds with bamboo tubes

Flexural (bending) strength: A trops analyzed common bamboos: diameter of tubes= 70-100 mm, wall thickness= 6-12 mm with a span of 3,60m. The elastically deflections were minimum =1/25under maximum 1/16, and as an average 1/20,1 of the spans. Where a deflection in the construction was unavoidable and annoying, one could bend the recently harvested tubes so that you get a super elevation, which later will be compensated under the working load.

Shearing strength: Especially for the construction of the bamboo tube joining it is important to consider the shearing resistance. The influence of the distance of the shearing surface decreases with growing length of shearing surface. At a wall thickness of 10 mm the shearing strength is about 11% lower than at a tube with a wall thickness of 6 mm; this could be explained by the distribution of the high-strength fibers per cross section surface.

3. THE FRACTURE BEHAVIOR:

The behavior of breaking of common building wood differs clearly from the breaking conditions of bamboo. Here you don't have a spontaneous break through the whole material after the tearing of single bamboo fibers like wood does. The appearing clefts are led off immediately in direction of the fiber and so they impair the critical region less. The energy transfer is delayed by diffusion. Especially the pressure-, shearing-, and inter laminar strength are raised by the knots. Those symptoms are titled as increasing factor of the fracture toughness. In the research of modern compound material it is less important to prevent the formation of cracks than to counteract the distribution of the clefts by finding a suitable material construction.

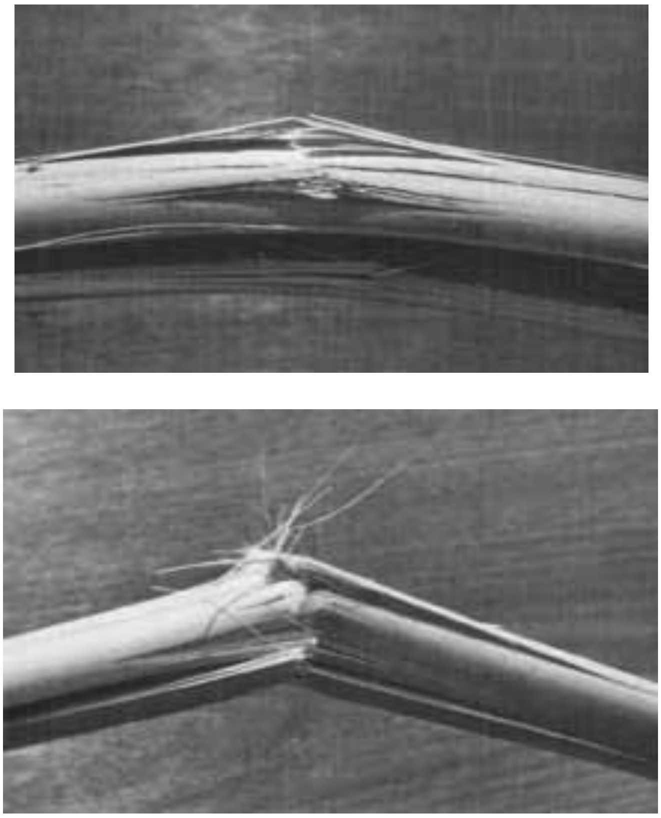

Fig8: The fracture behavior

The work that is needed for the punch of a bamboo tube is nearly the same whether the punch hits the knot or the intern odium. But the breaking conditions itself are totally different. If the punch hits the knot the tube will burst in axial stripes; that means a break as a result of the effort of the strength vertical to the fibers.. It is not comparable to the value of the spruce (0,5 mkp/cm²) because the bamboo is of course not solid but a tube.

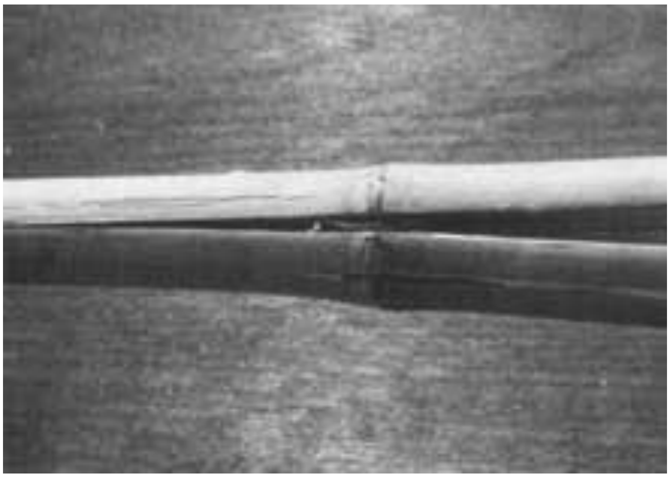

Fig9: The punch fracture behavior of bamboo

Table 1: Properties of bamboo

PROPERTIES	BAMBOO
Specific gravity	0.575 to 0.655
Average weight	0.625kg/m
Modulus of rupture	610 to 1600kg/cm2
Modulus of Elasticity	1.5 to 2.0 x105kg/cm2
Ultimate compressive stress	794 to 864kg/cm2
Safe working stress in compression	105kg/cm2
Safe working stress in tension	160 to 350kg/cm2
Safe working stress in shear	115 to 180kg/cm2
Bond stress	5.6kg/cm2

4. WORKING OF BAMBOO

Bamboo can be worked with the simplest tools which must be especially sharp because of the highly silicified outer zone. Tool wear is considerably high.

Splitting: very easy as long as you work along the cane axis. The cane is split in halves and quarters and the driven apart by a wedge. It can also be split with a knife frame into four or eight segments cutting with a machete-type or knife used for cutting.

Fig10: Bamboo Splitting

Shaping: Bamboo which grows in a box gets a square shape. So it can be better used for connections.

Bending: Freshly cut, bamboo can be bent and will keep this shape after drying. When heated above 150° C, bamboo keeps its shape after it goes cold.

Fig11: Bamboo Bending

5. PRESERVATION AND TREATMENT

As bamboo has less natural durability it requires chemical treatment for longer life. Bamboos have low natural durability (1 to 3 years) against attacks by fungi and insects. They are very difficult to be treated by normal preservative methods in dry condition since their outer and to some extent inner membranes are impermeable to liquids. The treatment of bamboo is, therefore, best carried out in green conditions.

5.1 TYPES OF PREVENTION

Coal Tar Creosote:

- This is a fraction of coal tar distillate with a boiling point range above 200°C and is widely used admixed with fuel oil in the ratio of 50:50.
- The fuel oil ensures stability to creosote against evaporation and bleeding from the treated bamboos.
- Creosote has high performance; it is non-corrosive and provides good protection from termites.

Boric Acid Borax*:*

- This has been used successfully against lyctus borers. A mixture of 2:5 percent of each is found more suitable.

5.2 METHODS OF TREATMENT

- ***Surface Application*****:** this is done by brushing, spraying or dipping of timber in preservative solution for the required period.
- ***Soaking process*****:** the debarked timber is submerged in the preservative solution for sufficient period till the desired absorption is obtained.

6. ADVANTAGES OF BAMBOO

- Bamboo is low weight material to be used for construction.
- It can be transported and worked easily.
- The use of cranes is mostly unnecessary.
- It is a very flexible plant.
- Grows back very rapidly once harvested.
- Raw material for paper making.
- Bamboo has a higher tensile strength.
- Usage of bamboos will reduce deforestation
- Composite material

7. DISADVANTAGES OF BAMBOO

- If not treated well it will get attacked by the fungi.
- Bamboo does not lend itself to being painted because of its waxy coating.
- Bamboo is not designed to bear weight width-wise.

CONCLUSION

- Bamboo is lighter in weight than bird but is stronger than steel.
- It takes carbon dioxide in and releases 30% more oxygen than tree. It grows a meter in one year and is mature in almost 3 years.
- Houses constructed using this bamboo are cool in summer and stays warm in winter and more over it can withstand earthquakes and can stand forever.
- The environmental and financial comparison demonstrates that bamboo can compete with building material.
- Bamboo is a natural product and will therefore always have some extent of irregularity.
- It is therefore suggested that the bamboo culm should be used in functions were the measurement requirements are not entirely precise or fixed, as in temporary buildings (e.g., pavilions and tents) or small civil projects.

- Furthermore, bamboo can play a role as a non-supporting or finishing material.

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COMPARATIVE ANALYSIS OF INTERNATIONAL CODES OF PRACTICES FOR BUILDINGS

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Abstract

Design procedures and assumptions relating to design of various structural components are governed by various codes of practices. American Standards are used in USA /Canada/ Latin America/ South American Countries. EU specifications are used in Europe and Scandinavia. IS standards govern Design in SAARC region while Australia and New Zealand. These Codes of practices govern building activities of about 80% of world human population. In the present Thesis comparative study of different Codes of Practices for Buildings and Bridges such as Euro & Indian [IS: 456-2000, American Concrete Institute, Hong Kong government concrete code of practice CP-04 will be done.

Introduction

Structural engineering is the back bone of Civil Engineering and infrastructure in any society. It is crucial that all the codes related to structural engineering are based on principles of mechanics, and experimentally verified. They should be logical, rational and efficient, and should be revised as has been conducted on various aspects in advanced countries. Factors affecting strength, stability and performance of R.C.C structures frequently as necessary. Performance based codes need to be introduced at the earliest so that our engineers may compete globally. Each society has developed its own guidelines. On how to construct safe houses/structures in its own ways from times immemorial based on its own experiences with materials, constructionPractices and nature. Over the last century each code has evolved based on scientific and technical inputs. In India, it is an open fact that there is no fundamental and focused research in structural engineering field including earthquake engineering.
Until 1950’s there were no engineering methods of designing R.C.C buildings and thickness of walls was based on Rule-of-Thumb tables given in Building codes and Regulations. As a result walls used to be very thick and R.C.C structures were found to be very uneconomical beyond 3 or 4 stories. Since 1950’s intensive theoretical and experimental research have been identified, which need to be considered in design.In India, there has not been much progress in the construction of tall load bearing R.C.C structures.

REVIEW OF LITERATURE

N. Subramanian, The Indian concrete journal (August 2005)The bond between concrete and reinforcement bars is very important to develop the composite behavior of reinforced concrete. Bond strength is influenced by several factors such as bar diameter, cover of concrete over the bar, spacing of bars, transverse reinforcement, grade and confinement of concrete around the bars, aggregates used in concrete, type of bars and coating applied on bars, if any, for corrosion prevention. In the Indian code on concrete structures which was revised in the year 2000, the provisions regarding development length remained unchanged. The effect of high strength concrete, self-consolidating concrete and fiber reinforced concrete on the development length is also discussed. A formula for inclusion in the Indian code is also suggested based on recent research..

Ashok K Jain, (MARCH 2014) in his paper related to the state of codes on structural engineering in India,Structural engineering is the back bone of Civil Engineering and infrastructure in any society. It is crucial that all the codes related to structural engineering are based on principles of mechanics, and experimentally verified. They should be logical, rational and efficient, and should be revised as frequently as necessary.It is recommended that India should adopt any of the code in full from amongst some of the finest international codes along with commentary. A national annexure can be appended to address the local practices and other issues. Performance based codes need to be introduced at the earliest.

SaravananRamalingam and Manu Santhanam in The Indian Concrete Journal, MAY 2012, The Indian Code IS 456:2000 for plain and reinforced cement concrete specifies five exposure classifications namely, mild, moderate, severe, very severe and extreme, which seem to be arbitrary and prescriptive in nature regarding durability requirements. Further, the classifications do not necessarily address the relevant mechanisms of concrete deterioration adequately. Given the importance of environmental effects on concrete service life and performance, it is necessary to have detailed classifications catering to all exposure conditions, which address the appropriate mechanisms of deterioration.

The shortcomings of the present system are described first, followed by a summarization of the international developments in exposure classifications. Concrete mix designs from several construction sites across India executed by Central Public Works Department are then presented and analyzed in the light of the prescriptions made by codal provisions from a number of countries across the world. The results of the analyses, along with the relevant features of international developments, are used to finally propose a rational system for classification of concrete exposure conditions.

SCOPE AND OBJECTIVES

The objective of the thesis is to study different codes from a number of countries on the design of reinforced cement concrete structures.

Various aspects considered in analysis are:

1. Methods for analysis & design
2. Specifications related to materials, design load combinations.
3. Design detailing.
4. Provisions for durability, serviceability, and sustainability.

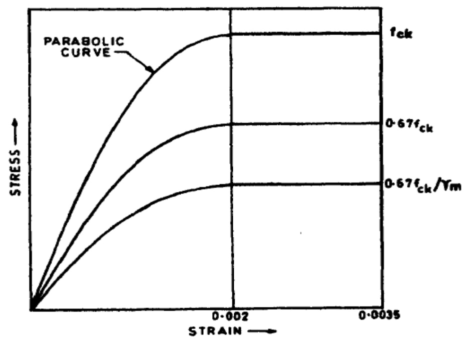
The different international construction codes of practices considered are

1. IS 456-2000(Indian Standard Code of Practice for Plain and Reinforced Concrete for General Building Construction)
2. BS 8110-97 (Structural Use of Concrete Code of Practice for design and construction
3. ACI 318-99 (Building Code Requirements for Structural Concrete and Commentary)
4. HONGKONG Government concrete code of practice (CP:04)

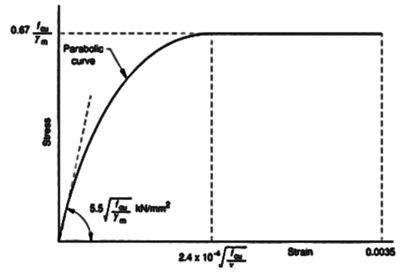
METHODS OF ANALYSIS & DESIGN

	IS456:2000	BS8110:97	ACI-318:99	HONGKONG CODE
1	Limit state method of design	Limit state method of design	Theory of elastic analysis	Limit state method of design
2	Working stress method of design			

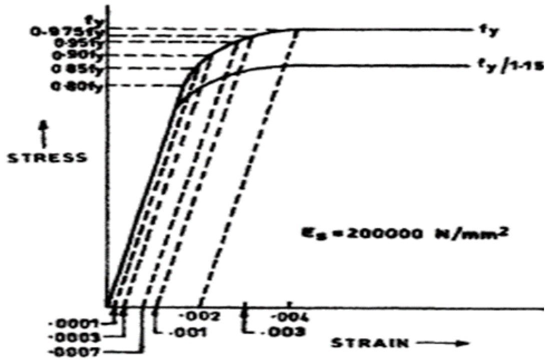
Stress Strain Curve for Concrete



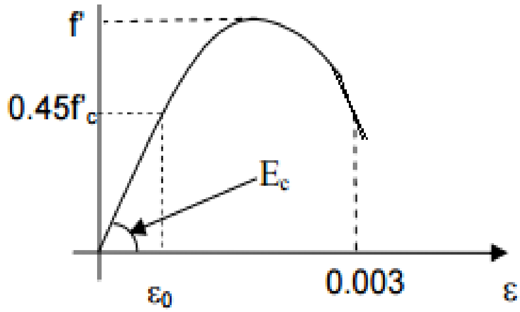
Stress Strain curve for concrete IS456



Stress Strain curve concrete BS8110



stress strain curve for steel IS456



Stress and strain curve for concrete ACI318

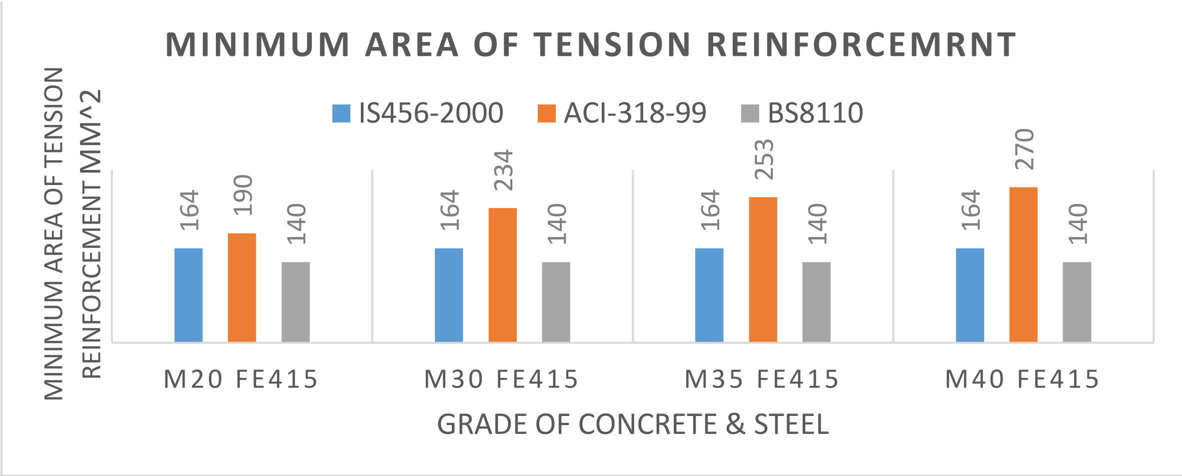
NOMINAL COVER		
IS 456 in (mm)	BS8110 in (mm)	ACI-318 in (mm)
Columns Not less than 40mm When dimension is 200mm or size of bar is 12mm then cover is 25mm **Footings** Not less than 50mm **Slabs** Not less than 15mm **Beams** not less than 20mm	**Beams** not less than 25 mm **Columns** not less than 40mm. **Footings** not less than 50mm. **Slabs** not less than 20mm.	**Footing** Resting on mat – 50 Resting on earth – 75 Top of piles – 50 **Beams and columns** **For dry conditions** Stirrups, spirals, ties – 38 Main reinforcement – 50 **Exposed to earth** Stirrups, spirals, ties – 50 Main reinforcement – 60 **Slabs** No. 11bars – 20mm No. 14 – 18 bars – 38mm

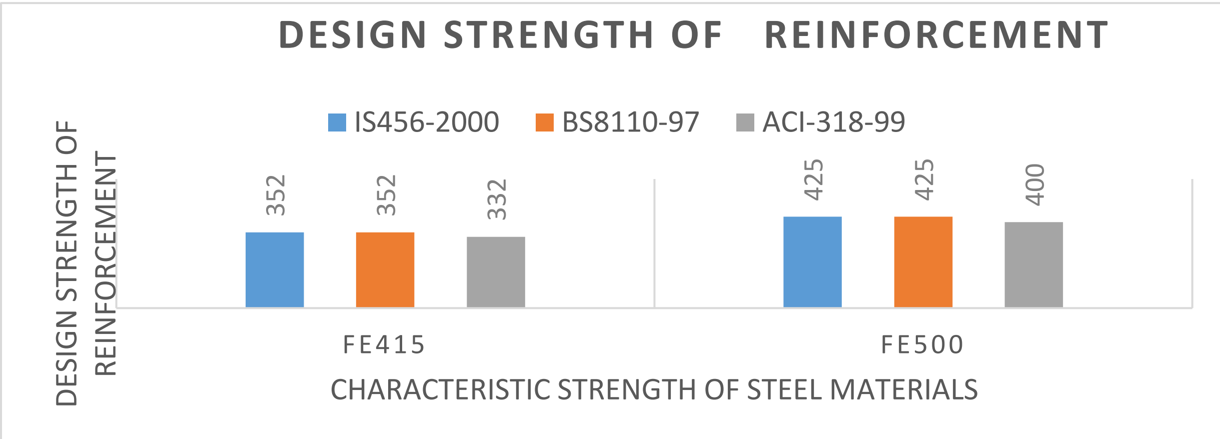
Exposure condition	**IS456-2000**	**BS8110-97**	**ACI-318-99**
Normal (mm)	0.3	0.3	0.33(inside) 0.44(outside)
Aggressive(mm)	0.1	0.1	0.1

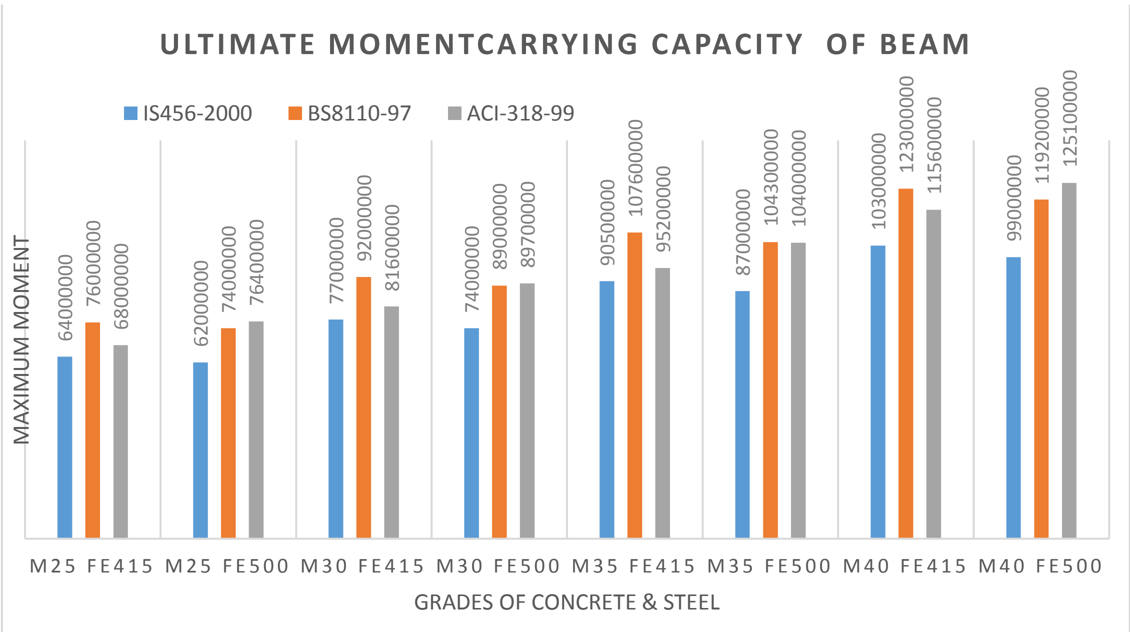
Exposure condition	**ACI**	**CEB-FIP**	**IS456-2000**
Low humidity, dry air or protective environment	0.4	0.4-0.6	0.3
High humidity, moist, air, soil	0.3	0.2-0.3	0.2
Deciding chemicals	0.2	0.1-0.15	0.1
Sea water and sea water spray	0.15	0.1-0.15	0.1
Water-retaining structures	0.31		

Control of deflection	**IS456**	**ACI318**	**BS8110**
Final deflection due to all loads including the effects of creep, temperature, shrinkage and measured from as cast level of the supports of floors, roofs and all other horizontal members, should not normally exceed	Span/250	Span/250	Span/240
The deflection including the effects of creep, temperature, shrinkage occurring after erection of partitions and the application of finishes soul dot normally exceed	Span/350(or)20mm (whichever is less)	Span/500(or)20mm (whichever is less)	Span/480

Load combinations		
IS456	BS8110	ACI318-99
1.5(DL+IL) 1.2(DL+IL+EL) 1.5(DL+EL) 0.9(DL)+1.5(EL) DL+LL+WL+TL	1.4(DL)+1.6(IL) 1.4(DL+WL) 1.4(DL+EL) 1.2(DL+IL+WL) 1.2(DL+IL+EL) 1.4DL + 1.6IL+ 1.6SL 1.2D+1.2L+1.2SL±1.2WL	U= 1.4DL+1.7IL U=0.7(1.4DL+1.7IL+1.7WL) U= 0.9DL+1.3WL U=1.4DL+1.7IL+1.7H U=0.7(1.4DL+1.7T+1.7IL) U= 1.2D + 1.0L + 0.5S ± 1.6WL U=1.05DL+1.28IL+1.4EL U= 0.9DL+1.43WL


MINIMUM AREA OF TENSION REINFORCEMRNT
IS456-2000
ACI-318-99
BS8110
MINIMUM AREA OF TENSION REINFORCEMENT MM^2
164
190
140
164
234
140
164
253
140
164
270
140
M20 FE415
M30 FE415
M35 FE415
M40 FE415
GRADE OF CONCRETE & STEEL


DESIGN STRENGTH OF REINFORCEMENT
IS456-2000
BS8110-97
ACI-318-99
DESIGN STRENGTH OF REINFORCEMENT
352
352
332
425
425
400
FE415
FE500
CHARACTERISTIC STRENGTH OF STEEL MATERIALS


ULTIMATE MOMENTCARRYING CAPACITY OF BEAM
IS456-2000
BS8110-97
ACI-318-99
MAXIMUM MOMENT
64000000
76000000
68000000
62000000
74000000
76400000
77000000
92000000
81600000
74000000
89000000
89700000
90500000
107600000
95200000
87000000
104300000
104000000
103000000
123000000
115600000
99000000
119200000
125100000
M25 FE415
M25 FE500
M30 FE415
M30 FE500
M35 FE415
M35 FE500
M40 FE415
M40 FE500
GRADES OF CONCRETE & STEEL

CONCLUSION

- ACI318 is frequently revised for at least every 6 years but IS456 & BS8110 are not revised frequently.
- ACI318 Code are more extensive for design requirements point of view than BS8110 &IS456 as it permitting using higher concrete strength.
- The principles contained in various codes are generally based on the similar considerations but they differ in detailspecifications and application consideration.
- IS456 & BS8110 allows more factor of safety allowing less economical structures.
- Ultimate moment carrying capacity of ACI-318 is high compared to other codes.
- Tension reinforcement value is also high compared to other codes.

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STUDIES OF NITRIC ACID ATTACK ON SCBA CONCRETE

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Abstract

The main objective of this paper is to study the influence of partial cement replacement with rice husk ash in concrete subjected to different curing environments. Experimental investigation was carried out to assess the acid resistance of concrete in HNO_3 solution. The variable factors considered in this study were concrete grade of M_{40} and curing periods of 7days, 28 days, 60 days, 90days, and 180 days of the concrete specimens. The parameter investigated was the time in days to cause strength deterioration factor of fully immersed concrete specimens in 1%, 2%, 3%, 4%, 5% HNO_3 solution. Bagasse ash has been chemically & physically characterized & partially replaced in the ratio of 0%, 5%, 10%, 15% and 20%. Fresh concrete tests like compaction factor test and hardened concrete tests like compressive strength at the age of 7days, 28 days, 90, 180 days was obtained.
***Keywords**: Sugar cane baggase ash, compressive strength, Acid attack, durability, HNO_3 solution.*

Introduction

Sugarcane production in India is over 300 million tons/year leaving about 10 million tons as unutilized and, hence, wastes material. The utilization of SCBA in concrete as a partial replacement of cement is gaining immense importance today; mainly it enhances the long term durability of concrete combined with ecological benefits. The use of SCBA in concrete as a supplementary cementitious material was tested as an alternative to traditional concrete. This SCBA is a great environment threat causing damage to the land and the surrounding area in which it is dumped. Lots of ways are being thought of for disposing them by making commercial use of this SCBA.

Experimental Programme

The experimental investigation carried out by producing concrete by replacing cement by SCBA in the range of 0%, 5%, 10%, 15%, and 20% by weight of cement and tested for 7, 28, 60, 90 and180 days for compressive strengths when exposed to HNO_3 solution of 1%, 2%, 3%, 4%, 5% to evaluate effect of the acid attack .

Test Results & Discussion

The following table gives the specifications of SCBA:

Table 1: Physical properties of SCBA

S. No.	Property	Test Result
1.	Density	96 kg/m^3
2.	Physical state	Solid non-Hazardous
3.	Appearance	Very fine powder
4.	Particle size	25 microns – mean
5.	Color	Gray
6.	Specific gravity	2.3

Table 2: Chemical composition of SCBA

Characteristic	Test Results %
(SiO_2) + $A1_2O_3$. Fe_2O_3 % by mass	85.14
SiO_2 % by mass	60.20
MgO % by mass	2.48
Total sulfuras SO_3 % by mass	0.10
Available alkali as sodium oxide (Na_2O) % by mass	4.32
Loss of ignition % by mass	5.10

Table 4.Workability

% Replacement of cement with SCBA	Workability compaction factor
0%	0.84
5%	0.82
10%	0.81
15%	0.80
20%	0.76

Table 5: Compressive Strengths of M35 SCBA Concrete

Sample Designation	% of SCBA	compressive strength at 7 days	compressive strength at 28 days	compressive strength at 60days	compressive strength at 90days	compressive strength at 180days
W-0	0	38.24	46.19	56.82	59.99	62.00
W-05	5	38.95	47.08	57.54	60.18	62.20
W-10	10	39.69	48.145	57.86	61.16	63.16
W-15	15	37.30	45.61	55.28	58.12	60.12
W-20	20	35.76	44.14	54.01	57.81	59.81

Table 6: Compressive Strength results for cubes cured in 1% HNO_3 solution:

Sample Designation	**% of SCBA**	**compressive strength at 7 days (f_{cu})**	**compressive strength at 28 days (f_{cu})**	**compressive strength at 60days (f_{cu})**	**compressive strength at 90days (f_{cu})**	**Compressive Strength at 180days (f_{cu})**
H_{11}	0	34.59	42.52	52.31	55.38	63.60
H_{21}	5	36.12	43.90	53.66	56.72	65.14
H_{31}	10	37.49	45.16	55.91	59.46	67.30

Sample Designation	% of SCBA	compressive strength at 7 days (f_{cu})	compressive strength at 28 days (f_{cu})	compressive strength at 60days (f_{cu})	compressive strength at 90days (f_{cu})	Compressive strength at 180days (f_{cu})
H_{13}	0	35.50	43.34	53.51	56.91	65.28
H_{23}	5	36.56	44.52	54.66	57.89	66.84
H_{33}	10	38.27	46.11	57.24	60.57	68.79
H_{43}	15	34.91	43.29	55.84	59.21	67.08
H_{53}	20	34.38	42.19	52.97	56.06	64.07
H_{41}	15	34.75	42.94	54.31	57.58	65.50
H_{51}	20	34.11	41.83	51.45	54.33	62.29

Table 7: Compressive Strength Results for cubes cured in 2% HNO_3 solution:

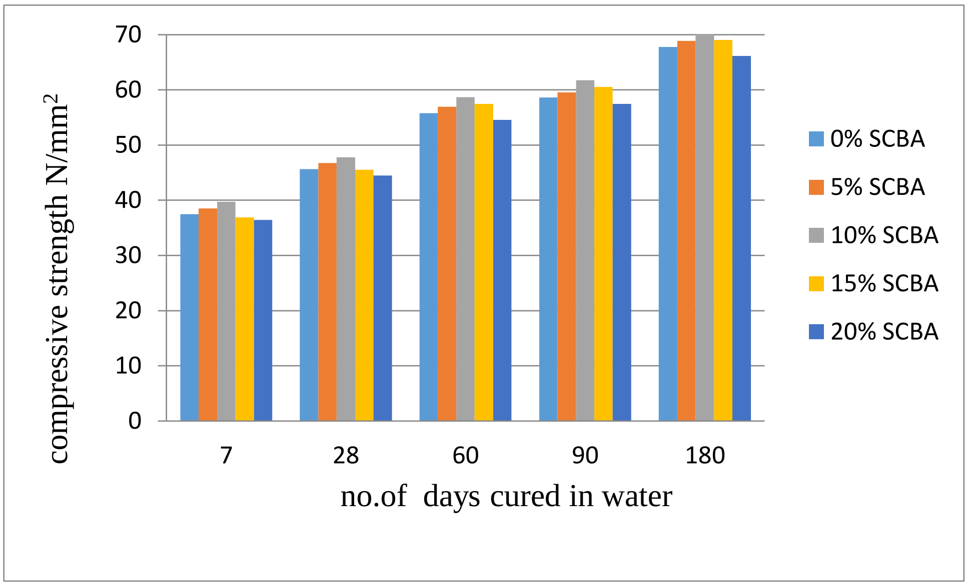
Sample Designation	% of SCBA	compressive strength at 7 days (f_{cu})	compressive strength at 28 days (f_{cu})	compressive strength at 60days (f_{cu})	compressive strength at 90days (f_{cu})	Compressive Strength at 180days (f_{cu})
H_{12}	0	35.27	42.98	52.68	56.05	64.61
H_{22}	5	36.42	44.44	54.64	57.54	66.02
H_{32}	10	37.94	46.29	57.13	60.47	68.41
H_{42}	15	34.87	43.77	55.55	58.74	66.85
H_{52}	20	34.40	42.64	52.48	55.47	63.53

Table 9: Compressive Strength Results for cubes cured in 4% HNO3 solution:

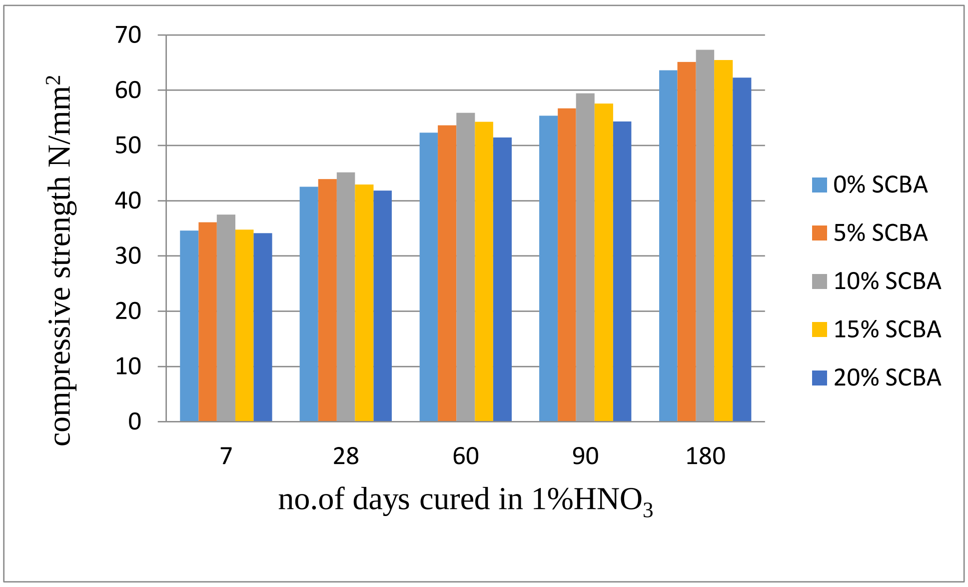
Sample Designation	% of SCBA	compressive strength at 7 days (f_{cu})	compressive strength at 28 days (f_{cu})	compressive strength at 60days (f_{cu})	compressive strength at 90days (f_{cu})	Compressive strength at 180days (f_{cu})
H_{14}	0	35.74	43.68	53.85	57.23	65.70
H_{24}	5	36.96	44.89	55.23	58.35	67.00
H_{34}	10	38.69	46.63	57.34	60.66	68.88
H_{44}	15	35.55	43.98	56.05	59.38	67.27
H_{54}	20	35.05	42.88	53.17	56.13	64.23

Table 10: Compressive Strength Results for cubes cured in 5% HNO_3 solution:

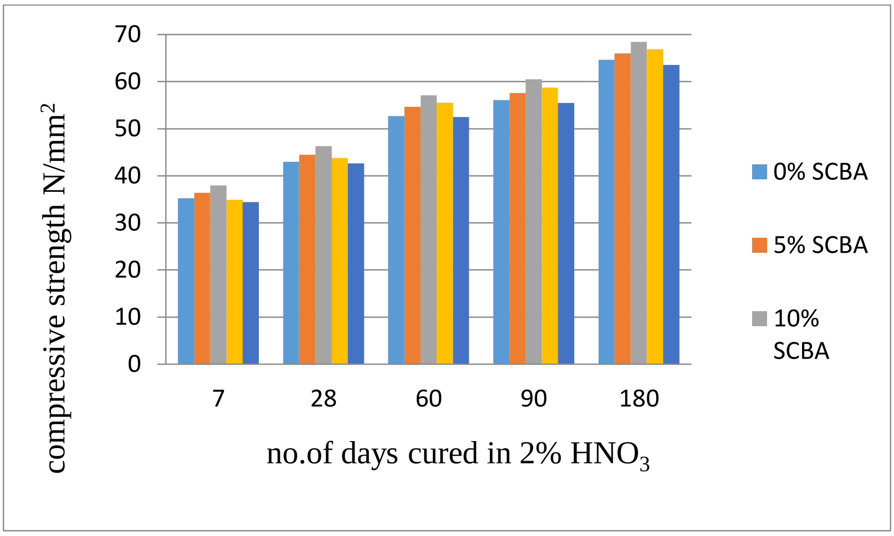
Sample Designation	% of SCBA	compressive strength at 7 days (f_{cu})	compressive strength at 28 days (f_{cu})	compressive strength at 60days (f_{cu})	compressive strength at 90days (f_{cu})	Compressive strength at 180days (f_{cu})
H_{15}	0	36.15	44.42	54.41	57.47	66.08
H_{25}	5	37.58	45.66	55.72	58.41	67.52
H_{35}	10	38.98	46.98	57.95	61.13	69.34
H_{45}	15	35.94	44.51	56.43	59.45	67.73
H_{55}	20	35.46	43.37	53.55	56.39	64.85



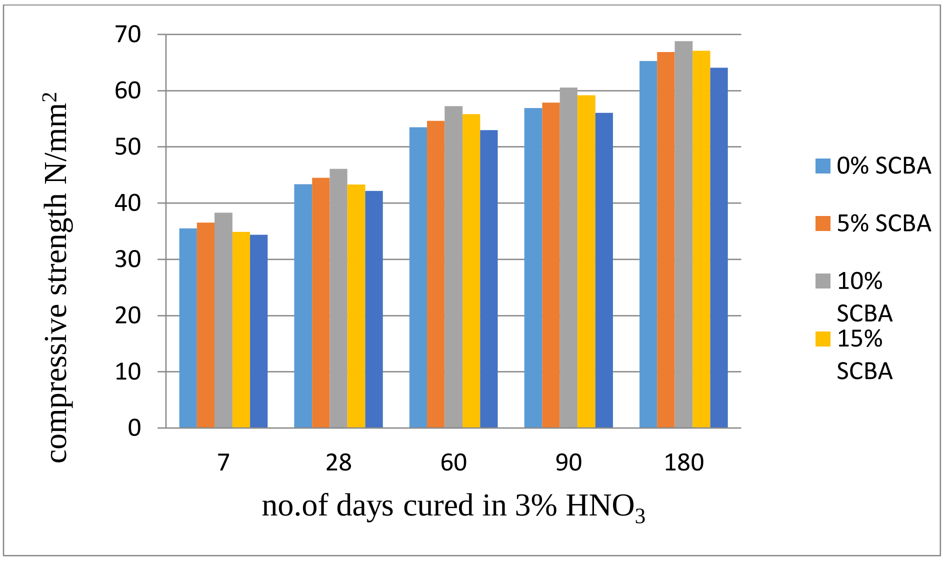
1.Compressive Strength Results of cubes Cured in Water



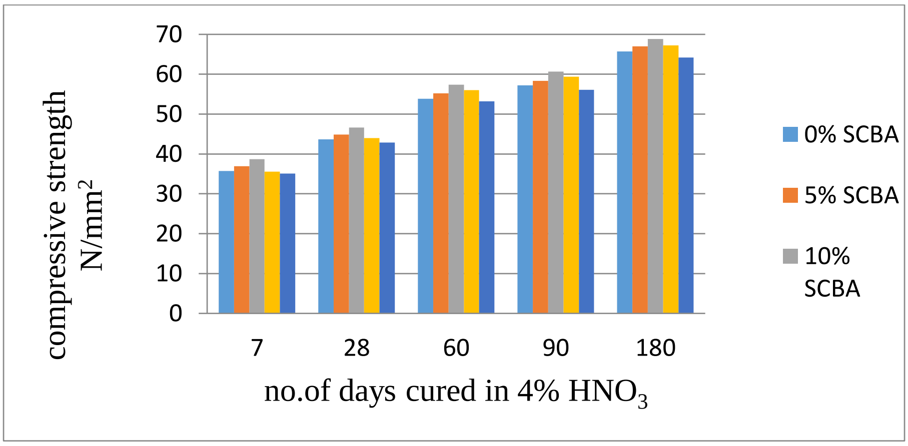
2. Compressive Strength Results of cubes cured in 1% HNO3



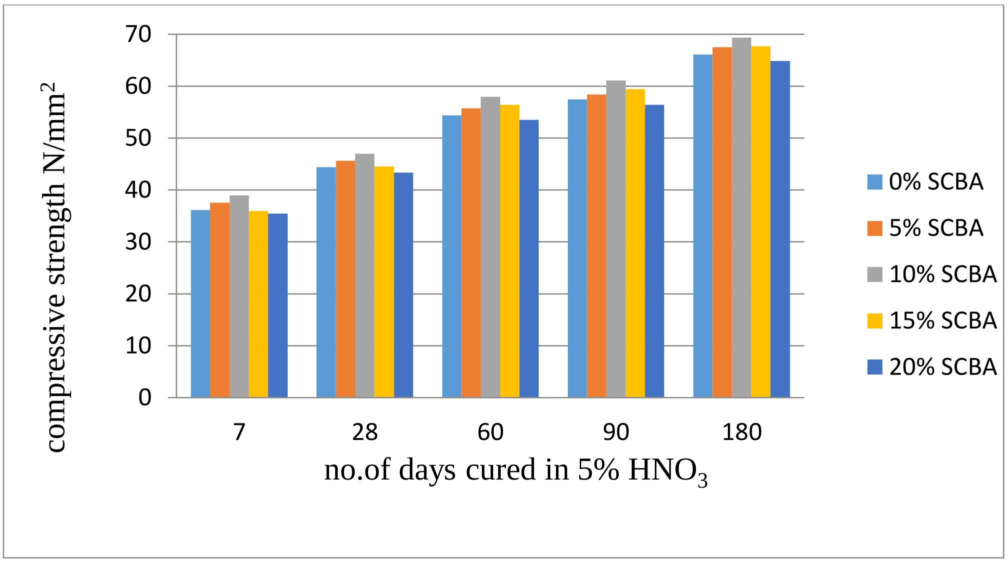
3. Compressive Strength Results of cubes cured in 2% HNO3



4. Compressive Strength Results of cubes cured in 3% HNO_3



5. Compressive Strength Results of cubes cured in 4% HNO_3



6. Compressive Strength Results of cubes cured in 5% HNO3

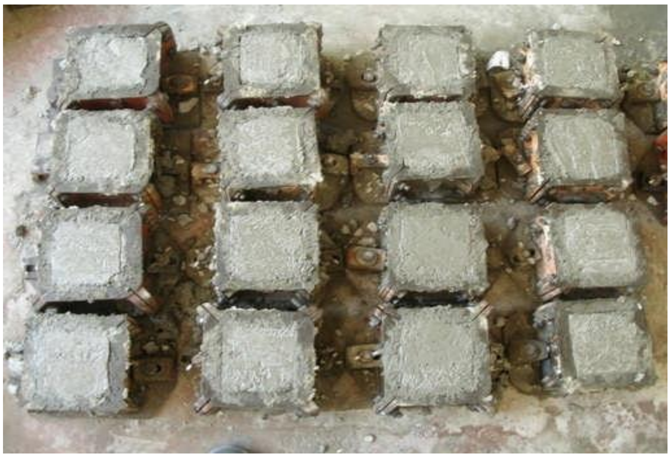


Fig 1: Casted cubes of M35 SCBA Concrete of 10% replacement

Fig 8: M35 SCBA Concrete of 10% replacement after acid attack

CONCLUSIONS

From the experimental studies following conclusions are drawn:

1. The specific surface area OF SCBA is 420 m^2/kg greater than 330 m^2/kg of cement. The workability of SCBA concretes have decreased in compared with ordinary concrete. It is inferred that reduction in workability is due to large surface area of SCBA.
2. The compressive strengths of concrete (with 0%, 5%, 10%, 15% and 20%, weight replacement of cement with SCBA cured in seawater & in 1%,2%,3%,4%,5% HNO3 solution for 7, 28, 60 ,90 and 180 days) have reached the target mean strength.
3. The compressive strengths of concrete (with 0%, 5%, 10%, 15% and 20%, weight replacement of cement with SCBA cured in seawater & in 1%, 2%, 3%, 4%, 5% HNO3 solution for 7, 28, 60, 90 and180 days), indicate that at 5% replacement there is decrease in strength, increased at 10% and 15% replacement but at 20% replacement loss in strength nearly equal to normal concrete.
4. Due to slow pozzolanic reaction the Sugar Cane Bagasse Ash (SCBA) concrete achieves significant improvement in its mechanical properties at later ages.
5. In concretes cement can be replaced with 20% SCBA without sacrificing strength.

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AN EXPERIMENTAL STUDY ON BEHAVIOUR OF HIGH PERFORMANCE GLASS FIBRE REINFORCED BEAM WITH PARTIAL REPLACEMENT OF CEMENT BY SILICA FUME

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Abstract

This paper presents the experimental investigation conducted to study the behavior of flexure for beams produced from High Performance Concrete (HPC) with silica fume and glass fibre as a partial replacement of cement in concrete . In this investigation HPC was manufactured by usual ingredients such as cement, fine aggregate, coarse aggregate, water and mineral admixtures such as Silica Fume (SF) of various replacement levels and the Super Plasticizer CONPLAST-430 were used by adopting water cement ratio (w/c) of 0.26. The concrete used in this investigation was proportioned to target a mean strength of 80 MPa. Specimens such as cubes, cylinders and beams were cast and tested for various mixes. Seven mixes M1 to M7 are cast with 0%, 5%, 7.5% and 10% replacement of cement using SF and another set of specimens with 0%, 5%, 7.5% and 10% replacement of cement using SF along with 0.2% constant replacement of Glass fibre to study the mechanical properties such as compressive strength, split tensile strength, flexural strength, water absorption and modulus of elasticity. The results shows that for the optimum replacement of silica fume 7.5% more cohesive concrete were obtained which is less prone to segregation.

Keywords*: High Performance Concrete (HPC), Silica fume , Cem Fil Anti crack fibers*

1.0 Introduction

The development of concrete technology has been a gradual process over many years. With the advent of new admixtures and cementitious materials, it becomes possible to produce highly workable concrete with superior mechanical properties and durability. This newly developed concrete is being called High Performance Concrete (HPC). HPC has very high degree of durability, which is obtained by using supplementary cementitious materials like fly ash, silica fume and GGBS etc., to replace a certain percentage of OPC. The use of these replacing materials improves the properties of concrete, both in fresh and hardened stages.

1.1 High Performance Concrete (HPC)

According to ACI “High Performance Concrete is defined as concrete which meets special performance and uniformity requirements that cannot always be achieved routinely by using conventional materials and normal mixing, placing and curing practices”.

According to A.M. Neville, “HPC is concrete selected so as to fit for the purpose for which it is required. There is no mystery about it, no unusual ingredients are needed, and no special equipment has to be used. All we use is an understanding of the behavior of the concrete and will, to produce a concrete mix within closely controlled tolerances”.

The High performance concrete is used for concrete mixture, which possesses high strength, high modulus of elasticity, high density, high dimensional stability, low permeability and resistance to chemical attack. In normal concrete, relatively low strength and elastic modulus are the result of high heterogeneous nature of structure of the material, particularly the porous and week transition zone, which exists at the cement paste- aggregate interface. To produce

HPC it is generally essential to use mineral and chemical admixtures such as silica fume, fly ash, and superplasticizer.

1.2 Pozzolanic Action

This is a chemical mechanism, silica fume reacts with the calcium hydroxide, which is liberated during process of hydration, about 22-24 percent and produces calcium-silicate-hydrate (C-S-H). The following are the chemical reactions that are taking place.

Portland cement reaction: C_3+H=C-S-H+CH

Portland reaction of silica fume: S+CH+H = C-S-H

2. MATERIAL PROPERTIES

This chapter deals with the study and testing of various materials before using for the experiment.

2.1 Materials Used

- Cement: Ordinary Portland cement, 43 Grade conforming to IS: 12269 – 1987.
- Fine aggregate: Locally available river sand conforming to Grading zone II of IS: 383 –1970.
- Coarse aggregate: Locally available crushed blue granite stones conforming to graded aggregate of nominal size 20 mm as per IS: 383 – 1970.
- Silica fume: Obtained from ELKEM India (P) Ltd., Navi Mumbai conforming to ASTMC 1240 as mineral admixture in dry densified form.
- Super plasticizer: CONPLAST 430 was used as chemical admixture to enhance the workability of the concrete.
- Water: potable water.
- Cem Fil Anti crack fibres

2.2 Cement

To produce high performance concrete, the utilization of high strength cements is necessary. Different types of cement have different water requirements to produce pastes of standard consistence. Different types of cement also will produce concrete have a different rates of strength development. The choice of brand and type of cement is the most important to produce a good quality of concrete. The type of cement affects the rate of hydration, so that the strengths at early ages can be considerably influenced by the particular cement used.

2.3.1 Tests on Cement

Tests were conducted to find the specific gravity, consistency, setting time and compressive strength of OPC and the results are tabulated in Table 2.1. This table also compares the results obtained and the requirement as per IS: 8112-1989

Table 2.1 Properties of 43 grade OPC

Test Particulars	**Result Obtained**	**As per IS:8112-1989**
Specific gravity	3.15	3.10-3.15
Normal consistency (%)	31	30-35
Initial setting time (min)	33	30 minimum
Final setting time (min)	540	600 maximum

4 Aggregates

Aggregates generally occupy about 70 to 80% of the volume of concrete and combine with the binder (cement and pozzolana) and water to produce concrete. Therefore, it can be expected to have an important influence on its properties. The aggregates chosen should not affect the durability of the hardened concrete.

2.4.1 Tests on Aggregate Properties

Tests were conducted to obtain specific gravity and fineness modulus of the fine aggregate and coarse aggregate used in this study as per IS: 2386-1983 and the results are tabulated in Table 2.2 and Table 2.3

2.4.2 Fine Aggregate:

Sand is naturally occurring organic material composed of finely divided rock and mineral particles; it is defined by size, being finer than gravel and coarser than silt. Sand from river with a specific gravity ranging from 2.63-2.67 can use for the experiments.

Table 2.2 Properties of river sand

S. No.	**Properties**	**Values**
1	Specific Gravity	2.63-2.67
2	Zone	II

2.4.3 Coarse Aggregate:

Crushed stone from Tirunelveli district with a size of 20mm and normal continuous grading can be used. The content of flaky and elongated particle is less than 3%, the crushing index less then are equal to 6% specific gravity is 2.52. Coarse aggregate are usually those particles which are retained on an IS 4.75mm sieve.

Table 2.3 Properties of Coarse Aggregate

S. No.	**Properties**	**Values**
1	Specific Gravity	2.60
2	Fineness Modulus	7.88

2.5 Silica Fume

Silica fume which is also known as micro silica is a very fine pozzolanic material, composed of amorphous silica produced by electric arc furnaces as a byproduct of the production of elemental silicon or ferrosilicon alloys. Silica fume can be used in a variety of cementitious products such as concrete, grouts and mortars.
As the quartz is heated to 2000˚C and an electric arc is fired through the furnace, it releases silicon monoxide gas. This gas rises and reacts with oxygen in the upper parts of the furnace and condenses as it cools, into the pure spherical particles of micro silica.

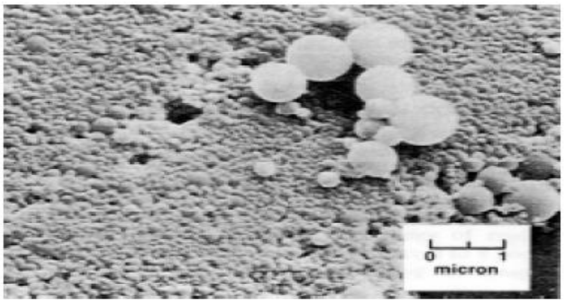


Fig 2.1 Micrograph from scanning electron microscope showing typical silica

Table 2.4 Chemical properties of silica fume

Constituent	**Percentage (%)**
SiO_2	90-96
Al_2O_3	0.5-0.8
MgO	0.5-1.5
Fe_2O_3	0.2-0.8
CaO	0.1-0.5
Na_2O	0.2-0.7
K_2O	0.4-1
C	0.5-1.4
S	0.1-0.4

Fig. 2.2 Silica Fume Sample

2.6. Glass Fibres

The use of fibres in concrete increases the mechanical properties such as compressive strength, tensile strength of concrete to some extent. Hence the flexural behaviour can be increased to some extent. It also possesses the ability to reduce plastic shrinkage in concrete.Cem-Fil Anti Crack Fibres used in this study is shown in Fig. 2.3

Fig. 2.3 Cem-Fil Anti Crack Fibres

2.6.1 Material Properties of Fibre

Density	- 2.6 t/m^3
Elastic modulus	- 73 GPA
Tensile strength	- 1700 MPa
Number of fibres	- 220 million / kg
Filament diameter	- 14µ
Specific gravity	- 2.6
Length	- 12mm
Aspect ratio	- 857:1
Specific surface area	- 105m^2/kg

3 RESULT AND DISCUSSIONS

3.1 Compressive Strength Test

Table:3.1 Compressive Strength Results

Mix	**% of SF**	**% of GF**	**3 Days MPa**	**7 Days MPa**	**28 Days MPa**
M1	0	0	40.33	53.67	69.33
M2	5	0	41.33	56.33	70.67
M3	**7.5**	**0**	**43.67**	**60.33**	**75.67**
M4	10	0	40.67	58.67	73.33
M5	5	0.2%	41.67	56.67	70.67
M6	**7.5**	**0.2%**	**45.33**	**62.67**	**77.33**
M7	10	0.2%	40.67	59.33	74.67

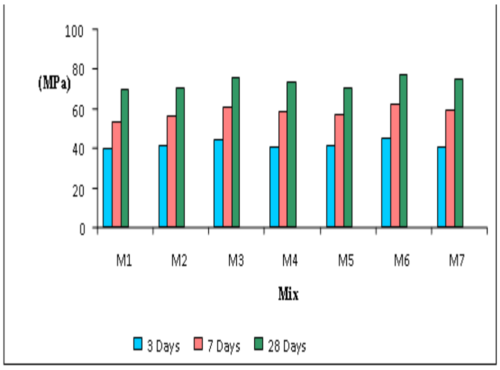


Fig.3.1 Variation of Compressive strength at various age

2.3 Split Tensile Strength Test

Table: 3.2 Split Tensile Strength Result

Mix	% of SF	% of GF	28 Days MPa	Theoretical Value MPa
M1	0	0	5.23	5.83
M2	5	0	5.78	5.88
M3	**7.5**	**0**	**5.97**	**6.09**
M4	10	0	5.84	5.99
M5	5	0.2%	6.12	5.88
M6	**7.5**	**0.2%**	**6.58**	**6.16**
M7	10	0.2%	6.35	6.05

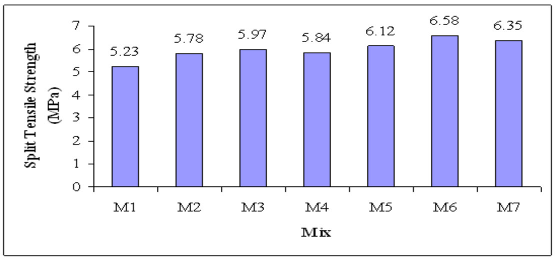


Fig .3.2 Split tensile strength of various mixes

3.3 Flexural Strength Test

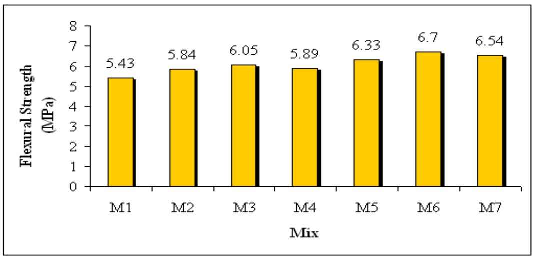


Fig.3.3 Flexural Strength of various mixes

Table 3.3 Flexural Strength Result

Mix	% of SF	% of GF	28 Days (MPa)	Theoretical Value (MPa)
M1	0	0	5.43	5.83
M2	5	0	5.84	5.88
M3	**7.5**	**0**	**6.05**	**6.09**
M4	10	0	5.89	5.99
M5	5	0.2	6.33	5.88
M6	**7.5**	**0.2**	**6.70**	**6.16**
M7	10	0.2	6.54	6.05

2.4 Water Absorption Test

Table 3.4 Water Absorption Test Result

Mix	% of SF	% of GF	Wet weight of cube (gms)	Dry weight of cube (gms)	Water Absorption (%) In 24 Hrs
M1	0	0	8625	8544	3.18
M2	5	0	8639	8561	3.05
M3	7.5	0	8645	8572	2.84
M4	10	0	8651	8583	2.63
M5	5	0.2%	8699	8622	2.94
M6	7.5	0.2%	8705	8635	2.66
M7	10	0.2%	8765	8698	2.48

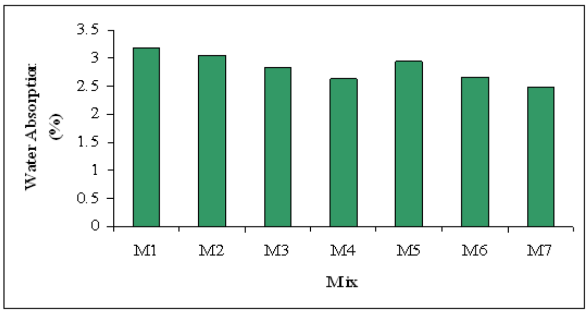


Fig.3.4 Water Absorption for various mixes

3.5 Modulus of Elasticity

In the case of concrete, since no part of the graph is straight, the modulus of elasticity is found out with reference to the tangent drawn to the curve at the origin. The modulus found from this tangent is referred as initial tangent modulus. This gives satisfactory results only at low stress value. For higher stress value it gives misleading picture.The modulus of elasticity is determined by subjecting a cube or cylinder specimen to axial compression and measuring the deformations by means of dial gauges fixed between certain gauge lengths.

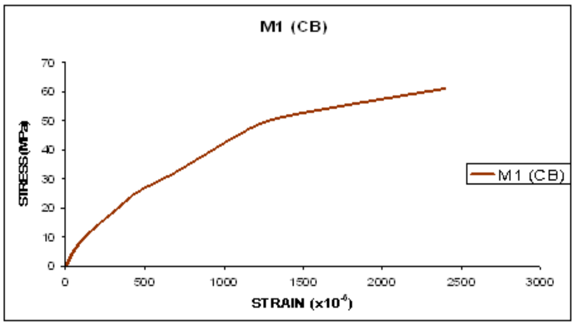
Modulus of elasticity = Stress / Strain N/mm^2

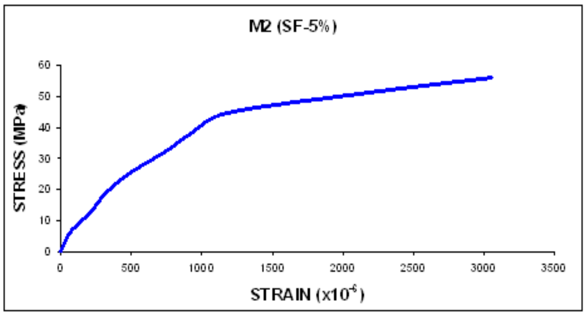
Table 3.5 Young's Modulus Test Result

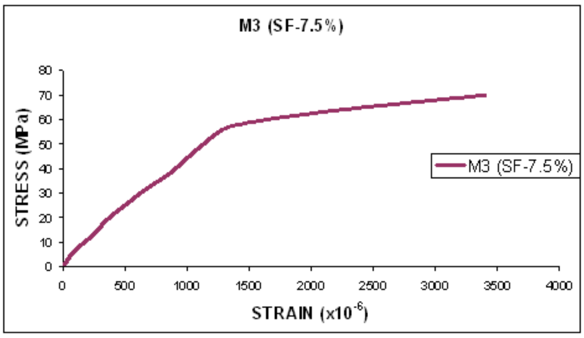
Mix	Stress N/mm^2	33% of Ultimate strength N/mm^2	Strain	Young's Modulus N/mm^2
M1	60.51	20.17	0.00048	42462.84
M2	57.32	19.11	0.00043	44960.65
M3	70.06	23.35	0.00045	51899.03
M4	63.69	21.23	0.00048	44697.73
M5	66.24	22.08	0.00053	42058.43
M6	71.34	23.78	0.00053	45293.7
M7	62.42	20.81	0.0005	41613.58

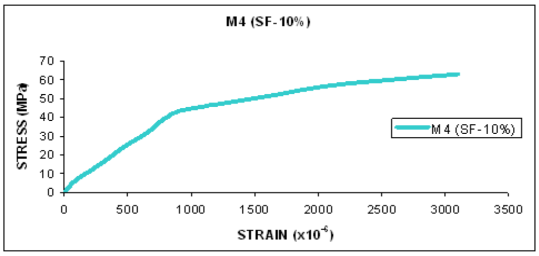
3.6 Stress-Strain Behavior

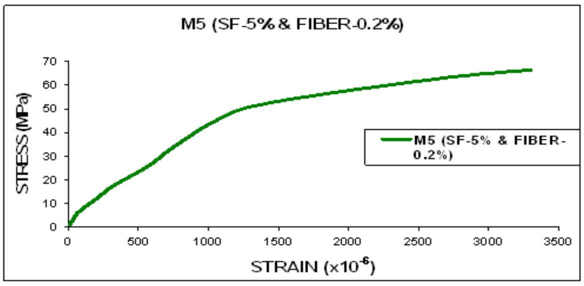
Tests were conducted to study stress-strain behavior of HPC on cylinder specimens of size 100x300mm for all the mixes in the compression testing machine of capacity 2000kN. These specimens were tested under axial compression and the values of axial stress and axial strain were recorded at regular intervals. The stress versus strain curves are shown for the mixes M1 to M7 in Fig3.5. The maximum stress and maximum strain values for various mixes are given in Table3.6.It is found that M3 (with 7.5% silica fume) shows higher strength and ultimate strain compared to normal concrete.

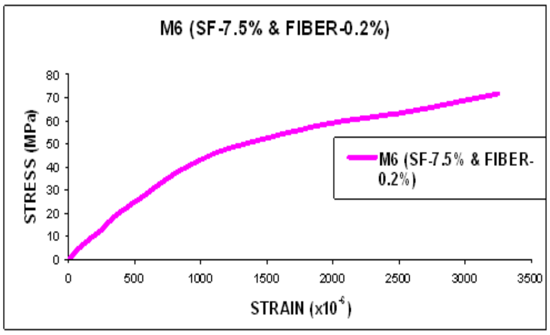

M1 (CB)
STRESS(MPa)
STRAIN (x10^-6)
M1 (CB)


M2 (SF-5%)
STRESS (MPa)
STRAIN (x10^-6)


M3 (SF-7.5%)
STRESS (MPa)
STRAIN (x10^-6)
M3 (SF-7.5%)


M4 (SF-10%)
STRESS (MPa)
STRAIN (x10^-6)
M4 (SF-10%)


M5 (SF-5% & FIBER-0.2%)
STRESS (MPa)
STRAIN (x10^-6)
M5 (SF-5% & FIBER-0.2%)



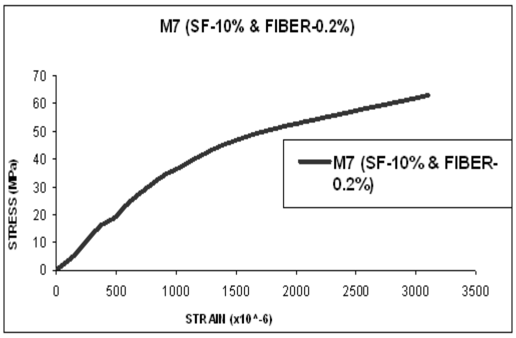


Fig. 3.5 Stress-Strain Curve for MixM1 -M7

Table 3.6 Maximum Stress Strain Values

Mix	**Stress (MPa)**	**Strain in Microns**
M1	60.51	2490
M2	57.32	3050
M3	70.06	3470
M4	63.69	3175
M5	66.24	3300
M6	71.34	3350
M7	62.42	3125

4. CONCLUSIONS

- The addition of silica fume results in a more cohesive concrete which results in a good concrete which is less prone to segregation.
- The M3 mix which is without fibre give maximum compressive strength of concrete without addition of fibre(ie) 75.67 MPa which is 9.14% greater than control mix.
- It is observed that M6 mix shows the maximum compressive strength. For M6 mix with 7.5% silica fume and 0.2 % glass fibre, the 28 days’ compressive strength is 77.33 MPa which is 11.54% greater than control mix.
- The M3 mix which is without fibre give maximum split tensile strength of concrete without addition of fibre (ie) 5.97 MPa which is 14.15% greater than control mix.
- The M6 mix which is with fibre give maximum tensile strength of concrete without addition of fibre (ie) 6.58 MPa which is 25.81% greater than control mix.

- The M3 mix which is without fibre give maximum flexural strength of concrete without addition of fibre (ie) 6.05 MPa which is 11.42% greater than control mix.
- The M6 mix which is with fibre give maximum flexural strength of concrete without addition of fibre (ie) 6.70 MPa which is 23.39% greater than control mix.

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OPTIMIZATION OF QUATERNARY BLENDED HIGH PERFORMACE CONCRETE MIX PROPORTIONING

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Abstract

High performance concrete can be made by partial substitution of cement by one or a combination of two or three supplementary cementitious materials (SCMs), when available at competitive prices, can be advantageous, not only from an economical point of view but also from a rheological and sometimes strength point of view. One of the main objectives of the present research work was to investigate synergistic action of binary, ternary and quaternary blended high strength grade (M80) concretes on its compressive strength. The present investigations are aimed to studythe effect of synergic action of Metakaolin(MK), Microsilica (MS) and fly ash (FA) in binary, ternary and quaternary blended high strength grade (M80) concretes on compressive strength. The findings of various trail mixes made with different combinations of metakaolin (MK), micro silica (MS) and fly ash (FA) as pozzolans, carried out in the laboratory to determine the optimum quantities of pozzolans (expressed in terms of percentages by weight of powder) to be used in the development of high performance blended high strength grade (M80) concretes.Based on the experimental observations, it can be concluded that when compared to ternary blended (MS+FA combination) high strength SCC mixes of M80 grade, the quaternary blended (MK+MS+FA) high strength SCC mixes are more advantageous from economical and sustainability point of view.

***Keywords**: High Performance Concrete, Metakaolin, Fly ash, Quaternary Blended Concrete, High Strength Concrete*

1.0 Introduction

In the present-day scenario to fulfill the demands of sustainable construction, concrete made with multi-blended cement system of Ordinary Portland Cement (OPC) and different mineral admixtures, is the sensible choice for the construction industry. One of the effective methods to conserve the Mother Nature's resources and also reduce the environmental impact is to use Supplementary Cementitious Materials (SCMs) by substituting OPC partly or totally in concrete. Since most of SCMs are pozzolanic in nature and hence they are helpful in increasing later strength of concrete. Blending of SCMs with cement has many advantages such as saving in cement, utilization of industrial by-products, enhancement of microstructural properties of concrete and reduces environmental impact through reduced greenhouse gases production. Most of the SCMs are industrial by-products which are considered as waste and pollutants when dumped into land or thrown into water bodies. So blending them in concrete becomes safe disposal method for them. Such SCMs are Fly ash (FA), Ground Granulated Blast furnace Slag (GGBS), Micro Silica(MS) or Silica Fume(SF), Copper slag (CS), Rice Husk Ash (RHA) etc.

2.0 Objectives of the Present work

The primary objectives of this paper is to quantitatively comprehend and assess the role of optimum metakaolin (MK) in development of strength in binary, ternary and quaternary blended concrete, made with optimal micro silica and fly ash, of High strength grade (M80)

3.0 Experimental Programme

To accomplish the defined objectives for the present work and to obtain specific experimental data which helps to understand the behaviour and significance of utilization of metakaolin in blended concretes, the total experimental programme was taken up in various phases.

They are framed into three phases:

PHASE 1: Determination of Physical and chemical Properties of materials used

The materials used in the experimental investigation are locally available cement, sand, coarse aggregate, mineral and chemical admixtures. The chemicals used in the present investigation are of commercial grade and properties are supplied by the manufacturer and are appropriately acknowledged.

- **Cement**

Ordinary Portland cement (OPC) of 53 grade [IS: 12269-1987,

- **Fine Aggregates (River Sand)**

The fine aggregate used was locally available river sand without any organic impurities and conforming to IS: 383 – 1970.

- **Coarse Aggregate**

The coarse aggregate chosen for blended concrete was typically round in shape, crushed granite metal of size of 20 mm and 10 mm graded obtained from the locally available quarries was used in the present investigation.

- **Water**

Water used for mixing and curing was potable water, which was free from any amounts of oils, acids, alkalis, sugar, salts and organic materials or other substances that may be deleterious to concrete or steel confirming to IS : 3025 – 1964 part22, part 23 and IS : 456 – 2000 [Code of practice for plain and reinforced concrete].

- **Fly Ash**

Fly ash used in this investigation was procured from Vijayawada Thermal Power Station, Andhra Pradesh, India. It confirms with grade I of IS: 3812 – 1981 [Specifications for flyash for use as pozzolana and admixture].

- **Micro silica (MS)**

Micro silica Grade 92D conforming to IS: 15388 -2003 is used. Silica fume has specific surface area of about 20,000m^2/kg.

- **Metakaolin (MK)**

Metakaolin obtained from KOAT manufacturing company, Vadodara, Gujarat has been used. Metakaolin is obtained by calcination of pure or refined kaolin clay at a temperature between 650^0C and 850^0C, followed by grinding to achieve a fineness of 15000 m2/kg (B.E.T).The specific gravity is found as 2.50.

PHASE 2: Determination of Quantities of materials

The mix proportioning was done based on the Erntroy and shaklock mix design approach for high strength grade (M80) of binary , ternary and quaternary blended concretes made with optimum combinations of fly ash (FA), microsilica (MS) and metakaolin(MK). Different trial mixes were attempted in the laboratory to get an optimum blended concrete mixes, which gives required strength property.

PHASE 3: Optimization of pozzolans quantities in blended concrete mixes

Several trial mixes are conducted on number of blended concrete mixes made with the different possible combinations of Fly Ash (FA), Microsilica (MS) and Metakaolin (MK) to develop various binary, ternary and quaternary blended concrete systems to determine the appropriate optimized quantities of Fly Ash (FA), Microsilica (MS) and Metakaolin (MK) on realization of desired strength.

3.0 Test Results and Discussions

The test results of experimental investigations carried out during the development of high strength grade (M80) binary, ternary and quaternary blended concrete mixes made with optimum proportions of fly ash (FA), microsilica (MS) and metakaolin (MK) combination are tabulated in the following sections.

3.1 Determination of Quantities of materials

Quantities required for 1 cu.m are evaluated for high strength grade (M80) binary, ternary and quaternary blended concretes made with optimum proportions of Fly Ash (FA), Microsilica (MS) and Metakaolin (MK) combination based on calculations from Erntroy and shacklock mix design method. Final quantities, for all blended concrete mixes considered, are assumed after several trial mixes on quantities computed.

Table 1– Quantities in kg per cu.m for high strength (M80) grade blended concrete obtained using Erntroy and shacklock Mix Design

	Cement	Fine Aggregate	Coarse Aggregate	Water
Quantity kg/m^3	700	644	966	150 L

The computed amount of OPC is 700 kg. But keeping in viewthe clause 8.2.4.2 of IS 456-2000, the maximum cement content is limited to 450 kg per cum of concrete. After trail mixes, revised quantities in kg per cu.m for high strength grade (M80) blended concrete mix are arrived without compromising the desired strength property.

The final quantities for high strength M80 grade blended concrete mix are tabulated in Table 2.

Table 2 – Final Quantities for trial mixes of high strength M80 grade blended concrete mix

	Cement	Total Pozzolana	Total Powder Content	Fine Aggregate	Coarse Aggregate	Water (water/powder =0.23)
Quantity kg/m^3	450	250	700	644	966	150L

Henceforth, the total amount of powder quantity (cement + pozzolanic mixture) adopted for high strength M80 concrete is 700 kg/m^3 and water/powder ratio is 0.23 for all blended high strength M80 concrete mixes.

3.2 Optimization of SCMs proportions in blended concrete mixes

This phase identifies the optimum proportions of fly ash, micro silica and metakaolin in binary, ternary and quaternary blended concrete mixes in order to obtain the enhanced performance of blended concrete at all ages. The details of the quantities of materials, replacement percentages and quantities (kg) of SCMs and OPC in total powder content and their corresponding fresh properties are shown in Table 3 to Table 5 respectively for high strength grade (M80) of binary, ternary and quaternary blended concrete made with optimum proportions of Fly Ash (FA), Microsilica (MS) and Metakaolin (MK) combination. Table 3 gives base quantities of high strength grade (M80) blended concrete mix derived after several trial mixes on the quantities calculated using Erntroy and shacklock mix design method. It can be observed that the total powder content is 700 kg/m3 with cement content restricted to 450 kg/m3 from durability of concrete point of view and rest of the powder is fly ash (250 kg/m3). Depending on the above calculated base quantities for high strength grade (M80), twenty nine (29) blended concrete mixes were designed in three groups of binary, ternary and quaternary.

Table 3 – Base Quantities for high strength M80 grade concrete mix

	Cement	Total Pozzolana	Total Powder Content	Fine Aggregate	Coarse Aggregate	Water (water/powder =0.23)
		Fly ash				
Quantity kg/m^3	450	250	700	644	966	150L

Table 4 shows various blended high strength grade (M80) blended concretemixtures made with different proportions of Fly Ash (FA), Microsilica (MS) and Metakaolin (MK). In Mix designation, number indicates percentage by weight of total powder content. Various binary, ternary and quaternary blended concrete mixes were prepared with different proportions of Fly Ash (FA), Microsilica (MS) and Metakaolin (MK). (B1 to B8, T1 to T8 and Q1 to Q12). Mix numbers B1 to B8 are binary blended concrete mixtures made of either fly ash (FA) or microsilica (MS) or metakaolin (MK) while Mix numbers T1 to T8 are ternary blended fly ash based concrete mixtures made of microsilica (MS) or metakaolin (MK) and Mix numbers Q1 to Q12 are quaternary blended fly ash based concrete mixtures made of microsilica (MS) and metakaolin (MK) combination.

In high strength grade (M80) concrete mix 'C1' developed with 100% OPC does not yield desired strength. So using 100% OPC in development of high strength grade (M80) concrete mix is completely ruled out. In binary blended high strength grade (M80) concrete mixtures, percentage replacement of fly ash by weight of total powder content is 35% i.e. 250 kg/m3 (B1) which is based on preliminary calculation from mix design. For the mix proportion C65+FA35, desired strength is not realized. For binary blended concrete mixtures made with percentage replacement of either micro silica (MS) or metakaolin (MK) or both combined, micro silica (MS) and metakaolin (MK) are limited to 5-15% and 5-20% respectively.

In ternary blended micro silica (MS) and fly ash (FA) blended high strength grade (M80) concrete mixtures (T1 to T4) percentage replacement of micro silica (MS) is limited to 5 -20%

by weight of total powder content. Similarly in ternary blended metakaolin (MK) and fly ash (FA) blended high strength grade (M80) concrete mixtures (T5 to T8), percentage replacement of metakaolin (MK) is limited to 5 -20% by weight of total powder content. In both the above ternary blended MS+FA blended concrete and MK+FA blended concrete mixtures (T1 to T8), the cement content is kept constant (65% by weight of total powder content).

In binary blended high strength grade (M80), for fly ash (FA) blended concrete mix (B1) and metakaolin (MK) blended concrete mixes (B5 to B8) desired strength is not realized. But in binary blended micro silica (MS) blended concrete mix, desired strength is attained, if the MS percentage replacement is limited to 5-10% by weight of powder. The optimal mix chosen for binary blended micro silica (MS) basedconcrete mix is 5% MS replacement (B2). Henceforth, for high strength grade (M80) mixes, Mix OPC95+MS5 (B2) is taken as reference mix.

In ternary blended metakaolin (MK) and fly ash (FA) blended high strength grade (M80) concrete mixtures (T5 to T8), desired strengths are not obtained for any of the mixes. But for micro silica (MS) and fly ash (FA) blended ternary blended concrete mixes (T1 toT4), up to 15% MS by weight of powder, desired strengths are attained satisfactorily. So C65+FA20+MS15 (T3) concrete mix is considered optimal in ternary blended high strength grade (M80) concrete mixes.

In quaternary blended high strength grade (M80) concrete mixtures (Q1 to Q12) made of microsilica (MS) and metakaolin (MK) combination, keeping cement content constant (65% by weight of total powder content), microsilica (MS) and metakaolin (MK) proportions are limited to 7 – 14%. For quaternary blended concrete mix (Q1), initially 7% MS and 7% MK replacements are assumed, keeping cement content constant i.e. 65% by weight of total powder content and rest of powder is fly ash, and required workability is not satisfied. So microsilica (MS) and metakaolin (MK) are gradually increased to 14% each yet workability is not achieved. Then author proposed to additionally increase fly ash content incrementally by 10% by weight of powder content (700 kg/m3), thereby incrementally increasing the powder quantity by 70 kg. With addition of 30% of fly ash (FA) to the C65+FA7+MS14+MK14 concrete mix (Q11), required workability and strength properties are achieved. So for quaternary blended concrete mix, the optimum combination of cement and pozzolanic mixture is revised as C50+FA28+MS11+MK11 concrete mix where final total powder content is 910 kg/m3 in which cement content is 455 kg/m3 and pozzolanic mixture is 455 kg/m3.

Table 4 presents several possible binary, ternary and quaternary blended high strength grade (M80) concrete mixes with the quantities of pozzolanic mixtures, their flow properties and achieved strengths. From this table, three optimally blended concrete mixes are selected.

From the experimental investigations, the mixes B2, T3 and Q11 are chosen as optimum binary, ternary and quaternary blended high strength grade (M80) concrete mixes made with fly ash (FA), microsilica (MS) and metakaolin (MK) where both desired workability and strength properties are met along with optimal usage of pozzolanic quantities. The following are mix designations of optimum combinations of binary, ternary and quaternary blended high strength grade (M80) desired mixes:

(1) C95+MS5 [B2]
(2) C65+FA20+MS15 [T3]
(3) C50+FA28+MS11+MK11 [Q11]

Table 4– Trail mixes of high strength grade (M80) blended concretemixes

Mix No.	Mix Designation (Values indicate percentage by weight of 'P'	Replacement % (bwp)*				Additional % of FA bwp*	Quantities kg per cu.m				Total Powder Content 'P'	Slump mm	Achieved Strength (MPa)
		OPC	FA	MS	MK		OPC	FA	MS	MK			
C1	C100	100	-	-	-	-	700	0	-	-	700	50	72.35
B1	C65+FA35	65	35	-	-	-	450	250	-	-	700	66	58.94
B2	C95+MS5	95	-	5	-	-	665	-	35	-	700	50	108.56
B3	C90+MS10	90	-	10	-	-	630	-	70	-	700	31	106.04
B4	C85+MS15	85	-	15	-	-	595	-	105	-	700	18	88.32
B5	C95+MK5	95	-	-	5	-	665	-	-	35	700	54	72,15
B6	C90+MK10	90	-	-	10	-	630	-	-	70	700	46	75.78
B7	C85+MK15	85	-	-	15	-	595	-	-	105	700	37	78. 82
B8	C80+MK20	80	-	-	20	-	560	-	-	140	700	48	69.35
T1	C65+FA30+MS5	65	30	5	-	-	455	210	35	-	700	55	81.23
T2	C65+FA25+MS10	65	25	10	-	-	455	175	70	-	700	55	94.20
T3	C65+FA20+MS15	65	20	15	-	-	455	140	105	-	700	55	100.54
T4	C65+FA15+MS20	65	15	20	-	-	455	105	140	-	700	46	78.91
T5	C65+FA30+MK5	65	30	-	5	-	455	210	-	35	700	54	76.23
T6	C65+FA25+MK10	65	25	-	10	-	455	175	-	70	700	52	77.34
T7	C65+FA20+MK15	65	20	-	15	-	455	140	-	105	700	51	78.12
T8	C65+FA15+MK20	65	15	-	20	-	455	105	-	140	700	43	67.21
Q1	C65+FA21+MS7+MK7	65	21	7	7	-	455	147	49	49	700	44	90.88
Q2	C60+FA28+MS6+MK6	65	21	7	7	10	455	217	49	49	770	54	82.34
Q3	C54+FA34+MS6+MK6	65	21	7	7	20	455	287	49	49	840	55	72.17
Q4	C65+FA14+MS14+MK7	65	14	14	7	-	455	98	98	49	700	41	80.16
Q5	C59+FA22+MS13+MK6	65	14	14	7	10	455	168	98	49	770	54	81.23
Q6	C54+FA28+MS12+MK6	65	14	14	7	20	455	238	98	49	840	54	83.65
Q7	C50+FA34+MS11+MK5	65	14	14	7	30	455	308	98	49	910	55	71.37
Q8	C65+FA7+MS14+MK14	65	7	14	14	-	455	49	98	98	700	33	90.94
Q9	C58+FA16+MS13+MK13	65	7	14	14	10	455	119	98	98	770	52	93.25
Q10	C53+FA23+MS12+MK12	65	7	14	14	20	455	189	98	98	840	54	94.72
Q11	C50+FA28+MS11+MK11	65	7	14	14	30	455	259	98	98	910	55	110.71
Q12	C46+FA34+MS10+MK10	65	7	14	14	40	455	329	98	98	980	56	79.91

Numbers in the above mix designations indicate percentage by weight of total powder content. Total powder content for binary, ternary is 700 kg/m3 and while for quaternary blended high strength grade (M80) it is 910 kg/m3. Thus, by incorporating metakaolin (MK) into micro silica (MS) and fly ash (FA) blended ternary blended desired mixes, the amount of fly ash used has almost doubled to achieve the requisite workability and therefore desired strength. From this observation, it can be understood that micro silica (MS) in blended desired mixtures imparts high strength but flow properties are marginally satisfied while metakaolin (MK) inclusion enhances the usage of high quantity of fly ash in blended concrete mixes for superior rate of gain of strength and more importantly for improved workability of concrete mix. The quaternary blended fly ash based concrete mix made of microsilica (MS) and metakaolin (MK) combination is found to be superior to ternary blended fly ash based concrete mix made with either microsilica (MS) or metakaolin (MK) due to reasons that for similar strength, better early strength, enhanced rate of gain of strength, improved flow properties and more use of fly ash quantity in developing blended high strength grade (M80) concrete.

In high strength grade (M80), three optimally blended binary and ternary concrete mixes (C95+MS5, C65+FA20+MS15, C50+FA28+MS11+MK11) are chosen based on desired compressive strength achievement. From the studies, it is observed that without inclusion of micro silica (MS), desired high strength cannot be attained. Further investigations have showed that metakaolin (MK) based quaternary blended high strength concrete mix yield better performance than ternary and binary blends in terms of (1) usage of high quantity of fly ash, (2) enhanced fresh properties and (3) reduction in quantity of cement used.

Table 5 presents the summary of all the optimal quantities of binary, ternary and quaternary blended M80 grade concrete mixes. The table also displays the replacement percentages of SCMs, total powder content in kg and water/powder ratios along with corresponding mix numbers and mix designations

From Table 5 , the total powder content for binary and ternary blended concrete mixes of high strength grade (M80) the total powder content adopted is 700 kg/m3 and whereas for quaternary blended concrete mixes of M80 grade, the total powder content adopted is 910 (additionally 30% of FA is added). It can be concluded that quaternary blended concrete mixes are more efficient that ternary blended concrete mixes for high strength grade (M80).

Based on the compressive strength attained at specified age of curing, the efficacy of pozzolans are understood. In this study, pozzolans used for blended concrete mixes are Fly Ash (FA), Microsilica (MS) and Metakaolin (MK). Age of curing specified for Fly Ash (FA) blended binary, ternary and quaternary blended concrete mixes of various grades is 60 days while it is 28 days for Microsilica (MS) and Metakaolin (MK) blended concrete mixes.

Metakaolin (MK) blended concrete mixes will set relatively quickly due to its high reactivity, which also prevents bleeding and settling of aggregates.In fresh state, tensile strengths have increased rapidly in order to prevent any internal stresses caused by drying shrinkage preventing cracks in the younger concrete. Metakaolin (MK) when compared to micro silica (MS) has similar particle density and surface area but different morphology and surface chemistry. Because of its hydrophilic surface, Metakaolin (MK) is easier to disperse into wet concrete. Metakaolin (MK) can be incorporated at any stage of concrete production; it should be mixed thoroughly to achieve even distribution; intensive mixing is not necessary like micro silica (MS) based concrete.Metakaolin (MK) concrete normally requires smaller super plasticizer dose than that required for the equivalent micro silica (MS) concrete. With no super plasticizer, it may be required to increase the water/binder ratio in order to maintain workability. This is partly due to fact that Metakaolin (MK) has a lower density than cement so that replacing, say, 10 mass % cement by Metakaolin (MK) decreases the water/binder volume ratio and the slurry rheology which is determined by the liquid/solid volume ratio.

The workability of fly ash based concrete mixes increases significantly with increase in fly ash content. For fly ash contents above 10% in concrete mixes workability falls. The reduction in workability is attributed to flocculation/coagulation at low fly ash concentration and the increase in workability at high concentration is attributed to neutralization of positive charges on cement particles and their resultant dispersal. When super plasticizer is used as a dispersing agent, no fall in workability is observed.Loss of workability due to the present of Metakaolin (MK) can be compensated for by the incorporation of fly ash (FA). The degree of restoration of workability, provided by fly ash, is influenced significantly by the cement replacement level, the MK/FA ratio and the W/b ratio. The addition of metakaolin increases the viscosity of blended concrete mixtures.

Fly ash addition reduces heat of hydration and slows the strength attainment at early stages. To overcome this shortcoming, metakaolin is added to fly ash based concretemixes to offset the delayed early strength attainment. Also incorporation of metakaolin (MK) to fly ash (FA) based concrete, enables to consume more amount of fly ash. With increased amount of pozzolanic content due to inclusion of metakaolin (MK) in fly ash (FA) based concretein turn reduces w/p ratio resulting in the increase of strength and flow properties.

Since metakaolin (MK) is cheaper than micro silica (MS), for same strength criteria, metakaolin (MK) blended fly ash (FA) basedconcrete is better in performance and economically viable than micro silica (MS) blended fly ash (FA) based M80 gradeconcrete.

Micro silica (MS) and metakaolin (MK) in blended fly ash (FA) basedconcrete mixes will increase the strength of concrete largely because it increases the strength of the bond between the cement paste and the aggregate particles. Addition of metakaolin (MK) to blended concrete mixes will enhance early hydration because of it high reactivity due to its glassy nature. The rate of pozzolanic reaction and calcium hydroxide (CH) consumption in metakaolin (MK) blended concrete mixes is higher than micro silica (MS) blended concrete mixes indicating high initial reactivity of metakaolin. Usage of metakaolin not only improves the workability but also makes the concrete microstructure denser. Thus with the inclusion of metakaolin in to concretemixes, super plasticizer dosage can be reduced noticeably so is the costs involved. So water demand is less for metakaolin blended concretemixes than micro silica (MS) blended concretemixes. It can be quantified that strength improvement in the metakaolin based concrete mixes is due to changes in the structure of the interfacial zone and the increased paste-aggregate bond strength. Also the dissolved Ca_{2+}, SiO_4^{4-} and OH^- ions readily in concrete combine with the metakaolin to give cementitious phases with a modified morphology.

Strength loss in the early ages, which was proportional to the cement replacement level, was probably due to the dilution effect of the pozzolan and as well as the slow nature of the pozzolanic reaction.

In blended concrete mixes, fly ash (FA) based mixes are assumed to yield desired strength at 60 days where as for micro silica (MS) or metakaolin (MK) or combination based concrete mixes desired strengths at 28 days are considered for assessment.

Metakaolin (MK) blended concrete mixes attain much higher early strength when compared to other concrete mixes while fly ash (FA) based concrete mixes are accomplishing strengths at later age. Nowadays usage of fly ash (FA) in concrete has become almost mandatory because (1) it enhances fluidity of concrete and (2) it is major part of powder content. So Metakaolin (MK) and fly ash (FA) concrete blends derive both the benefits of fly ash (FA) and Metakaolin (MK) in concrete by attaining early and later strengths consistently. So the rate of gain of strength attainment is steady in Metakaolin (MK) and fly ash (FA) blended concrete mixes. Metakaolin (MK) based ternary blended concrete mixes exhibit better performance than Metakaolin (MK) based binary blended concrete mixes due to the synergic action of blended pozzolans.

Table 5 - Final optimized mix proportions of blended concrete mixes

Grade of concrete Mix	**Mix No**	**Mix Designation (Values indicate percentage by weight of 'P'**	**Replacement % (bwp)***				**Additional % of FA bwp***	**Quantities kg per cu.m**								
			OPC	FA	MS	MK		OPC (i)	FA (ii)	MS (iii)	MK (iv)	Total Powder Content 'P' kg (i)+(ii)+(iii)+(iv)	Fine Aggregate	Coarse Aggregate	Water	W/P ratio
M80	B2	C95+MS5	95	-	5	-	-	665	-	35	-	700	644	966	150	0.23
	T3	C65+FA20+MS15	65	20	15	-	-	455	140	105	-	700	644	966	150	0.23
	Q11	C50+FA28+MS11+MK11	65	7	14	14	30	455	259	98	98	910	644	658	150	0.23

bwp* – By weight of Total Powder Content
W/P ratio – Water/Powder Ratio

Metakaolin cementing reaction rate is very rapid, significantly increasing compressive strength before first three days, which can have various benefits in fast paced construction industry.

Use of micro silica (MS) in high strength grade (M80) also acts as a micro-filler in making the concrete dense. Therefore it is established that micro silica (MS) is required for the development of high strength concrete. But metakaolin (MK) being less expensive and give better flow ability than micro silica (MS) is much preferred for the development of concrete mixes. Use of metakaolin accelerated the initial set time of concrete; however the final set time remained unchanged. This was caused mainly by the higher reactivity of the blended binder with metakaolin.

The addition of micro silica (MS) or metakaolin (MK) is advantageous in concrete mixes because in general, the strength at the transition zone between cement paste and coarse aggregate particles is lower than that of the bulk cement paste. The transition zone contains more voids because of the accumulation of bleed water underneath the aggregate particles and the difficulty of packing solid particles near a surface. Relatively more calcium hydroxide (CH) forms in this region than elsewhere. Without micro silica (MS) or metakaolin (MK), the calcium hydroxide (CH) crystals grow large and tend to be strongly oriented parallel to the aggregate particle surface. CH is weaker than calcium silicate hydrate (C-S-H), and when the crystals are large and strongly oriented parallel to the aggregate surface, they are easily cleaved. A weak transition zone results from the combination of high void content and large, strongly oriented CH crystals. Micro silica (MS) and metakaolin (MK) in blended concrete mixes will increase the strength of concrete largely because it increases the strength of the bond between the cement paste and the aggregate particles. The presence of micro silica (MS) and metakaolin (MK) in fresh concrete mix generally results in reduced bleeding and greater cohesiveness.

Metakaolin (MK) may be better alternative to micro silica (MS) especially in normal grades but in higher grades of concrete, its combination with micro silica (MS) will yield better performance.

Metakaolin (MK) is highly reactive alumino silicate whereas micro silica (MS) is reactive silicate so Metakaolin (MK) supplemented concrete mixes have high strengths because silica and alumina present in Metakaolin (MK) reacts with CH forms CSH (pozzolanic reaction) and CAH (aluminate hydration) respectively which contributes to additional strength than micro silica (MS). Fe_2O_3 in more in Metakaolin (MK) leading to high enhancement of strength in the blended concrete mixes due to the rapid consuming of $Ca(OH)_2$ which was formed during hydration of Portland cement specially at early ages related to the high reactivity of Fe_2O_3 particles. As a consequence, the hydration of cement is accelerated and larger volumes of reaction products are formed. Also Fe_2O_3 particles recover the particle packing density of the blended cement, directing to a reduced volume of larger pores in the cement paste.

Micro silica (MS) beyond 15% and Metakaolin (MK) beyond 20-25% should not exceed to preserve residual free CH in the paste to maintain pH of pore solution. High dosages of Metakaolin (MK) or micro silica (MS) in concrete mixes may lead to lower C/S ratio of the CSH gel formed from pozzolanic reaction resulting in high shrinkage of the formed gel.

Metakaolin (MK) blended fresh concrete mixes will set relatively quickly due Metakaolin (MK) when compared to micro silica (MS) has similar particle density and surface area but different morphology and surface chemistry.

Cement average size is 100 times larger than micro silica (MS) while Metakaolin (MK) average size is 20 to 30 times larger than micro silica (MS) so in Metakaolin (MK) + micro silica (MS) blended FA based quaternary concrete mix, coarser pores in fly ash (FA) based concrete can be reduced by inclusion of Metakaolin (MK) and finer pores are filled up by micro silica (MS).

So quaternary blended concrete mix made with micro silica (MS) and Metakaolin (MK) has improved microstructure which is dense and impermeable. Metakaolin (MK) blended concrete mixes improves workability, finishability, reduces surface dehydration and plastic cracking. Since Metakaolin (MK) does not increase the compressive strength of cement paste directly, it is concluded that strength improvement in the concrete is due to changes in the structure of the interfacial zone and the increased paste-aggregate bond strength. For the same reasons, Metakaolin (MK) has a beneficial effect on flexural strength, although the magnitude of the effect is less than that observed for compressive strength.

The cost of metakaolin is about three times the cost of ordinary Portland cement, thus using metakaolin alone as a supplementary cementitious material (SCM) may not be cost effective. On the other hand, the slow reaction rate of fly ash can make its use impractical when rapid early strength development is required. However, use of these materials in combination has the potential to overcome the higher cost associated with metakaolin concrete and the slower strength development associated with fly ash concrete.

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The cost of metakaolin is about three times the cost of ordinary Portland cement, thus using metakaolin alone as a supplementary cementitious material (SCM) may not be cost effective. On the other hand, the slow reaction rate of fly ash can make its use impractical when rapid early strength development is required. However, use of these materials in combination has the potential to overcome the higher cost associated with metakaolin concrete and the slower strength development associated with fly ash concrete.

4.0 Conclusions

Based on the systematic and detailed experimental study conducted on high strength grade (M80) of binary , ternary and quaternary blended Concrete mixes made with fly ash (FA), microsilica (MS) and metakaolin (MK) with an aim to develop high performance concrete mixes, the following are the conclusions arrived.

1. Metakaolin blended binary, ternary and quaternary concrete mixes attain early strengths due to its inherent faster reacting capability than microsilica (MS) blended concrete mixes.
2. For development of high strength concrete mixes (M80), use of micro silica is compulsory due to its inherent high reactive property and micro-filler capacity.
3. In development of high strength (M80) grade fly ash blended concrete mixes, both metakaolin and micro silica are required to be added to leverage the benefits of micro-filler capacity of micro silica and early strength attainment of metakaolin. Addition of metakaolin (MK) to blended concrete mixes will enhance early hydration because of it high reactivity.
4. Optimally blended high strength grades M80 quaternary concrete mixes made of 50%OPC+28%FA+11%MS+11%MKyields both required workability and desired compressive strengths. From this observation, it can be understood that micro silica (MS) in blended concrete mixtures imparts high strength while metakaolin (MK) inclusion enhances the usage of high quantity of fly ash in concrete mixes for superior rate of gain of strength. So it is evident that both metakaolin and micro silica are required in blended concrete mixes made with low water/powder ratio.
5. From the above observations it can be assumed for better flow and strength realization, in high strength grades (M80) blended fly ash based concrete mixes, the optimum percentage use of metakaolin is found to be 11%.
6. Metakaolin (MK) is highly reactive alumina silicate whereas micro silica (MS) is reactive silicate. Hence Metakaolin (MK) supplemented concrete mixes have high strengths at all ages because silica and alumina present in Metakaolin (MK) reacts with CH forms CSH (pozzolanic reaction) and CAH (aluminate hydration) respectively which contributes to additional strength than micro silica (MS). So quaternary blended concrete mix made with micro silica (MS) and Metakaolin (MK) has improved microstructure which is dense and impermeable.

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CASE STUDY ON GLASS FIBRE REINFORCED CONCRETE

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Abstract

Concrete being one of the majorly used construction materials, continuous efforts have been made to further improve its properties to meet the demands of the modern construction world. Concrete is a best material to handle compressive loads, it is very well known for its stiffness and durability properties but it is also brittle and is found to be very weak in tension, so efforts have been made in reinforcing concrete over the days in order to improve its tensile strength. Glass Fiber is a material consisting of numerous extremely fine fibers of glass. These fibres when added in certain percentage to the concrete are found to improve the strain properties well as crack resistance, ductility, flexure strength and toughness Glass fibers have high tensile strength and fire resistant properties thus reducing the loss of damage during fire accidents. The main aim of the study is to study the effect of glass fibre in the concrete and the properties like compressive strength, flexure strength, split tensile strength.

Keywords*: glass fibres, strength, durability*

1.0 Introduction

Concrete is one of the revolutionary building materials made by man, its ability to form a hard matrix with a binding property enabled engineers to tackle challenging designs. To improve its tensile strength reinforcement materials are used. Steel is one of the most widely used reinforcement material, it is an excellent material due to its ductility but it is affected by corrosion. The search for other materials which could overcome this limitation lead to the glass fibers. Glass fibre reinforced concrete or GFRC is a type of fiber-reinforced concrete. It is a material made of a cementatious matrix composed of cement, sand, water, and admixtures, in which short length glass fibers are dispersed. The effect of the fibers in this composite leads to an increase in the tension and impact strength of the material. They also have high tensile strength and fire resistant properties thus reducing the loss of damage during fire accidents. GFRC has been widely used in construction industry for non structural elements, like façade panels, piping, and channels.

Manufacturing process of glass fibers

Glass fibers are produced in a process in which molten glass is drawn in the form of filaments, through the bottom of a heated platinum tank or bushing. Usually, 204 filaments are drawn simultaneously and they solidify while cooling outside the heated tank; they are then collected on a drum into a strand consisting of the 204 filaments. Prior to winding, the filaments are coated with a sizing which protects the filaments against weather and abrasion effects, as well as binding them together in the strand.

Composition of GFRC

- Cement.
- Coarse aggregate the crushed aggregates used were maximum size of aggregate 20mm and minimum size of aggregate 12mm.

- Fine aggregate River bed sand was used as fine aggregate.
- Glass fibre (12mm, 15mm, 20mm in length) to provide tensile and flexural strength.

Types of Glass Fibres

A-glass: it is mainly used in the manufacture of process equipment.
C-glass: This kind of glass shows better resistance to chemical impact.
E-glass: This kind of glass combines the characteristics of C-glass with very good insulation to electricity.
AE-glass: Alkali resistant glass. Generally glass consists of quartz sand, sodium phosphate, soda, potash, feldspar and a number of refining and dying additives.

- Polymers in some cases to increase toughness.
- Plasticizers to enhance workability if necessary.

Mixing procedure

Mixing Procedure of Pre Mix GRC: The sand and cement are mixed dry and then the water/admixture and polymer (if used) are added. Generally a twospeed slurry/fiber blender mixer is used. With this type of mixer, the fast speed is designed to create smooth creamy slurry. This takes about one-two minutes. The mixer is switched to slow speed and fiber in the form of chopped strand (length approximately 13 mm) is added slowly. The fibre is blended into the mix for approximately 1 min. Once the mix is ready, it is poured into the moulds, which are vibrating using a vibrating table.

Casting procedure

After mixing in a pan mixer, the mix was cast in moulds for each % of fiber sufficient no of cubes and flexure beams were cast for testing at the ages of 28 days. The GFRC mix is left into the mould to set and is covered with polythene sheet to prevent moisture loss. The product is demoulded the next day and kept for curing.

Tests

- **Workability:** The workability tests are performed using standard sizes of Slump Moulds as per IS: 1199 - 1999 and Compaction Factor apparatus which was developed in UK and is described in IS: 1199 - 1999.
- **Compressive Strength:** The Steel mould of size 150 x 150 x 150 mm is well tighten and oiled thoroughly. They are allowed for curing in a curing tank for 28 days and they are tested in 200-tonnes electro hydraulic closed loop machine. The test procedures used are as per IS: 516-1979.
- **Flexural Strength:** The Steel mould of size 100 x 100 x 500 mm is well tighten and oiled thoroughly. They are allowed for curing in a curing tank for 28 days and they are tested in universal testing machine. The test procedures used are as per IS 516-1979.
- **Split Tensile Strength:** The specimens shall be cylinder with 150 mm in diameter and 300 mm long and is well tighten and oiled thoroughly. They are allowed for curing in a curing tank for 28 days and they are tested in universal testing machine. The test procedure used are as per IS 516-1979.

Applications of GFRC

- Architecture: Prefabricated architectural cladding, architectural moldings and Features, Environments & Landscaping.
- Building: Industrial and agricultural roofing, Walls and Windows, Renovation, Foundations and Floors, Modular Buildings.
- Engineering: Permanent Formwork, Utilities, Acoustics, Bridges and Tunnels, Water and Drainage.

Advantages of GFRC

- Light weight: With GFRC, concrete can be cast in thinner sections and is therefore as much as 75% lighter than similar pieces cast with traditional concrete. It is beneficial to use GFRC mix for countertops, a concrete countertop can be 1-inch thick with GFRC rather than 2 inches thick when using conventional steel reinforcement.
- High strength: GFRC can have flexural strength as high as 27.5N/mm2 and it has a very high strength-to-weight ratio.
- Reinforcement: Since GFRC is reinforced internally, there is no need for other kinds of reinforcement, which can be difficult to place into complex shapes.
- Consolidation: For sprayed GFRC, no vibration is needed. For poured, GFRC, vibration or rollers are easy to use to achieve consolidation.
- Equipment: Expensive equipment is not needed for poured or vibrated GFRC with a face coat, for sprayed GFRC, equipment generally costs about Rs. 50,000.
- Toughness: GFRC doesn't crack easily it can be cut without chipping.
- Surface finish: Because it is sprayed on, the surface has no bug holes or voids.
- Adaptability: Sprayed or poured into a mould can adopt
- Durability: It has been increased due to use of low alkaline cement and puzzolana.
- Sustainable: because it uses less cement than equivalent concrete and uses a recycled material.

Conclusions

- GFRC has more tensile strength when compared to steel reinforced concrete. The addition of glass fibers prevents crack formation in concrete. Unlike steel reinforcement, glass fibers are not corrodible, they are also resistant to fire, alkali attacks and hence GFRC results in a more durable structure.
- GFRC is mostly used for architectural purposes, the compatibility of glass fiber with concrete or mortar helps in using it easily in daily projects such as facade of buildings, as AR glass fibers have good resistance to alkalinity
- The fatigue performance of GFRC is observed to be better than plain concrete.
- Cement, when reinforced with glass fiber, produces precast elements much thinner and therefore lighter elements than that would be possible with steelreinforced precast concrete, where 30mm or more concrete cover to the steel is essential as protection against corrosion. GFRC is an eco-friendly material because it consumes less energy during production and can be used to control pollution and carbon dioxide which is dangerous to human life.

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EFFECT OF PACKING FACTOR (PF) ON WORKABILITY AND MECHANICAL PROPERTIES OF SELF-COMPACTING CONCRETE OF M60 GRADE WITH FLY ASH AND MICRO SILICA AS MINERAL ADMIXTURE

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Abstract

This paper presents the results of an experimental research on the workability and mechanical properties of self-compacting concrete. The work focused on concrete mixes having Packing Factors of 1.11, 1.12, 1.13, 1.14 and 1.15 for a Water Cement ratio of 0.33The Concrete mixes contains different proportions of fine aggregate, coarse aggregate and constant proportions of Cement, Micro Silica, fly ash, Super plasticizers, VMA, water binder ratios for different Packing factors ratios. The percentage of Micro Silica and fly ash added is 7% for all mixes. The proportions of mix design are obtained on the basis of NAN-SU mix design. All the mixes contain Cement of 518 kg/m^3 but with different total binder content. The workability tests utilized in this research were the Slump flow, Slump flow T500mm, V-funnel, V-funnel T5min and J-ring, which can be used to evaluate the filling ability, segregation resistance and passing ability of self-compacting concrete Based upon the experimental results, it is observed that when Packing Factor is less than 1.11 the mix requires more binders there by affecting the workability, whereas when Packing Factor is more than 1.15 the required strengths and workability are not achieved.

Keywords: *Self-Compacting Concrete, Packing Factor, Workability, Fly Ash, Micro silica, Super plasticizer and VMA.*

Introduction

Self-compacting concrete having advanced viscosity and workability properties can easily fill the moulds without compaction.High volume of mineral powder is a necessity for a proper self-compacting concrete design. For this purpose, mineral admixtures such as limestone powder, Fly Ash, micro silica, rice husk ash, blast furnace slag and kaolinite can be used. In this study, investigation had been done on fresh and hardened properties of self -compacting concrete(SCC). It is worth noting that extensive investigations on the workability of self-compacting concrete have been made recently. Kayat et al reported that the L-box, U-box, and J-ring tests can be performed to evaluate the passing ability of self-compacting concrete. When combined with the slump flow test and the L-box test is very suitable for the quality control of on-site self-compacting concrete. It is apparent that workability depends on a number of interacting factors such as water content, aggregate type and grading, fine aggregate to coarse aggregate ratio, packing factor, kind and the fineness of cement and dosage of super plasticizers. The main factors on self-compacting concrete are water and super plasticizer contents, the inter particle lubrication is increased by simply adding them. In this research mix design used is based on NAN-SU method. His design is based on packing factor(PF) of aggregate. In this research W/C ratio and FA/CA used are 0.33 and 58/42 for different Packing Factors. Proportions of Coarse aggregate, Fine aggregate and water binder ratios are different and proportions of Cement, Micro Silica, Fly ash, Super plasticizers and VMA are constant for different Packing Factors. The percentages of Micro Silica, Fly ash added are 7%, super plasticizer is 0.02 and VMA is 0.004% for all mixes.

Materials and mix proportions:

This part of the paper presents the specifications of the mixes used for obtaining the compressive strength, split tensile strength, flexural strength andworkability of self-compacting concrete. OrdinaryPortland Cement (OPC 53 grade), Fly Ash and Micro silica were used as cementitious materials. Natural river sand with a fineness modulus of 2.96 and crushed gravel with a nominal maximum size of 10 mm were used as the aggregates. Chemical admixtures used were polycarboxylated ether as super plasticizer and glenium stream 2 as VMA.

Table 1: Mix proportions of concrete containing different Packing Factors.

	Concrete Mixes for W/C =0.33									
	M1 P.F=1.11		M2 P.F=1.12		M3 P.F=1.13		M4 P.F=1.14		M5 P.F=1.15	
	Qty. (kg/m^3)	Prop.	Qty. (kg/m^3)	Prop.	Qty. (kg/m^3)	Prop.	Qty. (kg/m^3)	Prop.	Qty. (kg/m^3)	Prop.
Cement	518.87	1	518.87	1	518.87	1	518.87	1	518.87	1
Fly ash	36.32	0.07	36.32	0.07	36.32	0.07	36.32	0.07	36.32	0.07
Miro Silica	36.32	0.07	36.32	0.07	36.32	0.07	36.32	0.07	36.32	0.07
F.Aggregate	1036.51	1.99	1045.85	2.01	1055.19	2.02	1064.53	2.05	1073.87	2.07
C.Aggregate	614.91	1.18	620.45	1.99	625.99	1.20	631.53	1.21	637.07	1.22
Water/Binder	176.39	0.33	170.74	0.33	170.78	0.33	168.43	0.32	165.89	0.32
Super Plasticisers	17.74	0.02	17.74	0.02	17.74	0.02	17.74	0.02	17.74	0.02
VMA	2.95	0.004	2.95	0.004	2.95	0.004	2.95	0.004	2.95	0.004

Workability and compressive strength:

The strict definition of workability is the amount of useful internal work necessary to produce full compaction. The physical property of concrete and energy required to overcome the internal friction between the individual particles of the mixture. Because of the very high workability of self-compacting concrete, it needs no external vibration and can spread into place, fill the framework and encapsulate reinforcement without any bleeding or segregation. In other words, to ensure that reinforcement can be encapsulated and that the framework can be filled completely, a favourable workability is essential for self-compacting concrete. Moreover, aggregate particles in self-compacting concrete are required to have uniform distribution in the specimen and the minimum segregation risk should be maintained during the process of transportation and placement.

Because the strength of concrete is adversely and significantly affected by the presence of voids in the compacted mass, it is vital to achieve a maximum possible density. This requires a sufficient workability or virtually full compaction. The presence of voids in concrete reduces the density and greatly reduces the strength, which means the presence of 5 percent of voids can lower the strength by as much as 30 percent. This research compares the compressive strength, split tensile strength and flexural strength of self-compacting concrete mixtures for different water cement ratios.

Results and discussion:

The experimental results of self-compacting concrete mixes related to compressive strength, split tensile strength, flexural strength and workability are discussed for different fine aggregate and coarse aggregate contents. The workability tests performed in this research were as EFNARC methods. They are Slump flow, Slump flow T500, V-funnel, V-funnel T5min and J-ring.

Experimental Results:

The results of workability tests on self-compacting concrete are shown in Table 2. The results of compressive strength, split tensile strength and flexural strength are shown in Table 3.

Fresh and hardened state properties of Self Compacting Concrete:

Slump flow (SF) decreases as the Packing Factor (PF) increases. When PF increases from 1.11 to 1.15(3.5%) slump flow decreases from 680mm to 650mm (4.41%).T500, V-funnel, T5 and J-ring values are increasing as the PF increases. When PF increases from 1.11 to 1.15(3.5%), T500 time increases from 3.11 sec to 4.64 sec (32.97%), V-funnel time increases from 6.92sec to10.59sec (34.65%), T5 time increases from 11.66sec to 14.29sec(18.40%) and J-ring value increases from 2mm to 9mm(77.77%). It is observed that workability decreases as the PF increases.Compressive strength decreases as the Packing Factor increases. When PF increases from 1.11 to 1.15(3.5%), 7days Compressive strength decreases from 51.2MPa to 43.47MPa (15. 09%).Whereas 28days Compressive strength decreases from 71.92MPa to 69.18MPa (3.08%).Split tensile strength decreases as the Packing Factor increases. When PF increases from 1.11 to 1.15(3.5%) 7days Split tensile strength decreases from 10.49MPa to 8.46MPa (19.35%). Whereas 28 days Split tensile strength decreases from 12.22MPa to 10.26MPa (16.03%).Flexural strength decreases as the Packing Factor increases. When PF increases from 1.11 to 1.15(3.5%), 7days Flexural strength decreases from 4.98MPa to 4.23MPa (15.06). Whereas 28days Flexural strength decreases from 5.76MPa to 4.53MPa (21.35%). It was observed that the Compressive and Split tensile strengths decreases at higher rate for 7days strength when compared to 28days strength, whereas the Flexural strength decreases at higher rate for 28days strength when compared to 7days strength.

The relation between the strengths and Packing Factor are as given below.

$$\mathbf{f_{ck} = -181(PF) + 250.4}$$

$$\mathbf{f_{t} = -51.2(PF) + 69.05}$$

$$\mathbf{f_{cr} = -29.9(PF) + 38.83}$$

Where

f_{ck} = 28 days compressive strength in MPa.
f_t = 28 days split tensile strength in MPa.
f_{cr} = 28 days flexural strength in MPa.
PF= Packing Factor.
The relation between the flow values and Packing Factors are as given below.
S.F in mm = -1850(PF) + 2785
T500 in sec = 39.4(PF) - 40.68
V-funnel in sec = 91.7(PF) - 95.02
T5 in sec = 66.4(PF) - 61.89
J-ring in mm =180(PF) - 198.2
The relation between Compressive strength and Flexural strength is shown in Fig.10 and the relation between them is obtained as given below.

$f_{cr} = 0.437(f_{ck}) - 25.75$

The relation between Compressive strength and Split tensile strength is shown in Fig.11 and the relation between them is obtained as given below.

$f_t = 0.738(f_{ck}) - 40.75$

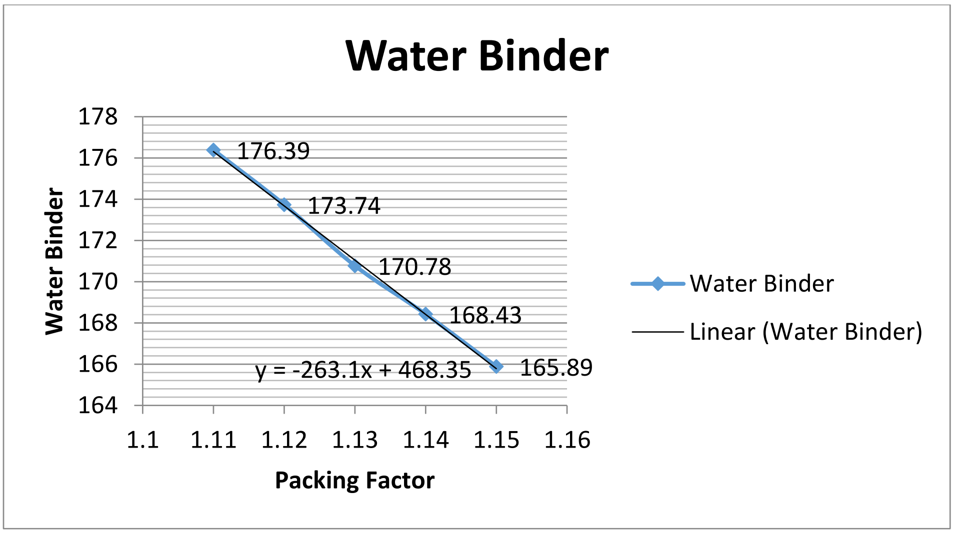

Figure 1: Packing Factor Vs Water Binder

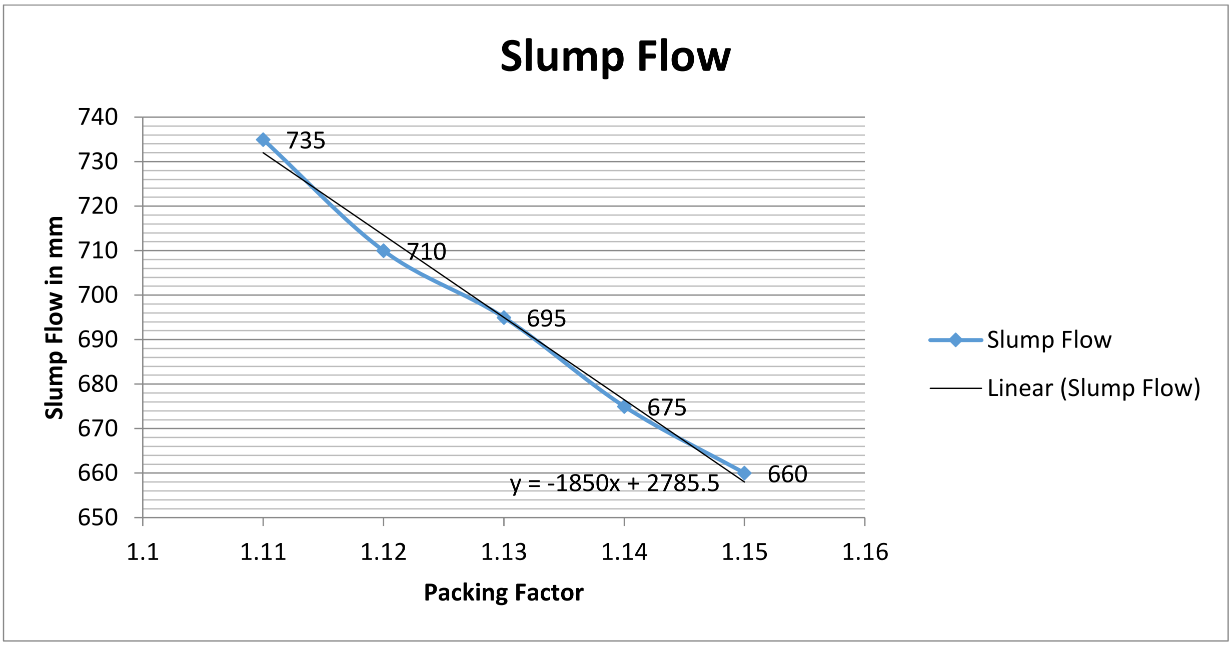

Figure 2: Packing Factor Vs Slump flow

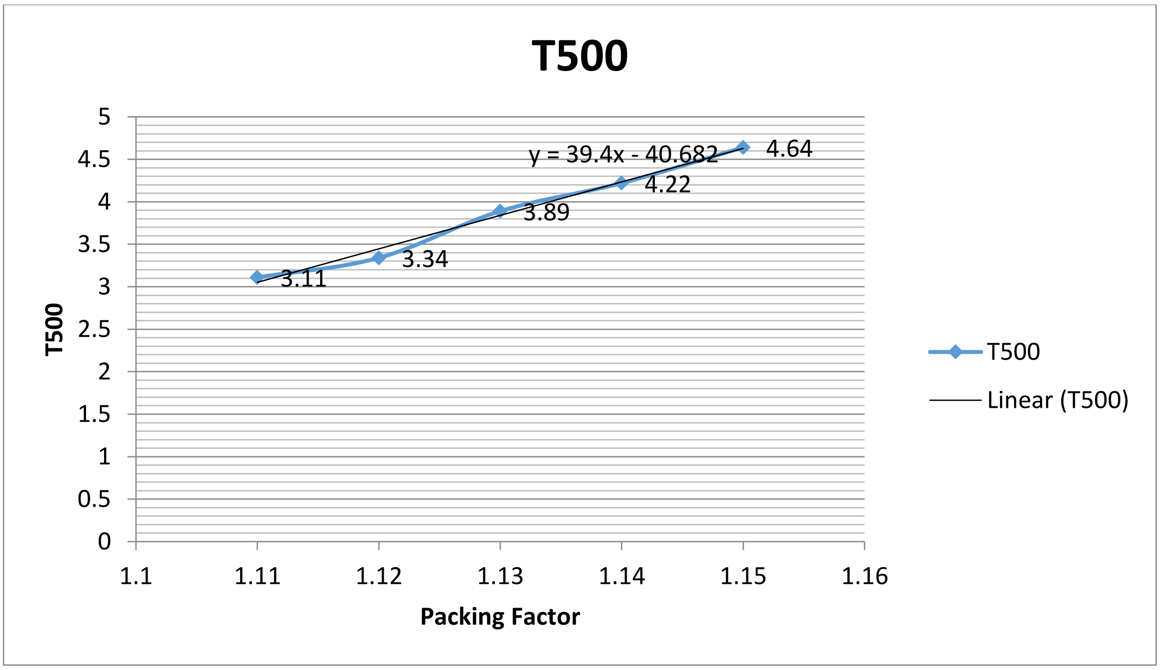

Figure 3: Packing Factor Vs T500

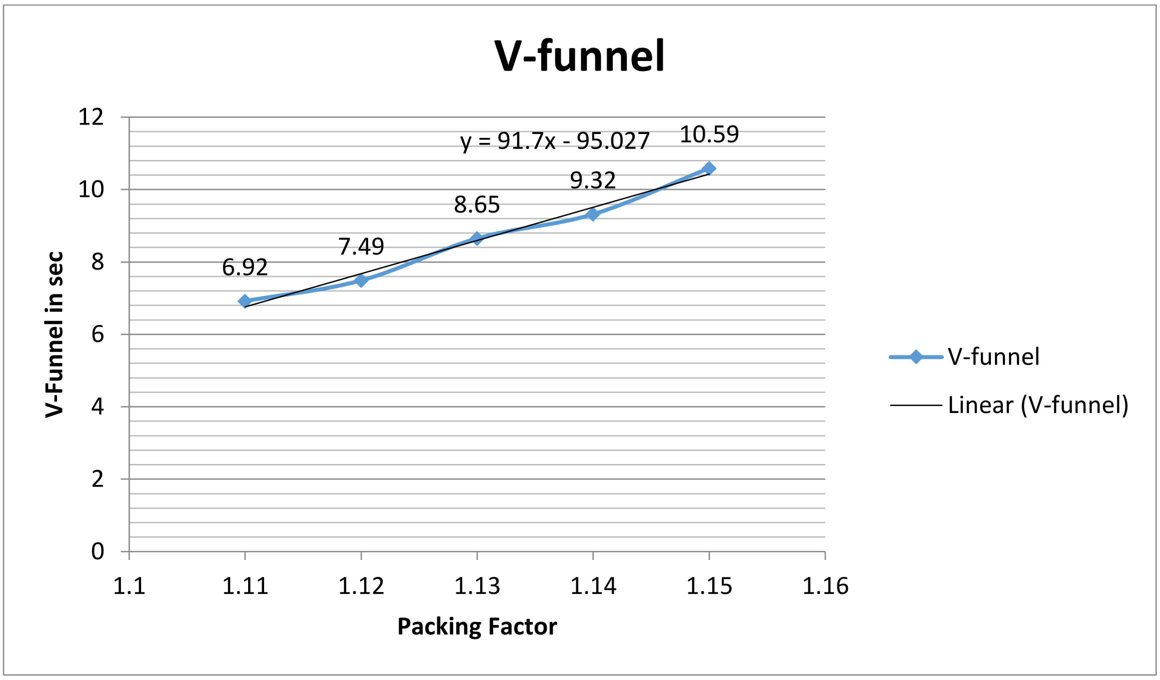

Figure 4: Packing factor vs V-funnel

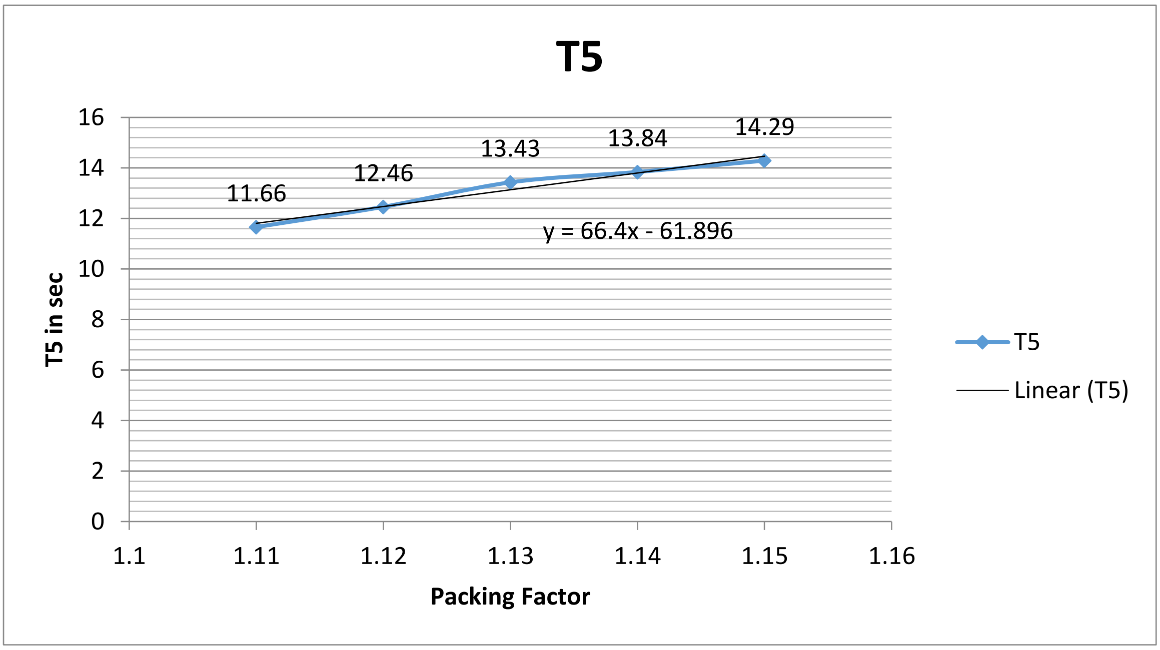

Figure 5: Packing Factor Vs T5

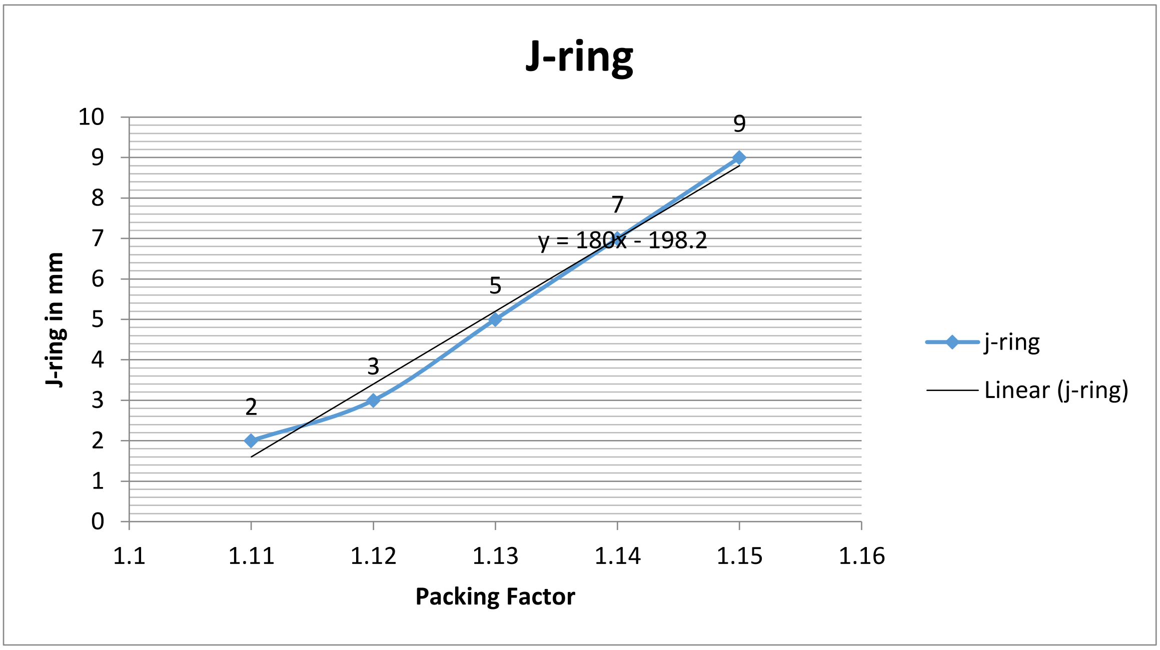

Figure 6: Packing Factor Vs J-ring

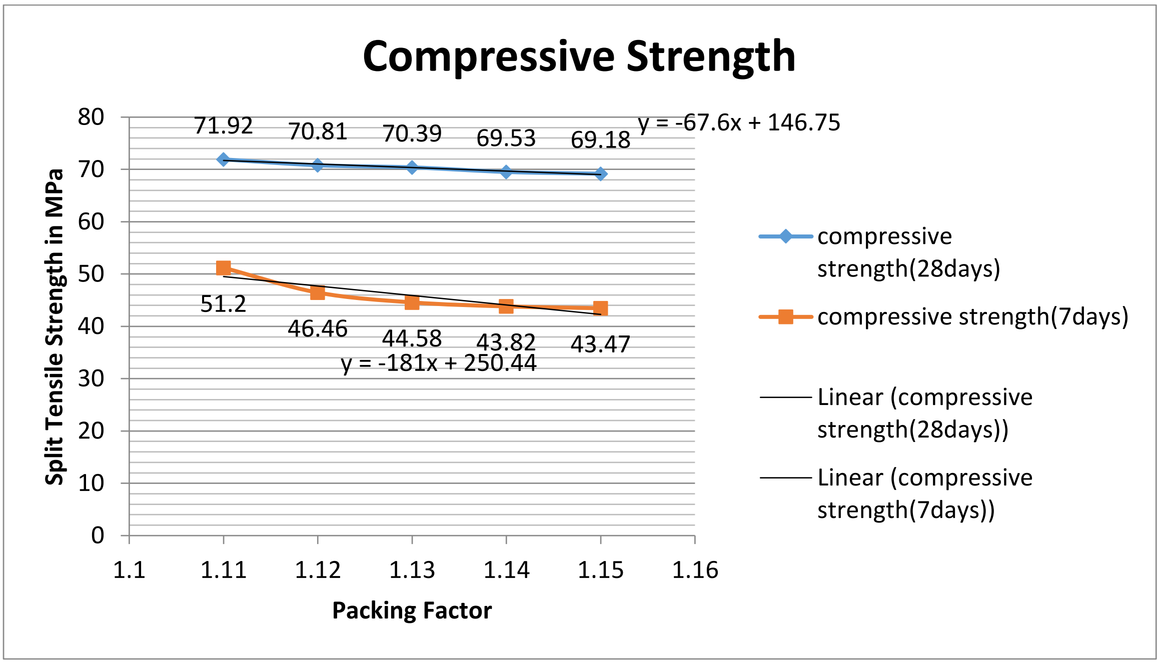

Figure 7: Packing Factor Vs Compressive strength

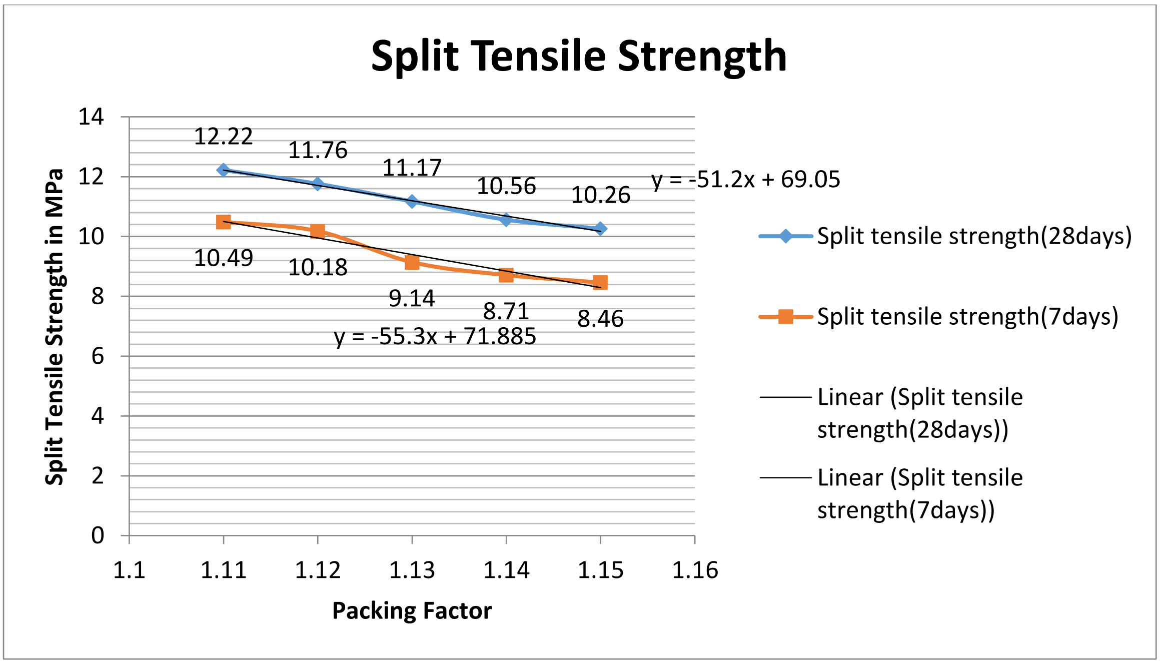

Figure 8: Packing Factor Vs Split tensile strength

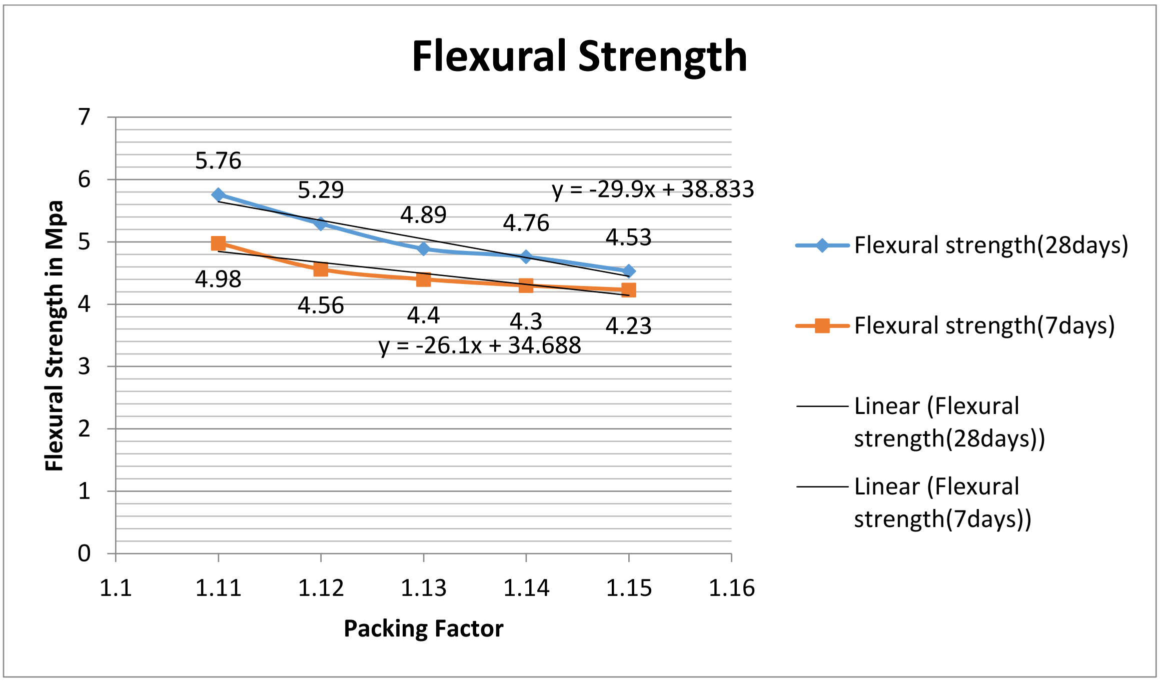

Figure 9: Packing Factor Vs Flexural strength

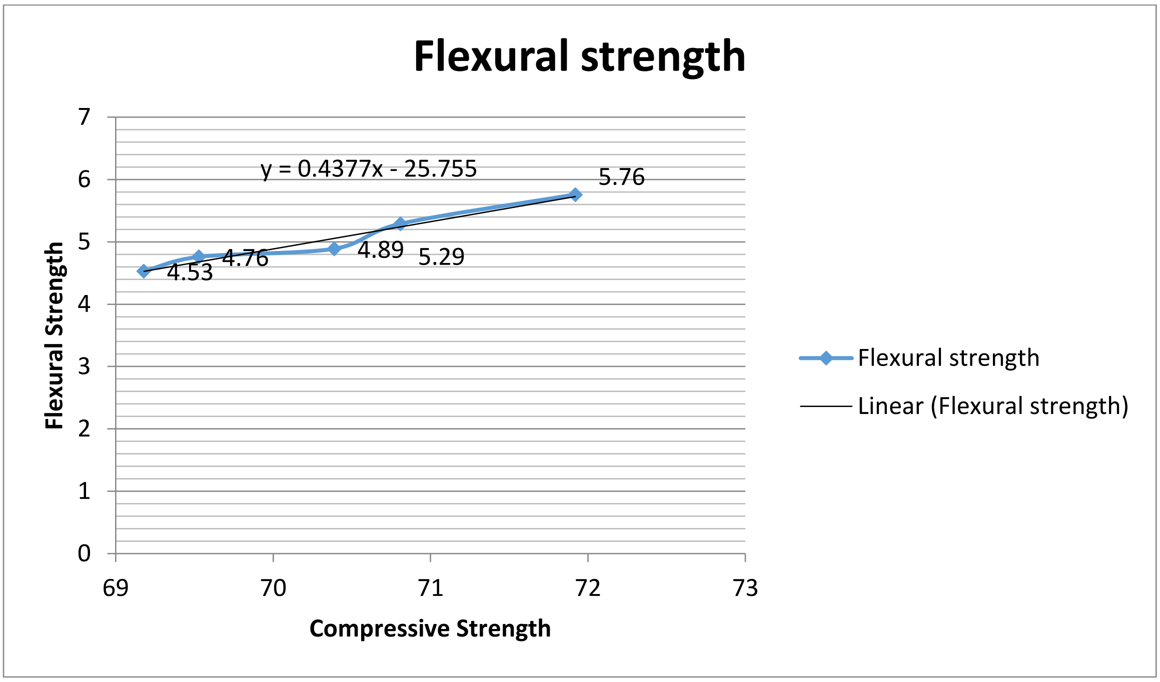

Figure 10: Compressive Strength Vs Flexural strength

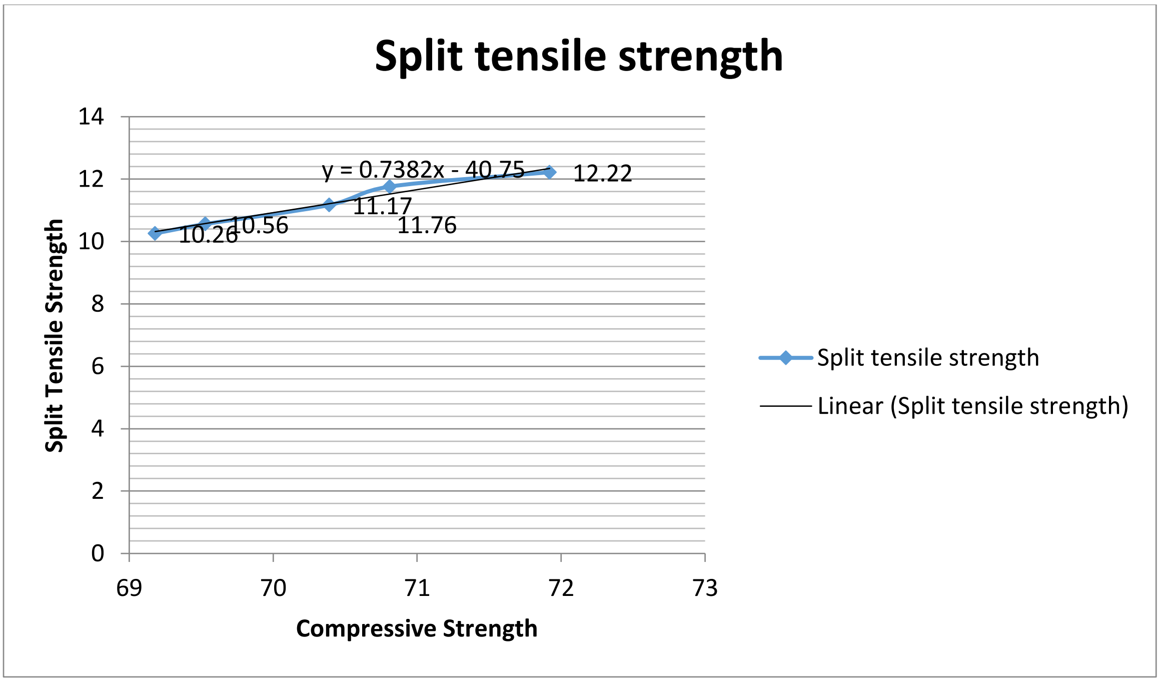

Figure 11: Compressive Strength Vs Split Tensile strength

Conclusions

Based on the experimental work conducted on SCC mixes of M60 grade, with the aim to study effect of different Packing Factors for constant water cement ratio, the main conclusions are

- As the packing factor increases, the workability decreases and also strengths are decreasing.
- Maximum strengths are achieved for a Packing Factor of 1.11 with optimum slump for M60 grade high strength self-compacting concrete.
- These values are obtained for a Water Cement ratio of 0.33 with addition of 7% micro silica and 7% of flyash.
- It is observed that when Packing Factor is less than 1.11 the mix requires more binders there by affecting the workability. Whereas when Packing Factor is more than 1.15 the required strengths and workability are not achieved.
- There is increase in Compressive strength and split tensile strength with decrease in Packing Factor.

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SPECTRAL REFLECTANCE OF BRINJAL CROP CANOPY WITH STAGES

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Abstract

The reflectance spectrum or spectral reflectance curve is the plot of the reflectance as a function of wavelength. Spectral signature is the difference in reflectance or emittance characteristics with respect to wavelength .Spectroradiometer is an instrument used for measurement of reflectance in percentage values from the objects. It has been using in many fields such as vegetative stress analysis, Forestry Analysis, Marine and Wetlands studies measurement etc. The graph given by Spectroradiometer is between percentages of reflectance on Y-axis and Wavelength in nanometer on X-axis. Graphs suggest how much percentage of energy reflects from incident energy and the drawn graph which is not a straight line or smooth curve. Rise and fall indicates the water absorption bands. Dips vary with material to materials which also depends on molecules present in them. Healthy vegetation will absorb more light than unhealthy or stress condition vegetation. If the reflectance is more than 26% then the crop is in healthy condition if it is less than 26% means crop is in stress condition. For any vegetation dips can be found at 400nm, 500nm, 550nm, 670nm, 770nm, 870nm and 920nm. There may be the same dips for a particular crop irrespective stages but percentage of reflectance will be vary because of chlorophyll , leaf internal structure and moisture content. In the present study, the spectral reflectance of brinjal crop for maturation and harvesting stages in Sri KondaLaxman Horticultural University of Telangana (Rajendra Nagar) farmhouse from the visible/near infrared spectra of sensitive spectral band was applied to develop a method for rapid detection of spectral reflectance.

Keywords: *Horticulture, controlled condition, Spectral reflectance, emittance*

1.0 Introduction

Remote sensing is the collection of information about an object or phenomenon without making physical contact with the object and thus in contrast to on-site observation. Remote sensing is used in numerous fields, including geography and most Earth Science disciplines for example, hydrology, ecology, oceanography etc. In Optical Remote Sensing, optical sensors detect solar radiation reflected or scattered from the earth, forming images resembling photographs taken by a camera high up in space. Spectroradiometer is basically emitting the light which has the properties such as electromagnetic radiation light exists as photons displaying both wave and particle properties, discrete quanta of energy wavelength measurements in 350nm-1050nm range of EM spectrum. The interaction of electromagnetic radiation with materials on a macroscopic level, including the refraction, diffraction and scattering effects formed the basis of traditional remote sensing theory. The hyperspectral spectral analysis is a fast and non-destructive method and has been used in many fields such as oil industry, food industry, Vegetative stress analysis, Surface color measurement. In most of the vegetation, dips can be found at 400nm, 450nm, 500nm, 550nm, 670nm, 770nm and 920nm wavelengths. This is due to presence of chlorophyll content in the leaves in visible region more chlorophyll present so lesser the reflectance whereas in near infrared region chlorophyll content is medium and in middle infrared and lesser the chlorophyll content so by observing that it is found that the spectral response is rising trend in vegetation with dips at specific wavelengths. For the purpose of study data captured for mainly, two stages at maturity stage. There were same dips for particular crop irrespective stages of the crop. Only difference is percentage of reflectance value varied because of chlorophyll content within the crop this change is due to leaf internal structure, moisture content etc. Rise and fall of trend can be noticed which provides the information about water absorption dips varies with material to materials which also depends on molecules present which them. There are various objects which can reflects different range of percentage mostly for simple study some of the major curves are scientifically drawn such as vegetation, water, soil and snow.

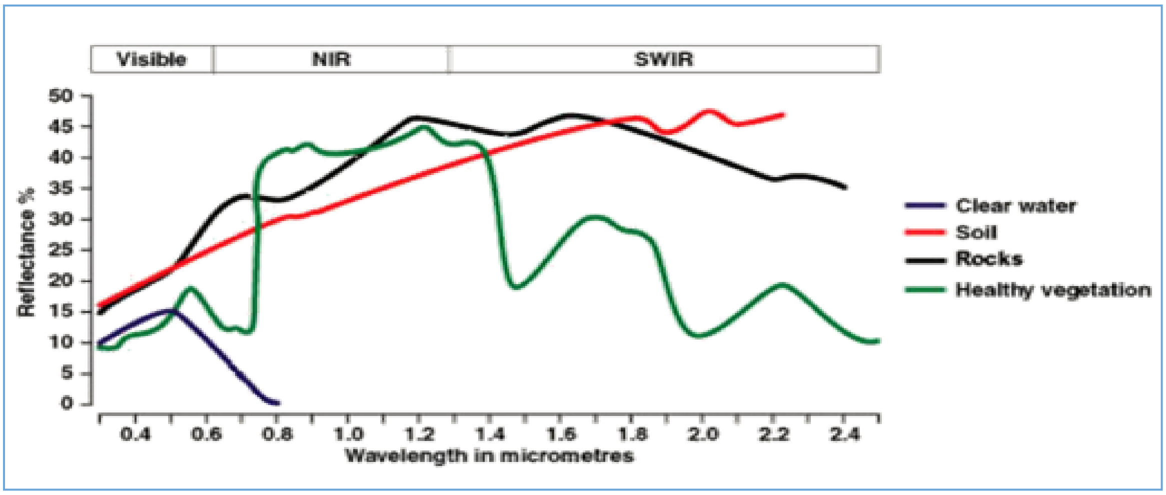


Fig: 1.1.Spectral Signature of different materials

2.0 LITERATURE OF REVIEW

Dhaval*et al* (2013) had attempted to derive canopy level estimation of chlorophyll and Leaf Area Index (LAI) for tropical species using the Hyperion data. 50 leaves were picked up from different individuals. Leaf area and leaf dry weight were measured, SLA was calculated. Hyperion data was collected with cloud cover less than 25%. Atmospheric correction was carried out using ACORN 1.5 software. Subset extraction and image processing were performed using ENVI V.4.6 software.

Dhaval*et al* (2013) had attempted to derive canopy level estimation of chlorophyll and Leaf Area Index (LAI) for tropical species using the Hyperion data. 50 leaves were picked up from different individuals. Leaf area and leaf dry weight were measured, SLA was calculated. Hyperion data was collected with cloud cover less than 25%. Atmospheric correction was carried out using ACORN 1.5 software. Subset extraction and image processing were performed using ENVI V.4.6 software.

Adam *et al* (2010) he studied ground based hyperspectral analysis is carried out by to acquire the spectral analysis is carried out to acquire the spectral signatures using ASD and first derivative reflectance at fresh leaf scale. The correlations between spectral signatures, first derivative reflectance and three biochemical variables were established using curve fitting analysis.

Jitendra*et al* (2010) identified the aphid infestation in mustard by hyperspectral remote sensing. Spectral reflectance was taken in healthy as well as in aphid-infested canopies of mustard in field as well as in laboratory. The filed experiments were conducted during 2009-2010 rabbi season at research farm of IARI New Delhi India.

Rama Rao *et al* (2007) investigated the utility of space borne hyper spectral imaging for the development of a crop specific spectral library and automatic identification and classification of 4 cultivars for each of rice, chilly, sugarcane and cotton crops. The classification of crops at cultivator level using two spectral libraries developed using hyper spectral reflectance data at canopy scale (in-situ measurements) and at pixel scale (Hyperion data). The data has been collected from different sources such as Hyperion data, in situ hyperspectral data using field spectroradiometer (GER 3700, Spectra Vista Crop. New York, NY, USA) and other ancillary data.

Sebastian *et at* (2006) presented a method to extract a subset of individual bands from a hyperspectral data set of Hymap sensor, by performing the classification for previously mapped agricultural areas for which ground truth data had been collected in the same vegetation period in Germany, by applying the algorithms for feature extraction method.

A.L.Kaleita*et al* (2005) their study was relationship between soil moisture content and soil surface reflectance. They minimize both the effect of time of day on the spectral data and the effect of drying time on the moisture data, every effort was made to perform the data collection within a consistent and minimal period. Data collections were limited to approximately a 2hr window.

Zarco*et al* (2005) discussed the several new narrow band hyperspectral indices calculated from Air borne visible and near Infrared (AVNIR) hyperspectral sensor over a cotton field in California (USA) collected over an entire growing season at 1m spatial resolution. Within field variability of yield monitor spatial data collected during harvest was correlated with hyperspectral indices related to crop growth and canopy structure, chlorophyll concentration and water content.

3.0 METHODS AND MATERIALS

Spectroradiometer is an instrument for measurement of radiometric quantities in narrow wavelength intervals over a given spectral region. Spectra Vista Corporation proudly offers the SVC HR-512i. Here 512 indicate number of bands that are used in this instrument. This instrument combines the latest technology required to produce exceptional spectral data while capturing digital photographic, GPS and external sensor data. Hyper spectral hundreds to thousands number of spectral band and spectral resolution narrow, few nm; its capability detects and identifies solids and liquids. One person by first setting up instrument parameters through the touch screen display and then initiating a measurement easily acquires measurements. Spectral resolution and low noise ensure that the collected data is of the highest quality and vice versa. Hyper spectral images provide each and every aspects can be analyze easily, spectral information to identify and distinguish between spectrally similar materials.

3.1. Taking a Reference

1. Check that the LED Power Indicator is illuminated, indicating that the power is enabled. On the LCD’s “Setup Screen A”, ensure that the trigger field is set to “Laser + Scan” by pressing the “TRIGGER:” button until the correct setting appears.
2. On the LCD’s “Main Screen”, ensure that the scan type is set to “Reference” by pressing the “SCAN:” button until the correct setting appears.

3. Press and hold the LASER/SCAN button to activate the sighting laser and verify the placement of the reference plate relative to the laser spot as required filling the field of view. Then LASER/SCAN button to record the measurement.
4. Repeat the Same process for Target measurement.

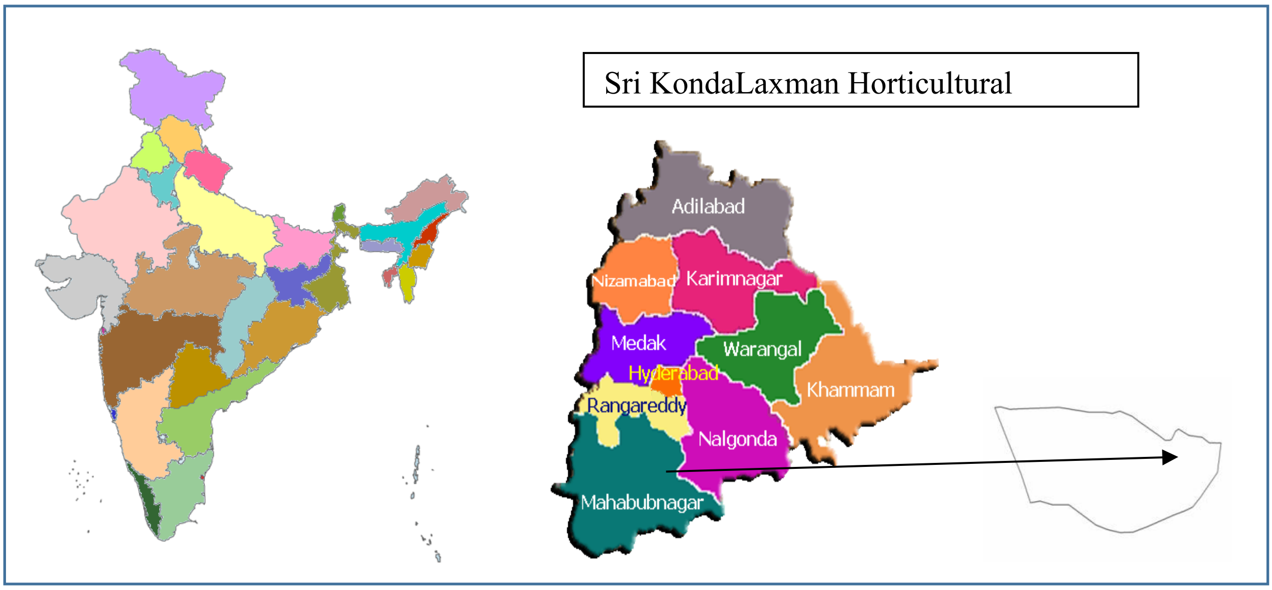
3.2. Downloading Stand-alone Measurements

Following steps are to data acquisition software in order to download the stand-alone measurements to the PC: See "**Control**->**Setup Instrument**…" for basic software connection instructions. See "**Control**->**Setup Overlap/Matching**…" for instructions on how to configure these settings, as they are applied to both downloaded and newly acquired spectral data. See "**Control**->**Read Memory**…" for downloading instructions.

4.0 STUDY AREA

For the purpose of study, area chosen which is belongs to Ranga Reddy district of Sri Konda Laxman Telangana State Horticultural University Rajendranagar, Hyderabad, Telangana State, India. This is enclosed with geographical regions of 17°19'17"N, 78°25'23"E. My study area boundary delineated from BHUVAN website for the study area using Arc Map 10.2 version. The study area map has been prepared and shown in Fig.4.1 Sri KondaLaxman Horticultural University the crops are identified in which the research is to be continued. Sri KondaLaxmanTelangana State Horticultural University (SKLTSHU), a dedicated & splendid institute of horticultural learning. It is the first Horticultural University in the state of Telangana and fourth in the Country. The mission goal & primary mandate of the University is to excel as Centre of Excellence in Education, Research and Extension Education in the field of Horticulture and allied sectors.

Fig 4.1 Sri KondaLaxman Horticultural University boundary Study Area Map



The spectral signature of following vegetation were studied in this research

- ✓ Brinjal crop (maturation to harvesting stage)

5.0 RESULTS AND DISCUSSIONS

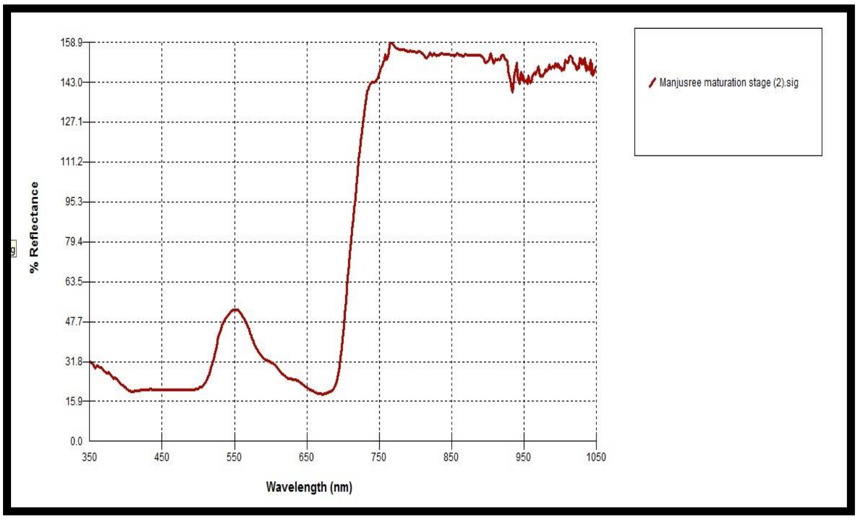
The crop identification and the village identification is done by direct visit to the study area and the direct analysis of the crops in direct view. The photographs of the crops are taken during the maturation and harvesting stages of the crops. Brinjal crop taken for study results drawn for at two stages of crop maturation and harvesting stage. It is found that at the initial stage of crop the chlorophyll content is higher in amount due to that spectral reflectance is less in percentage at visible range, later chlorophyll content reduces due to aged crop so reflectance increases in near infrared regions after 920nm reflectance got disturbance.

5.1. Ground truth data of Brinjal crop

The egg-shaped glossy purple fruit has white flesh with a meaty texture. The cut surface of the flesh rapidly turns brown when the fruit is cut open.

Fig: 5.1. Image of Eggplant (Brinjal) at maturation stage

Fig: 5.2. Spectral reflectance of Manjusree (Eggplant) at maturation stage



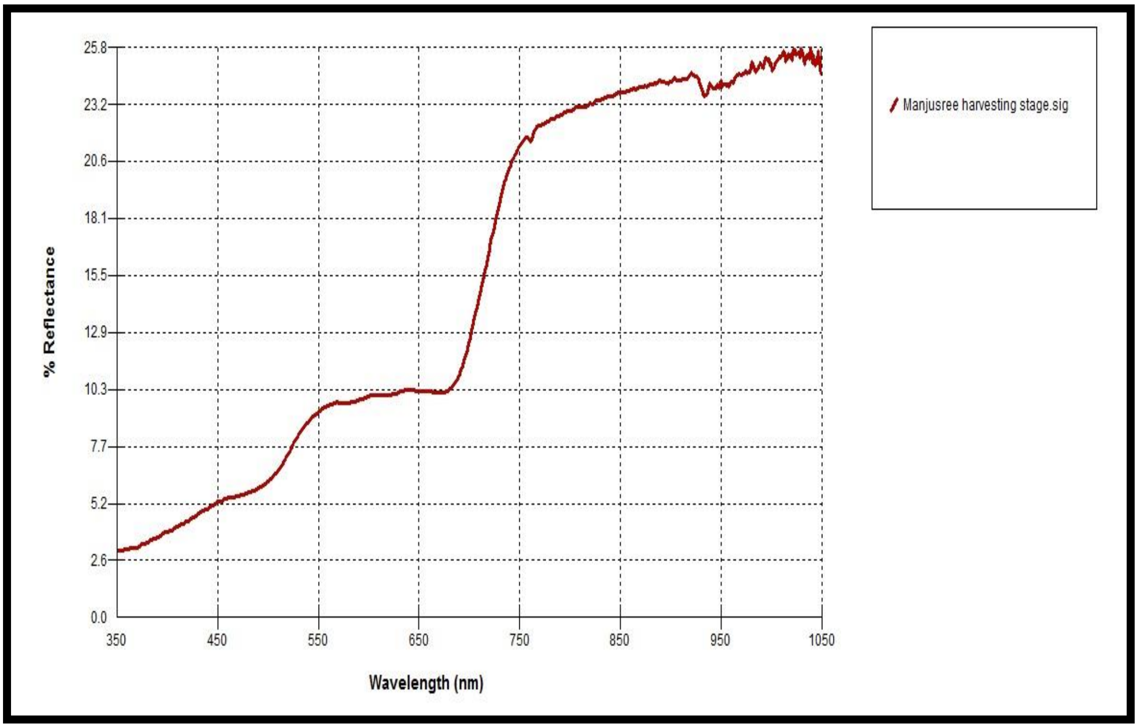


Fig: 5.3. Spectral reflectance of Manjusree (Eggplant) at harvesting stage

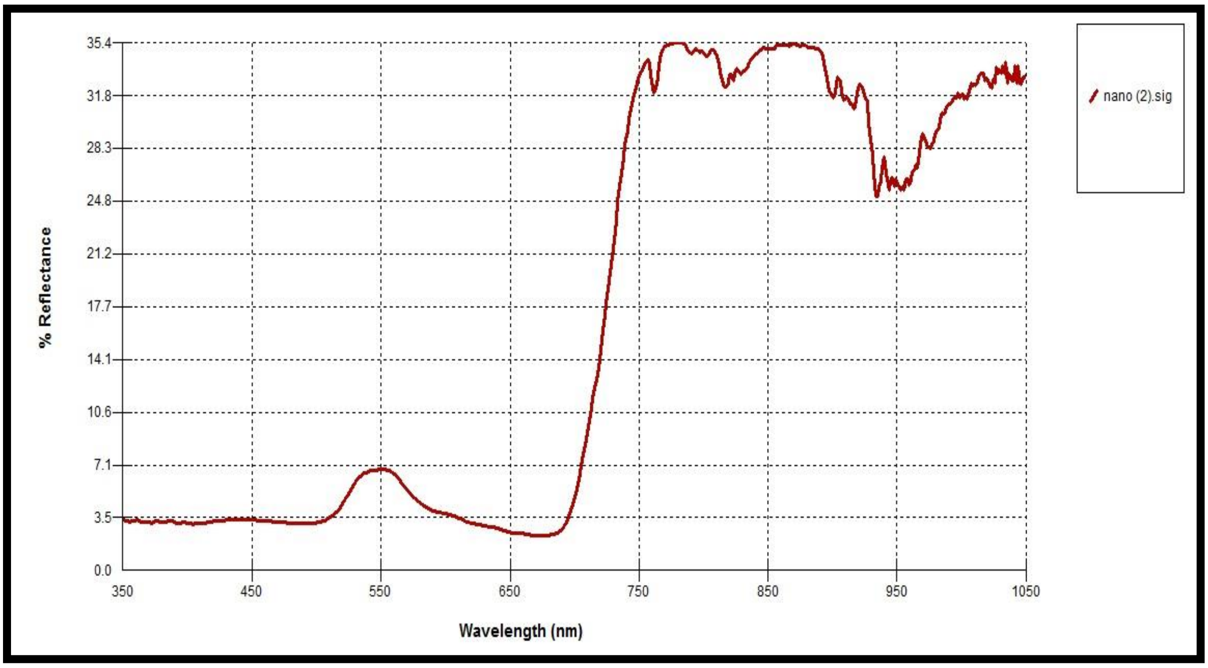


Fig: 5.4. Spectral reflectance of Nano (Eggplant) at maturation stage

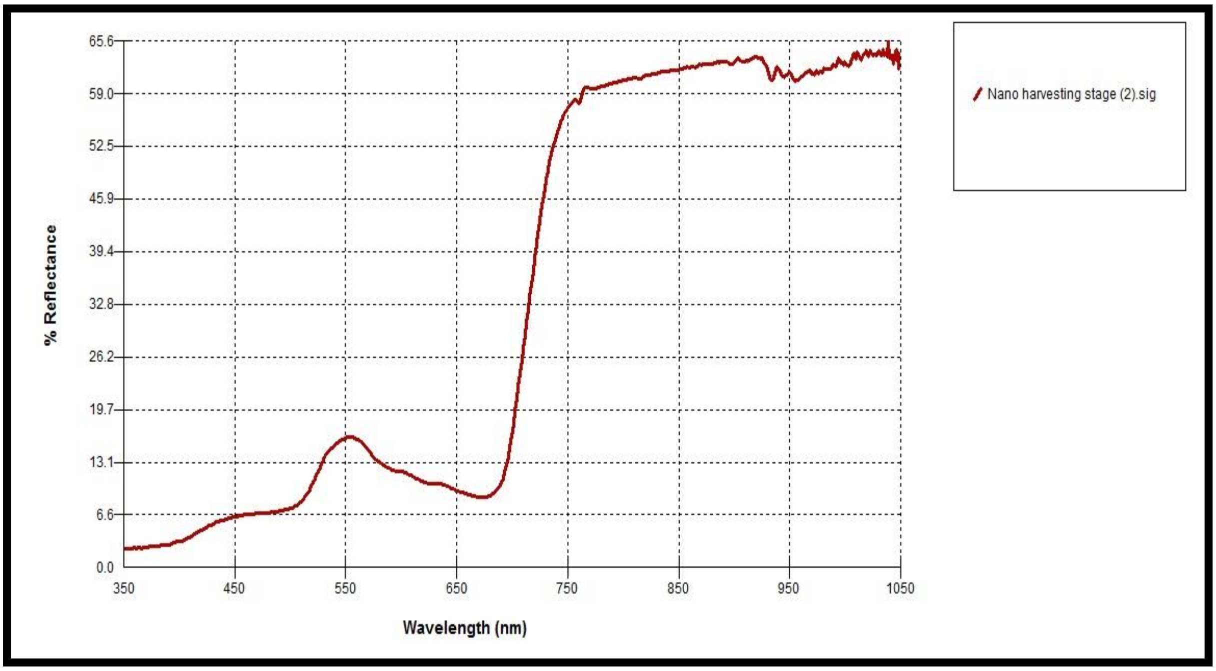


Fig: 5.5. Spectral reflectance of Nano (Eggplant) at harvesting stage

Brinjals numerous varieties are available in the world but for the study only four types was selected such as nano, manjusree, and they sowed on 06/06/2016, Planted on 27/06/2016.For the crop Brinjals (eggplant) spectral response is at 400nm, 450nm, 500nm down dip, 550nm up dip, 690nm down dip, 770nm up and finally at 920nm dips can be observed these dips are common with irrespective of stages and within same varieties such as Manjusree, Nano.

- ✓ At 450nm reflectance is about 5.2% and at 920nm is about 23.2% at maturation stage of manjusree
- ✓ At 450nm reflectance is about 3.5% and at 920nm is about 24.8% at maturation stage of nano

Though many varieties present in the brinjal vegetables crops the dip formation is common for them but only percentage of reflectance value varies due leaf internal structure mainly thickness of leaf, moisture content, amount of chlorophyll present in the leaf.

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ANALYSIS OF SHEAR BEHAVIOR OF BFRP STRENGTHENED RC BEAMS

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Abstract

This study deals with experimental investigation for enhancing and strengthening structurally deficient T-beams by using an externally bonded fiber reinforced polymer (FRP). The rehabilitation of existing reinforced concrete (RC) bridges and building becomes necessary due to ageing, corrosion of steel reinforcement, defects in construction/design, demand in the increased service loads, and damage in case of seismic events and improvement in the design guidelines. Fiber-reinforced polymers (FRP) have emerged as promising material for rehabilitation of existing reinforced concrete structures. The rehabilitation of structures can be in the form of strengthening, repairing or retrofitting for seismic deficiencies. RC T-section is the most common shape of beams and girders in buildings and bridges. Shear failure of RC T-beams is identified as the most disastrous failure mode as it does not give any advance warning before failure. The shear strengthening of RC T-beams using externally bonded (EB) FRP composites has become a popular structural strengthening technique, due to the well-known advantages of FRP composites such as their high strength-to-weight ratio and excellent corrosion resistance. This study assimilates the experimental works of glass fiber reinforced polymer (GFRP) retrofitted RC T-beams under symmetrical four-point static loading system. An innovative method of anchorage technique has been used to prevent these premature failures, which as a result ensure full utilization of the strength of FRP.

***Keywords**: FRP, GFRP, CFRP, AFRP, CB*

1. Introduction

The deterioration of civil engineering infrastructures such as buildings, bridge decks, girders, offshore structures, parking structures are mainly due to ageing, poor maintenance, corrosion, exposure to harmful environments. These deteriorated structures cannot take the load for which they are designed. A large number of structures constructed in the past using the older design codes in different parts of the globe are structurally unsafe according to the new design codes and hence need up gradation. The conventional retrofitting techniques available are concrete-jacketing and steel-jacketing. The concrete- jacketing makes the existing section large and thus improves the load carrying capacity of the structure. But these techniques have several demerits such as construction of new formworks, additional weight due to enlargement of section, high installation cost etc. The steel-jacketing has proven to be an effective technique to enhance the performance of structures, but this method requires difficult welding work in the field and have potential problem of corrosion which increases the cost of maintenance. With increase in research and introduction of new materials and technology there are new ways of retrofitting the structure with many added advantages. Introduction of Fiber Reinforced Polymer (FRP) Composite is one of them. FRP composites comprise fibers of high tensile strength embedded within a thermosetting matrix such as epoxy, polymer or vinyl ester. The most widely used matrix is epoxy. FRP was originally developed for aircraft, helicopters, space-craft, satellites, ships, submarines, high speed trains because of its light weight. The application of FRP in the civil engineering structures has started in 1980s.

The first application of FRP strengthening was made to reinforce the concrete beams. The beams are load bearing structural elements that are designed to carry both vertical gravity loads and horizontal loads due to seismic or wind. The structurally deficient beams fail during such events. There are mainly two types of failure of beams i.e., flexural and shear. Hence, the strengthening of such beams is needed in flexure or shear or both zones and the use of external FRP strengthening to beams may be classified as:

i) Flexural strengthening ii) Shear strengthening

2. Objective of the Present Work

In the light of the literature survey presented above, the following objectives of are identified for the present work:

1) To study the structural behaviour of reinforced concrete (RC) T-beams under static loading condition.
2) To study the contribution of externally bonded (EB) Fiber Reinforced Polymer (FRP) sheets on the shear behaviour of RC T-beams.
3) To investigate the effect of a new anchorage scheme on the shear capacity of the beam.

3. Design of Material Properties

The material properties reported by the manufacturers, such as the ultimate tensile strength, typically do not consider long-term exposure to environmental conditions and should be considered as initial properties. Because long-term exposure to various types of environments can reduce the tensile properties and creep-rupture and fatigue endurance of FRP laminates, the material properties used in design equations should be reduced based on the environmental exposure condition. Eq.s (1) through (3) gives the tensile properties that should be used in all design equations. The design ultimate tensile strength should be determined using the environmental reduction factor given in ACI 440.2R-02 document for the appropriate fiber type and exposure condition:

$$\text{Design ultimate tensile strength} = f_{fu} = C_E \, f^*_{fu} \qquad (1)$$

Where,

f_{fu} = design ultimate tensile strength of FRP, (MPa)

C_E = environmental reduction factor

f^*_{fu} = ultimate tensile strength of the FRP materials as reported by the manufacturer, (MPa)

Similarly, the design rupture strain should also be reduced for environmental-exposure conditions:

4. Nominal shear strength

The nominal shear strength of a RC beam may be computed by basic design equation presented in ACI 318-95 and given as in Eq. (4)

$$V_n = V_c + V_s \qquad (4)$$

In this equation the nominal shear strength is the sum of the shear strength of the concrete (which for a cracked section is attributable to aggregate interlock, dowel action of longitudinal reinforcement, and the diagonal tensile strength of the un-cracked portion of the concrete) and the strength of the steel shear reinforcement.

In the case of beams strengthened with externally bonded FRP sheets, the nominal shear strength may be computed by the addition of a third term to account for the contribution of FRP sheet to the shear strength. This is expressed in Eq. (5)

$$Vn = Vc + Vs + Vf \qquad (5)$$

5. FRP system contribution of shear strength

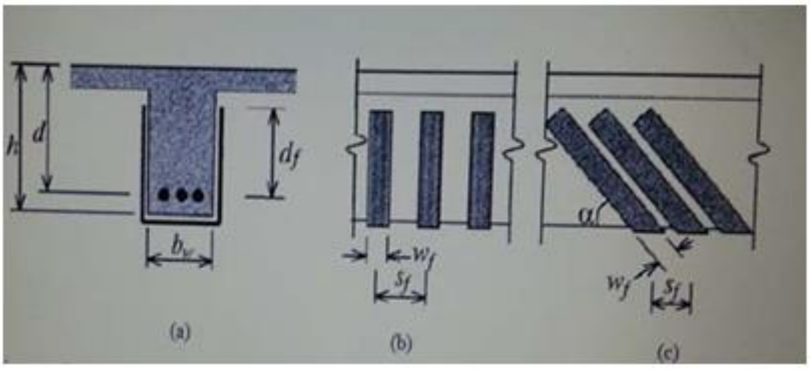

Figure 1: Illustration of the dimensional variables used in shear-strengthening calculations for repair, retrofit, or strengthening using FRP laminates.

Where,

Design rupture strain = εfu = CE ε*fu (2)

$$Efu = ffu / εfu \qquad (3)$$

(a) Cross-section,

(b) Vertical FRP strips,

(c) Inclined FRP strips

Because FRP materials are linearly elastic until failure, the design modulus of elasticity can then be determined from Hook's law. The expression for the modulus of elasticity, given in Eq. (3), recognizes that the modulus is typically unaffected by environmental conditions. The modulus given in this equation will be the same as the initial value reported by the manufacturer.

Figure 1 illustrates the dimensional variables used in shear- strengthening calculations for FRP laminates. The contribution of the FRP system to shear strength of a member is based on the fiber orientation and an assumed crack pattern [Khalifa et al. 1998]. The shear strength provided by the FRP reinforcement can be determined by calculating the force resulting from the tensile stress in the FRP across the assumed crack.

6. Conclusion

The material used for this present work is glass fiber and epoxy resin, and the exposure condition is internal exposure. For present calculation the environmental reduction factor (CE) is used as 0.75.

In this experimental investigation the shear behavior of RC T-beams strengthened by GFRP sheets are studied. The test results illustrated in the present study showed that the external strengthening with GFRP composites can be used to increase the shear capacity of RC T-beams, but the efficiency varies depending on the test variables such as fiber orientations, wrapping schemes, number of layers and anchorage scheme. Based on the experimental and theoretical results, the following conclusions are drawn:

1) Externally bonded GFRP reinforcement can be used to enhance the shear capacity of RC T-beams.
2) The test results confirm that the strengthening technique of FRP system can increase the shear capacity of RC T- beams.
3) The initial cracks in the strengthened beams are formed at a higher load compared to the ones in the control beam.
4) Strengthening of T-beam on the webs with GFRP is most vulnerable to de bonding with premature failure.
5) The beam strengthened with a U-wrap configuration is more effective than the side-wrap configuration.
6) Among all the GFRP strip configurations (i.e. vertical strips, strips inclined at 45° and strips inclined at +45° in one direction and +135° in another direction making an “X-shape”), the X-shape is more effective than the others.
7) Applying GFRP to the beam with end anchorage is better than strengthening without end anchorage.
8) The use of anchorage system eliminates the debonding of the GFRP sheet, and consequently results in a better utilization of the full capacity of the GFRP sheet.
9) The test results indicated that the most effective configuration was the U-wrap with end anchorage among all the configurations.
10) The load-deflection behaviour was better for beams retrofitted with GFRP inclined strips than the beams retrofitted with GFRP strips on the sides alone.

Finally, the use of GFRP sheets as an external reinforcement is recommended to enhance the shear capacity of RC T- beams with anchorage system.

7. Future scope

Based on the finding and conclusions of the current study the following recommendations are made for future research in FRP shear strengthening:

1. Study of bond mechanism between CFRP, AFRP and BFRP and concrete substrate.
2. FRP strengthening of RC T-beams with different types of fibers such as carbon, aramid & basalt.
3. Strengthening of RC L-beams with FRP composite.
4. Strengthening of RC L-section beams with web opening.
5. Effects of web openings of different shape and size on the shear behaviour of T & L-beams.
6. Effects of shear span to depth ratio on shear strengthening of beams.
7. Numerical modelling of RC T & L-beams strengthened with FRP sheets anchored at the end.

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EXPERIMENTAL STUDIES OF BLENDED SCC MADE WITH GRANULATED BLAST FURNACE SLAG AS PARTIAL FINE AGGREGATE REPLACEMENT

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Abstract

The River sand is the major material for preparation of mortar and self-compacting concrete(SCC) but the problem with river sand is, it is the largest basic consumer non-renewable resource .Hence, it is our responsibility to safeguard the sand for future generation. Today we all experience the scarcity of natural sand due to continuous excavation of river beds which seriously effects our environment and ecology. It's time to think for another alternative to replace river sand and for any mix of SCC. In the present study we put an effort to replace the river sand with GBFS. Metakaolin and fly ash are used as replacement for cement in SCC to investigate the synergistic action of binary, ternary self –compacting concrete (SCC) on flow properties and strength properties of SCC (M20) made with partial replacement of fine aggregate with granulated blast furnace slag (GBFS).Quantities required for 1cu.m are evaluated for M20 grade SCC made with Fly ash (FA), Metakaolin(MK) based on calculations from Nan Su mix design Method. From the experimental investigations, the mixes C100,C50+FA50,C75+MK25,C25+FA60+MK15 are chosen as optimum binary and ternary blended SCC M20 grade where both flow and desired strength properties are met along with optimal usage of pozzolanic materials.

Keywords: *self-compacting concrete, Metakaolin, Fly Ash, granulated blast furnace slag, Ternary Blended SCC.*

1 Introduction

The advent of technology and globalization with the expansion of population has resulted in depletion of building materials to meet the demands of global market. River sand is a basic ingredient used as fine aggregate in concrete production .The growing demand of competitive market demands construction materials on a large scale directed to the over utilization of river sand which has severe detrimental effect like the raise in river bed depth lowering of water table and increase in saline content of water. High reactivity Metakaolin (MK) is supplementary cementing material developed recently for improving the performance of concrete. All though some works have reported that it improves properties information about the properties of Metakaolin blended self-compacting concrete (SCC) is still limited and somewhat contradictory which retards its application in the construction practice. The use of appropriately proportion Metakaolin in fly ash in blended SCC revealed the benefits of their synergic effect in improving the fresh and hardened properties of fly ash based SCC. The present work aim said determining the most suitable mix proportion that can produce Metakaolin based binary and ternary blended M20 grade SCC of desired flow properties and strength. Also studies are carried out to understand the use of granulated blast furnace slag (GBFS) as fine aggregate replacement in Metakaolin based binary and ternary blended SCC without compromising on engineering performance and quality.

2. EXPERIMENTAL INVESTIGATIONS

2.1 Determination of Quantities of materials for SCC mixes

Based on Nan Su mix design method, quantities required for 1 cu.m are evaluated for standard grade (M20) of ternary blended Self Compacting Concrete (SCC) made with SCMs such as Fly Ash (FA), and Metakaolin(MK). Final quantities, for all SCC mixes considered, are assumed after several trial mixes on quantities computed using Nan Su mix design method subjected to satisfaction of EFNARC flow properties. This phase identifies the optimum proportions of fly ash and metakaolin in ternary blended SCC for enhanced performance of SCC at all ages.

The following are the quantities of materials calculated using Nan Su mix design method for standard grade (M20) based Self Compacting Concrete (SCC) and also presented the final quantities of materials after various trial mixes.

Table-1: Quantities per 1 cu.m for standard (M20) grade SCC obtained using Nan Su method of Mix Design

Grade of SCC Mix		Cement	Pozzolana (Fly ash)	Fine aggregate	Coarse aggregate	S.P.	Water
M20	Quantity kg/m^3	190	257	904	812	8.1L	192.34L

The amount of total powder (i.e., OPC+FA) computed is 447kg/m3 and computed total weight of fly ash is 257kg/m3 (57% of total powder). For the above powder content, flow properties are not achieved as per EFNARC guidelines, so several trail mixes were carried out of satisfy the flow properties. The final SCC mix proportions shown in Table 1 are arrived at after several trail mixes by adjusting Cement, Pozzolan (fly ash) and super plasticizer till the mix conforms to EFNARC specifications for the required fresh properties. Hence forth the total amount of powder quantity adopted for ordinary M20 grade self – compacting concrete is 486kg/m3 and water powder ratio 0.45 for all blended ordinary M20 grade Concrete.

Table-2: Final Quantities per 1 cu.m for high strength (M20) grade SCC mix after trail mixes

Grade of SCC Mix		Cement	Pozzolana (Fly ash)	Fine aggregate	Coarse aggregate	S.P.	Water	Water/Powder Ratio
M20	Quantity kg/m^3	225	261	904	812	9.34L	221L	0.45

Table 3: Optimum Mix of various ordinary grade (M20) blended SCC after several trail mixes

Grade of SCC Mix	Mix No	Mix Designation	Replacement Percentage			Quantities Kg per cu.m		
			OPC	FA	MK	OPC	FA	MK
M20	C1	C100	100	-	-	486	-	-
M20	B1	C50+FA50	50	50	-	243	243	-
M20	B2	C75+MK25	75	-	25	365	-	121
M20	T1	C25+FA60+MK15	25	60	15	123	290	73

Table 4: Compressive Strengths of M20 grade of optimally blended SCC

Grade of SCC Mix	Mix No	Mix Designation (Values indicate percentage by weight of total powder)	Compressive Strength (MPa)			
			3 Days	7 Days	28 Days	60 days
M20	C1	C100	10.07	16.36	25.18	25.34
M20	B1	C50+FA50	5.78	11.60	20.29	28.98
M20	B2	C75+MK25	18.54	20.16	33.77	34.35
M20	T1	C25+FA60+MK15	15.67	19.88	27.11	33.29

Table 5: Flow Properties of M20grade of optimally blended SCC

Grade of SCC Mix	Mix No	Mix Designation (Values indicate percentage by weight of total powder)	Slum Flow		V Funnel		L Box
			Slump Diameter mm	T-50 Sec	T-0 min	T-5 min	Blocking Ratio H2/H1
M20	C1	C100	707	3.22	6.50	8.31	096
M20	B1	C50+FA50	740	3.19	6.29	7.77	0.96
M20	B2	C75+MK25	660	4.43	7.94	15	0.87
M20	T1	C25+FA60+MK15	712	3.63	6.74	8.20	0.90

Binary (60%), Ternary (70%) with metakolin blended SCC mixes are superior because of the reduction in the cement content quality and increase in wastage (Fly ash)

Table 6: Strengths of M20grade of blended SCC mix made with optimum percentage replacement of River Sand with GBFS:

Grade of SCC Mix	Mix No	Mix Designation (Values indicate percentage by weight of total powder)	Optimum percentage Replacement of Natural Sand with GBFS	Compressive Strength(MPa)
M20	C1	C100+GBFS50	50	26.90
M20	B1	C50+FA50+ GBFS60	60	33.15
M20	B2	C75+MK25+ GBFS60	60	34.56
M20	T1	C25+FA60+MK15+GBFS70	70	37.89

3. DISCUSSIONS

The amount of total powder that is (OPC+FA) computed is 447kg/cu.m computed total weight of fly ash is 257kg/m3 (57% of total powder). For the above powder content, flow properties are not achieved as per EFNARC guidelines, so several trial mixes are carried out to satisfy the flow properties .Table-1 presents the quantities per 1 cu.m for M20 grade SCC obtained using Nan Su method of Mix Design The final powder quantities for Standard grade (M20) SCC mix are arrived at after several trail mixes conforming EFNARC specifications for the required fresh properties as shown in Table-2.

Total amount of powder quantity (Cement+pozzolanic)mixture adopted for standard Grade From these final quantities, for various percentage replacement of cement by FA, MK and their combinations are tried to optimize the quantities for ternary blended SCC mixes of grades considered. The percentage replacements, their corresponding desired strengths are tabulated in Table-3.

For M20 grade SCC made with 100%OPC, EFNARC flow specifications and target compressive strength at 60 days can be accomplished. Equivalent compressive strengths can be achieved at 60 days for FA based ternary blended SCC systems. In ternary blended M20 grade SCC system, for OPC+FA and OPC+FA+MK combinations, compressive strengths comparable to that of 60 days target strength of 100% OPC high strength grade SCC can be achieved. For all the above ternary combinations of SCMs, EFNARC flow specifications are satisfied.

Compressive strengths are achieved early in Metakaolin based ternary blended SCC. Due to synergy effect, the interaction of two or more admixtures has a greater effect than the sum of their individual effects. The findings of the present work endorses the recommendation that use of Metakaolin in fly ash based SCC enhances both strength and replacement percentages of cement by mineral admixtures found to be cost effective in terms of less cement usage, increased use of fly ash and also plays a major role in early strength development of FA based SCC Metakaolin based ternary blended SCC reduces the setting times and imparts early strength when compared to reference mix ternary blend SCC with Fly ash.

4. CONCLUSIONS

1. Binary (60%), Ternary (70%) with metakolin blended SCC mixes are superior because of the reduction in the cement content quality and increase in wastage (Fly ash)
2. Fresh properties of SCC replacing upto 60% of sand with GBFS were found to be good
3. Fresh properties of concrete mix decreases above 60% replacement of sand with GBFS
4. Fine particles of GBFS of 150µ and 75µ are more hence 100% replacement of GBFS is not possible
5. Terenary blended SCC is superior then binary blended scc
6. It is advisable to mix at least 30*40% river sand with GBFS

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EVALUATION OF CEMENTING EFFICIENCY OF TERNARY BLENDED HIGH STRENGTH SELF-COMPACTING CONCRETE

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Abstract

The utilization of Supplementary Cementitious Materials (SCM) is well accepted because of the several improvements possible in the concrete composites, and due to the overall economy. From the present investigations it is found that the compressive strength of metakaolin based high strength Self Compacting Concrete (SCC) depends both on the age and the percentages of replacement levels of Metakaolin, and Fly ash combination. The proposed work attempts to quantify the strength of Metakaolin, and Fly ash combination at the various optimum replacement levels and evaluate their efficiencies in SCC. The present work proposes to report the results of an experimental study conducted to evaluate the synergic effects of Metakaolin, and Fly ash combination compressive strength of hardened ternary blended high strength SCC of M40grades at different ages. The strength efficiency factor 'k' are evaluated using Bolomey's empirical expressions which are frequently used to predict the strength of concrete theoretically. The overall strength efficiency was found to be a combination of general efficiency factor, depending on the age and a percentage efficiency factor, depending upon the percentage of replacement of Metakaolin and Fly ash combination. The experimental work on Metakaolin (MK) concrete found that the computed efficiency factor varied with pozzolan type, replacement level and age at 28 days. The value of k ranges from 1.4 to 1.82 for Fly ash for 7 days. 1.56 to 1.87 for 14 days and 1.88 to 1.97 for 28 days in ternary blended SCC. For MK & FA are 1.29 for 7 days, 1.27 for 14 days and 1.05 for 28 days.

***Keywords**: self-compacting concrete, Metakaolin, Fly Ash, Efficiency factor, Ternary Blended SCC.*

1 Introduction

The Extensive research work for decades is in progress throughout the globe in concrete technology in finding alternative materials which can partially or fully replace ordinary Portland cement (OPC) and can also meet the requirements of strength and durability aspects. Amongst the many alternative materials tried as partial cement replacement materials, the strength, workability and durability performance of industrial by-products like fly ash (FA), blast furnace slag, metakaolin, rice husk ash, etc., now termed as Supplementary cementitious materials (SCMs) are quite promising. Subsequently, these have led to the development of ternary concretes depending on the number of SCMs and their combinations used as partial cement replacement materials. The use of appropriately proportioned ternary blended concretes allows the effect of one SCM to compensate for the inherent shortening of another. The one of the main objectives of this research was to investigate synergistic action of ternary self-compacting concretes (SCC) on rheological properties, strength and their cementing efficiencies in SCC. Several research studies have reported on the performance of Rice Husk Ash (RHA) blended SCC. However, very limited research information was reported on the synergic action of Metakaolin and fly ash in SCC for high performance concretes. The present investigations are aimed to study the cementing efficiencies of FA and MK in ternary blended SCCs.

2. RESEARCH SIGNIFICANCE

Research work till date suggests that Metakaolin (MK), improve many of the performance characteristics of the SCC such as strength, workability, permeability, and durability. From the present investigations it is found that the compressive strength of MK based SCC depends both on the age and the percentage replacement level. It is felt that efficiency concept can be used to understand the behavior of MK in SCC. In the present studies, Nan Su mix design method is adopted to determine the quantities of materials in kg per cu.m for standard strength grade (M40) of Self Compacting Concrete (SCC). This paper presents a study on the behavior of FA and MK in SCC performance by evaluating the efficiency of MK at different percentages of replacement for standard grade (M40) grades in terms of efficiency factor “k”. The strength efficiency factors for MK, MK+FA, dosages (in terms of percentage replacement) in SCC Mixes are computed based on the compressive strength of OPC only SCC mixes.

3. BOLOMEY’S EMPIRICAL EXPRESSION

The Bolomey’s empirical expression frequently used to predict the strength of concrete is theoretically well founded when applied to hardened concrete. Efficiency factors found from this strength equation are used to describe the effect of the MK replacement. Efficiency factors are generally used to describe the impact of MK and FA replacement on the compressive strength of SCC Mixes.

The Bolomey’s equation is:

$S = A\,[(c/w)] + B$ ------ (1)

‘S’ is the compressive strength in MPa, ‘c’ is the cement content in kg / m3, ‘w’ is the water content in kg/m3 and ‘k’ denotes efficiency factor. By knowing the amounts of ‘c’, ‘w’ and the strength ‘S’ achieved for each slag dosage from the finally arrived experimental values, efficiency factor ‘k’ has been computed for each of the dosages. Equation (1) has been shown to practically reduce to following two equations

$S = A\,[(C/W) – 0.5]$ ...... (2)

$S = A[(C/W) + 0.5]$ ...... (3)

These two equations represent two ranges of concrete strengths and it is due to the often observed fact that a change in slope occurs at about w/p = 0.25, when P/W (powder-water ratio) is plotted against strength. However, it is found that the equation (2) is useful for most of the present day concretes when an analysis was done on test results available and the extensive data published by Larrard who also mentions this equation in his famous book, on 'Concrete Mix Proportioning – A scientific approach’. Therefore, equation (2) can be generally used for re-proportioning. The value of constant ‘A’ can be found out for the given concrete ingredients, by considering a concrete mix of any w/c ratio.

3.1 Cementing efficiency factor, k

This factor describes the mineral admixture’s ability to act as cementing material recognizing that mineral admixture’s contribution to concrete strength which comes mainly from its ability to react with free calcium hydroxide produced during cement hydration. The rate of this reaction, called as pozzolanic reaction (PR), when compared to cement hydration rate (CHR) determines the value of k. When k=1, both PR and CHR would be same and the water-binder ratios of concretes with and without mineral admixture could be almost same. When k<1, PR would be slower than CHR and for equal strengths, the water-binder ratio of concrete with mineral admixture need to be less than that of concrete without mineral admixture. When k>1, PR would be faster than CHR and for equal strengths, the water-binder ratio of concrete with mineral admixture would to be more than that of concrete without mineral admixture.

4. EXPERIMENTAL INVESTIGATIONS

4.1 Determination of Quantities of materials for SCC mixes

Based on Nan Su mix design method, quantities required for 1 cu.m are evaluated for standard grade (M40) of ternary blended Self Compacting Concrete (SCC) made with SCMs such as Fly Ash (FA), and Metakaolin(MK). Final quantities, for all SCC mixes considered, are assumed after several trial mixes on quantities computed using Nan Su mix design method subjected to satisfaction of EFNARC flow properties. This phase identifies the optimum proportions of fly ash and metakaolin in ternary blended SCC for enhanced performance of SCC at all ages.

The following are the quantities of materials calculated using Nan Su mix design method for standard grade (M40) based Self Compacting Concrete (SCC) and also presented the final quantities of materials after various trial mixes.

Table-1: Quantities per 1 cu.m for standard (M40) grade SCC obtained using Nan Su method of Mix Design

Grade of SCC Mix		Cement	Pozzolana (Fly ash)	Fine aggregate	Coarse aggregate	S.P.	Water
M40	Quantity kg/m^3	344	180	891	738	9.43L	190.35L

The amount of total powder (i.e., OPC+FA) computed is 531 kg/m3 and computed total weight of fly ash is 133 kg/m3and weight of metakaolin is 53 kg/m3(35% of total powder). For the above powder content, flow properties are not achieved as per EFNARC guidelines, so several trail mixes were carried out of satisfy the flow properties. The final SCC mix proportions shown in Table 1 are arrived at after several trail mixes by adjusting Cement, Pozzolan (fly ash) and super plasticizer till the mix conforms to EFNARC specifications for the required fresh properties.

Table-2: Final Quantities per 1 cu.m for high strength (M40) grade SCC mix after trail mixes

Grade of SCC Mix		Cement	Pozzolana (Fly ash)	Fine aggregate	Coarse aggregate	S.P.	Water	Water/Powder Ratio
M40	Quantity kg/m^3	317	214	891	786	9.34L	185L	0.35

Table 3: At various replacement of Percentage of Fly ash

MIX No.	B-1	B-2	B-3	B-4	B-5	B-6	B-7
Fly Ash % (bwc)	0	10	15	20	25	30	35
Cement (Kg)	531	478	451	425	398	371	345
Water (Kg)	198	198	198	198	198	198	198
Fly Ash (Kg)	0	53	79	106	133	159	186

For the above powder content, compress strength values are maximum for B-5 sample, so considering the same sample and Metakaolin is replaced for further studies.

Table 4:At various replacement percentage of Fly ash and Metakaolin (10%)
2.0 Project Significance

Mix No.	B-5
Fly ash %	25
Metakaolin %	10
Cement KG	345
Fly ash	133
Metakaolin	53
Water	198

Table 5: At various replacement percentage of Fly ash and Metakaolin (10%)

Mix No.	% FA Replacement	% MK Replacement Compressive Strength			Efficiency Factor		
		7	14	28	7	14	28
A5	25	32	51	67	0.99	1.19	0.99

5. DISCUSSIONS

The amount of total powder that is (OPC+FA+MK) computed is 531kg/cu.m and computed total weight of FA is 133 kg/cum. weight of MK is 53 kg/cum. (35% of total powder). For the above powder content, flow properties are not achieved as per EFNARC guidelines, so several trial mixes are carried out to satisfy the flow properties Table-1 presents the quantities per 1 cu.m for M40 grade SCC obtained using Nan Su method of Mix Design The final powder quantities for Standard grade (M40) SCC mix are arrived at after several trail mixes conforming EFNARC specifications for the required fresh properties as shown in Table-2.

Total amount of powder quantity (Cement+ pozzolanic) mixture adopted for standard grade from these final quantities, for various percentage replacement of cement by FA, MK and their combinations are tried to optimize the quantities for ternary blended SCC mixes of grades considered. The percentage replacements, their corresponding desired strengths are tabulated in Table-3.

For M40 grade SCC made with 100%OPC, EFNARC flow specifications and target compressive strength at 28 days can be accomplished. Equivalent compressive strengths can be achieved at 28 days for FA based ternary blended SCC systems. In ternary blended M40 grade SCC system, for OPC+FA and OPC+FA+MK combinations, compressive strengths comparable to that of 28 days target strength of 100% OPC high strength grade SCC can be achieved. For all the above ternary combinations of SCMs, EFNARC flow specifications are satisfied.

Compressive strengths are achieved early in Metakaolin based ternary blended SCC. Due to synergy effect, the interaction of two or more admixtures have a greater effect than the sum of their individual effects. For calculating the efficiency of Metakaolin, and fly ash combination in ternary blended SCC, an equation on the principle of Bolomey's equation has been considered for predicting the strength of concrete containing mineral admixtures. The efficiency factors evaluated can be used for proportioning of blended SCC.
In ternary blended SCC compressive strength of FA and MK based SCC mixes, K values are 1.29 for 7 days, 1.27 for 14 days and 1.05 for 28 days, which means that in a given SCC mix 1KG of FA or 1KG of MK based pozzolanic materials may replace 35% of cement without impairing the compressive strength. This may be valid, provided that the water content is kept constant. Bolomey's coefficients 'A' are calculated from the control mixes. Using computed 'A' value, calculate strength efficiency factors k at all ages for all percentage replacement levels of MK and FA combination in SCC.
It is observed that the efficiency factor K for fly ash is 1.97 for 28 days and it is observed that the efficiency factor K for metakaolin is 1.05 for 28 days. The Metakaolin blended SCC mix reduce the amount of cement and increases the quantity of fly ash compare to binary blended SCC mix.

6. CONCLUSIONS

1. Compressive strengths are achieved early in Metakaolin based ternary blended SCC.
2. For calculating the efficiency of Metakaolin and fly ash combination in ternary blended SCC, an equation has been proposed by Bolomey's equation for predicting the strength of concrete containing mineral admixtures.
3. In ternary blended SCC compressive strength of MK based SCC mixes, K is in the range of 1.29 to 1.05, which means that in each SCC mix 1KG of MK based pozzolanic materials may replace 1.29 kg of cement without impairing the compressive strength. This may be valid, provided that the water content is kept constant. Bolomey's coefficients 'A' are calculated from the control mixes. Using computed 'A' value, calculate strength efficiency factors k at all ages for all percentage replacement levels of MK+FA, MK and FA combination in SCC.
4. Ternary blended mix is found to be more efficient because of high usage of waste byproduct FA (35%) with equal dosage of MK
5. Due to substantial saving in quantity of cement, ternary and quaternary blended MK, FA based SCC is considered as Green High-Performance Concrete, because cement was replacing to maximum level with improvement of the performance properties. Therefore, ternary blended SCC reduces environmental and helps in sustainable development
6. The findings of the present work endorse the recommendation that use of Metakaolin in fly ash based SCC enhances both strength and replacement percentages of cement by mineral admixtures found to be cost effective in terms of less cement usage, increased use of fly ash and also plays a major role in early strength development of FA based SCC
7. Metakaolin based ternary blended SCC reduces the setting times and imparts early strength when compared to reference mix ternary blend SCC with Fly ash.
8. The contribution of MK to the blended SCC mix give early strength.
9. By incorporating MK deduction in cement quantity and increases in use of industrial waste by product without compromising the performance and quality.
10. The addition of MK to binary blended SCC mixes will enhance early hydration because of its high reactivity. The high reactivity of MK is due to its glassy nature.

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STRENGTH APPRAISAL OF FIBRE REINFORCED CONCRETE WITH PARTIAL REPLACEMENT OF OPC WITH MINERAL ADMIXTURES (FLY ASH, GGBS, METAKAOLIN)

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Abstract

Concrete consumption has becomes multi-fold over last few decades, and such usage of concrete has increased on large scale world over. Concrete ingredients used are becoming more costly day by day and also demand for the same is increasing widely all over. The present experimental investigation is aimed to compare the glass fibre reinforced concrete on partial replacement opc with mineral admixtures (Fly ash, GGBS &Metakaolin) with concrete grade of M30. The mix proportioning for M30 grade was done according to the IS:10262-2009. The work is focused on replacing the opc i.e. Fly ash20%+GGBS20%, Fly ash20%+Metakaolin20%, Matakaolin20%+GGBS20% by mineral admixtures with and without Alkali Resistant (AR) glass fibres0.5%, 1.0% content. And also maintaining less water-cement ratio used water reducing admixture (super plasticizer). The maximum compressive strength and split tensile strength of concrete is observed for the mix with opc replaced by mineral admixtures Fly ash20%+GGBS20% @1.0% glass fibres(Mix M2) is increased by 2.55% and 9.11%. The split tensile strength of concrete mix is increased by 8.01% and31.33%. when compared with compressive strength and split tensile strength of control concrete at 7 and 28 days respectively.

***Keywords**: Alkali Resistant(AR)glass fibres, Workability,Pozzolana, Fly Ash, GGBS, Metakaolin, Super Plasticizer, Compressive Strength, Split Tensile Strength.*

1 Introduction

Concrete is the most widely used man-made construction material in the world. It is obtained by mixing cementitious materials, water, aggregate and sometimes admixtures in required proportions. Fresh concrete or plastic concrete is freshly mixed material which can be moulded into any shape hardens into a rock-like mass known as concrete. The hardening is because of chemical reaction between water and cement, which continues for long period leading to stronger with age. The utility and elegance as well as the durability of concrete structures, built during the first half of the last century with ordinary Portland cement (OPC) and plain round bars of mild steel, the easy availability of the constituent materials (whatever may be their qualities) of concrete and the knowledge that virtually any combination of the constituents leads to a mass of concrete have bred contempt. Strength was emphasized without a thought on the durability of structures.

FIBRE REINFORCED CONCRETE

Concrete is strong in compression and weak in tension. Concrete is brittle and will crack with the application of increasing tensile force. Once concrete cracks it can no longer carry tensile loads. In order to make concrete capable of carrying tension at strains greater than those at which cracking initiates, it is necessary to increase the tensile strength. To increase the tensile and flexural strength, fibres are added in concrete. The addition of fibres to concrete will result in a composite material that has properties different from those of un-reinforced concrete. The extent of this variation depends not only on the type of fibres, but also on the fibre dosage. The incorporation of fibres into a brittle concrete can have the effect of controlling the growth and propagation of micro cracks as the tensile strain in the concrete increases. Care is needed in using fibre as additive in concrete. The use of fibres in concrete has increased with the development of fast-track construction. In fact, nearly 65 per cent of the fibres produced worldwide is currently used in concrete. It offers increasing toughness and ductility, tighter crack control and improved load-carrying capacity. Different types of fibres are available in the market for reinforcing concrete and they are: steel, glass, acrylic, aramid, carbon, nylon, polyester, polyethylene, polypropylene, etc..

GLASS FIBER REINFORCED CONCRETE

Glass fibre–reinforced concrete is (GFRC) basically a concrete composition which is composed of material like cement, sand, water, and admixtures, in which short length discrete glass fibers are dispersed. Inclusion of these fibres in these composite resultsin improved tensile strength andimpact strength of the material.GFRC has been used for a period of 30 years in several construction elements but at that time it was not so popular, mainly in non-structural ones, like facingpanels (about 80% of the GRC production), usedin piping for sanitation network systems, decorativenon-recoverable formwork, and other products.At the beginning age of the GFRC development, one of the most considerable problems was the durability of the glass fiber, which becomes more brittle with time, due to the alkalinity of the cement mortar. After some research, significant. Improvement have been made, andpresently, the problem is practically solved with the new types of alkali-resistant (AR resistance) glass fibers and with mortar additives that prevent the processes that lead to the embrittlement of GFRC.

Improvement in Concrete Properties by Glass Fibers

- Compressive strength –Increased about 20 – 30%
- Tensile strength–It is improved compared to conventional concrete
- Flexural Strength–Increased about 25 – 30%
- Split Strength –Increased up to 25 – 30%

Applications of fiber reinforced concrete

1. Runway, Aircraft Parking, and Pavements
2. Tunnel Lining and Slope Stabilization
3. Thin Shell, Walls,4444 Pipes, and Manholes
4. Dams and Hydraulic Structure

2.EXPERIMENTAL PROGRAMME

MATERIAL PROPERTIES

CEMENT: Normal consistency-32%, Specific gravity-3.15, Compressive strength-3,7,28 days 25.3MPa,36.8MPa,52.5 MPa,Fineness-1.3%, Soundness-1.5mm, Initial setting time-90min, Final setting time-270min

FINE AGGREGATE,COARSE AGGREGATE-Specific gravity-FA 2.85,CA-2.85, Bulk density loose-FA-1728kg/m^3 CA -1544kg/m^3,Bulk density compacted -FA-1805kg/m^3, CA-1605kg/m^3 ,Water absorption –FA- 1.05% ,CA –NIL, Fineness modulus-FA-2.64, CA-7.62

SUPER-PLASTICIZER (MasterRheobuild920SH)- State – Liquid,Color - Dark Brown,Density - 1.20,Chloride content - 0.074,Chemical Name - Naphthalene formaldehyde polymers,P^H - 8.40,Dry material content - 39.36

Alkali Resistant (AR) Glass Fibers-Appearance –White, Size-18mm, Diameter - 14µm,Aspect ratio - 857:1, Density - 2.7 g/cm^3 ,Tensile strength - 1700 MPa,Elastic modulus -72 GPa, Elongation at break - 2.3%, Zirconia content - 16.7%, Melting point- 1450^{0}C, Refractive index - 1.561

Fly ash – Color- Dark gray, Bulk density -1041kg/m^3 , Sp. Gravity-2.1, Fineness-336 m^2/kg
GGBS – Color-Off white, Bulk density- 1280kg/m^3 , Sp. Gravity- 2.8, Fineness-340 m^2/kg
Metakaolin – Color-Off white, Bulk density- 785kg/m^3 , Sp. Gravity- 2.7, Fineness -356 m^2/kg

Table1: CHEMICAL PROPERTI44E4S OF FA,GGBS,MK,ARGF

PROPERTY	**FLY ASH**	**GGBS**	**METAKAOLIN**	**AR GLASS FIBERS**
SiO_2	61.5%	37.73%	52.86%	54.88%
Al_2O_3	21.80%	14.42%	44.10%	15.38%
Fe_2O_3	8.50%	1.11%	0.45%	10.54%
CaO	2.68%	37.74%	0.28%	8.39%
MgO	0.6%	8.71%	0.20%	4.9%
LOI	1.1%	1.41%	0.85%	-

3.TEST RESULTS

Table3: Compressive, Split tensile strength values of various mixes

MIX	**COMPRESSIVE STRENGTH N/mm^2**		**SPLIT TENSILE STRENGTH N/mm^2**		**Slump value (mm)**
	7 days	**28 days**	**7days**	**28days**	
C1	24.5	36.29	2.58	2.92	32
A1	26.44	41.02	2.80	3.19	30
A2	28.21	44.12	3.083	3.731	31
M1	26.04	46.81	3.22	4.410	45
M2	28.93	48.14	3.330	4.90	42
M3	25.45	46.5	3.194	4.378	43
M4	28.84	46.62	3.325	4.486	39
M5	26.23	45.7	3.168	4.163	40
M6	28.83	46.52	3.24	4.593	38

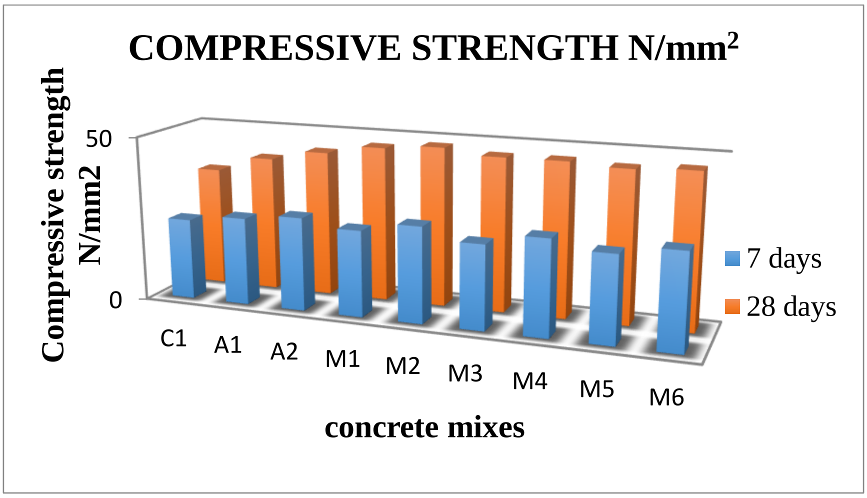

Fig 1:7,28-day Compressive strength of concrete with different mixes

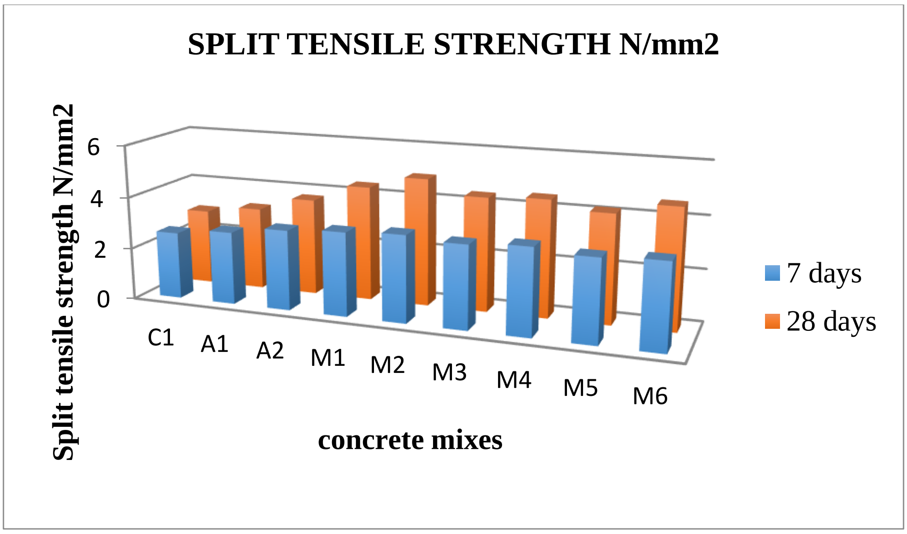

Fig 2:7,28-day Split tensile strength of concrete with different mixes

The nine different types of concrete mixes are as described below

C1 (Conventional mix): OPC100% + F.A + C.A

A1: OPC100% + F.A + C.A + Fibers @ 0.5% by weight of binder
A2: OPC100% + F.A+ C.A + Fibers @ 1.0% by weight of binder

Mixes with cement replaced Mineral Admixtures by (Flyash + GGBS +Metakaolin)

M1: OPC60% + 20%Flyash + 20%GGBS + F.A + C.A +Fibers @0.5% by weight of binder
M2: OPC60% + 20%Flyash + 20%GGBS + F.A+C.A +Fibers @1.0% by weight of binder
M3: OPC60% + 20%Flyash + 20%Metakaolin + F.A+C.A +Fibers @0.5% by weight of binder
M4: OPC60% + 20%Flyash + 20% Metakaolin + F.A+C.A +Fibers @1.0% by weight of binder
M5: OPC60% + 20% Metakaolin + 20% GGBS + F.A+C.A +Fibers @0.5% by weight of binder
M6: OPC60% + 20% Metakaolin + 20% GGBS+ F.A+C.A +Fibers @1.0% by weight of binder

4. DISCUSSIONS

For conventional mixes with fiber content

- As the Conventional concretemix(C1),glass fibers are added mixes (A1,A2)withrespect to the @0.5%,1.0% compressive strength is increases by 7.91% & 15.14% for 7 days respectively.
- As the Conventional concretemix(C1),glass fibers are added mixes (A1,A2) with respect to the @0.5%,1.0% compressive strength is increases by 13.03% & 21.57% for 28 days respectively.
- As the Conventional concrete mix(C1),glass fibers are added mixes (A1,A2) with respect to the @0.5%,1.0% split tensile strength is increases by 8.52% & 19.49% for 7 days respectively.013852002015641
- As the Conventional concrete mix(C1),glass fibers are added mixes (A1,A2) with respect to the @0.5%,1.0% split tensile strength is increases by 9.24% & 27.73% for 28 days respectively.

For mixes partial replacement of opc by mineral admixtures(Flyash, GGBS, Metakaolin) with glass fibers@0.5%,1.0% respectively

- As compare the opc (A1) and replacement by mineral admixturesFlyash+20% of GGBS (M1), 20% of Flyash + 20% of Metakaolin (M3) & 20% of Metakaolin +20% of GGBS (M5**)** including fiber content added @0.5% by weight of binder compressive strength is decreases by 1.52%, 3.75%, 0.8% and split tensile strength increases by 15%, 14.07%, 13.14% for 7 days respectively.
- As compare the opc (A1) and replacement by mineral admixturesFlyash+ 20% of GGBS (M1), 20% of Flyash + 20% of Metakaolin (M3) & 20% of Metakaolin +20% of GGBS (M5)including fiber content added @0.5% by weight of binder compressive strength is increases by 14.11%, 13.35%, 11.4% and split tensile strength increases by 38.24%, 37.24%, 30.5% for 28 days respectively.
- As compare the opc (A2) and replacement by mineral admixturesFlyash+ 20% of GGBS (M2), 20% of Flyash + 20% of Metakaolin (M4) & 20% of Metakaolin +20% of GGBS (M6)including fiber content added @1.0% by weight of binder compressive strength is increases by 2.55%, 2.23%, 2.19% and split tensile strength increases by 8.01%, 7.84%, 5.87% for 7 days respectively.
- As compare the opc (A2) and replacement by mineral admixturesFlyash+20% of GGBS (M2), 20% of Flyash + 20% of Metakaolin (M4) & 20% of Metakaolin
- +20% of GGBS (M6)including fiber content added @1.0% by weight of binder compressive strength is increases by 9.11%, 5.66%, 5.43% and split tensile strength increases by 31.33%, 20.23%, 23.1% for 28 days respectively.

Table3: Cost analysis for all the mixes (kgs)

Mix	C1	A1	A2	M1	M2	M3	M4	M5	M6	Actual cost
Cement	425	425	425	255	255	255	255	255	255	50
Cost	2975	2975	2975	1785	1785	1785	1785	1785	1785	350
F.A	740	740	740	740	740	740	740	740	740	45
Cost	2137	2137	2137	2137	2137	2137	2137	2137	2137	130
C.A	1350	1350	1350	1350	1350	1350	1350	1350	1350	60
Cost	2025	2025	2025	2025	2025	2025	2025	2025	2025	90
Fly ash	-	-	-	85	85	85	85	-	-	80
Cost	-	-	-	121.42	121.42	121.4	121.4	-	-	85
MK	-	-	-	-	-	85	85	85	85	70
Cost	-	-	-	-	-	121.4	121.4	121.4	121.42	100
GGBS	-	-	-	85	85	-	-	85	85	70
Cost	-	-	-	109.28	109.28	-	-	109.28	109.28	90
Fiber @0.5%	-	2.125	-	2.125	-	2.125	-	2.125	-	0.5
Cost	-	1700	-	1700	-	1700	-	1700	-	800
GF @1.0%	-	-	4.25	-	4.25	-	4.25	-	4.25	1
Cost	-	-	3400	-	3400	-	3400	-	3400	800
Total cost	7137.77	8787.77	10487.77	7847.365	9547.365	7859.5	9559.5	7878.475	9578.475	1645
% Difference	-	23.11%	46.93%	10.71%	8.96%	10.57%	8.86%	10.35%	8.68%	

COST ANALYSIS
FOR MIXES WITH FIBER CONTENT @0.5%

- For mix with fibres added @0.5% of weight of binder to conventional concrete(C1) cost increased by 23.11%
- As the opc(A1) is replaced by mineral admixtures 20%Fly Ash+20%GGBS (Mix M1) cost decreases by 10.71%
- As the opc (A1) is replaced by mineral admixtures 20%Fly Ash +20%Metakaolin (Mix M3) cost decreases by 10.57%
- As the opc(A1) is replaced by mineral admixtures 20%Metakaolin+20%GGBS (Mix M5) cost decreases by 10.35%

FOR MIXES WITH FIBER CONTENT @1.0%

- For mix with fibres added @1.0% of weight of binder to conventional concrete(C1) cost increased by 46.93%
- As the opc (A2) is replaced by mineral admixtures 20%Fly Ash+20%GGBS (Mix M2) cost decreases by 8.96%

- As the opc (A2) is replaced by mineral admixtures 20%Fly Ash+20%Metakaolin (Mix M4) cost decreases by 8.86%

As the opc (A2) is replaced by mineral admixtures 20%Metakaolin+20%GGBS (Mix M6) cost decreases by 8.68%

5. CONCLUSIONS

- Based on the research studies the following conclusions can be made:
- It has been observed that the workability of concrete decreases with the addition of mineral admixtures and further decreasing by adding glass fibers. However, requisite workability has been achieved by using super – plasticizers.
- The glass fibers are added in the wet concrete i.e. after adding water to the dry mix, otherwise it will stick to the surface of mixer. Care should be taken while mixing the glass fibers with concrete.it should be not allowed to mix more than 2 minutes, otherwise it will segregate.
- To compare the compressive strength of conventional concrete with adding glass fibers and opc replaced by mineral admixtures concrete mix of compressive strength is decreases of at the 7 days because pozzolans obtained early strength is very less.
- But 28 days compressive strength mineral admixtures concrete mix get more strength compare to conventional concrete mix because of Long term process pozzoloans are getting more strength.
- The percentage increases of compressive strength of concrete mix with replacement of opc(Mix A2) by 20% flyash + 20%GGBS @1.0% glass fibers (Mix M2)when compared with 7,28 days compressive strength of control concrete withglass fibers is observed as 2.55% , 9.11%
- The percentage increases of split tensile strength of concrete mix with replacement of opc(Mix A2) by 20% flyash + 20%GGBS @1.0% glass fibers (Mix M2)when compared with 7,28 days compressive strength of control concrete with glass fibers is observed as 8.01% ,31.33%
- It was observed that, the percentage increase in the strength of glass fibers reinforced concrete increases with the age of concrete.
- Plain concrete fails suddenly once the deflection corresponding to the ultimate flexural strength is exceeded, on the other hand, fiber-reinforced concrete continue to sustain considerable loads even at deflections considerably in excess of the fracture deflection of the plain concrete.
- It shows that the presence of fibers in the concrete acts as the crack arrestors.
- The ductility characteristics have improved with addition of glass fibers. The failure of fiber concrete is gradual as compared to that of brittle failure of plain concrete.

SCOPE FOR THE FUTURE WORK

- The research work on pozzolanic materials and fiber along with pozzolanas is still limited. But the promises a great scope for future studies.
- Percentage and actual fineness of mineral admixtures requires as partial cement replacement for good strength development.
- The benefit of using fiber is that it is non-corrosive. The strength is very good. The heat resistance power is very good which is extremely impotant of every structure.

- The partial replacement of opc with mineral admixtures gives the required strength values for the concrete construction purpose and the cost of the mineral admixtures becomes very cheap if available locally or near places.
- Mineral admixtures and glass fibers percentage increases and also observed the compressive strength and split tensile strength properties and also observe durability properties.

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EFFECT OF ELONGATION INDEX ON THE BITUMINOUS MIXES

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Abstract

The desirable properties of bituminous mix i.e. Stability, flow, voids in mineral aggregate (VMA), voids filled with bitumen (VFB) and voids in total mix (VTM) are significantly depends on size and shape of aggregates. Bituminous concrete (BC) mixes were analyzed with different concentration of elongated aggregates (0%,10%,20%,30%,40% and 50%). It has been observed that voids filled with bitumen increases with increase in concentration of elongated aggregates in bituminous mixes. Reaming properties such as stability flow, voids in mineral aggregate (VMA), voids in total mixes (VTM) has been decreases. Marshal stability tests were conducted to evaluate the performance of bituminous mixes with different concentration of elongated aggregates (0%, 10%, 20%, 30%, 40% and 50%). And the type of binder selected for bituminous concrete was Crumb rubber modified bitumen (CRMB). It was found that the elongation index should not exceed 30 % for a better performance.

I. Introduction

Natural aggregates are available from open excavation (quarry) in the form of larger rocks .These rocks are crushed into required shape and size of aggregates. While crushing the larger rocks there is chance of getting the flaky and elongated aggregates which are consider undesirable for construction of bituminous pavements. The shape of aggregates has a significant influence on the performance of flexible pavements. The flaky and elongated aggregates have tendency to break down during the compaction and subsequent vehicle movements .blending of bituminous mixes with elongated aggregates have more voids subsequently its reduces the workability .By considering all this it is very essential to study the effect of elongated index on bituminous mixes

II. OBJECTIVES OF THE STUDY

a) To study the effect of different proportions of elongated particles (0 %, 10%, 20%, 30%, 40%, and 50%) taking 5.2% as optimum bitumen content of CRMB.
b) To study the basic properties of aggregates
c) To study the basic properties of CRMB.
d) To study the properties of stability & flow by varying the percentages of elongated aggregates using Marshall stability test
e) To study the bulk density and volumetric properties (VTM, VFB, VMA) 0f Marshall Specimens.

III.ANALYSIS AND RESULTS

The experimental investigation has been done in the following manner to carry out final object of this project.

The test of results of aggregates and bitumen are given below table 1 and table 2

Table 1: Aggregates Tests

S.NO	TEST NAME	STANDARD	UNIT	RESULT	ACCEPETD LIMIT
1	LOS ANGLES ABRASION TEST	IS:2386	%	26	30
2	IMPACT VALUE	IS:2386	%	24	30
3	CRUSHING VALUE	IS:2386	%	23	30
4	STRIPPING OF AGGREGATES	IS:2386	%	97	95
5	SPECIFIC GRAVITY	IS:2386	%	2.61	2.5-3.2

Table 2: Bitumen Tests (CRMB-Crumb Rubber Modified Bitumen)

S.NO	TEST NAME	STANDARD	RESULT	LIMITS AS PER IS:15462 -2004	LIMITS AS PER IRC:SP:53 -2002
1	PENETRATION at 25 0C, 0.01 mm, 100g, 5s.	IS:1203	38.15	<50	<50
2	SOFTENING POINT (R&B),^{0}C,Min	IS:1205	60.4	60	60
3	DUCTILITY	IS:15462,Annex -A	51.7	50	50

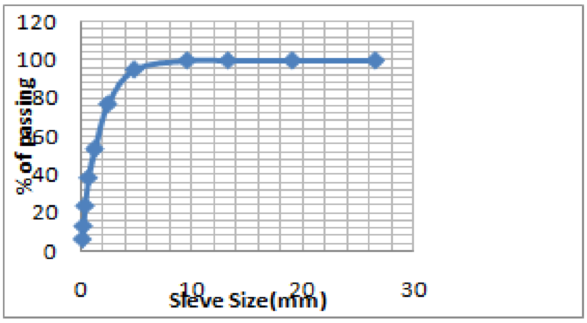
Sieve Analysis and Blending of aggregate

The test of Sieve analysis is a simple test consisting of sieving a measured quantity of material through successively smaller sieves. The weight of material retained on each sieve is weighted and expressed as a percentage of the total weight of the sample. In this study a weight of 5 kg from each portion of aggregate is taken such as Course, fine and dust, then passed through Bituminous concrete (B.C) of grade -1 gradation, sieves which starts from (26.5-0.075) mm as per the MORT&H table no 500-18. The following graphs-1, 2 & 3 contain individual sieve analysis results of different kinds of nominal sizes i.e. 20mm, 12mm and dust. After getting final results from sieve Analysis blending process is carried out by Trial and error method, in order to get the required amount by percentage of each component like course, fine and dust.

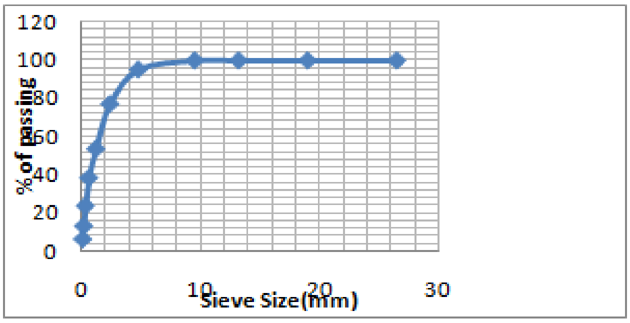
Table 3: Combined Sieve analysis Results

Is sieve (mm)	**Weight retained(kg)**	**Retained %**	**Cum Retained %**	**Cum Passing %**
26.5	0	0	0	100
19	2.424	44.42	44.42	55.58
13.2	2.594	47.535	91.855	8.145
9.5	0.395	7.238	99.093	0.807
4.75	0.026	0.476	99.56	0.44
2.36	0.018	0.329	99.898	0.102
1.18	0	0	99.9	0.1
0.6	0	0	99.9	0.1
0.3	0	0	99.9	0.1
0.15	0	0	99.9	0.1
0.075	0	0	99.9	0.1

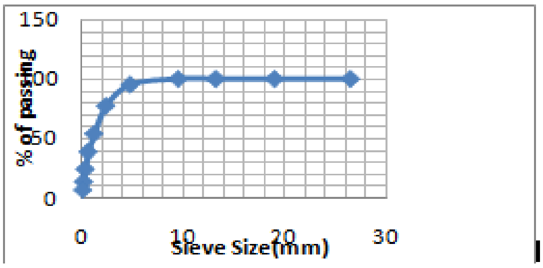
The individual results of sieve analysis are shown below with their respective graphs for different nominal size of aggregates graph (1), graph (2) and graph (3).



Graph 1: sieve size v/s % of passing for C.A



Graph 2: sieve size v/s % of passing for F.A



Graph 3: sieve size v/s % of passing for stone dust

The combination of the above 3 graphs are carried out to perform blending of the required gradation of Bituminous concrete (B.C) by trial and error method. The final composition of aggregates of different sizes and optimum bitumen content of 5.2% by weight as shown in table (4) for preparing the Marshall Stability sample

Table: 4 Aggregates of different sizes and optimum bitumen content

Sieve Size		% passing	Wt. of aggregates
26.5		100	0
26.5	19	1.22	14.7
19	13.2	21.11	253.3
13.2	9.5	11.85	142.2
9.5	4.75	23.62	283.5
4.75	2.36	7.92	95.0
2.36	1.18	10.48	125.8
1.18	0.6	6.00	72.0
0.6	0.3	5.96	71.5
0.3	0.15	4.48	53.8
0.15	0.075	3.42	41.0
Passing 0.075			47.3
Total Weight Aggregates in Grams			**1200**
Optimum Bitumen Content 5.2% by Weight in Gram			**62.4**

Preparation of Marshall Samples

A set of 3 no's Marshall Specimens were cast according to table no: 4 by varying the percentages of elongated particles from 0 to 50% with the constant bitumen content of 5.2%. Marshall Specimen's bulk density and volumetric properties were computed as per Asphalt

Institute Manual Series – 2 (MS-2) and then specimens were tested at 60^0C to find the Marshall Stability and flow

Table 5: Marshall Test Results.

% of Elongated particles	Air voids (Vv) %	Voids in Min.Agg. (VMA) %	Voids filled with Bit.(VFB) %	Density (g/cc)	Marshall Stability (KN)	Flow (mm)
0	4.33	15.23	71.56	2.34	14.75	7.97
10	3.83	14.93	74.39	2.298	13.56	6.77
20	3.60	14.82	75.69	2.281	12.01	5.63
30	3.51	14.83	76.37	2.270	10.70	3.97
40	3.35	14.78	77.31	2.273	8.92	1.70
50	2.47	14.11	82.55	2.226	7.00	0.83

IV. CONCLUSIONS

From the results that are obtained from this study it has been concluded that elongation index should not exceed 30 % for a better strength and durability based on varies recommendations like the minimum stability is 9KN, the acceptable flow limit is between 2-4, the acceptable air voids is between 3%-6%, the standard limit for VFB is between 65-75%, the VMA limit is between 12-14% for nominal maximum size of aggregate 19 mm..

V. SCOPE FOR FUTURE WORK

1) Different bituminous mix designs can be observed other than B.C, such as stone mastic asphalt (SMA), dense graded bituminous macadam (DBM)…etc.
2) Different grades and other modified binders can be investigated.
3) Individual flaky aggregates, and also combined elongated & flaky aggregates can be tested.
4) Different types and percentages of filler can be adopted.

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STUDY ON PROPERTIES OF CONSTRUCTION MATERIALS

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Abstract

This paper reports on the study of the building materials like burnt clay bricks, cement bricks and fly ash bricks, this includes the physical and mechanical properties. This study provides an essential compendium of case studies for practicing engineers, designers, researches and other practitioners who are interested in all aspects construction materials. Selection and suitability of good materials in the construction is most important task. It reduces the time and cost. This report can helps in selection of suitable building materials and testing procedures, practice in lab & site itself. And also number of bricks required for a one m^2 area.
***Keywords**: Physical properties, Mechanical Properties and Procedure*

Introduction

Building materials have an important role to play in this modern age of technology. Although their most important use is in construction activities, no field of engineering is conceivable without their use. Also, the building materials industry is an important contributor in our national economy as its output governs both the rate and the quality of construction work.
There are certain general factors which affect the choice of materials for a particular scheme. Perhaps the most important of these is the climatic background. Obviously, different materials and forms of construction have developed in different parts of the world as a result of climatic differences. Another factor is the economic aspect of the choice of materials. The rapid advance of constructional methods, the increasing introduction of mechanical tools and plants, and changes in the organisation of the building industry may appreciably influence the choice of materials.

Tests conducted on burnt clay bricks

Water absorption test

Table 1: Water absorption of burnt clay bricks
Table 2: Water absorption test of cement bricks

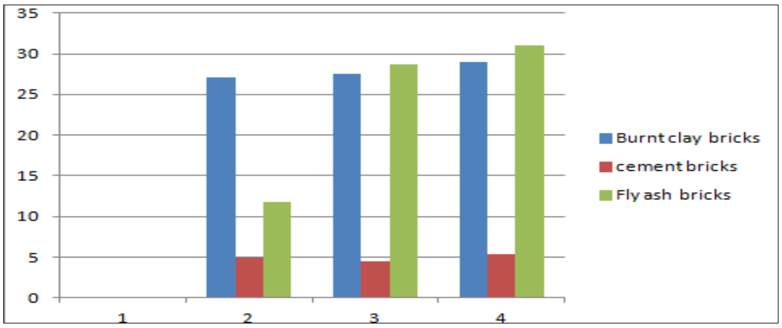
S.no	Dry weight Kg	Wet weigtht Kg	% of water absorption
1	2.47	3.14	27.12
2	2.58	3.29	27.51
3	2.48	3.20	29.03
Average absorption is 27.88 %			

S.no	Dry weight (kg)	Wet weight (kg)	Increase in weight (kg)	% of water absorption
1	13.10	13.74	0.64	4.88
2	13.86	14.47	0.61	4.40
3	13.39	14.11	0.72	5.33
Average				4.73

S.NO	DRY WEIGHT (kg)	WET WEIGHT (kg)	INCREASE IN WEIGHT (kg)	% OF WATER ABSORPTION
1	18.06	20.18	2.12	11.73
2	15.63	20.11	4.48	28.66
3	15.92	20.85	4.93	30.96
Average				23.78

Table:3 Water absorption of burnt of fly ash bricks

Water Absorption The bricks, when tested in accordance with the procedure laid in IS 3495 (Part 2) : 1992 after immersion in cold water for 24 hours, water absorption shall not be more than 20 percent by weight



- The strength of bricks decreases by about *25* per cent when soaked in water.
- Strength of sun-dried (un burnt) bricks is from 15 to 25 kg/cm^2
- Water absorption of bricks after 24 hours immersion,
- First class bricks—20%
- Second class bricks—22 %,
- Third class bricks—25%. Heavy duty machine made bricks should not absorb more than 5% of their weight.

COMPRESSION TEST

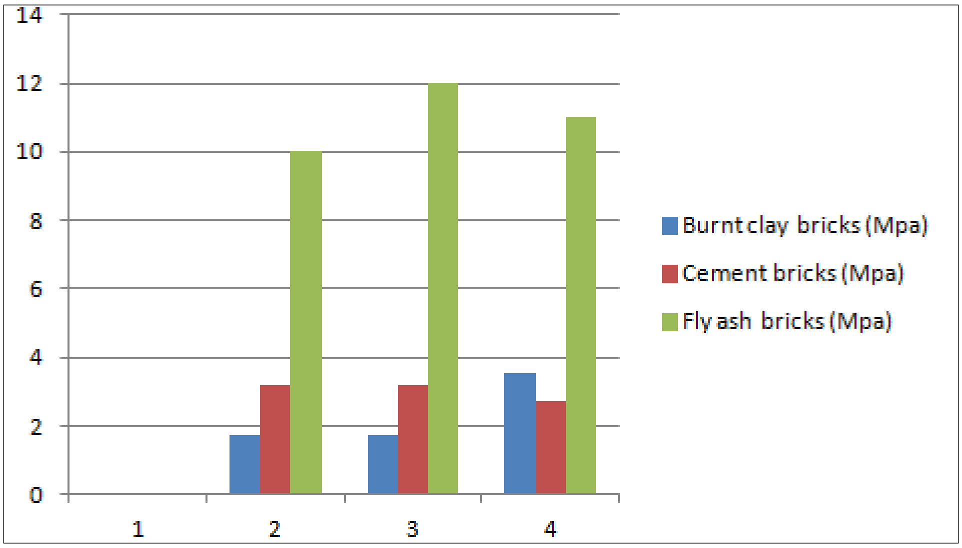
S. No	Load (kg)	Comp.Strength kg/cm^2
1	390	18.57
2	400	19.05
3	790	37.63

Average compression strength of brick = 25.09 kg/cm^2

Table4: compression test on burnt clay bricks

S.no	Load (kn)	Compressive strength (N/mm^2)
1	140	3.17
2	140	3.17
3	120	2.71
Average		3.01

Table 5:compressive strength of cement bricks



EFFLORESCENCE TEST

Result: Moderate

This test is conducted for finding out the presence of soluble salts in a brick when it is immersed in water for 24 hours and taken out and allowed to dry in shade.

Absence of grey or white deposits on its surface indicates absence of soluble salts. If the white deposits cover about 10%surface, the efflorescence is said to be slight and it is considered as moderate, when the white deposits cover about 50% of surface. If grey or white deposits are found on more than 50% of surface, the efflorescence becomes heavy and it is treated as serious, when such deposits are converted into powdery mass

The liability to efflorescence shall be reported as ‘nil’, ‘slight’, ‘moderate’, ‘heavy’ or ‘serious’ in accordance with the following definitions:

a) Nil - When there is no perceptible deposit of efflorescence.
b) Slight - When not more than 10 percent of the exposed area of the brick is covered with a thin deposit of salts.
c) Moderate - When there is a heavier deposit than under ‘slight’ and covering up to 50 percent of the exposed area of the brick surface but unaccompanied by powdering or flaking of the surface.
d) Heavy - When there is a heavy deposit of salts covering 50 percent or more of the exposed area of the brick surface but unaccompanied by powdering or flaking of the surface.

e) Serious - When there is a heavy deposit of salts accompanied bp powdering and/or flaking of the exposed surfaces.

SOUNDNESS TEST

The Soundness test for clay bricks and fly ash bricks was conducted and the results were compared in which two bricks are struck with each other, It was found that a normal brick shows good results when struck with each other but fly ash bricks show clear ringing sound.

Sample	Length(cm)	Width(cm)	Heigtht (cm)
Average	21.82	9.7	7.47

Table 6: Average sound ness test on clay bricks

DISCUSSION

For burnt clay bricks :

S no	Name of the test	Test result	As per code book	Remarks
1	Water absorption	27.8%	Not more than 20%	Above the acceptable limits
2	Compression test	25.09 kg/cm^2	35 kg/cm^2	<10 kg/cm^2 , below Class III Grade
3	Efflorescence	Moderate	Slight for top class	Good
4	Dimension test	L=218.26mm,W=97mm H =74.79mm	For modular L=190mmW=90mmH=90mm	Good
5	Internal structure	Colour difference White colour spots	uniform	Moderate

Table 7:Test Results Of Burnt Clay Bricks

For fly ash bricks:

1. Fly Ash Bricks were found to be sufficiently hard as scratching by the finger nail on the surface left no impression on it as compared to normal bricks.
2. The Efflorescence of all bricks tested were found to be slight as white or grey deposits were less than 10% on surface of the bricks which is almost same as that in the normal bricks.
3. A ringing sound in the Fly ash Bricks was observed to be far better than that in normal bricks.
4. Structure of the bricks was found to be compact, homogeneous and free from any defects like holes, lumps etc as compared to normal bricks.
5. The average absorbed moisture content of clay bricks is found to be 28% and for fly ash bricks are found to be 23.58%. Thus there is net 5% decrease in moisture 6.5absorbed for fly ash bricks as a part to clay bricks.

6. The crushing strength of clay bricks is found to be 8.14 N/mm2 and for fly ash bricks is found to be 18.81 N/mm2. Thus there is net 56.72% increase is crushing strength for fly ash bricks as a part to clay bricks.

For cement bricks:

The compressive strength of **11 × 6.5 × 6** inches cement bricks is in between the 5-7 N/mm^2. The value obtained is 3.01 N/mm^2. It is not in the limits. The water absorption is lesser than the normal bricks and fly ash bricks.

The compressive strength of cement bricks is measured as per the height of the bricks or 1 m^3 volume brick .the strength ranges from 2.8 N/mm^2 to 20 N/mm^2.

Conclusion

After analyzing the results it can be concluded FLY ASH BRICKS are the optimal material for the construction purpose which is having the highest compressive strength ,and absorbing optimum moisture content .

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COMPATIBILITY OF SULPHONATED NAPHTHALENE FORMALDEHYDE AND LIGNOSULPHONATES BASED SUPERPLASTICIZER WITH PORTLAND SLAG CEMENTS

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Abstract

At present, most of the constructions require pumpable and workable concrete. To achieve high workability, chemical admixtures like superplasticizers are required to be added to the concrete mix in fresh state. Granulated blast furnace slag (GGBS) based Portland slag cement is a promising material having less environmental impact and showing superior durability when used in concrete than ordinary Portland cement. Portland slag cements are required for marine and offshore structures, sewage treatment plants etc., to have high resistance to chlorides and sulphates. Three Brands of GGBS based Portland slag cements are considered in this study for concretes with design strengths of 25 MPa and 40 MPa. Early age strength of concrete and slump loss have been studied. To achieve high workability in the fresh state and desired characteristic compressive strength in the hardened state of the normal concretes, sulphonated naphthalene formaldehyde and lignosulphonates based chemical admixture has been incorporated. It was observed that the setting behavior varies significantly from one brand of cement to another, though the cement type and the chemical admixture are the same. Hence, it is required to study the compatibility between the superplasticizer and Portland slag cement before a suitable combination is used in concrete, especially when high workability, slump retention and early age strength are required.

Keywords: *Pumpable concrete, workability, slump-retention, superplasticizers, setting time.*

1.0 Introduction

In the construction industry pumpability and high workability of concrete play an important role especially where concrete is produced away from actual job site, for example as 'ready mixed concrete (RMC)' is produced at a batching plant. To achieve high workability in the fresh state and to have considerable slump-retention apart from cement, aggregates and water, a fourth ingredient namely a superplasticizer is introduced in the concrete. Aitcin (1998) has stated that modern concretes almost always possess additives, either in the mineral or chemical form. Particularly, chemical admixtures such as water reducers and set controllers are invariably used to enhance the properties of fresh and hardened concrete. If a particular brand of cement and superplasticizer are not compatible, it may lead to adverse effect on performance of concrete.

Common problems include flash setting, delayed setting, rapid slump loss, improper strength gain, inordinate cracking etc. These issues in turn affect the hardened properties of concrete, primarily strength and durability. The properties of blended cements like Portland slag cements (PSC) are mentioned in Tables 1 and 2. PSCs are relatively cheaper when compared with ordinary Portland cements (OPC).

Table 1 Chemical analysis of cements

S.No	Constituents determined	Percentage by mass			Specified limits as per IS 455 (1989)
		Cement A	Cement B	Cement C	Maximum Value
1	Loss in ignition(%)	0.81	1.01	1.34	5.0
2	Magnesia (Mgo)(%)	6.27	4.83	3.51	8.0
3	Insoluble residue(%)	0.99	0.84	1.34	4.0
4	Sulphuric anhydride (So_3)	1.94	2.1	1.13	3.0
5	Total chlorides (%)	0.007	0.003	0.003	0.05
6	GGBFS (%)	70	50	70	As stated by manufacturer

Table 2 Physical properties of cements

S.No	Property	Cement A	Cement B	Cement C	Specified limits as per IS 455 (1989)
1	Specific gravity	2.765	2.81	2.82	---
2	cosistency of cement(%)	28	30	31	---
3	Initial setting time(minutes)	48	56	65	30Minutes(Minimum)
4	Final setting time(minutes)	188	203	208	600Minutes (Maximum)
5	3 days strength (MPa)	17.1	16.76	17.11	16MPa(Minimum)

Ramachandran et al (1998) have stated that superplasticizers are used to enhance the properties of fresh and hardened concretes. In cement pastes addition of superplasticizer results in a decrease in viscosity, change in other rheological parameters. See the review of Jaysree et al (2011), for the mechanism of action of the superplasticizer and the interaction between cement and superplasticizer. Mehta (1997) and Neville (1997) have stated that setting times of concrete differs widely from setting times of cement. The setting time of concrete depends upon the water cement ratio, temperature conditions, type of cement, mineral admixture and superplasticizers. Janardhana et al., (2004) have studied the effect of combination of admixtures on setting times of concrete. Superplasticizing and retarding admixtures were added in different dosages to the cement concrete made up of with ordinary Portland cement. They stated that the initial setting times of concrete decrease with increase in grade of concrete and superplasticizers prolong the setting times of concrete. The setting time of concrete are determined as per IS 8142 (1976) using penetration resistance apparatus. Mortar passing through 4.75mm sieve is separated from concrete for this test. According to IS 8142 (1976), the initial setting time (IST) is the elapsed time, after initial contact of cement and water, required for the motor (sieved from the concrete) to reach a penetration resistance of 3.43 N/mm^2 and the final setting time (FST) is the elapsed time, after initial contact of cement and water, required for the mortar (sieved from the concrete) to reach a penetration resistance of 26.97 N/mm^2. For concrete mix design, the water cement ratio selected was lower than the maximum value of 0.50 for M25 and 0.40 for M40 as per IS 456 (2000).Water cement ratios of 0.45 for M25 grade concrete and 0.35 for M40 grade concrete are chosen. Two grades of concretes of compressive strengths of 25MPa and 40 MPa are considered for this study. Factory blended superplsticizer of 'SNF and lignosulphonate' based chemical admixture was mixed with fresh concrete to achieve a high workability of 150 to 160 mm slump in fresh state. Tests were conducted for evaluating workability, slump retention, compressive strengths and setting times of concrete and their results are discussed.

EXPERIMENTAL INVESTIGATIONS

Three brands of GGBFS based PSCs, conforming to IS 3812 (1981), were used in the present study and are denoted by Cement A, Cement B, Cement C. One variety of superplasticizer conforming to IS 9103 (1978) is used in the study. Chemical composition of cements was determined at the National Council for Cement and Building Materials (NCB), Hyderabad. The physical properties of cements are carried in-house at the JNTUH College of Engineering, Hyderabad. Two concrete mix design mixes as given in Table- 3 for compressive strengths 25 MPa and 40 MPa were prepared with 10-20 mm and 5-10 mm coarse aggregates, sand conforming to zone- II and with water - cement ratio 0.45and 0.35 respectively. Concrete mix design was carried out as per IS 10262 (2009). Dosage of superplastizer (percentage by weight of cement) as given in Table 4 was chosen by trial and error, for each brand of PSCs to achieve an initial slump of 150 to 160 mm and a workable slump of 50 to 60 mm after one hour. Experiments were carried for finding out the compressive strength as per IS 9013 (1978) by resorting to accelerated curing and also with normal curing. The compression test on concrete cubes was carried out as per IS 516 (1959). The 28 day compressive strength of concrete were predicted as per IS 9013 (1978) and conventional strength of concrete at 3, 7 and 28 days was determined. The initial and final setting times of concrete estimated as per the IS 8142 (1976) as shown in the Table 5.

Table 3 Mix proportion per cu.m of concrete in kgs

Brand of cement	concrete design strength (MPa)	Cement	Fine aggregate	Coarse aggregate		Water cement ratio
				10mm	20mm	
A	25	370	684.488	464.372	691.341	0.45
	40	430	582.814	491.732	732.07	0.35
B	25	370	685.08	464.5	691.53	0.45
	40	430	582.3	491.3	731.43	0.35
C	25	370	686.54	465.49	693.01	0.45
	40	430	584.51	493.16	734.2	0.35

Table 4 Dosage of chemical admixture

Brand of cement	Design compressive strength of concrete (MPa)	Dosage (percentage by weight of cement)of superplsticizer to achive about 150mm to 160mm slump
		SNF and lignosulphonated based
A	25	1.1
	40	1.25
B	25	1
	40	1.35
C	25	0.8
	40	1

Table 5 Initial and final setting times of concrete

Brand of cement	Design compressive strength of concrete (MPa)	Type of chemical admixture	
		SNF and lignosulphonated based	
		IST(Hours)	FST(Hours)
A	25	15.33	18.3
	40	12.08	16.15
B	25	10.32	13.39
	40	16.03	21.25
C	25	8.59	13.32
	40	10.36	14.16

DISCUSSION OF TEST RESULTS

It was observed that the dosage of superplasticizer was different for three brands of cement though they are of the same category (i.e., GGBFS based PSCs). It was observed that the cement C required relatively lesser dosage of superplasticizer when compared with other two brands, to achieve an initial slump of 150mm. It was also observed that slump-retention after 30 and 60 minutes varied significantly among the three brands of cements for given type of superplasticizer (corresponding results are shown in the Table 6). It was observed that slump-retention after 30 minutes for cements A and B was comparable whereas it was relatively higher for cement C. The initial and final setting times of the three types of cements are different. It indicates that the interaction with the chemical admixture influences the setting response of concrete considerably. The initial and final setting times of cement concrete with cement C are the lowest among the remaining cement concretes.

Table 6 Slump retention test of concrete with SNF and lignosulphonate based superplasticizer

Brand of cement	Design compressive strength of concrete(MPa)	Dosage of superplasticizer(%)	Slump (mm)		
			Initial	After 30 minutes	After 60 minutes
A	25	1.1	172	104	76
	40	1.25	168	101	64
B	25	1	154	93	51
	40	1.35	150	92	53
C	25	0.8	168	98	55
	40	1	150	82	51

Table 7 Compressive strengths of concrete with SNF and Lignosulphonates based superplasticizer

Brand of cement	Design compressive strength of concrete (MPa)	Pridicted strength (accelerated curing) N/mm^2	Conventional curing of concrete		
			3 day strength N/mm^2	7 day strength N/mm^2	28 day strength N/mm^2
A	25	31.65	17	23.2	32
	40	48.74	23.1	34	49.6
B	25	34.82	17.2	23	32.5
	40	49.2	22.7	32.2	48.27
C	25	34.47	18.39	24.2	34.2
	40	48.65	25.1	35	49.8

Compressive strengths of concretes are shown in Table 7 for different ages of concrete. The predicted 28 day compressive strengths of concrete, by resorting to accelerated curing, as well as the strength obtained with normal curing were also reported. By resorting to accelerated curing, it will be convenient to assess the strength of concrete within 30 hours from the time of casting. If required the concrete mix design may be revised. It was observed that the early age strengths of concrete for the three brands of cement were comparable. However, there was considerable difference in the 28 days compressive strengths between one brand of cement to another. It was observed that predicted compressive strength of concrete obtained from accelerated curing of concrete was not always comparable with that of the strengths obtained from conventional curing. Hence, it can be concluded that care should be exercised while predicting the 28 day strength of PSC concrete by adopting accelerated curing.

CONCLUSIONS

On the basis of experimental investigations and studies carried out in the present work, it is concluded that though the concrete mix proportion is approximately the same in all the mixes, the workability, compressive strengths and setting times of cement concretes are considerably different depending on the brand of the cement. The dosage of superplsticizer required is less for cement C when compared with cements A and B**.** The setting times of cements A and B are relatively higher when compared with that of cement C. The compressive strengths of concrete made with cements A and B are relatively lesser when compared with that of cement C. The reason could be requirement of higher dosages of superpalsticiser for concretes made with cements A and B. It was observed that the different brands of cements (GGBFS based PSCs) behaved differently even if the coarse and fine aggregates, water and family of chemical admixture and the method of concrete mix design were kept constant. Hence, it is essential to know the interaction of the superplasticizer with the cement, even if they are of similar kind, and trial concrete mixes have to be studied in a laboratory before actually using them at site.

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REPLACEMENT OF SAND WITH QUARRY DUST IN M30 GRADE CONCRETE

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Abstract
Every year the consumption of concrete is increasing due to rapid growth in infrastructure. Concrete is a major building material which is used in construction throughout the world. It is extremely versatile and is used for all types of structures. The present investigation aims in the study of properties of concrete in which quarry dust is used as a partial replacement for natural sand. The basic strength properties of concrete were investigated by replacing natural sand by quarry dust at replacement levels of 20%, 30%, and 50%. It is clearly observed that for 20% of quarry dust (3 days) the average compressive strength is 20.16N/mm² where as for natural sand it is 13 N/mm². For 30% quarry dust the value increased to 29.6N/mm² and for sand is 27 N/mm². Finally for 50% the strength increased to 37.76N/mm². As the properties are good as and,the quarry dust is used as fineaggregate in replacement with and in the cement concrete .This study reveals that in case of cement mortars, the natural sand can be replaced by quarry dust. The strength of concrete containing 20%, 30%, 50% mix of quarry dust proportions is much higher than normal concrete containing only sand as fine aggregate. It is better to use Quarry dust without removing the finer particles.Finally the compressive strength of quarry dust results says that the natural sand can be replaced with respect to various mix proportions (20%, 30%, and 50%).

Keywords*: Alternate building materials, Concrete, Quarry Dust, aggregates*

1.0 Introduction

Concrete is a widely used construction material consisting of cementing material, fine aggregate, coarse aggregate and required quantity of water, where in the fine aggregate is usually natural sand,. The use of sand in construction results in excessive sand mining which is objectionable .due to rapid growth in construction activity, the available sources of natural sand are getting exhausted. (Palaniraj, 2003) Also, good quality sand may have to be transported from long distance, which adds to the cost of construction .In some cases, natural sand may not be of good quality.

Therefore, it is necessary to replace natural sand in concrete by an alternate material either partially or completely without compromising the quality of concrete. (Dhir & carthy, 2000) quarry dust is one such material which can be used to replace sand as fine aggregate. The present study is aimed at utilizing quarry dust as fine aggregate in cement mortar and cement concrete, replacing natural sand. The study on mortar includes determination of compressive strength of different mortar mixes. (Nadgir & Bhavikatti, 2006) the study on concrete includes determination of compressive strength of concrete.

Experimental program

In this investigation, the compressive strengths of cement mortar are observed by replacing natural sand by quarry dust at different levels of replacement namely 20%, 30%,and 50 %.Mix design of M30 was chosen for the study. Moulds of size 150mm x150mmx150mm were used

.the compressive strength of three percentages types of mortars are obtained at age of 3days, 7days and 28days.the grade of concrete M30 is selected for the study. The study includes determination of compressive strength at the ages of 3days, 7days and 28days. (jaafer et al.,2002) the strength properties of concrete with quarry dust replacement are compared with that of normal concrete(NC)which does not contain quarry dust

Experimental procedure

Materials used

In this study, 53 grade ordinary 0portland cement conforming to is 12269 1987 is used. Natural sand belonging to zone III as per IS 383-1970 is used in this investigation.

Test procedures

Compressive strength of quarry dust mortars:

The materials required for the number of specimens were dry mixed and then mixed with calculated amount of water. the quantity of water is obtained as per IS4032-1988.it is given by percentage of water equal to (p/4 +3) percent of combined weight of cement and fine aggregate, where p is the percentage of water required to produce a cement paste of standard consistency.

While preparing the specimens for each proportion, a reference mix using cement and natural sand is prepared. This is done in order to compare quarry dust mortar with the normal mortar. For each quarry dust replacement, the total fine aggregate quantity is obtained as the combination of natural sand and quarry dust .for example, the first set of specimens consist of 20%qurrry dust and 80% of natural sand.

For each mortar mix and for each replacement level of Quarry Dust, 18 specimens were casted. The results were obtained by testing 3 specimens each at 3 days, 7 days and 28 days. The testing of specimens was carried out as per IS 4031-1988. Specimens were tested with a gradually increasing compressive load until they fail by crushing. Compressive strength of Quarry Dust mortars: (Sahu et al., 2003) the specimens were prepared by replacing sand by Quarry Dust at same levels of replacement as in natural sand mortar. The specimens were tested at the end of desired curing period to get the compressive strength. Tests on Quarry Dust concrete: The mix design of M30 grade concrete was obtained as per IS10262- 1982. The mix proportion for M30 concrete was 1:1.05:2.52 with a water-cement ratio of 0.4.
The workability of concrete was measured using slump test. For M30 grade of concrete, the slump valuesfor different Quarry Dust Concretes were maintained good and the values obtained are impressive. (Shahul Hameed & Sekar, 2009) The compressive strength is obtained as per IS-516-1959. For the compressive strength, cube specimens of size 150mm were casted. Required quantity of water was added to get a homogeneous mixture. The fresh concrete was poured into the moulds in three layers and compacted using vibrator. After 24 hours, the specimens were remolded and were kept for curing. The specimens were subjected to gradually increasing compressive load till the failure in a Compression Testing Machine of 100T capacity.

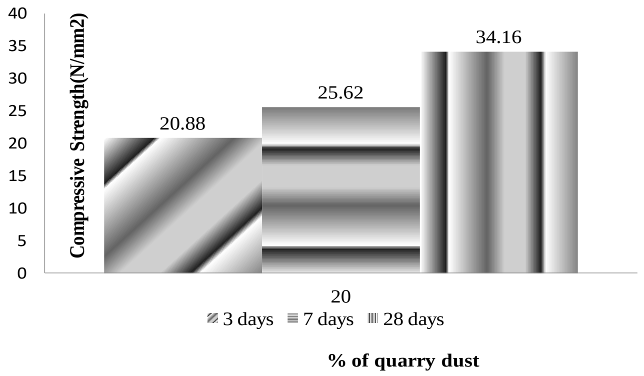

Fig 1: Compressive Strength of Natural Sand after 3days, 7days&28days

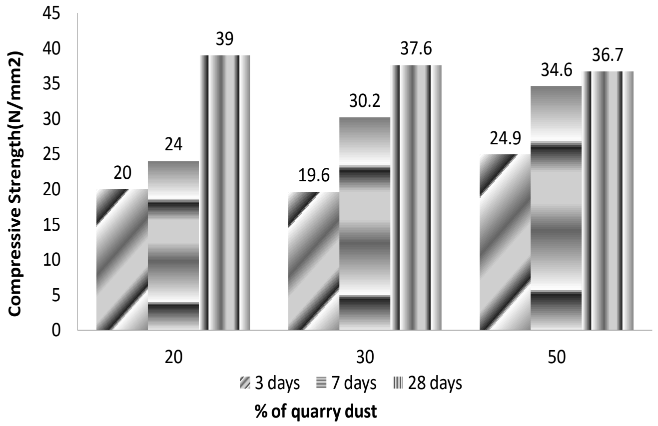

Fig 2: Compressive Strength of quarry dust after 3days, 7days& 28days

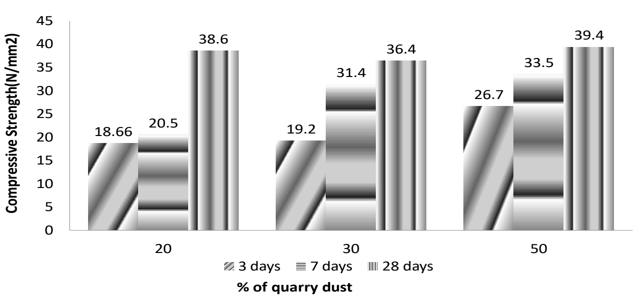

Fig 3: Compressive Strength of quarry dust after 3days, 7days&28days

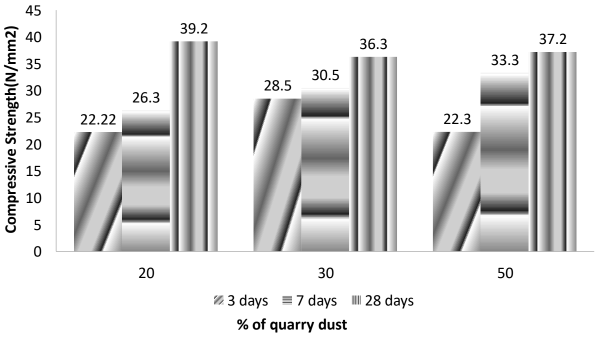

Fig 4: Compressive Strength of quarry dust after 3days, 7days&28days

Specific gravity

The Specific gravity of quarry dust and sand is 3.3&2.68respectively.

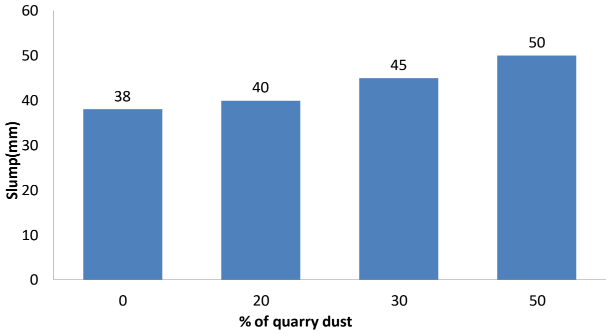

Fig 5: slump values for various proportions of quarry dust

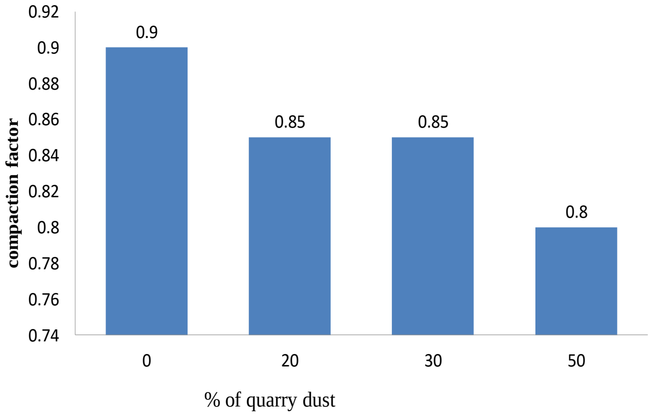

Fig 6 compaction factor for various proportions of quarry dust

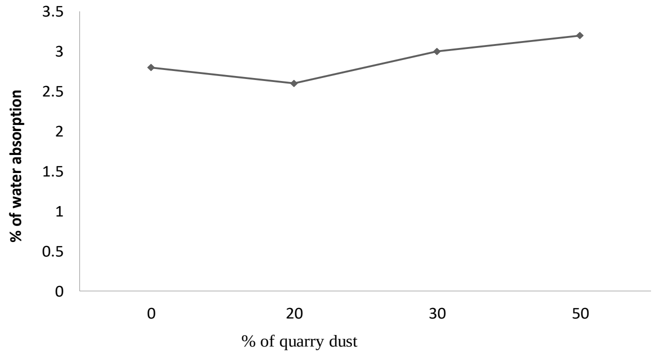

Fig 7:- water absorption for various proportions of quarry dust

Results and discussions

The results of this investigation are shown in Fig 1 to 7. The results are discussed with respect to different parameters. Effect of replacement of quarry dust compressive strength of mortar: The results of compression test on quarry dust and compressive strength ratios with respect to normal mortars. The results of Quarry dust mortars are represented in Fig 2, 3 and 4. From Fig 2, 3 and 4, it is observed that the compressive strength of concrete increases with the increase in percentage of proportions of quarry dust. The values of compressive strengths of concrete with mix proportions of 20%, 30% and 50% gave good and positive results. The compressive strength of concrete for 28 days is more than the natural sand as shown in Fig 1. Fig 1 shows that the compressive strengths of concrete increases with the increase in quarry dust compare with Natural sand. It is clearly observed that for 20% of quarry dust(3 days) the average compressive strength is 20.16N/mm² where as for natural sand it is 13 N/mm².For 30% quarry dust the value increased to 29.6N/mm² and for sand is 27 N/mm².Finally for 50% the strength increased to 37.76N/mm².Fig 3 shows the compressive strength variation with respect to various mix proportions(20%,30%,50%).For 20% of quarry dust the compressive strength of concrete is 21.23N/mm².For 30% of quarry dust the compressive strength of concrete is 28.14N/mm².For 50% of quarry dust the compressive strength of concrete attains 38.13N/mm². Fig 4 shows the compressive strength variation with respect to various mix proportions (20%, 30%, and 50%).For 20% of quarry dust the compressive strength of concrete is 23.167N/mm². For 30% of quarry dust the compressive strength of concrete is 28.4N/mm². For 50% of quarry dust the compressive strength of concrete attains 37.56N/mm² same as natural sand. Average specific gravity values of 3.3 for quarry dust and 2.68 for fine aggregate are obtained. As the fineness of quarry dust increases the specific gravity increases. In case of slump value, slump increases within the increase in the proportions of the quarry dust. For 20% of quarry dust it is observed to be 40 mm, which is desirable. For 30% of quarry dust it is observed to be 45 mm. For 50% of quarry dust it is observed to be 50 mm. *Fig 5* shows the slump test conducted in laboratory. Compaction factor which effects i.e. (workability), decreases with the increase in the values of the quarry dust proportions in concrete. For 20% of quarry dust in concrete the compaction factor was observed to be 0.85%. For 20% of quarry dust in concrete the compaction factor was observed to be 0.85 %. For 20% of quarry dust in concrete the compaction factor was observed to be 0.8%.As the rate of mix proportions of quarry dust increases, the workability also increases.

Conclusions

1. All the experimental data shows that the addition of the industrial wastes improves the physical and mechanical properties.
2. These results are of great importance because this kind of innovative concrete requires large amount to fine particles
3. Due to its high fines of quarry dust it provided to be very effective in assuring very good cohesiveness of concrete. From the above study it is concluded that the quarry dust may replacement material for fine aggregate.
4. Quarry dust has been used for different activities in the construction industry such as for road construction and manufacture of building materials
5. This study reveals that in case of cement mortars, the natural sand can be replaced by quarry dust. The strength of concrete containing 20%, 30%, 50% mix of quarry dust proportions is much higher than normal concrete containing only sand as fine aggregate. It is better to use Quarry dust without removing the finer particles. For lean mortar mixes, quarry dust can be replaced up to 100%.
6. As the quarry dust particles are finer there is an increase in the value of specific gravity which is more than fine aggregate.
7. For rich mortar mixes, quarry dust can be replaced up to 40%.Water absorption in concrete increases as the rate of quarry dust increases. The water absorption percentage of quarry dust concrete decreased for dust content from(0-20) % and the started to increase for 20%,30%,and 50% of dust contents.
8. It is found that there is enough workability with the w/c ratio provided. It is concluded that the compressive strength of concrete are not affected with the replacement of sand by quarry dust as fine aggregate up to 50%.
9. It is clearly observed that for 20% of quarry dust (3 days) the average compressive strength is 20.16N/mm² where as for natural sand it is 13 N/mm². For 30% quarry dust the value increased to 29.6N/mm² and for sand is 27 N/mm². Finally for 50% the strength increased to 37.76N/mm² whereas for natural sand it is 35.6N/mm².
10. Finally the compressive strength of quarry dust results says that the natural sand can be replaced with respect to various mix proportions (20%, 30%, and 50%).
11. Hence, quarry dust can be effectively used to replace natural sand, without reduction in the strength of concrete with sand replacement level up to 50%.

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A STUDY ON REPLACEMENT OF ADDITIVES WITH CCR+FA TO IMPROVE BEARING CAPACITY OF SOIL

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Abstract

Black cotton soil is the locally available soils, with Volume change behaviour and low bearing capacity which are the major problems that pose to the geotechnical engineer. Soil stabilization using industrial waste materials has been widely recommended for developing countries, for the construction of various elements of pavements. This paper deals with identification of Calcium carbide residue (CCR) and Fly Ash (FA) as waste products from acetylene gas and power plant production to study the influence on Black cotton soil. The optimum content of CCR and CCR+FA blender has been found by carrying out laboratory tests - Standard Proctor compaction, unconfined compression test (UCC) and California bearing ratio (CBR) test for a curing period of 3, 14, 21, 28 days. From this study, it is revealed that for mixes of BC Soil + (6% CCR or 8% CCR)+10% FA are considered as optimum to improve the black cotton soil for pavement construction.

Keywords: *CBR, UCC, Calcium carbide residue, Fly Ash.*

Introduction

The main purpose of this work is waste reduction. Disposal of waste in landfills is not an effective solution in the long term because of limited availability of land resources. The most suitable long term sustainable solution is to reduce the quantity of waste being produced and eventually become a "zero-waste" society. Industrial waste reduction can be achieved by recycling or through reuse of the waste material generated. If soil could be replaced by waste material in some of the applications like use in earth dams, road embankments etc., would help in reduction/ reuse of these waste materials. Some of these waste materials which meet the criteria of geotechnical characteristics are considered for this soil stabilization purpose. The waste materials considered for this purpose are fly ash and calcium carbide residue.

SOIL SAMPLE

Black Cotton soil

BC soil has been obtained from Amalapuram, komaragiri patnam. The soil has been collected from a depth of 1.0m below ground level. According to IS classification system, the soil was classified as high plasticity clay (CH).The results of standard proctor test shows that optimum moisture content was 28% corresponding to a maximum dry density of 1.67g/cc.

Calcium Carbide Residue

Calcium carbide residue is a waste material that is collected from gas welding shop. The specific gravity of Calcium carbide residue is 2.23.

$$CaC_2 + 2H_2O \rightarrow C_2H_2 + Ca(OH)_2$$

Table 1: Elemental chemical composition of CCR (%)

Constituent	CCR
SiO_2	18.4%
Al_2O_3	10.17%
Fe_2O_3	8.04%
CaO	62.3%
TiO_2	0.65%
MgO	0.44%

FLY ASH

Fly ash is a by product of coal fired electric power generation facilities; it has little cementations properties compared to lime and cement. However, in the presence of a small amount of activator, it can react chemically to from cementations compound that contributes to improved strength of soft soil. There are two main classes of fly ashes; classes C and class F. The reduction of swell potential achieved in fly ashes treated soil relates to mechanical bonding rather than ionic exchange with clay minerals.

Table 2 Elemental chemical composition of Fly ash (%)

Constituent	Fly ash
SiO_2	62%
Al_2O_3	10.17%
Fe_2O_3	3.04%
CaO	0.92%
TiO_2	0.99%
MgO	0.44%

**Courtesy – Vijayawada Thermal Power Station
From the Table 2 it is clear that percentage of CaO is less than 20% and SiO_2 +Al_2O_3 + Fe_2O_3 more than 70%. From this, Fly Ash is classified as class F.

METHODOLOGY

The experimentation program of the present work was conducted in two phases.

Phase 1:

First phase dealt with finding the optimum content of CCR from Atterberg's limits and UCS, which involved addition of different percentages (2%, 4%, 6%, 8%, and 10%) of CCR to BC soil.All CCR-treated soil specimens were tested for UCS after curing time of 7, 14 , 21 and 28 days. Soaked CBR test are carried out after 3 days curing period. Although the optimum content of CCR obtained from experimental results is 6%, for phase 2 the percentage of CCR is fixed as 8% in order to get pozzolanic reaction between free lime in CCR and fly ash.

Phase 2:

The strength of the blended CCR and FA-stabilized BC soil was investigated with fixed CCR content of 8% with varying fly ash contents (5%, 10%, 15%, 20%). UCS test is carried out for curing periods of 7, 14, 21 and 28 days, CBR tests are carried out for curing period of 3 days and soaked for 4 days.

RESULTS AND DISCUSSION

Atterberg's Limits

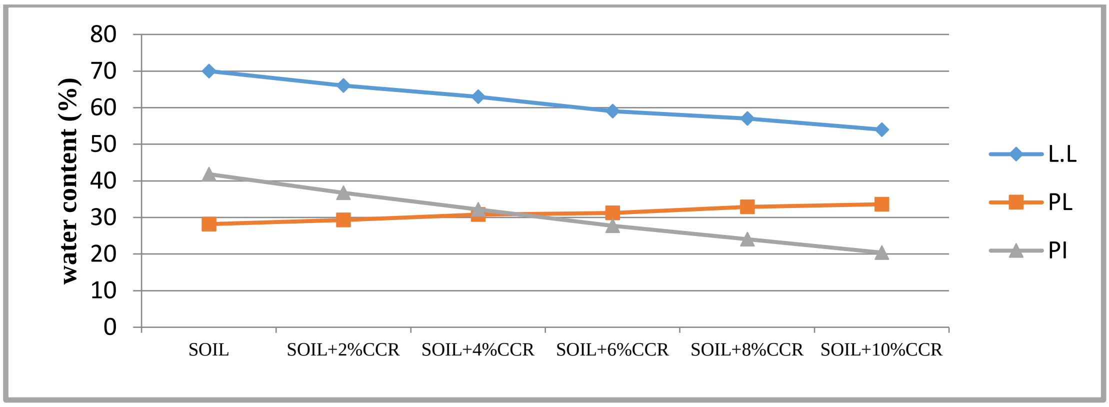

Fig 1: variations of L.L, P.L and P.I with addition of CCR to BC soil

2%CCR+FA			
	LL	PL	PI
0 CCR	70	21.18	41.82
2 CCR	66	29.31	36.69
2CCR+5FA	65	28.20	36.80
2CCR+10FA	65	29.00	36.00
2CCR+15FA	64	29.31	34.69
2CCR+20FA	64	29.50	34.50

Table 3: variations of L.L, P.L and P.I with addition of 2 %CCR with FA to BC soil

4%CCR+FA			
	LL	PL	PI
4CCR	63	30.80	32.20
4CCR+5FA	63	31.00	32.00
4CCR+10FA	62	31.00	31.00
4CCR+15FA	61	30.51	30.49
4CCR+20FA	60.51	30.06	30.45

Table 4: variations of L.L, P.L and P.I with addition of 4 % CCR with FA to BC soil

6%CCR+FA			
	LL	PL	PI
6CCR	59	31.24	27.76
6CCR+5FA	58	31.80	26.20
6CCR+10FA	57.50	32.12	25.38
6CCR+15FA	57	32.91	24.09
6CCR+20FA	56	33.15	22.85

Table 5: variations of L.L, P.L and P.I with addition of 6 % CCR with FA to BC soil

8%CCR+FA			
	LL	PL	PI
8CCR	57	32.91	24.09
8CCR+5FA	52	33.20	18.80
8CCR+10FA	51	34.00	17.00
8CCR+15FA	51	34.00	17.00
8CCR+20FA	50	35.00	15.00

Table 6: variations of L.L, P.L and P.I with addition of 8 % CCR with FA to BC soil

10%CCR+FA			
	LL	PL	PI
10CCR	56	32.00	24 .00
10CCR+5FA	52	31.75	20.25
10CCR+10FA	52	29.51	22.49
10CCR+15FA	50	26.08	23.92
10CCR+20FA	48	25.37	22.63

Table 7: variations of L.L, P.L and P.I with addition of 10 % CCR with FA to BC soil

Standard Proctor Test

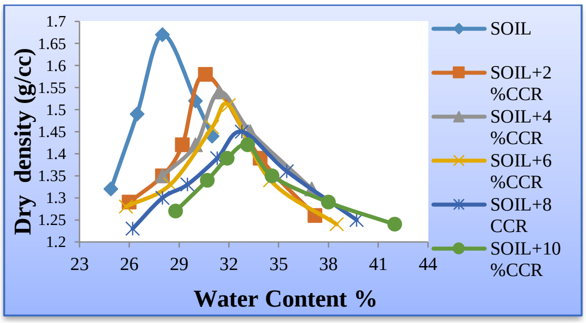

Fig 2: Showing changes in OMC and MDD with addition of CCR to BC soil.

SOIL+8%CCR+FA		
Composition	**OMC (%)**	**MDD (g/cc)**
SOIL+8%CCR	32.80	1.45
SOIL+8%CCR+5%FA	33.76	1.50
SOIL+8%CCR+10%FA	34.20	1.46
SOIL+8%CCR+15%FA	34.62	1.39
SOIL+8%CCR+20%FA	35.00	1.31

Table 8: Variation of OMC and MDD with addition of 8% CCR and different percentages of FA to BC Soil

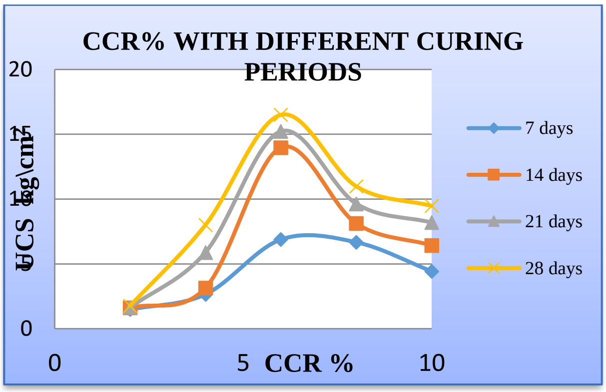

Fig 3: Unconfined compressive strength of BC soil treated with CCR for curing periods

Unconfined Compressive Strength

The unconfined compressive strength of CCR stabilized soil improved up to 6% of CCR content which falls under active zone. In the active zone, strength increases remarkably with increased CCR content. All the input $Ca(OH)_2$ is consumed by the natural pozzolanic material in the soil to produce a pozzolanic reaction. This active zone can be determined from the CCR fixation point, which is obtained simply from the index test. CCR fixation is defined as the CCR content at which the plasticity index of the CCR-soil mixture changes insignificantly with the CCR input. Strength development in the inert zone tends to slow down; the incremental gradient becomes nearly zero and does not make any further significant improvement. A decrease in strength, which appears when the CCR content is in the deterioration zone, is caused by unsoundness due to free lime.

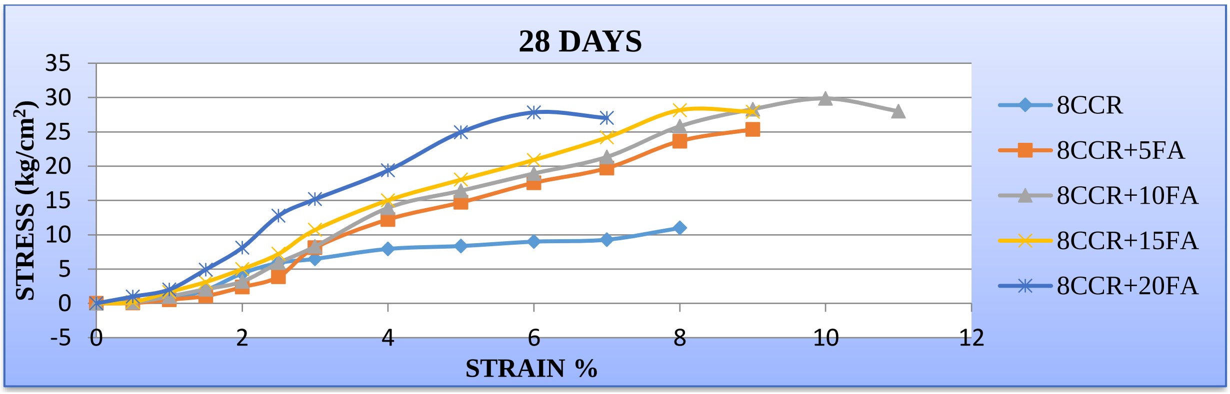

Fig 4: Stress-Strain behaviour with 8%CCR with different percentages of Fly ash

The stress-strain behaviour of the stabilized soil samples for 8% CCR with different FA contents at the OMC are shown in Fig.4 for 28 days curing period. Strength development in the inert zone is essentially dependent on the FA content. The strength increases sharply as the FA content increases up to the optimal value and then decreases. In the deterioration zone i.e., after optimum CCR content the strength of CCR stabilized clay is lower than that of the inert zone. The reduction in strength with increasing CCR content is caused by unsoundness due to the free lime content. Thus FA can improve this detrimental effect as indicated by the increase in strength with increasing FA content. However, the strength increase is gradual up to an optimal FA content at about 10%. The maximum unconfined compressive strength obtained for 8% CCR is about

10.98kg/cm^2 while the BC soil stabilized with 8% CCR and 10% FA blender is 29.87 kg/cm^2. Thus, the main objective to improve the strength of BC soil for utilizing it as pavement construction material is achieved.

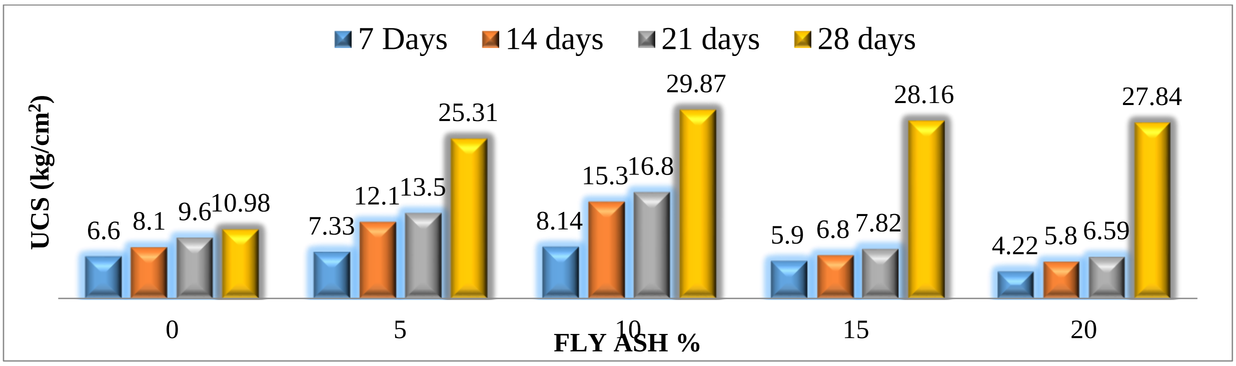

Fig 5 : Unconfined compressive strength of BC soil treated with 8%CCR with different percentages of Fly ash for curing periods of 7,14,21 and 28 days respectively

California Bearing Ratio

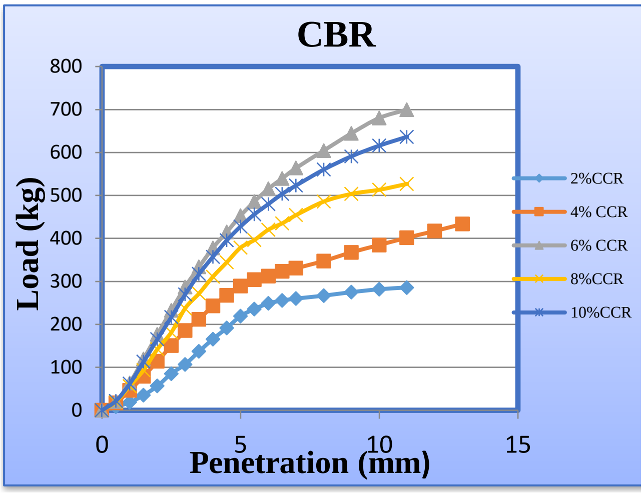

Fig 6: Load Vs Penetration graph for BC soil Stabilized with different percentages of CCR

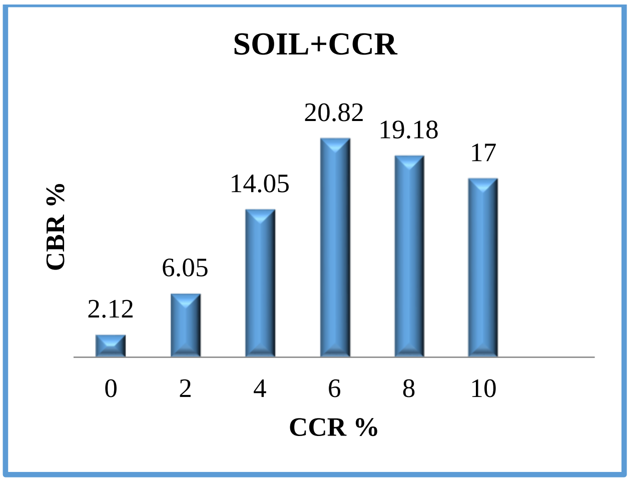

Figure 4.14: Variation of soaked CBR values for BC soil stabilized with different percentages

Based on the results as shown, BC soil was initially poor sub grade material with CBR of 2.12%, improved substantially with the addition of calcium carbide residue and became suitable for laying pavements. Even the mix having BC Soil+8%CCR, BC Soil+10%CCR and BC Soil+12%CCR are giving a soaked CBR value more than 20%, which is desirable for sub base material as per IRC specifications.

In second phase, CBR tests were carried out for Black cotton soil stabilized with 8%CCR and different percentages of Fly ash. From the Fig 6, it was observed that CBR value increasing up to 10% fly ash content thereafter it started decreasing. The improved CBR may be attributed to the presence of the free lime content in CCR which will react with silica and alumina of fly ash to cause pozzolanic reactions. With further increase in fly ash, CBR is decreasing because there is no free lime content to react with fly ash.

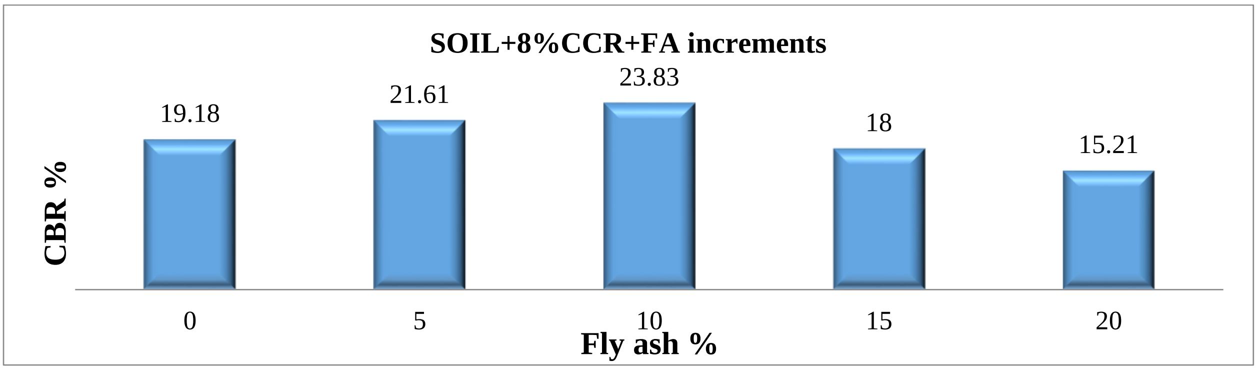

Fig 7: Soaked CBR values for BC soil stabilized with 8%CCR and fly ash of different percentages i.e., 5%, 10%, 15% and 20%

Optimum utilization of CCR and FA with 10% CCR to obtain the target value of CBR for BC soil

CCR (%)	LL (%)	PI (%)	OMC (%)	MDD (g/cc)	CBR (%)
0	70	41.82	28.00	1.67	2.12
2	66	36.69	30.60	1.58	6.05
4	63	32.20	31.42	1.54	14.05
6	59	27.76	32.00	1.51	20.82
8	57	24.09	32.80	1.45	19.18
10	54	20.39	33.14	1.42	17.00

Table 9: Data obtained from experiments conducted on CCR stabilized soil

FA (%)	LL (%)	PI (%)	OMC (%)	MDD (%)	CBR (%)
0	57	24.09	32.80	1.45	17.00
5	52	21	33.76	1.50	21.61
10	51	17	34.20	1.46	23.83
15	51	17	34.62	1.39	18.00
20	50	15	35.00	1.31	15.21

Table 10: Data obtained from experiments conducted on 8% CCR+ FA stabilized soil

CONCLUSIONS

The following conclusions are drawn based on the laboratory studies carried out in this work.

i. The unconfined compressive strength and California bearing ratio of CCR treated BC soil improved substantially with increasing percentages of CCR content.
ii. The natural black cotton soil stabilized with calcium carbide residue and fly ash mix also obtained strength enhancement in both UCS and CBR.
iii. From the Atterberg limits it is observed that with the addition of CCR the plasticity of black cotton soil is reduced.
iv. By increasing the fly ash content to the optimum CCR % there is an increase in strength up to certain percentage and then the strength decreased.

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A STUDY ON REPLACEMENT OF RIVER SAND WITH COMBINATION OF ROBO SAND, FLYASH AND GGBS IN FIBRE REINFORCED CONCRETE

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Abstract

In Recent Trends, the volume of concrete consumed by the construction industry is very large. Almost 40% volume of concrete is comprised of sand. The fine aggregates or sand used is usually acquired from natural sources especially river beds or river banks. These days we have shortage of river sand the only selection to this problematic is the use of artificial sand or M - sand. The material is achieved by crushing the rocks. The M sand is certainly available and has displayed better outcomes as compared to the natural sand. In this project I have substituted the river sand with 50 % of the M- sand and other 50% with Fly Ash and Ground Granulated Blast furnace Slag (GGBS) in two different trials. In this Experiment I had worked on Fiber Reinforced Concrete as it can be defined as a composite material consisting of mixtures of cement, mortar or concrete and discontinuous, discrete, uniformly dispersed suitable fibers in which the different types and properties with many advantages. I had carried out test on Glass fiber reinforced concrete in which sand replacement is also done to check the strength incremental and influence of fibers on strength of concrete. Initially I conducted experiment on the Cement, Aggregates and the workability later the Tests were conducted on specimens with two different fiber volume fractions i: e: 0.5 % and 1 %. The Cube and Cylindrical were Casted For the 7 , 28 days test with 6 different trials by varying the glass fiber proportion and sand replacement portion .It was observed that GFRC specimens with complete Replacement of Sand with i) GGBS – M sand and ii) Fly ash – M – sand showed enhanced properties compared to that of normal specimens .

Keywords: Fiber reinforced concrete, Robo Sand, M sand, GGBS, Fly ash, AR-Glass Fibers

1 Introduction

Concrete is the utmost used man-made construction material in the world. It is achieved by mixing cementations materials, water, aggregate and sometimes admixtures in necessary proportions. Fresh concrete or plastic concrete is freshly mixed material which can be cast into any shape hardens into a rock-like mass known as concrete. "Glass Fiber Reinforced Concrete" is actually cement mortar with uncountable strands of embedded glass fiber; it is a proper compound material. It does not have the categorized rock aggregates or steel-reinforcing bars routinely associated with concrete This tensile strength characteristic also creates drastically improved impact strength. It shares equally the two primary assets of conventional concrete, which are compressive strength and longevity. Conventional concrete has the trait known as "brittle failure" because it has a semi crystalline structure, which tends to shatter on impact. In Recent Trends, the volume of concrete consumed by the construction industry is very large. Almost 40% volume of concrete is comprised of sand. The fine aggregates or sand used is usually acquired from natural sources especially river beds or river banks. For this , the Solution is to replace the sand with following materials like M Sand, GGBS and Fly ash.

2. Experimental Program

2.1 Materials Used

The Materials used for this Experiment are cement, aggregates, GGBS, Robo Sand (M sand), fly ash, glass fibers and admixtures

Cement: It is the substance attained by burning a well proportion mix of material such as lime stone, argillaceous materials such as clay at very high temperature. It has adhesive and cohesive properties and it is shown in below table 1. It is a binding material used with stone, brick, building blocks, etc. Ordinary Portland cement of 53 grade was used in this experimentation conforming to I.S. – 12269- 1987.

Table 1

Sl.no	Property	Result value
1	Specific gravity	3.15
2	Compressive strength of cement	53 N/sq.mm
3	Fineness of cement	1.3%
4	Soundness of cement	1.3mm
5	Normal Consistency	32%
6	Initial Setting time	32 mins
7	Final Setting time	195 mins

Sand: It is mainly packing material for the spaces in concrete. Increasing the proportion of sand in the total mix increases cement demand because of the relatively very large surface area that needs to be coated by cement paste. The properties of sand is shown in below table2. Locally available river sand having bulk density 1860 kg/m3 was used. River sand is sieved to 4.75 mm and the passed out sand is used.

Table 2

S.no	Property	Result Value
1	Surface Texture	Smooth
2	Specific Gravity	2.65
3	Fineness Modulus	2.65
4	Water absorption	4.5

Coarse aggregate: It properties are shown in below table 3. Beyond this, a smaller sized aggregate may have strength advantages in that internal weak planes may be less likely to exist or would be smaller and irregular. A rough angular surface such as in crushed aggregates will increased for a given slump, the water and cement content per cubic meter of concrete are decreased. Coarse aggregate having bulk density 1691 kg/m3. Coarse aggregate is sieved to 20 mm and the passed out is used.

Table 3

S.no	Property	Result Value
1	Water Absorption	2.65
2	Specific Gravity	2.8
3	Fineness Modulus	7.17

Manufacture Sand (M-sand) / Robo Sand*:* It is popularly known is made by powdering hard granite rocks using hefty equipment. Its particles are cubical in shape and finely graded and hence provides greater durability and higher strength to concrete by overcoming deficiencies like segregation, bleeding, honey combing, voids and capillary

Table 4

S.no	Property	Result value 1
1	Specific gravity	2.7
2	Water Absorption	.42
3	Fineness Modulus	3.42
4	Type	Crushed

Water: Potable water was used for the experimentation.

Alkai Resistance (AR) Glass Fibers: The Fibers used were Alklai Resistance Glass Fibers which are uniformly and randomly distributed in the concrete mix. Two different fiber percentages were chosen 0.5%, 1% for each mix.

S.no	Property	Result Value
1	Specific Gravity	2.68
2	Elastic Modulus	72 G.Pa
3	Lengths	6 -12 -18 mm
4	Filament Diameter	14 Microns
5	No.of Fibers	200 Millions/Kg
6	Aspect Ratio	857:1
7	Specific Surface Area	105 sq./kg

Flyash: It is a fine powder which is a byproduct from burning pulverized coal in electric generation power plants. Fly ash is a pozzolan, a substance containing aluminous and siliceous material that forms cement in the presence of water. When mixed with lime and water it forms a compound similar to Portland cement.

Table 6

S.no	Property	Result Value
1	Specific Gravity	2.1
2	Bulk Density	1041Kg/Cum
3	Fineness	336Sq.m/Kg
4	Soundness	0.14

GGBS: Ground-granulated blast-furnace slag (GGBS or GGBFS) is obtained by quenching molten iron slag (a by-product of iron and steel-making) from a blast furnace in water or steam, to produce a glassy, granular product that is then dried and ground into a fine powder.

Table 7

S.no	Property	Result Value
1	Specific Gravity	2.8
2	Bulk Density	1280 Kg/Cu.m
3	Fineness	340Sq.m/Kg
4	Soundness	0.14

1. Preparation of Specimens

Keep the moulds on a porous surface and apply oil to surface of moulds. Weigh the required quantity of sand and coarse aggregate for given concrete ratio. Mix the sand and cement to form mortar. Then mix this mortar with coarse aggregate. Again mix this dry concrete with required quantity of water to provide a normal consistency. Place the wet concrete inside the mould in layers and compact well. Level the top surface of concrete to top level of mould. Mark the date and number on top surface of concrete. Hence the specimen details are shown in

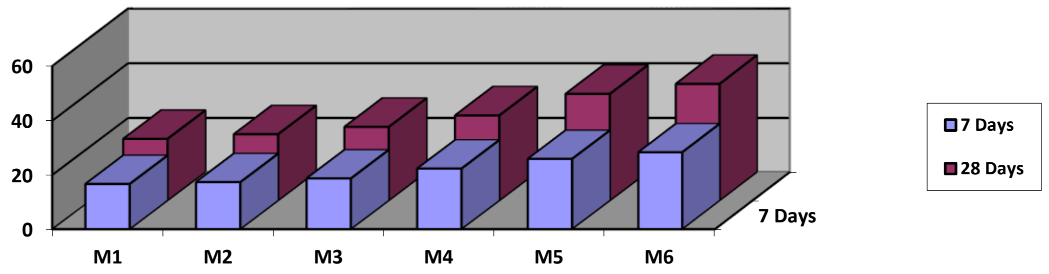
Table 8

S.NO	Trial Mix	Glass Fiber %	Notation
1	M-Sand (50%) +River Sand (50%)	0.5	M1
2	M-Sand (50%) +River Sand (50%)	1	M2
3	M-Sand (50%) + FLYASH (50%)	0.5	M3
4	M-Sand (50%) + FLYASH (50%)	1	M4
5	M-Sand (50%) + GBBS (50%)	0.5	M5
6	M-Sand (50%) + GBBS (50%)	1	M6

4. Tests and Results

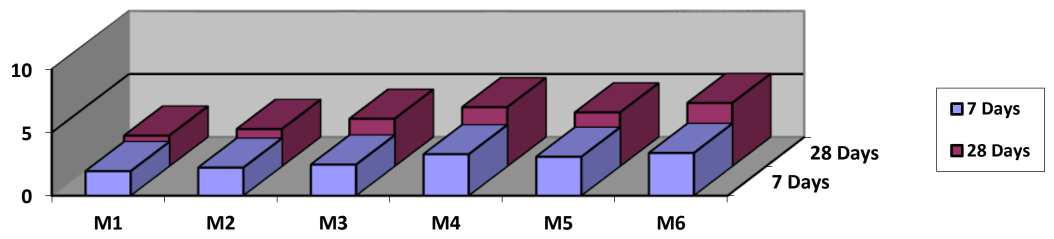
Compressive Strength: At the end of curing period take the cubes from curing tank and wipe them clean with cloth or waste cotton. Measure the dimension of the compression face one by one. Place the cube in between compression plates of the U.T.M / compression testing machine. After the initial adjustments are over apply the load gradually over the cubes. Note the load scale reading at the time of first crack and at the time of the failure. Repeat the procedure of testing for two or more cubes and take the average values

Mix	Compressive Strength (N/Sq.mm)	
	7 Days	28 Days
M1	16.66	22.8
M2	17.33	24.5
M3	18.7	27.2
M4	22.33	31.4
M5	25.9	39.3
M6	28.34	42.94



Spilt* Tensile Strength: *At the end of curing period take the cylinders from curing tank and wipe them clean with cloth or waste cotton. Place the cylinder in between compression plates of the U.T.M / compression testing machine. After the initial adjustments are over apply the load gradually over the cylinder. Test is carried out by placing a cylindrical specimen horizontally between the loading surfaces of a compression test machine and the load is applied until failure of the cylinder along the vertical diameter. The magnitudes of the tensile stress are given by 2P/πDL, were P is the applied load, D and L are the diameter and length of the cylinder respectively. The above table showing the trial mixes where the fine aggregate is completed replaced by the following options

Mix	Spilt Tensile Strength (N/Sq.mm)	
	7 Days	28 Days
M1	1.94	2.44
M2	2.22	2.96
M3	2.46	3.77
M4	3.28	4.69
M5	3.08	4.27
M6	3.38	5.02



4. Discussions

The results obtained in the present investigation that it is feasible to replace the fine aggregates by industrial waste i:e; Robo Sand (M sand/Stone Dust), Fly ash and GGBS for improving the strength characteristics of concrete, thus the industrial waste can be used as an alternative material for the production of concrete to address the waste disposal problems and to minimize the cost of construction with usages industrial waste which almost available at reasonable price in factory

- With the partial replacement of sand by Robo Sand (M sand/Stone Dust) and River Sand the compressive strength of concrete increased up to 6% replacement at curing age of 28 days for 0.5% and 1% proportions of glass fibers
- With the partial replacement of sand by Robo Sand (M sand/Stone Dust) and Fly ash the compressive strength of concrete increased up to 6 % replacement at curing age of 28 days for 0.5% and 1% proportions of glass fibers.
- With the partial replacement of sand by Robo Sand (M sand/Stone Dust) and GGBS the compressive strength of concrete increased up to 3 % replacement at curing age of 28 days for 0.5% and 1% proportions of glass fibers.
- On Addition of Glass Fibers of 0.5% in the concrete, the compressive strength of the concrete for the Mix M3 and M5 when compared to the M1 Mix it gets increased up to 16% and 42% at 28 Days;
- On Addition of Glass Fibers of 1% in the concrete, the compressive strength of the concrete for the Mix M4 and M6 when compared to the M2 Mix it gets increased up to 22% and 42% at 28 Days;
- With the partial replacement of sand by Robo Sand (M sand/Stone Dust) and River Sand the Spilt Tensile strength of concrete increased up to 17% replacement at curing age of 28 days for 0.5% and 1% proportions of glass fibers.
- With the partial replacement of sand by Robo Sand (M sand/Stone Dust) and River Sand the Spilt Tensile strength of concrete increased up to 20% replacement at curing age of 28 days for 0.5% and 1% proportions of glass fibers.
- With the partial replacement of sand by Robo Sand (M sand/Stone Dust) and River Sand the Spilt Tensile strength of concrete increased up to 15% replacement at curing age of 28 days for 0.5% and 1% proportions of glass fibers.
- On Addition of Glass Fibers of 0.5% in the concrete, the Spilt Tensile strength of the concrete for the Mix M3 and M5 when compared to the M1 Mix it gets increased up to 35% and 43% at 28 Days;
- On Addition of Glass Fibers of 1% in the concrete, the Spilt Tensile strength of the concrete for the Mix M4 and M6 when compared to the M2 Mix it gets increased up to 42 % and 42% at 28 Days;

However for the mixes M5, M6 having 42% the compressive strength was comparable to the compressive strength of M1 and M2 mix. The increase in compressive strength due to replacement of sand by GGBS at the constant cement content with different glass fiber percent is attributed to the contribution of GGBS to the hydration process and thus enhancing the compressive strength of concrete.

5. Conclusions

1. The workability of concrete decrease with the addition of glass fibers and can be resolved by adding super plasticizers.
2. On Replacement of Robo Sand it Decrease the Workability.
3. On Replacement with Fly ash and GGBs the workability can be achieved.
4. The Percentage rate of increasing of compressive strength and spilt Tensile strength was higher for fly ash when compared to the GGBS.
5. Workability was more with GGBS when compared with Flyash.
6. The glass fibers are added in wet concrete i.e: after adding water to the dry mix, otherwise it will stick to the surface of mixer. Care should be taken while mixing the glass fibers with concrete. It should not allowed to mix more than 2 minutes, otherwise it will segregate.
7. It was observed that the strength was increased with river sand replacements trials with robo sand along GBBS was slightly more than Robo sand along Fly ash
8. Increase in percentage of fiber has also increased the strength of concrete.
9. Use of Fly ash, Robo sand, GGBS in concrete causes saving of cement and address the problem of disposal of GBBS and Fly ash and resolve environmental problems. Use of Fly ash and GGBS minimize the greenhouse gases and lead to sustainable construction.
10. It reduces the metal industrial waste and disposal problem related to it.
11. Economical concrete.
12. Procuring higher grade of concrete by mixing GGBS /Fly ash/Robo Sand in M-20 concrete
13. Completely river sand is replaced with robo sand, flyash and GGBS and have achived good compressive and spilt tensile strength.

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STRENGTH OF EXPANSIVE SOIL TREATED WITH LIME, GYPSUM AND COIR FIBRE

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Abstract

This paper presents the effect of coir fibers on the compaction and unconfined compressive strength of expansive soil-lime-gypsum mixture. The coir fiber content varied from 0.5 to 2 %. The results indicated that the dry unit weight and the optimum moisture content of expansive soil-lime mix increased with the addition of gypsum. The unconfined compressive strength of the expansive soil increased with the increase in the lime content up to 8%, but beyond 8 % the unconfined compressive strength decreased. The dry unit weight of the expansive soil-lime-gypsum mix increased, and the optimum moisture content decreased with the addition of coir fibre .The unconfined compressive strength increased for the mix of expansive soil and 8 % lime with addition of 4 % gypsum, but beyond 4 % addition of gypsum the unconfined compressive strength decreased. The unconfined compressive strength of the the expansive soil-lime-gypsum mix increased with the addition of coir fibre up to a fibre content of 1.5 %. The unconfined compressive strength of the expansive soil increased with the addition of lime and gypsum and with the increase in the curing period.

Keywords: *Expansive soil, lime, gypsum, coir fibre and unconfined compressive strength.*

I.Introduction

Expansive soils are present in most of the places in India and around the world. These soils are very weak in bearing capacity. These soils in India are highly problematic, as they swell and shrinkage on evaporation.

Expansive soils pose serious problems for temporary roads constructed over them in terms of differential settlements, poor strength, and high compressibility, especially during a rainy season. Several states in India have vast deposits of expansive soils. The current approach adopted to treat such territories is to modify the properties with admixtures such as lime and gypsum to make them suitable for the construction of overlying temporary roads.

Expansive soils absorb water heavily, swell, become soft and lose strength. These soils are easily compressible when wet and possesses a tendency to heave during wet condition. Expansive soils shrink in volume and develop cracks during summer. They are characterized by extreme hardness and cracks when dry. These properties make them poor foundation soils and earth construction material. For developing a good and durable road network in expansive soil areas, the nature of soils shall be properly understood. On such soils suitable construction practices and sophisticated methods of design need to be adopted.

Soil stabilization is a collective term for any physical, chemical, or biological method, or any combination of such methods that may be used to improve certain properties of a natural soil to make it serve adequately an intended engineering purpose. It is the process of blending and mixing materials with a soil to improve certain properties of the soil. The main benefits of using lime to stabilize clays are improved workability, increased strength, and volume stability. Workability is improved because flocculation makes the clay more friable. Lime increases the optimum water content for compaction, which is an advantage when dealing with wet soil.

Lime increases the strength of clayey soil. Soil stabilization occurs when lime is added to a reactive soil to generate long-term strength gain through a pozzolanic reaction. The strength of lime mixture depends to a great extent on the quantity of lime added above lime fixation point. It is generally found that beyond a certain % of lime the increase in strength ceases & in fact a lowering strength may result due to present of unreacted free lime indicating that there exists optimum lime content for maximum strength grain. So after a certain limit of lime content no development of strength but cost increases.

To further improve the mechanical properties of these stabilized soils, a variety of materials are being used as reinforcements. They are polymeric in composition, have a long life, do not undergo biological degradation, and are liable to create environmental problems from their manufacture till their end use. The use of coir fibres are therefore gaining in popularity as they too are biodegradable in nature and do not cause any environmental problems. In the present paper, an attempt has been made to study the compaction and unconfined compressive strength of a expansive soil-lime-gypsum mixture reinforced with coir fibres for possible use in improving soil.

II. MATERIALS AND METHODOLOGY

A. Materials

Locally available expansive soil was used in this study. The physical and engineering properties of the expansive soil are given in Table 1. Hydrated lime and gypsum procured from Vijayawada, Andhra Pradesh, India. And coir fibre (coconut fibre) was procured from a locally available in Tadepalligudem, Andhra Pradesh, India. The specific gravity of the lime, gypsum and coir fibre was 2.37, 2.89 and 1.9, respectively.

Table 1: Physical Properties of Expansive soil

Property	**Value**
Specific gravity	2.68
Liquid limit, %	82
Plastic limit, %	47.22
Optimum moisture content, %	24.3
Maximum dry density, kN/m^3	15.48
Type	CH

B. Methodology

Unconfined compressive strength (UCS) tests were conducted in accordance with IS: 2720, Part X (1991). The strain rate was kept 1.2 mm/min in all the experiments. The proving ring of capacity 2 kN

III. Results and Discussion

Studies were carried out to obtain maximum dry unit weight and unconfined compressive strength for expansive soil and stabilized with lime, gypsum and coir fibre. The content of lime was varied from 2 % to 10 % by dry weight of expansive soil. The content of gypsum was varied from 2 % to 8 % by dry weight of expansive soil. The content of coir fibre was varied from 2 % to 10 % by dry weight of expansive soil.

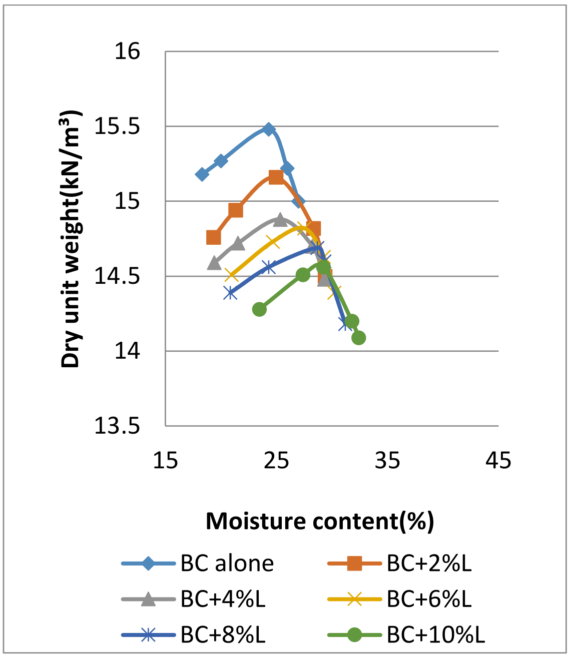

Figure 1: Compaction curves for expansive soil with varying percentage of lime

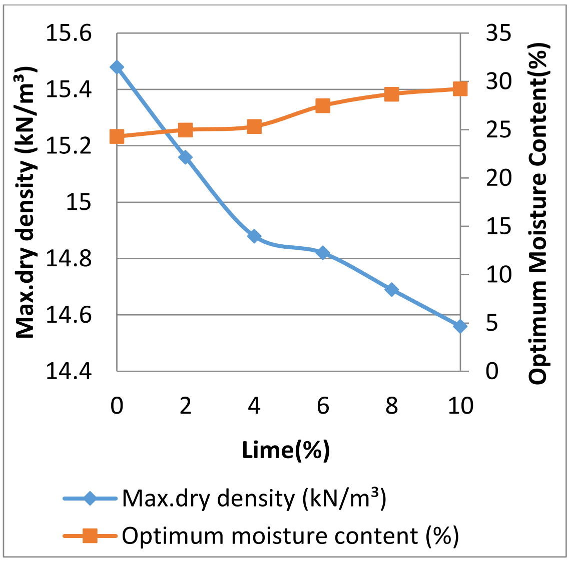

Figure 2: Variation of maximum dry unit weight and optimum moisture content of expansive soil with varying percentage of lime

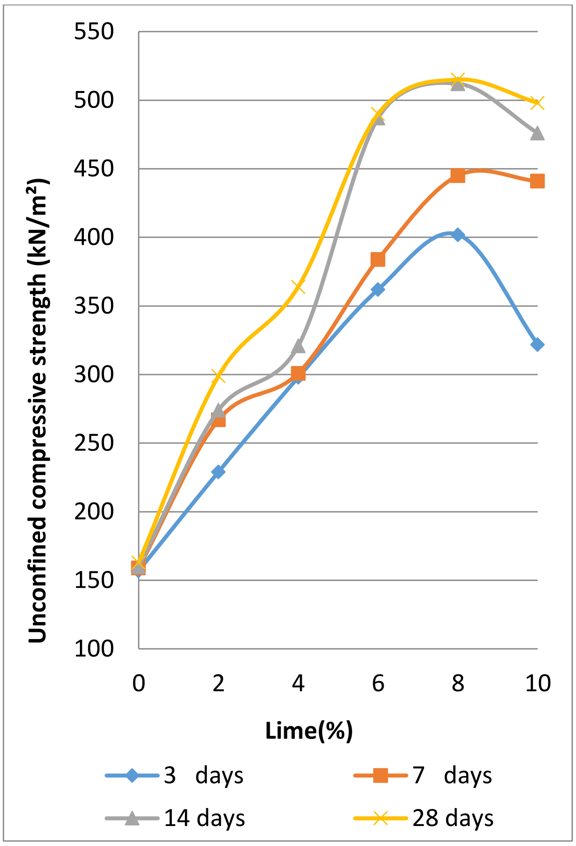

Figure 3: Variation of unconfined compressive strength of expansive soil with varying percentage of lime and curing period

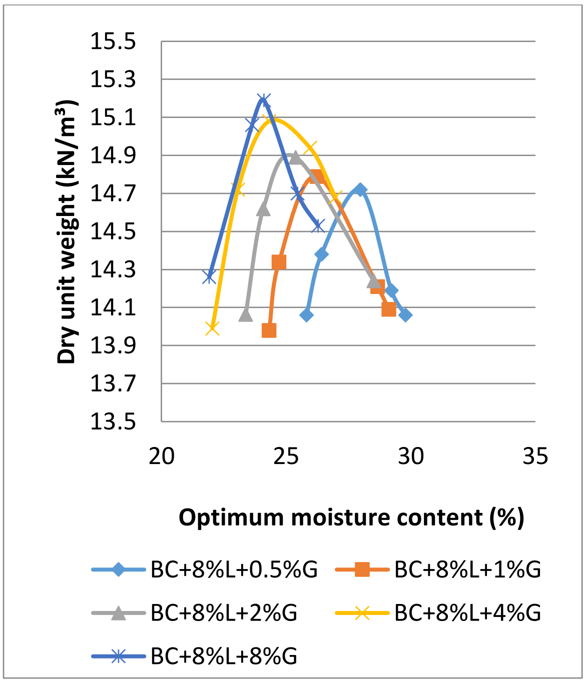

Figure 4: Compaction curves for expansive soil + 8 % lime with varying percentage of gypsum

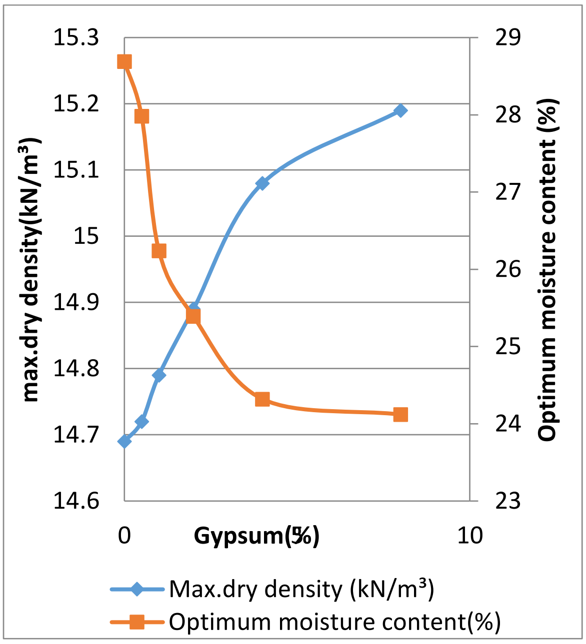

Figure 5: Variation of maximum dry unit weight and optimum moisture content of expansive soil + 8 % lime with varying percentage of gypsum

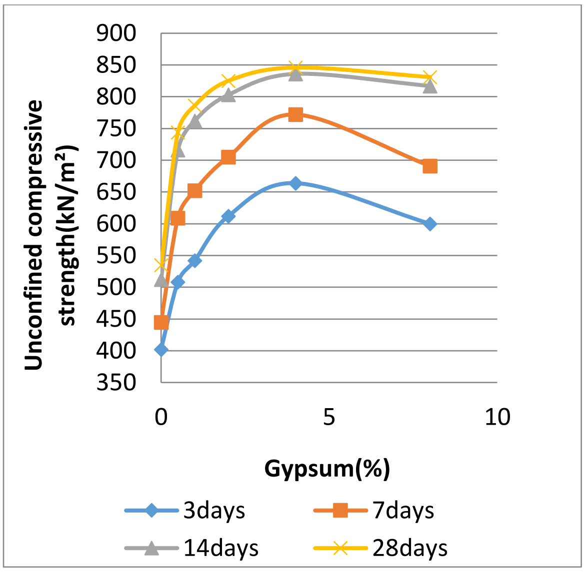

Figure 6: Variation of unconfined compressive strength of expansive soil + 8 % lime with varying percentage of gypsum and curing period

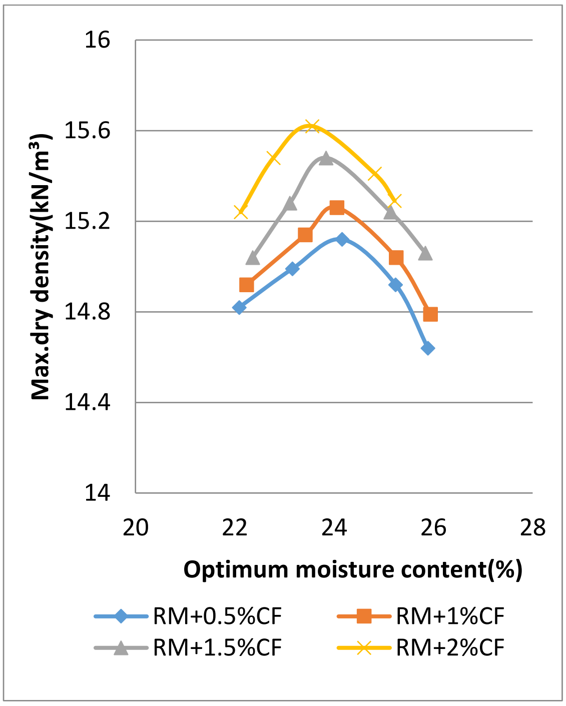

Figure 7: Compaction curves for expansive soil-lime-gypsum mix with varying percentage of coir fibres

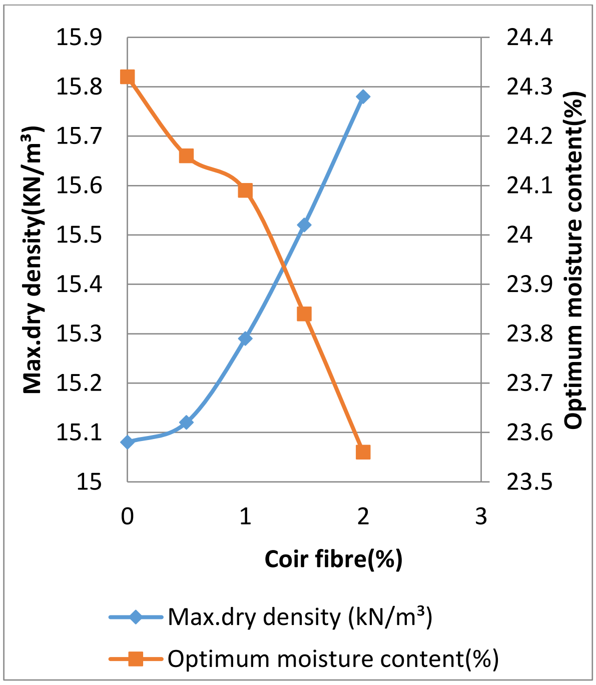

Figure 8: Variation of maximum dry unit weight and optimum moisture content of expansive soil-lime-gypsum mix with varying percentage of coir fibres

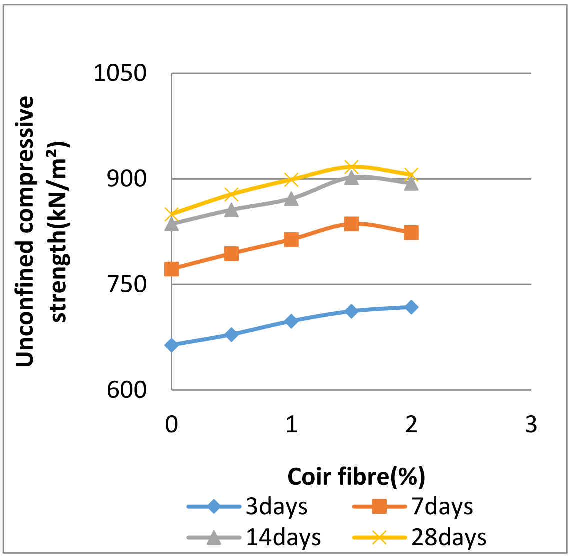

Figure 9: Variation of unconfined compressive strength of expansive soil-lime-gypsum mix with varying percentage of coir fibres and curing period

IV. CONCLUSION

An experimental study is carried out to investigate the strength characteristics of expansive soil treated with lime, gypsum and coir fibre. The study brings forth the following conclusions.

1. Addition of lime to the expansive soil caused a reduction in maximum dry density and increase in optimum moisture content. Considering a optimum amount of 8% lime content the MDD reduced from its original value of 15.48 kN/m^3 to 14.69 kN/m^3 and OMC increase from 24.3% to 28.69%.

2. Lime is effective in increasing the UCS value. For an optimum amount of 8% lime the UCS value of soil has increased from its original value of 157 kN/m^2 to 402 kN/m^2. Further improvement was observed with curing period. An optimum curing period of 14 days, the UCS value increased to 512 kN/m^2.

3. The combined effect of lime and gypsum to the expansive soil caused a increase in MDD and reduction in OMC. Considering a optimum amount of 8% and 4% lime and gypsum contents respectively, the MDD increased and OMC reduced.

4. Addition of optimum amount of lime (8%) and gypsum (4%) to the expansive soil has increased the UCS value. For an optimum curing period of 14 days the UCS increased upto 836 kN/m^2. And further addition of gypsum caused a decrease in UCS.

5. Addition of coir fibre to the lime + gypsum treated expansive soil caused additional increase in MDD. Coir fibre content of 1.5% shows the MDD increased from 15.08 kN/m^2 to 15.48 kN/m^2. Similarly the UCS value increased from 836 kN/m^2 to 902 kN/m^2.

Finally this study show an insight of the effect Lime, Gypsum and Coir fibre on the strength properties of the expansive soil. And the results reveals that 8%, 4% and 1.5% respectively are optimum contents and a curing period of 14 days was found to be effective.

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EFFECT OF RECYCLED AGGREGATE ON FRESH AND HARDENED STATE PROPERTIES OF SELF-COMPACTING CONCRETE

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Abstract

Concrete is the most widely used construction material in which Aggregates take maximum share. This poses the problem of acute shortage of aggregate and scouring of Granite Quarry. At the same time, the quantity of recycled Concrete aggregates from old Construction Demolished waste is piling up in many areas. If it is possible to use this RCA in fresh concrete by partial/complete replacement of Natural Coarse aggregates, then this will not only save the cost of construction at the same time it will solve the problem of disposal of this CDW waste. Therefore, the objective of this research work is to develop sustainable self Compacting Concrete (SCC) of various grades using Recycled Concrete Aggregate (RCA), fly ash etc. This paper discusses the fresh and hardened state properties of SCC of M30 grade using Natural and Recycled Concrete Aggregates. Quantification and Characterization was done using Modified Nan Su Mix design analysis.

Keywords*: Self Compacting Concrete (SCC), Recycled concrete aggregates (RCA), Fresh Properties, Mechanical Properties, Modified Nan Su Method.*

Introduction

The term Self-Compacting Concrete (SCC) refers to a “new” special type of concrete mixture, characterized by high resistance to segregation that can be cast without compaction or vibration. It flows like “honey”, de-aerates, self-compacts, and has nearly a horizontal concrete level after placing. Products made with SCC have an excellent finish, and are virtually free of bug holes The basic components of the mix composition of SCC are the same as those used in conventional concrete. However, to obtain the requested properties of fresh concrete in SCC, a higher proportion of ultrafine materials and the incorporation of chemical admixtures, particularly an effective superplasticizer, are necessary. Because of this, self-compatibility can be largely affected by the characteristics of materials and mix proportion. No standard or all-encapsulating method for determining mixture proportions currently exists for SCC. However, many different proportion limits have been listed in various publications. Therefore, a rational mix-design method for NASCC and RASCC using variety of materials is necessary. The proposed Modified Nan Su Mix design of SCC must satisfy the criteria on filling ability, pass ability and segregation resistance.

Mix Design Method: Initially EFNARC first approach for Modified Nan Su Mix design is used, and then the proportions of materials modified after the evaluation by fresh tests was done. The modifications are made according to EFNARC guidelines.

Sustainable Design: Sustainability in general terms is to create an economic system with enhanced performance with long term safety. Sustainability is the one which mainly focuses on the climate change, energy conservation, protection of natural resources and environmental enhancement.

Sustainability in Concrete Production: The use of natural resources in consideration of ecological and economical aspects forms the basis for sustainable developments. Low energy consumption during production and the use of by-products as well as waste materials are essential to reduce environmental and financial impacts.

EXPERIMENTAL INVESTIGATION

NECESSITY OF INVESTIGATION:

- Cost of Natural Coarse Aggregates is increasing day-by-day.
- Acute shortage of Coarse Aggregates in many areas.
- Using /Removal of more Natural Coarse Aggregates causes scouring of quarry and loss of natural minerals present in the river.
- The properties of recycled concrete aggregates RAC are already established and demonstrated successfully through several experimental and field projects. However, its use is restricted to standard grade concretes only in Indian scenario.

NEED FOR EDUCATION & POLICIES FOR ADOPTION OF RECYCLED AGGREGATES:

Government policies and initiatives promoting or encouraging waste minimization/recycling and proposed landfill levies can, to a large extent, drive the adoption of recycled concrete. In countries where the cost to produce RCA may be greater than the cost of acquiring virgin aggregates, government incentives and efforts to educate the public on the environmental benefits associated with the use of RCA are key to promoting increased and alternative applications Educating contractors and owner-agencies about methods for effectively incorporated RCA into new concrete mixtures could help the industry overcome this limitation. Such an education would require understanding the characteristics of RCA, revising specifications for aggregate as necessary, understanding the impact of the use of RCA on fresh and hardened concrete properties and on the durability of pavements, and making appropriate adjustments in the development of mixture proportions and production methods. This education would show contractors that RCA can be a cost- effective material that will not compromise the quality and result in an industry wide change in perception about the use of RCA

ENGINEERING PROPERTIES OF RCA: Properties like specific gravity, hardness, strength, physical and chemical stability, pore structure and colour depends on CDW/BDW.

Fine and Coarse Aggregates: The physical properties of Sand, NCA and RCA used in the present experimental investigations are carefully studied and results are tabulated in Table no (1). All the values for the Coarse aggregate and RCA have been found by conducting respective test in the laboratory as per IS code provision.

Table no (1): Properties of Sand, Natural Coarse Aggregate and Recycled Concrete Aggregate

Property	Sand	NCA	RCA	Method
Type	Natural	Crushed	Crushed	--
Specific gravity	2.62	2.65	2.45	IS:2386 Part-3-1963
Total water absorption	1.0%	0.3 % (12.5mm)	2.4 % (12.5mm)	IS:2386 Part-2-1963
Moisture content	0.15%	0.8% (12.5mm)	0.45% (12.5mm)	IS:2386 Part-2-1963
Bulking	5% wc	--	--	IS:2386 Part-3-1963
Bulk Density (Loose)	1567kg/m^3	1380 kg/m^3	1355 kg/m^3	IS:2386 Part-3-1963
Bulk Density (Compacted)	1713kg/m^3	1532 kg/m^3	1540 kg/m^3	IS:2386 Part-3-1963
Fineness Modulus	2.63 (Zone II)	5.94 (12.5mm)	4.65 (12.5mm)	IS:2386 Part-2-1963
Elongation Index	--	7.10% (12.5mm)	11.27% (12.5mm)	IS:2386 Part-2-1963
Flakiness Index	--	6.15% (12.5mm)	7.85 % (12.5mm)	IS:2386 Part-2-1963

Table no (2): Water Absorption, Specific Gravity and Packing factor for various proportions

Percentage of NA & RCA	Water absorption (%)	Specific gravity	Packing factor
0% (100%NA+0% RCA)	0.3	2.65	1.12
25% (75%NA+25% RCA)	1.0	2.60	1.12
50% (50%NA+50% RCA)	1.6	2.53	1.13
75% (25%NA+75% RCA)	2.0	2.48	1.13
100% (0%NA+100% RCA)	2.4	2.45	1.14

Table no (3): Quantities of different ingredients of RASCC & NASCC

MIX	W/P =	Cement (Kg)	Fly ash (kg)	River Sand (Kg)	CA - 12.5mm (NA) (Kg)	CA - 12.5mm (RA) (Kg)	Admixture (kg)	VMA (kg)	Water (lts)
NASCC-M30-0%	0.37	1.000	0.394	1.927	1.812	0.000	0.017	0.002	0.521
RASCC-M30-25%	0.38	1.000	0.394	1.927	1.359	0.453	0.017	0.002	0.526
RASCC-M30-50%	0.38	1.000	0.394	1.927	0.906	0.906	0.017	0.002	0.531
RASCC-M30-75%	0.38	1.000	0.394	1.927	0.453	1.359	0.017	0.002	0.535
RASCC-M30-100%	0.39	1.000	0.394	1.927	0.000	1.812	0.017	0.002	0.540

MATERIALS MIXING METHODOLOGY: Two stage mixing approach (TSMA)is employed, in which proportional amount of cement is added to the recycled aggregate in the premix stage. All the other quantities of ingredients are added subsequently as per recommended procedure of mix design of NASCC & RASCC.

Table no (4): Fresh properties of M 30 grade NASCC & RASCC with Different proportion of RCA

M30	Flow Table	T50	V-Funnel Test (Sec)		U- Box Test	L- Box Test
	(mm)	(Sec)	T0	T5	(mm)	(h2/h1)
Normal Range	650-800	2.-5	6. -12	6.-15	0-30	0.8-1.0
MIX IDENTIFICATION	Filling Ability	Filling Ability	viscosity		Passing Ability	Passing Ability
NASCC-M30-0%	720	2.5	8.0	10.5	28	0.95
RASCC-M30-25%	700	3.5	8.28	10.1	26.5	0.93
RASCC-M30-50%	690	4.0	8.5	9.6	24	0.90
RASCC-M30-75%	680	5.5	8.8	9.4	22	0.88
RASCC-M30-100%	650	6.0	9.0	9.26	21	0.85

Table no (5): Hardened Concrete Test Results of M30 grade having different Replacement Ratio at 7 days & 28 days of Curing.

Mix identification	Compression (Result in MPa)		Spilt Tensile (Result in MPa)		Flexural (Result in MPa)	
	7 Days	28 Days	7 Days	28 Days	7 Days	28 Days
NASCC-M30-0%	28.36	41.36	2.1	4.22	3.65	4.98
RASCC-M30-25%	28.20	39.77	1.9	4.02	3.32	4.70
RASCC-M30-50%	27.10	37.80	1.84	3.75	3.12	4.49
RASCC-M30-75%	26.40	35.30	1.76	3.54	3.08	4.18
RASCC-M30-100%	25.65	35.03	1.64	3.46	2.69	3.15

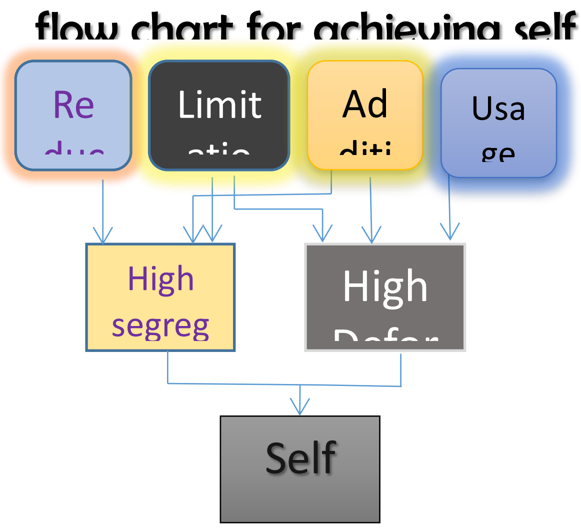

Fig: (1)- Flow chart of Self Compacting Concrete

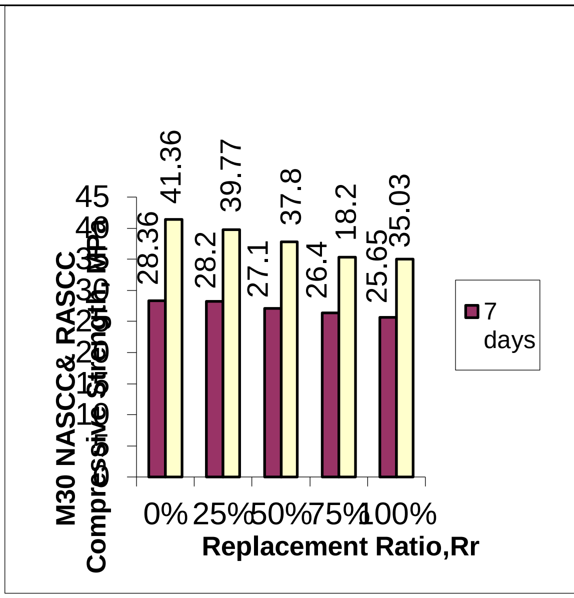

Fig: (2)- Compressive Strength of M30 Grade Concrete having different Replacement Ratio at Different Ages of Curing.

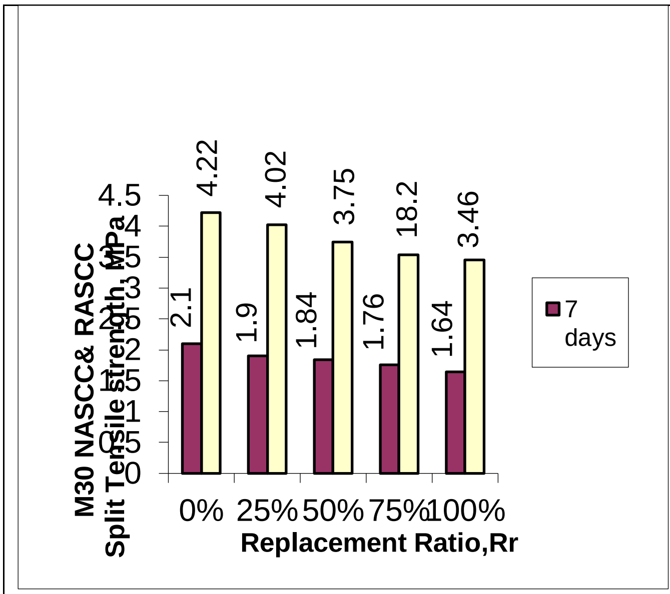

Fig: (3)-Split Tensile Strength of M30 Grade Concrete having different Replacement Ratio at Different Ages of Curing.

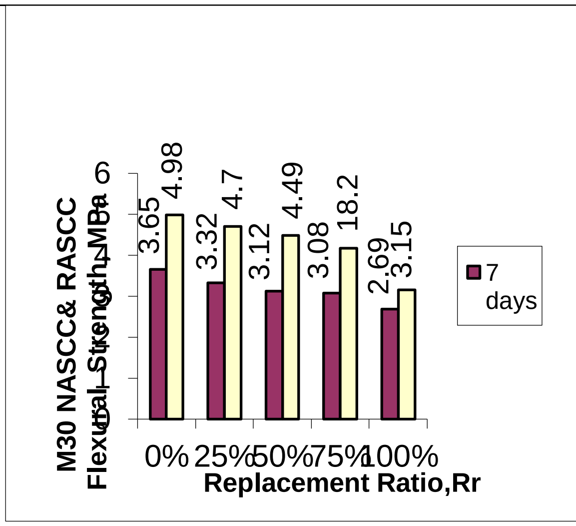

Fig: (4)- Flexural Strength of M30 Grade Concrete having different Replacement Ratio at Different Ages of Curing.

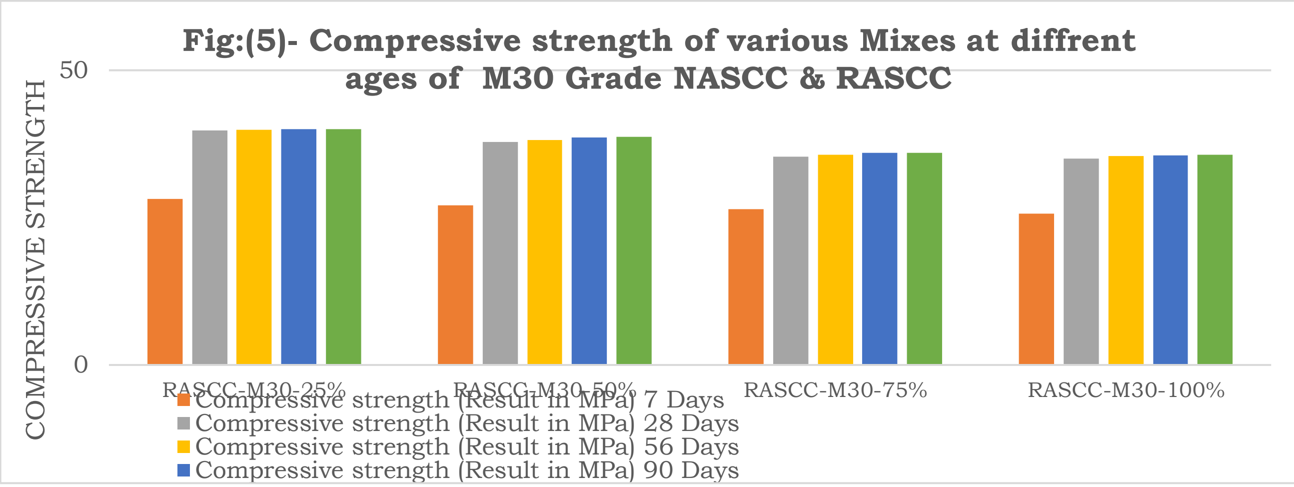

		
Plate:(1)-Demolished Concrete Material	**Plate:(2)-JAW crusher**	**Plate:(3)-Testing self compacting concrete for workability.**

DISCUSSIONS AND CONCLUSIONS

DISCUSSIONS:

Slump flow diameters ranging from 650 and 720 mm were obtained for the recycled self-compacting concretes. According to EFNARC limitation the reference mixture was within the limits. Although increasing the plastic content resulted in reduction of slump flow diameters of concretes, the results were acceptable for many normal application of self-compacting concrete. Using Recycled Concrete Aggregates as partial and full replacement of Coarse aggregate increased both T50 slump flow and V-funnel flow range limits. The L-box height ratio was also affected by the content and size of using Recycled Concrete Aggregate in to SCC. Increasing the RCA content caused systematical decrease in the L-box height ratio. However, all mixtures have the L-box height ratio values more than 0.8 which is the lower limit specified by EFNARC.

Self-compacting concrete with RCA having more than 35 MPa compressive strength could be produced easily. The strength results indicated that the utilization of RCA in SCC manufacturing resulted in systematical decrease of the compressive strength.
There is no difference in strengths of M30 Grade concrete at all ages as natural aggregate is replaced by 25% of RCA. A marginal decrease of strength is observed at 50% replacement of natural aggregate by RCA. The reduction strength increased with age for M30 grade concrete with 75% RCA up to 28 days and there after remained constant; The maximum reduction in strength being 14%. The strength values are more or less same for 75% and 100% replacement.

CONCLUSION:

The absence of an established industrial standard for SCC allows more creativity in tailoring a mix to specific job requirements. At the same time, the lack of standards means devising a successful mix depends on the expertise of the producer and contractor. Therefore, it is clear that educating manufacturers and contractors is the crucial step in expanding the use of SCC's extremely promising technology. The following conclusions are drawn based on the experimental investigations carried out:

1. Test results indicate that only partial replacement of natural aggregate by recycled concrete aggregate (50%) is giving reasonably good strength.
2. There is not much change in the compressive strengths of NASCC & RASCC of M30 grade at 3 days and 7 days for all replacements of NA by RCA. However, the loss of strength of around 14% is observed for RCA 75% and 100% at the ages of 28 days and beyond.
3. Split tensile strength and flexural strength of RASCC followed more or less similar trend and the decrease in these values compared to NASCC was found to be around 10%.
4. From the experimental results, it is observed that the optimum percentage replacement of natural aggregate by RCA is 50% to get reasonable strength in compression, tension and flexure.
5. There is no doubt that recycled concrete aggregate is cost free material and also reduces waste produced from construction or demolition works.

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FEASIBILITY STUDIES ON INFLUENCE OF FIBRES IN SELF-CURING CONCRETE

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Abstract

Today Water is the most required substance in the era. In common, Curing of concrete is maintaining moisture in the concrete during early ages specifically within 28 days of placing concrete, to develop desired properties. Proper curing of concrete is essential to obtain maximum durability, especially if the concrete is exposed to serve conditions where the surface will be subjected to excessive wear, aggressive solutions and severe environmental conditions. Poor curing practices adversely affect the desirable properties of concrete which makes a major impact on the permeability of a given concrete. Unexpected shrinkage and temperature cracks can reduce the strength, durability and serviceability of the concrete. The surface zone will be seriously weakened by increased permeability due to poor curing. The development of concrete shrinkage is proportional to the rate of moisture loss in concrete. When concrete is properly cured, water retained in concrete would help continuous hydration and development of enough tensile strength to resist contraction stresses. The continuous development of strength reduces shrinkage and initial cracks or micro-cracks. As a part of research initiative, this investigation of Fibrous Self Curing Concrete, proportion and addition of Polyolefin Macro Monofilament Fibre resulted in the formation of micro cracks in order to reduce the autogenous shrinkage and improvement of durability.

***Keywords**: Water scarcity; Autogenous shrinkage; Temperature cracks; Internal curing; Super Absorbent Polymer; Polyolefin Macro Monofilament Fibre.*

Introduction

Concrete is a blend of Cement, Aggregates and water with or without appropriate admixtures. To achieve alluring quality and different properties, curing is fundamental. Curing is the way toward keeping up the correct dampness substance to advance ideal bond hydration instantly after arrangement. Proper moisture conditions are critical because water is necessary for the hydration of cementitious materials. As a result, adequate curing is essential for concrete to obtain advanced structural and durability properties and therefore is one of the most important requirements for optimum concrete performance in any environment or application. Curing techniques and Curing durability significantly affect curing efficiency. As per IS 456: 2000, Curing is the process of preventing the loss of moisture from the concrete whilst maintaining a satisfactory temperature regime.

Review of Literature

Álvaro Paul and Mauricio Lopez, (2011),[1] internal curing (IC), which has been extensively investigated in the last decade, has been shown to enhance hydration, diminish autogenous shrinkage, and mitigate early-age cracking due to self desiccation in high-performance concrete. It also increases the internal porosity of concrete, however, which might reduce mechanical properties.

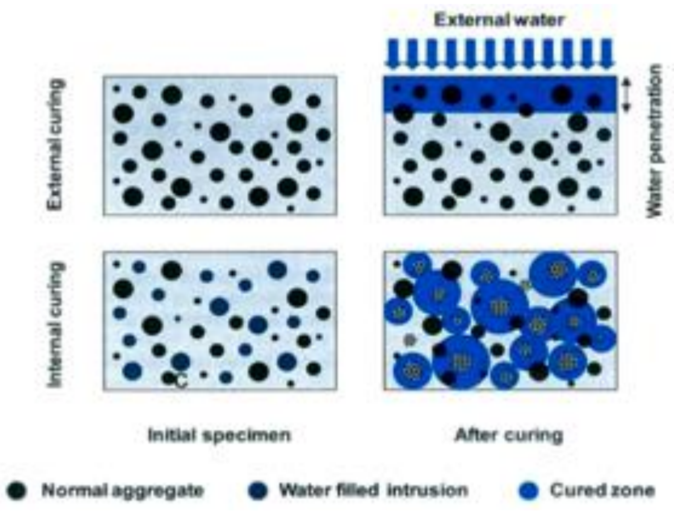
Ambily and Rajamane, (2009),[2] studied the different aspects of achieving optimum cure of concrete without the need for applying external curing methods excessive evaporation of water (internal or external) from fresh concrete should be avoided, otherwise, the degree of cement hydration would get lowered and thereby concrete may develop unsatisfactory properties. Curing operations should ensure that adequate amount of water is available for cement hydration to occur. Dale P.Bentz, (2007)[7] in the twenty-first century, most high-performance **co**ncretes, and many other ordinary concretes, are now based on blended cements that contain silica fume, slag, and/or fly ash additions. Because the chemical shrinkage accompanying the pozzolanic and hydraulic reactions of these mineral admixtures is generally much greater than that accompanying conventional Portland cement hydration, these blended cements may have an increased demand for additional curing water**.** The review of various literatures mentioned above has helped to understand the properties of self-curing concrete and addition of Super Absorbent Polymers and fibre improve the concrete's durability rather than conventional concrete. Thus based on these the methodology was developed to carry out present behavior studies.

Scope and Objectives

A comprehensive investigation has been undertaken to study the effects of self-curing agents such as Super Absorbent Polymer and the addition of Polyolefin Macro Monofilament Fibre on the mechanical and non-mechanical properties of concrete and its durability. The considered concrete properties are compressive, tensile strength, volumetric water absorption, P^H value and mass loss. The test program was performed on concretes containing cement content, water cement ratio (0.4) cured in air 25°C and elevated temperature 50°C. Special attention was given to the enhancement in self-curing concrete properties cured in normal 25°C and elevated temperature 50°C, as well as its durability and wet/dry cycles in water as affected by the type and doses of self-curing agent along with the addition of Super Absorbent Polymer.

Self-Curing

The concept of self-curing is to reduce the water evaporation from concrete and hence increase the water retention capacity of the concrete compared to conventional concrete.



External and Internal curing

Benefits from Internal Curing

Internal Curing can be anticipated when

- Cracking of concrete provides passageways resulting in deterioration of Reinforcing steel
- Low early-age strength is a problem and Need for reduced construction time,
- Quicker turnaround time in precast plants, lower maintenance cost,
- Greater performance, predictability, Permeability/durability must be improved,
- Rheology of concrete mixture, modulus of elasticity of the finished product or durability of high fly-ash concretes are considerations

Effect Of Fibres In Concrete

- Usually used in concrete to control cracking due to both plastic shrinkage and drying shrinkage.
- Reduce the permeability of concrete and thus reduce bleeding of water.
- Fibres produce greater impact, abrasion and shatter resistance in concrete.
- Fibres do not increase the flexural strength of concrete, and so cannot replace moment resisting or structural steel reinforcement

The amount of fibres added to a concrete mix is expressed as a percentage of the total volume of the composite (concrete and fibres), termed volume fraction (Vf). volume fraction typically ranges from 0.1 to 3%.

Benefits of Fibre Reinforced Concrete

- Improved impact & freeze-thaw resistance
- Improved resistance to explosive spalling in severe fire
- Increased resistance to plastic shrinkage during curing
- Improved structural strength
- Reduced steel reinforcement requirements & improved ductility
- Reduced crack widths and segregation

Material Properties

As per IS 383– 1970, ‘Indian Standard Code of practice for Specification for Coarse and fine aggregates from natural sources for concrete.

Material	Properties	Values
Cement	Fineness of Cement	7.5%
	Grade of Cement	43
	Specific Gravity	3.15
	Initial Setting time	28 min
	Final Setting time	600 min
Fine Aggregate	Specific Gravity	2.65
	Fineness Modulus	2. 25
Coarse Aggregate	Specific Gravity	2.77
	Size of Aggregates	20 mm
	Fineness Modulus	5.96
Water	Potable Water	6-6.5 P^H

Super Plasticizer

The inter particle friction between fibres and aggregates controls the orientation and distributions of the fibres and consequently the properties of concrete. Naphthalene-Formaldehyde Sulphonate based super plasticizer is added as a friction reducing admixture to improve the cohesiveness of mix.

A composition of matter useful for retarding the set time of concrete formulations significantly without adding more water thereto comprises a high molecular weight condensation product of naphthalene sulfonic acid and formaldehyde.

Super Absorbent Polymer

The common SAPs are added at rate of 0–0.6 wt. % of cement. The SAPs are covalently cross-linked. They are Acrylamide/acrylic acid copolymers. One type of SAPs are suspension polymerized, spherical particles with an average particle size of approximately 200 mm; another type of SAP is solution polymerized and then crushed and sieved to particle sizes in the range of 125–250 mm. The size of the swollen SAP particles in the cement pastes and mortars is about three times larger due to pore fluid absorption. The swelling time depends especially on the particle size distribution of the SAP. It is seen that more than 50% swelling occurs within the first 5 min after water addition. The water content in SAP at reduced RH is indicated by the sorption isotherm.

During the hydration process, the conventional concrete with low w/c ratio experiences a considerable amount autogenous shrinkage deformation lead to an early age cracks and these premature cracks severely reduce the durability of a concrete. External water curing is one of the most conventional and well known applied curing methods to mitigate the autogeneous shrinkage however once the capillary pores depercolate, it will be more difficult to provide adequate external water for curing.

The main objective of this study is to examine the effect of internal curing as a complement to traditional curing in conventional concrete. Internal curing was achieved by super absorbent polymer (SAP) and the experimental parameter was percentage of SAP substitution to regular sand.

Super Absorbent Polymer

SAP before and after addition of Water

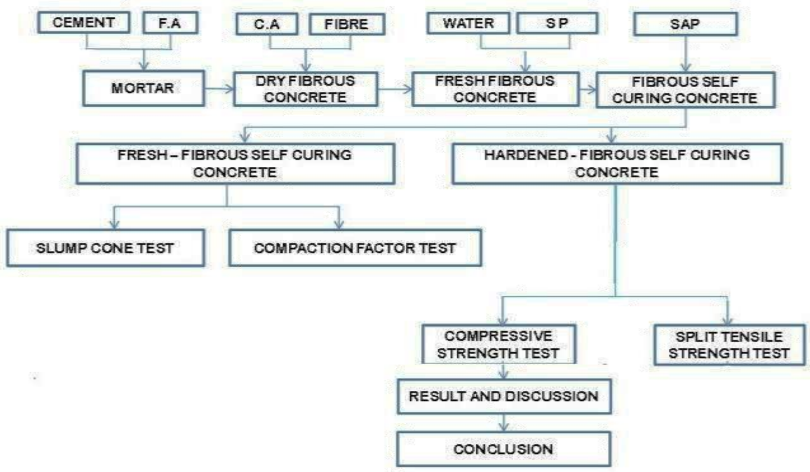
Properties	**Values**
FORM – dry	Crystalline white powder / granules
FORM – wet	Transparent gel
Particle size	0 – 1 mm
Water absorption with distilled water	800 g for 1 g
Water absorbed with sea water	40 g for 1 g
pH of absorbed water	Neutral
Density	1.08
Bulk density	0.85
Hydration / Dehydration	Reversible
Stability of dry product	5 years
Stability of wet product	4 years
Decomposition in sun light	6 months
Available water	95% approx.

Polyolefin Macro Monofilament (PMM)Fiber

PMM fibre is a macro-synthetic fibre designed specifically for the reinforcement of concrete and other cementitious mixes. These fibres have an engineered contour profile, which serves to effectively anchor the fibres into concrete thus resisting matrix pullout and enhancing the concrete's performance even after it has developed stress cracks.

Properties	**Values**
Material	Virgin Homo polymer
Length	45 mm
Type/Shape	Macro/ monofilament
Colour	White
Sp.Gravity	0.91
Acid and Salt resistance	High
Tensile Strength	620-758 MPa
Absorption	Nil
Fibre content	4-9 kg/m3 of concrete
Melting point	164° C (328° F)
Young's modulus	3.5 kN/mm2
Alkali resistance	Alkali Proof

Methodlogy



Conclusion

Being the part of my research initiative, Super Absorbent Polymer was used as self-curing agent with that Polyolefin Macro Monofilament Fibre of M25 grade of concrete is adopted for the investigation. Based on this investigation carried out, the conclusions were drawn:

1. Water retention for the concrete mixes by incorporating self-curing agent is higher when compared to conventional concrete mixes, as found by the weight loss with respect to time.
2. The effectiveness of internal curing by means of SAP applied to concrete is higher when 45 kg/ m3 water is added by means of 1 kg/m3 of SAP.
3. The Self-cured concrete using SAP was more economical than conventional cured concrete. The Performance of the self-curing agent is mainly affected by the cement content and the w/c ratio.

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AUTOCLAVED AERATED AND CELLULAR FLYASH CONCRETE BLOCKS FOR BUILDING CONSTRUCTIONS

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Abstract

The wide use of the flyash for building construction reduces the environmental pollution by flyash and this contributes to the environmental sustainability. Four Autoclaved and Aerated Cellular flyash-concrete Blocks (or Aerocon flyash concrete blocks) of Hyderabad Industries Limited of sizes in millimeters; 600 x 200 x 150, 600 x 200 x 200, 600 x 200 x 230, 600 x 200 x 100, are tested as per the Bureau of Indian Standards IS – 6441-1972 Part I, Part V and Part VI, for bulk density, moisture content, compressive strength and flexural strength. The bulk density ranges from 666.07kg/m3 to 731.88 kg/m3. The compressive strength range from 3.8 N/mm2 to 4.1N/mm2. The moisture content for the block 600x200x200 is found to be 0.94. The Flexural Strength for the block 600x200x150 is found to be 2.6 N/mm2 .The density values show that they have light weight because the density of the clay bricks is three times more than the Aerocon blocks. So therefore Aerocon blocks are ideal for additional floors in existing building. Aerocon blocks have the ease of the workability as they are easy to cut, shape, chisel with conventional carpenter's tools. The compressive strength and flexural strengths indicate that Aerocon blocks are suitable for load bearing and non-load bearing walls. Wide use of Aerocon and Aerated Cellular Flyash Concrete Blocks in building construction contributes to environmental sustainability as the environmental pollutant flyash is consumed.
***Keywords**: Autoclaved and Aerated Cellular flyash-concrete Blocks, Compressive strength, light weight, density, flexural strengths.*

1.0 Introduction

Clay bricks, the mainstay of building construction, are steadily giving way to a new, vastly improved building Blocks concept in India. The term Autoclaved means high pressure steam cured for added strength and dimensional stability to blocks. Aerocon blocks are an excellent substitute for clay bricks and hollow concrete blocks for construction of walls. Aerocon Blocks are eco-friendly and are ideal for modern day buildings, assuring quality construction and comfortable living. As the fly ash is the material which is available in abundance, the idea of introducing fly ash into bricks along with some other binding materials has added strength to the concrete block. The addition of flyash has reduced the weight of the block. Aerocon blocks are available for different thickness and with fine shape.

Aerocon blocks are made with a mixture of cement, fly ash, lime and water involving an aeration process that imparts its unique cellular structure. Aerocon Blocks are ideal material for all types of partitions due to their light weight, fire resistance and superior acoustic properties. Standard & Chartered Bank, Citi Bank, Infosys Technologies, Pentafour Technologies, Polaris, Sterling Software, HSBC, BANN, etc. are some of the reputed companies that have used Aerocon partition walls for their offices.

1.2 Types of AAC Blocks Available In the Construction Industry

The types of Autoclaved aerated concrete blocks available in the construction industry are as follows:

1. Aerocon HQ Blocks
2. Aerocon jumbo blocks
3. Aerocon infill blocks

1.3 Aerocon HQ Blocks

Aerocon HQ Blocks are made with a mixture of cement, fly ash, and special additives along with water involving areation process that imparts its unique cellular structure.
Aerocon Blocks are autoclaved i.e. high pressure steam cured. Autoclaving gives strength and dimensional stability to blocks. (Hyderabad Industried Ltd.,2007)

1.4 Aerocon Jumbo Blocks

Aerocon Jumbo Blocks are made with a mixture of cement, fly ash, lime and water involving an aeration process that imparts its unique cellular structure. Aerocon Jumbo Blocks are designed to get the benefits of their unique size in terms of mortar consumption, speed of construction, thermal insulation and ease of handling. They are ideal for internal divider and external load bearing or non load bearing walls. By using Jumbo Blocks, the saving in mortar is tremendous. The use of jumbo units greatly increases the speed of construction, as two units are equal to three regular blocks.

Some practical tests abroad have shown that productivity can be increased by up to 200% with jumbo blocks when compared with conventional block laying. The thermal
Performance of the wall is further enhanced when using jumbo blocks. Because larger block size means there are fewer joints, resulting in less heat loss through the mortar.

Applications of jumbo blocks :
i) Suitable for both load-bearing and non load-bearing walls.
ii) Ideal for multi-storeyed buildings.
iii) Most suitable where thermal comforts at home and savings in air-conditioning cost are desired.
iv) Suitable for all fire-rated applications.

1.5 Aerocon Infill Blocks

Aerocon Infill Block is mixer of cement, fly ash, lime, aluminum powder and water involving an aeration process that imparts its unique cellular structure. Aerocon Infill
blocks are Autoclaved i.e. High-pressure steam cured which gives strength and dimensional stability to blocks.

Applications:
• Any Roof with large column free space constructions.
• Commercial and multistoried buildings.
• Extension of unplanned floors.
• Fire rated buildings.
• Air-conditioned buildings.

1.6 Properties of Aerocon HQ Blocks

The following are the properties of Aerocon HQ Blocks.

1.6.1 Light Weight

i) 1/3rd the density of clay bricks
ii) Ideal material for additional floors in existing buildings

iii) Economic design: savings in cement and steel
iv) Enables faster construction
v) Suitable for low-soil bearing capacity & seismic zones

1.6.2 High Thermal Insulation

i) Low thermal conductivity
ii) Thermal performance 5 times better than clay brick & 10 times better than RCC.
iii) Interiors remain cool in summer and warm in cold wintry days.
iv) Savings in recurring energy costs in air-conditioning.
v) Ideal material for applications in cold storage rooms

1.6.3 Good Sound Insulation

i) Can fulfill required STC (Sound Transmission Class) rating.
ii) Upto 37-42dB sound reduction based on thickness.
iii) Ideal material for wall construction in hotels, auditoriums, studios, hospitals etc.

1.6.4 Fire Resistance

Aerocon Blocks are appropriate for fire-rated applications for desired safety. 100 & 200 mm Aerocon Blocks, tested at CBRI, Roorkee, withstood standard fire test for 4 hours.

1.7 Advantages of Aerocon Blocks

The following are the advantages of Aerocon Blocks.

1.7.1 Perfect Finish

Aerocon Blocks are factory-finished with precise edges and shape. Precision cut results in economical finish in internal partition walls. Accurate size and shapes help in reducing plastering costs.

1.7.2 Big Savings

i) Enormous saving of time and labour.
ii) Substantial savings in cement & steel consumption.
iii) More carpet area.
iv) Minimal wastages

1.7.3 Ease of Workability

Aerocon Blocks are easy to cut, shape and chisel with conventional carpenter's tools to achieve the desired wall pattern. Concealed wiring and plumbing can be carried out with ease

1.7.4 Flexible Wall Thickness by Design

Aerocon is an engineered building product that sets free from routine use of 230mm (9") and 115mm (4-1/2") thickness for external and internal walls. It offers a wide range of thicknesses ranging from 75mm to 230mm. Aerocon provide you the flexibility to design your wall thickness as per your needs. The standard face size is 600mm x 200mm.

1.7.5 Fixing

Aerocon walls provide one of the most amenable of backgrounds for the application of fixtures and fittings. It does not suffer either the flimsiness of wood partition walling or the difficult-to-work characteristics of brick/concrete walls. Aerocon offers the perfect degree of solidity combined with supreme workability. Aerocon is an excellent substrate for strong, reliable fixtures and fittings when hanging even moderately heavy items such as radiators and wall cupboards. Cut nails can be driven directly into Aerocon for fixing skirting boards, timber battens or lightweight doors linings, etc. For light duty and heavy duty fixings, plastic plugs with screws can be easily driven into Aerocon.

1.8 Relevant Codes Of Bureau Of Indian Standards Dealing With Aerocon Blocks

The following are the relevant codes of Bureau of Indian Standard for Autoclaved Cellular Concrete.

1.8.1 IS 2185(Part-III) 1984: Specification for Auto Claved Cellular Concrete (AAC)

The specifications for AAC are described in this code.

1.8.2 IS6041-1985 (Reaffirmed 1990): Code Of Practice For Construction Of AAC Block Masonry

The construction practices of AAC block masonry are described in this code.

1.8.3 IS6441-1972 (Part-I): Determination of Unit Weight Or Bulk Density And Moisture Content

This code gives the procedure to find Bulk Density and Moisture content of Autoclaved Cellular Product.

1.8.4 IS6441-1972 (Part-II): Determination of Drying Shrinkage

This code gives the procedure to find Drying shrinkage of Aerocon blocks

1.8.5 IS6441-1972 (Part-III): Determination Of Thermal Conductivity

The Thermal conductivity of AAC blocks can be determined by following the procedure enclosed in this IS code.

1.8.6 IS6441-1972 (Part-IV): Corrosion Protection Of Steel Reinforcement In Autoclaved Cellular Concrete

In this code, the testing procedure to find the corrosion protection of steel Reinforcement in AAC block.

1.8.7 IS6441-1972 (Part-V): Determination of Compressive Strength

This code specifies the testing procedure for Aerocon cellular product including preparation of specimen for determining compressive strength.

1.8.8 IS6441-1972 (Part-VI): Strength, Deformation And Cracking Of Flexural Members Subject To Bending-Short Duration Loading Test.

The procedure to find the flexural strength of AAC is specified in this code.

1.9 Testing Of Aerocon Flyash Blocks In the Structural Laboratory of Mufffakham Jah College of Engineering and Technology (MJCET)

The four Aerocon blocks of Hyderabad Industries limited of sizes in millimeters 600*200*100, 600*200*200, 600*200*150, 600*200*230 are tested in the structural Laboratory of Muffakham Jah College of Engineering and Technology as per the Bureau of Indian standards IS6441-1972, part-I, part-V and part-VI for determining the bulk density, moisture content, compressive strength and flexural strength. The results are tabulated in Table 1.9.1 and Table 1.9.2.

S.No.	Size of Block	Dry weight (kgs)	Volume in m^3	Bulk density (kg/m^3)	Compressive Strength (N/mm^2)
1	600*200*100	8.6	0.012	716.67	4.1
2	600*200*150	12	0.018	666.067	4
3	600*200*200	16.3	0.024	679.17	3.9
4	600*200*230	20.2	0.0276	731.88	3.8

1.9.2 Result Of Test For Moisture Contents And Flexural Strength

The flexural strengths and moisture contents of tested Aerocon flyash - concrete blocks are shown in Table 1.9.2

S.No.	Size of Block	Moisture content	Flexural strength
1	600*200*150	——-	2.6
2	600*200*200	0.94	——-

2.0 Conclusion

The following conclusions are drawn from the study on characteristics of Autoclaved and Aerated Cellular flyash-concrete Blocks. Four Aerocon blocks available in the market (Hyderabad Industries Limited) are procured. The block sizes in 'mm' are 600x200x100, 600x200x200, 600x200x150, 600x200x230. They are tested in the structural engineering laboratory of Muffakham Jah college of Engineering and Technology as per the code IS 6441-1992 part-I, part- V, part-VI to evaluated the bulk density, moisture content, compressive strength and flexural strength (Table 1.9.1 and 1.9.2).

1. The bulk density ranges from 666.07kg/m3 to 731.88 kg/m3.
2. The compressive strength range from 3.8 N/mm2 to 4.1N/mm^2.
3. The moisture content for the block 600x200x200 is found to be 0.94. The Flexural strength for the block 600x200x150 is found to be 2.6 N/mm2.
4. The density values show that they have light weight because the density of the clay bricks is three times more than the Aerocon blocks.
5. The compressive strength and flexural strengths indicate that Aerocon blocks are suitable for load bearing and non-load bearing walls.

Therefore Aerocon blocks are ideal for additional floors in existing building. Aerocon blocks have the ease of the workability as they are easy to cut, shape, chisel with conventional carpenter's tools.

Recommendations

Wide use of Aerocon and Aerated Cellular Flyash Concrete Blocks in building construction contributes to environmental sustainability as the environmental
Pollutant flyash is consumed.

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AN EXPERIMENTAL INVESTIGATION ON SILICA FUME ON STEEL SLAG CONCRETE

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Abstract

Concrete is prepared by mixing various constituents like cement, aggregates, water, etc. which are economically available. Concrete is a composite material composed of granular materials like coarse aggregates embedded in a matrix and bound together with cement or binder which fills the space between the particles and glues those together (NSA, 1982). Production of sand and gravel has increased at an annual rate of less than 1 percent. In essence the amount of crushed stone to be produced in the next 20 years will equal the quantity of all stone produced during the previous century, i.e., about 36.5 billion metric tons. Therefore the use of alternative sources for natural aggregates is becoming increasingly important (NSA, 1982). In the present study the mechanical properties of concrete by replacing cement with different percentages of Silica fume and aggregate by different percentages of Steel slag are studied. The results thus obtained are analyzed using Regression analysis. Results indicated that the replacement of cement by silica fume to the extent of 15% exhibited improved mechanical properties. Further, it has been also observed that with 15% replacement of cement by silica fume and replacement of natural aggregates by steel slag aggregates to the extent of 25% to 50% have shown improved strength compared normal concrete i.e. the concrete with 0% silica fume and 0% steel slag aggregates. Regression analysis has been carried out to compare the experimental results.

Keywords*: Concrete, Steel slag, silica fume, properties of concrete, regression analysis*

Introduction

Concrete plays a critical role in the design and construction of the nation's infrastructure. Almost three quarters of the volume of concrete is composed of aggregates. To meet the global demand of concrete in the future, it is becoming a more challenging task to find suitable alternatives to natural aggregates for preparing concrete (Maslehuddin *et al*., 2003). Abdullah (2004) has concluded that supplementary cementing materials, such as silica fume may be advantageously used in situations where good quality aggregates are not available. Saud al Otaibi (2008) has studied on replacement of sand by steel mill has concluded that 40% replacement of sand with steel mill scale gave the highest increase in compressive strength. Li Yun feng (2009) found that concrete with good performance can be produced using mineral admixtures consisting of steel slag powder and blast furnace slag. In the present paper an experimental investigation on the mechanical properties of concrete has been made with 0% to 20% replacement of cement by silica fume and 0% to 75% replacement of natural aggregate by steel slag. Results exhibited that concrete with 15% silica fume and up to 50% replacement of natural aggregate by steel slag, show better mechanical properties.

Material and Methods

A wide range of materials can be used as an alternative to natural aggregates. When any new material is used as a concrete aggregate, three major considerations are relevant: (1) economy, (2) compatibility with other materials and (3) concrete properties. The economical use of non-traditional materials in concrete depends on various factors, like transportation required to bring the materials from industry to the site of construction, quantity available, and the mix design requirements. In many situations sources are located very far from their potential markets for concrete with high transportation costs. The separation of any useful materials from undesired substances would be costly as well. Crushing the aggregates to particular sizes is also an important issue. The aggregates should not react adversely with other constituents of the concrete mixture. They should not change the properties of the concrete adversely. The aggregates have vital role in concrete and provide strength and durability to concrete. The use of industrial byproducts in the concrete has received increasing attention in the recent years. Blast furnace slag has been used as an aggregate for asphalt concrete and also as a cementitious material in concrete. Steel slag which is also an industrial byproduct has a potential to be utilized as an aggregate in concrete as well.Steel slag is a byproduct obtained either from conversion of iron to steel in a Basic Oxygen Furnace (BOF), or by the melting of scrap to make steel in the Electric Arc Furnace (EAF). The molten liquid is a complex solution of silicates and oxides that solidifies on cooling and forms steel slag. Steel slag is defined by the American Society for Testing and Materials as a non-metallic product, consisting essentially of calcium silicates and ferrites combined with fused oxides of iron, aluminum, manganese, calcium and magnesium that are developed simultaneously with steel in basic oxygen, electric arc, or open hearth furnaces. The two states producing the most steel slag in the U.S. are Ohio and Indiana. The chemical composition and cooling of molten steel slag have a great effect on the physical and chemical properties of solidified steel slag (Tables 1 and 2) (Maslehuddin *et al.*, 2003).

Silica fume is a by-product resulting from the production of silicon or ferrosilicon alloys or other silicon alloys. Silica fume is light or dark gray in colour containing high content of amorphous silicon dioxide. Silica fume powder as collected from waste gasses without further treatment is sometimes referred to as undensified silica fume to distinguish it with other forms of treated silica fume. Undensified silicon fume consists of very fine vitreous spherical particles with average diameter about 0.1micro meter, which is 100 times smaller than the average cement particle. The undensified silica fume is almost as fine as cigarette ash and the bulk density is only about 200-300 kg/m^3 and relative density of typical silica fume particle is 2.2 to 2.5. Because the extreme fineness and high silicon content, silica fume is a highly effective Pozzolona (Table 3) (Dotto *et al*., 2004).

The mix design procedure adopted to obtain a M25 grade concrete is in accordance with IS 10262- 2009. The specific gravities of the materials used were as tabulated in the Table 4. The mix proportion adopted was given in Table5.

Table 1: Sieve Analysis

Sieve size (mm)	**Wt Retain (g)**	**Cum Wt Retain (kg)**	**% Cu Wt Retain (kg)**	**% of Passing**
20	270	0.270	5.4	94.6
12.5	3522	3.792	75.84	21.16
10	790	4.582	91.64	8.36
4.75	334	4.916	98.62	1.68
Total	5000			

Table 2: Physical Composition of Steel slag

Property	Value
Specific Gravity	3.2-3.6
Water Absorption	0.85%
Clay lumps and friable particles	0.12%
Loss on abrasion	11.60%

Table 3: Chemical Composition of Silica Fume

SiO_2	92.85
Al_2O_3	0.61
Fe_2O_3	0.94
CaO	0.39
MgO	1.58
K_2O	0.87
Na_2O	0.5

Table 4: Specific gravities of materials used

Material	Specific Gravity
Cement	3.15
Fine aggregate	2.64
Coarse aggregate	2.72

Table 5: Mix Proportions

W/C ratio	Cement	Fine Aggregate	Coarse Aggregate
0.50	372	662 kg/m^3	1212 kg/m^3
0.50	1	1.77	3.25

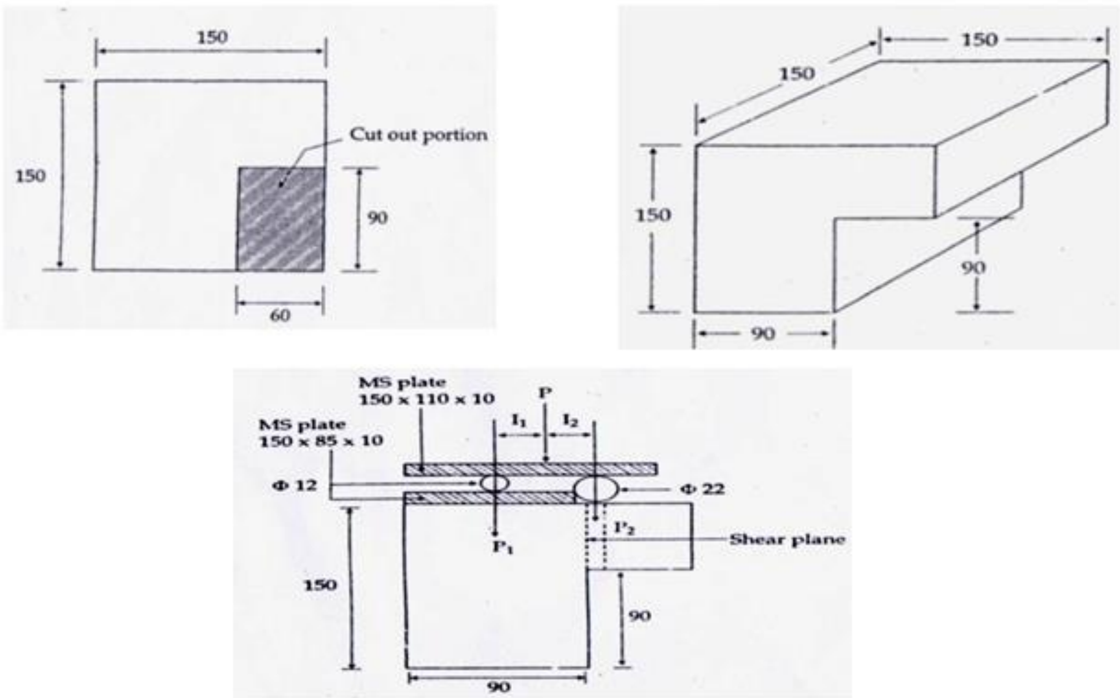
Casting of Specimens and Testing Procedure

Cement, sand and aggregate were taken in mix proportion 1:1.77:3.25 which correspond to M25 grade of concrete. Cement has been replaced with Silica fume (0%, 5%, 10%, 15%, and 20%) and coarse aggregate has been replaced with Steel slag (0%, 25%, 50%, 75%, and 100%). All the ingredients were dry mixed homogeneously. To this dry mix, required quantity of water was added (w/c = 0.48) and the entire mix was again homogeneously mixed. This wet concrete was poured into the moulds which was compacted through hand compaction in three layers and then kept into the vibrator for compaction. After the compac- tion, the specimens were given smooth finishes and were covered with gunny bags.

After 24 h, the specimens were demoulded and transferred to curing tanks where in they were allowed to cure for 28 days. To study the effects of replacements of cement and natural aggregates, have been studied on compressive strength, splitting tensile strength, flexural strength and shear strength as per IS 5816-1999 and IS 516-1959.

For evaluating the shear strength, L shaped specimens were prepared. A diagrammatic representation of the specimen is as shown in Figure 1. These specimens were tested on 2000 kN capacity compression testing machine. A loading arrangement was made such that a direct shearing force was applied on the shorter arm of the 'L' shaped specimen (i.e. over an area of × 60mm). The maximum applied load (P) was noted down.

Figure 1: Shear Test Specimen Arrangement



The failure load (F) due to the applied shear force is obtained by using the relation steel slag in addition to the replacement of cement by silica fume, it has been observed

Failure load (F) = $Pl_1 / (l_1 \times l_2)$

where, P = Failure load in kN

l_1 = 25mm

l_2 = 25mm

The shear strength is given by the relation Shear strength = F/A

where, F= Failure load

A = Area on which shear force is applied = 150mm 60mm

RESULTS AND DISCUSSION

Compressive Strength

- From Figure 2 it has been observed that there is increase in compressive strength with the increase in silica fume up to 15% beyond which it has shown downward trend. From Table 6 it has been observed that maximum increase in compressive strength of 11.69% was observed at 15% replace- ment of cement with silica fume alone. When natural aggregate were also replaced with that the compressive strength is more than the reference concrete up to 50% replace- ment of natural aggregates.

- It may be due to the fact that silica fume has smaller particle size which fills the voids and increases the strength or it may due to pozzolanic reaction between cement and silica fume.
- The compressive strength decreases when the percentage of steel slag increases from 0% to 75% to replace the natural aggre- gates. This is because of the porous nature of the steel slag.

Split Tensile Strength Test Results

- From Figure 3 it has been observed that the 15% replacement of cement with silica fume and 25% replacement of natural aggre-gates with steel slag show higher percentage of improvement in splitting tensile strength than compressive strength.
- Figure 3 shows similar variation in splitting tensile strength of concrete with replacement of cement with silica fume and natural aggregate with steel slag, but show less reduction in strength at higher steel slag compared to compressive strength.
- The splitting tensile strength decreases when the percentage of steel slag increases from 0% to 75% to replace the natural aggregates. This is because of the porous nature of the steel slag aggregates.

Figure 2: Variation of Compressive Strength

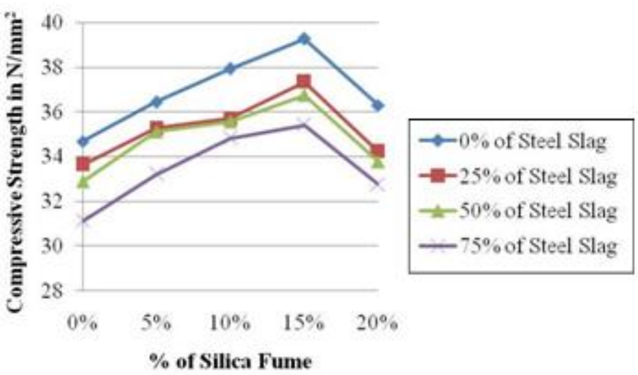


Table 6: Variation of Compressive Strength of M25 Concrete with Different Percentage of Replacement of Cement By Silica Fume and Natural Aggregate by Steel Slag Content

Percentage Replacement of cement by Silica	Percentage Replacement of NA by Steel Slag	Average Compressive	Percentage Variation w.r.t Normal Concrete
0%	0%	34.67	0
	25%	33.63	-2.99
	50%	32.89	-5.13
	75%	31.11	-10.26
5%	0%	36.44	4.85
	25%	35.26	1.67
	50%	35.11	1.25
	75%	33.19	-4.26
10%	0%	37.93	8.59
	25%	35.70	2.88
	50%	35.56	2.50
	75%	34.81	0.40
15%	0%	39.26	11.69
	25%	37.33	7.12
	50%	36.74	5.63
	75%	35.41	2.08
20%	0%	36.30	4.49
	25%	34.22	-1.29
	50%	33.78	-2.56
	75%	32.74	-5.56

Flexural Strength Test Results

- From Figure 4 it has been observed that there is increase in flexural strength with replacement of cement by silica fume alone and with natural aggregates by steel slag at 25% and 50% replacement, compared to normal concrete.
- From Table 8 it has been observed that 15% replacement of cement by silica fume and 25% replacement of natural aggregates by steel slag show highest increase in flexural strength. It has been observed that the contribution of silica fume and steel slag in increasing flexural strength at this replacement is marginal compared to the increase in compressive and splitting tensile strength.

Figure 4: Variation of Flexural Strength

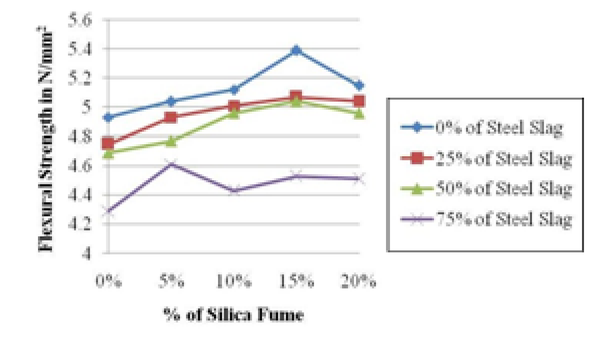


Table 7: Variation of Splitting Tensile Strength of M25 Concrete With Different Percentage of Replacement of Cement by Silica Fume and Natural Aggregate by Steel Slag Content

Percentage Replacement of cement by Silica	Percentage Replacement of NA	Average Compressive	Percentage Variation w.r.t Normal Concrete
0%	0%	3.21	0
	25%	3.16	-1.55
	50%	3.02	-5.91
	75%	2.93	-8.72
5%	0%	3.54	9.32
	25%	3.4	5.58
	50%	3.26	1.53
	75%	3.02	-5.91
10%	0%	3.63	11.53
	25%	3.54	9.32
	50%	3.3	2.27
	75%	3.07	-4.36
15%	0%	3.77	14.85
	25%	3.63	11.53
	50%	3.35	4.17

Table 8: Variation of Flexural Strength of M25 Concrete With Different Percentage of Replacement of Cement by Silica Fume and Natural Aggregate by Steel Slag Content

Percentage Replacement of cement by Silica	Percentage Replacement of NA by Steel Slag	Average Compressive	Percentage Variation w.r.t Normal Concrete
0%	0%	4.93	0
	25%	4.75	-3.65
	50%	4.69	-4.86
	75%	4.29	-
5%	0%	5.04	2.18
	25%	4.93	0
	50%	4.77	-3.24
	75%	4.61	-6.49
10%	0%	5.12	3.17
	25%	5.01	1.59
	50%	4.96	0.60
	75%	4.43	-
15%	0%	5.39	8.53
	25%	5.07	2.76
	50%	5.04	2.18
	75%	4.53	-8.11
20%	0%	5.15	4.27
	25%	5.04	2.18
	50%	4.99	1.20
	75%	4.43	-10.14

Shear strength test results

- From Figure 5 it has been observed that the maximum shear strength is achieved at the 15% replacement of cement by silica fume as observed in the case of compressive strength and tensile strength of M25 concrete. But the replacement of cement by silica fume and natural aggregates by steel slag, show higher strength at 15% and 50% replacement respectively.

  The Shear strength decreases when the percentage of steel slag increases from 0% to 75% to replace the natural aggregates. This is because of the porous nature of the steel slag.
- It has been observed that the maximum decrease in shear strength is obtained for 75% replacement of aggregates by steelslag.

Table 9: Variation of Shear Strength of Concrete With Different Percentage of Replacement of Cement By Silica Fume and Natural Aggregate by Steel Slag Content

Percentage Replacement of cement by Silica Fume	Percentage Replacement of NA	Average Compressive Strength in N/mm^2	Percentage Variation w.r.t Normal Concrete
0%	0%	5.74	0
	25%	5.3	-7.66
	50%	5.15	-10.27
	75%	4.93	-14.11
5%	0%	5.93	3.20
	25%	5.52	-3.83
	50%	5.26	-8.36
	75%	5.04	-12.19
10%	0%	6.07	5.43
	25%	5.74	0
	50%	5.37	-6.44
	75%	5.07	-11.67
15%	0%	6.3	8.88
	25%	5.85	1.88
	50%	5.44	5.22
	75%	5.26	-8.36
20%	0%	6.04	4.96
	25%	5.63	-1.91
	50%	5.3	-7.66
	75%	5.07	-11.67

REGRESSION ANALYSIS

Correlation between experimental strength and computed strength in the form of linear equation.

The linear equation is computed using Microsoft excel which uses the transformed regression model method of analysis.

The equation of the linear curve is

$$Z = a + bX_1 + cX_2 + dX_3 + eX_4$$

The normal equations are given by

$$S\Sigma = an + b\Sigma X_1 + c\Sigma X_2 + d\Sigma X_3 + e\Sigma X_4$$

$$\Sigma ZX_1 = a\Sigma X_1 + b\Sigma X_1^2 + c\Sigma X_1X_2$$

% of Cement X_1	% of Silica Fume X_2	% of NA X_3	% of Steel Slag X_4	Exp. Strength Z	X_1^2	X_2^2	X_3^2	X_4^2	X_1X_2	X_1X_3
100	0	100	0	34.67	10000	0	1000	0	0	10000
100	0	75	25	33.63	10000	0	5625	625	0	7500
100	0	50	50	32.89	10000	0	2500	250	0	5000
100	0	25	75	31.11	10000	0	625	562	0	2500
95	5	100	0	36.44	9025	25	1000	0	475	9500
95	5	75	25	35.26	9025	25	5625	625	475	7125
95	5	50	50	35.11	9025	25	2500	250	475	4750
95	5	25	75	33.19	9025	25	625	562	475	2375
90	10	100	0	37.93	8100	100	1000	0	900	9000
90	10	75	25	35.7	8100	100	5625	625	900	6750
90	10	50	50	35.56	8100	100	2500	2500	900	4500
90	10	25	75	34.81	8100	100	625	5625	900	2250
85	15	100	0	39.26	7225	225	10000	0	1275	8500
85	15	75	25	37.33	7225	225	5625	625	1275	6375
85	15	50	50	36.74	7225	225	2500	2500	1275	4250
85	15	25	75	35.41	7225	225	625	5625	1275	2125
80	20	100	0	36.3	6400	400	10000	0	1600	8000
80	20	75	25	34.22	6400	400	5625	625	1600	6000
80	20	50	50	33.78	6400	400	2500	2500	1600	4000
80	20	25	75	32.74	6400	400	625	5625	1600	2000
1800	180	1250	750	702.08	163000	3000	93750	43750	17000	11250

X_1X_4	X_2X_3	X_2X_4	X_3X_4	Z^2	X_1Z	X_2Z	X_3Z	X_4Z	Computed Strength
0	0	0	0	1202.01	3467	0	3467	0	35.82
2500	0	0	1875	1130.98	3363	0	2522.2	840.75	34.74
5000	0	0	2500	1081.75	3289	0	1644.5	1644.5	33.66
7500	0	0	1875	967.83	3111	0	777.75	2333.2	32.57
0	500	0	0	1327.87	3461.8	182.2	3644	0	36.28
2375	375	125	1875	1243.27	3349.7	176.3	2644.5	881.5	35.19
4750	250	250	2500	1232.71	3335.45	175.5	1755.5	1755.5	34.11

7125	125	375	1875	1101.58	3153.05	165.9	829.75	2489.2	33.03
0	1000	0	0	1438.68	3413.7	379.3	3793	0	36.73
2250	750	250	1875	1274.49	3213	357	2677.5	892.5	35.65
4500	500	500	2500	1264.51	3200.4	355.6	1778	1778	34.57
6750	250	750	1875	1211.74	3132.9	348.1	870.25	2610.7	33.48
0	1500	0	0	1541.35	3337.1	588.9	3926	0	37.19
2125	1125	375	1875	1393.53	3173.05	559.9	2799.7	933.25	36.10
4250	750	750	2500	1349.83	3122.9	551.1	1837	1837	35.02
6375	375	1125	1875	1253.87	3009.85	531.15	885.25	2655.75	33.94
0	2000	0	0	1317.69	2904	726	3630	0	37.64
2000	1500	500	1875	1171.01	2737.6	684.4	2566.5	855.5	36.56
4000	1000	1000	2500	1141.09	2702.4	675.6	1689	1689	35.48
6000	500	1500	1875	1071.91	2619.2	654.8	818.5	2455.5	34.39
67500	1250	7500	3125	24717.6	63096.	7111.	44556	25652	702.15

Compressive Strength

From the above linear equation, correlation coefficient for compressive strength equation is

$Z_c = -0.0911X_1 + 0.4493X_3 + 0.406X_4$ ***where, Z = Computed 28-day strength,***

Standard Deviation(S_d)

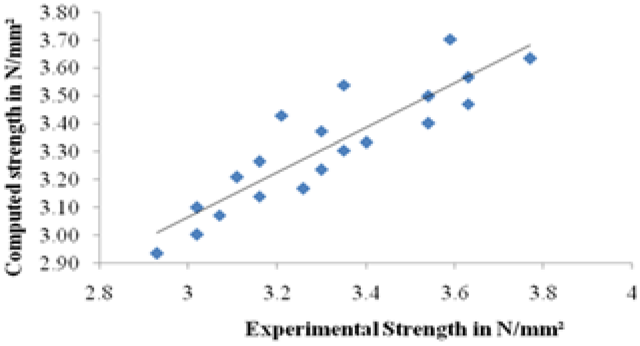
The standard deviation is calculated as follows

$$S_d = \sqrt{\frac{\Sigma Z^2}{n} - \left(\frac{\Sigma Z}{n}\right)^2}$$

$$S_d = \sqrt{\frac{24717.69}{20} - \left(\frac{702.08}{20}\right)^2}$$

$S_d = 1.895$

Figure 6: Best fit curve for Compression strength



Split Tensile Strength

The best-fit linear equation for split tensile strength is

Standard Deviation

SD = 0.23

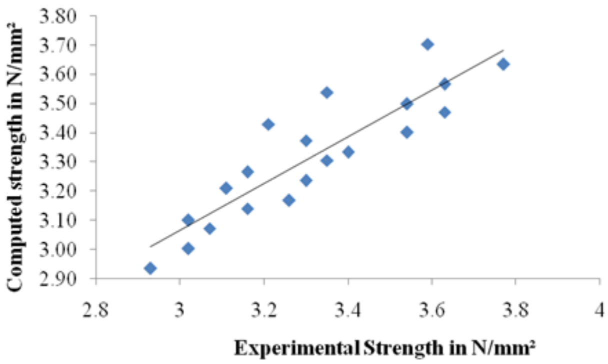
Standard Error

S = 0.011

Correlation coefficient

r = 0.89

Figure 7: Best fit curve for Split Tensile Strength



Flexural Strength

The best-fit linear equation for flexural strength is

Standard Deviation

SD = 0.27

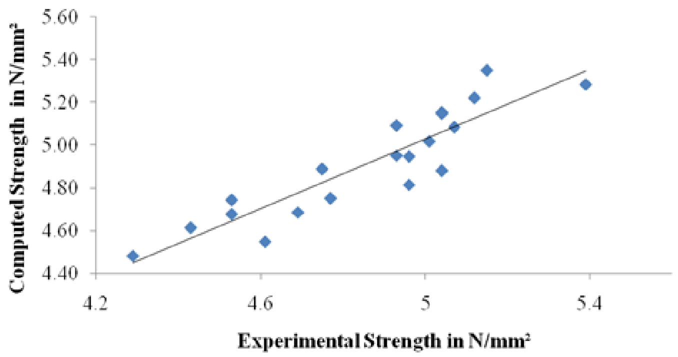
Standard Error

S = 0.12

Correlation coefficient

r = 0.91

Figure 8: Best fit curve for flexural Strength



CONCLUSION

Based on the experimental results obtained for different percentages of replacement of cement and aggregates in concrete the following conclusions may be drawn.

1. Replacement of cement with silica fume up to 15% increases the compressive strength, splitting tensile strength, flexural strength and shear strength of M25 concrete.
2. Replacement of cement by 15% with silica fume and natural aggregates by up to 50% Steel slag shows improved compressive strength, Tensile strength, Flexural strength and Shear strength than the normal concrete.
3. The specimens are failed in 100% replacement of aggregate by steel slag.
4. As the percentage of Steel slag increases there is decrease in workability of concrete. Especially replacement beyond 50%
5. Regression analysis of the experimental results, it can be seen that linear relationship gives permissible coefficients of correlation and it may be preferred for its simplicity and suitability to statistical analysis.
6. The standard deviation of the steel slag concrete is in the range of 0.23 to 1.89. This standard deviation of the experimental results is well within the permissible limits.

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COMPRESSIVE STRENGTH INVESTIGATION ON TERNARY BLENDED CONCRTE AND NORMAL CONCRETE

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Abstract

The main aim of the project is to study the strength properties of ternary blended concrete using Fly Ash and Metakaolin. The compressive strength of ternary blended concrete using 5%, 10% & 15% Metakaolin and 20%, 30%, 40% & 50% Fly Ash is added by the weight of the cement as additional ingredients and Super Plasticizer in concrete mixes with water/cement ratio 0.45 for 3 days, 7 days, 28 days and 56 days is determined respectively. The results shows that for 15% Metakaolin and 50% Fly Ash combination gives better strength out of all the combinations.

Keywords: *Fly Ash, Metakaolin, Compressive strength, Super Plasticizer, Ternary Blended Concrete.*

1. Introduction

Metakaolin is a natural pozzolanic material. High-reactivity metakaolin (MK) is a supplementary cementitious material developed recently for standard grade of concrete. It is obtained from KOAT manufacture company, Vadodara , Gujarat. It is a natural mineral kaolinite. Metakaolin is produced by heating the kaoline clay at the temperature of 6500 C to 8500 C.

Fly Ash is an artificial pozzolanic material. Fly Ash used in this investigation was procured from Vijayawada thermal power station, Andhra Pradesh. It confirms with grade I of IS 3812-1981.Fly Ash is an industrial byproduct obtained from thermal power stations.

2.1Physical properties of Metakaolin used in this investigation:

Colour	White
Bulk density	356 gm/litre
Moisture	0.22 %
Fineness	12000 m^2/kg
Average particle size	1 µm
Specific gravity	2.52

2.2Chemical properties of Metakaolin used in this investigation:

Characteristics	Percentage(%)
Silica	52.1
Alumina	36.1
Iron Oxide	4.3
Lime	0.1
Magnesia	0.84

Alkalis	1.38
Loss on Ignition	0.4

2.3Physical properties of Fly Ash used in this investigation:

Fineness	364 m^2/kg
Specific Gravity	2.15

2.4Chemical properties of Fly Ash used in this investigation:

Characteristics	**Percentage(%)**
Silica	61
Alumina	30.3
Iron Oxide	3.93
Lime	0.80
Magnesia	0.4
Sulphur Trioxide	<0.01
Alkalis	1.16
Loss on Ignition	0.82

3.OBJECTIVE:

The present work aims to determine the most suitable mix proportions that can produce metakaolin and fly ash based ternary blended standard grade concrete to obtain the early strength, later strength and enhanced performance also studies are carried out to understand the combination of fly ash and metakaolin as a partial replacement of cement without compromising on engineering performance and quality.

4.TERNARY BLENDED CONCRETE:

Ternary blended concrete is a concrete made up of three materials i.e cement and two supplementary binding materials with aggregates, water and superplasticizers(if necessary).The present experimental work is carried out to form concrete in two groups i.e 1. By using cement only and 2. By using partial replacement of cement with fly ash and metakaolin. Here reference mix is only cement, afterwards these results are compared with combination of cement, fly Ash and metakaolin.

In ternary blends of standard grade, metakaolin blended, fly ash based mix is considered as more efficient than reference mix in terms of cost, more usage of industrial byproduct and achieve early compressive strength due to metakaolin and later strength due to fly ash.

5.RESULTS AND DISCUSSIONS:

The test results of experimental investigation carried out during the development of standard Ternary blended concrete mixes made with metakaolin and Fly Ash tabulated below.

5.1 Quantities of Materials Required per 1 m^3 of normal Concrete

Sl.No.	w/c Ratio	Cement(Kgs)	F.A(Kgs)	C.A(Kgs)	Water(Ltrs)
1.	0.45	338	731	1102	152

5.2 Quantities of Materials Required per 1 m^3 of Ternary Blended Concrete (5%M.K+20, 30, 40, 50%FLYASH)

Sl.No	w/c Ratio	Percentages Of Binder materials	Cement (Kgs)	Metakaoline (Kgs)	Flyash (Kgs)	F.A (Kgs)	C.A (Kgs)	Water (Ltrs)
1	0.45	5%M.K+20% FLYASH	253.5	16.9	67.6	731	1102	152
2	0.45	5%M.K+30%FLYASH	219.7	16.9	101.4	731	1102	152
3	0.45	5%M.K+40%FLYASH	185.9	16.9	135.2	731	1102	152
4	0.45	5%M.K+50%FLYASH	152.1	16.9	169	731	1102	152

5.3 Quantities of Materials Required per 1 m^3 of Ternary Blended Concrete

(10%M.K+20, 30, 40, 50%FLYASH)

Sl.No	w/c Ratio	Percentages Of Binder materials	Cement (Kgs)	Metakaoline (Kgs)	Flyash (Kgs)	F.A (Kgs)	C.A (Kgs)	Water (Ltrs)
1	0.45	10%M.K+20% FLYASH	236.6	33.8	67.6	731	1102	152
2	0.45	10%M.K+30%FLYASH	202.8	33.8	101.4	731	1102	152
3	0.45	10%M.K+40%FLYASH	169	33.8	135.2	731	1102	152
4	0.45	10%M.K+50%FLYASH	135	33.8	169	731	1102	152

5.4 Quantities of Materials Required per 1 m^3 of Ternary Blended Concrete

(15%M.K+20, 30, 40, 50%FLYASH)

Sl.No	w/c Ratio	Percentages Of Binder materials	Cement (Kgs)	Metakaoline (Kgs)	Flyash (Kgs)	F.A (Kgs)	C.A (Kgs)	Water (Ltrs)
1	0.45	15%M.K+20% FLYASH	219.7	50.7	67.6	731	1102	152
2	0.45	15%M.K+30%FLYASH	185.9	50.7	101.4	731	1102	152
3	0.45	15%M.K+40%FLYASH	152.1	50.7	135.2	731	1102	152
4	0.45	15%M.K+50%FLYASH	118.3	50.7	169	731	1102	152

6. Compressive Strength of Normal Concrete and Ternary Blended Concrete of w/c ratio 0.45

6.1 Compressive Strength of Normal Concrete at 3 Days,7Days, 28 Days, 56 Days

SL.NO	w/c Ratio	Compressive Strength(N/mm^2)			
		3 DAYS	7 DAYS	28 DAYS	56 DAYS
1.	0.45	11.4	23.52	49.56	52.20

6.2 Compressive Strength of Ternary Blended Concrete at 3 Days,7Days, 28 Days, 56 Days

SL.NO	w/c Ratio	%METAKAOLINE	%FLYASH	Compressive Strength(N/mm^2)			
				3DAYS	7DAYS	28DAYS	56 DAYS
1	0.45	5%	20%	12.78	23.82	48.2	52.88
2	0.45	5%	30%	13.2	24.39	47.66	53.04
3	0.45	5%	40%	13.96	25.6	47.29	53.96
4	0.45	5%	50%	14.65	26.2	46.06	54.87

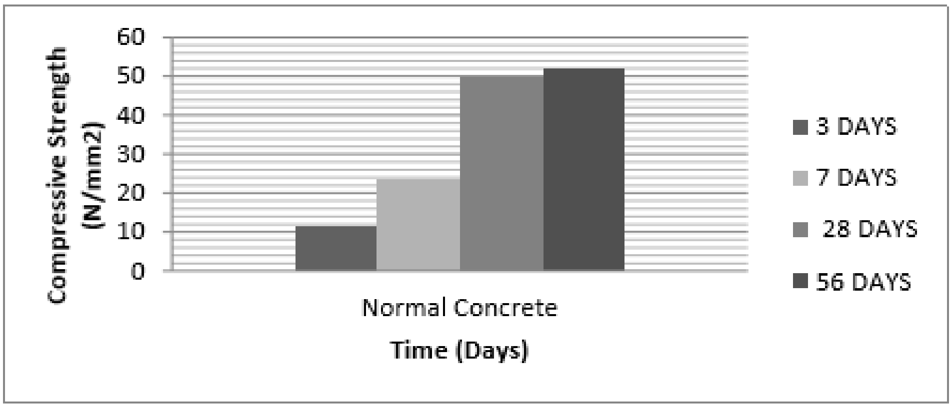
6.3 Compressive Strength of Ternary Blended Concrete at 3 Days,7Days, 28 Days, 56 Days

SL.NO	w/c Ratio	%METAKAOLINE	%FLYASH	Compressive Strength(N/mm^2)			
				3DAYS	7DAYS	28DAYS	56 DAYS
1	0.45	10%	20%	13.4	24.33	49.7	55.03
2	0.45	10%	30%	13.87	25.3	51.06	56.47
3	0.45	10%	40%	14.59	25.9	52.77	56.9
4	0.45	10%	50%	15.07	26.6	53.61	57.8

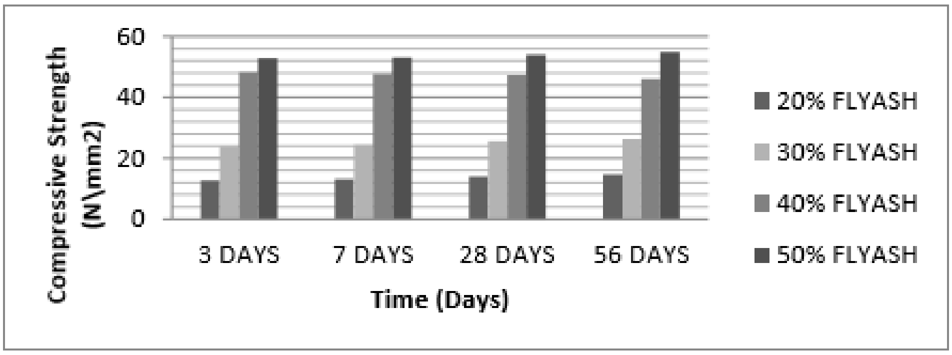
6.4 Compressive Strength of Ternary Blended Concrete at 3 Days,7Days, 28 Days, 56 Days

SL.NO	w/c Ratio	%METAKAOLINE	%FLYASH	Compressive Strength(N/mm^2)			
				3 DAYS	7 DAYS	28 DAYS	56DAYS
1	0.45	15%	20%	14.7	26.03	51.2	58.06
2	0.45	15%	30%	15.26	26.54	52.46	59.67
3	0.45	15%	40%	16.65	27.11	54.3	60.9
4	0.45	15%	50%	17.1	27.87	55.5	61.24

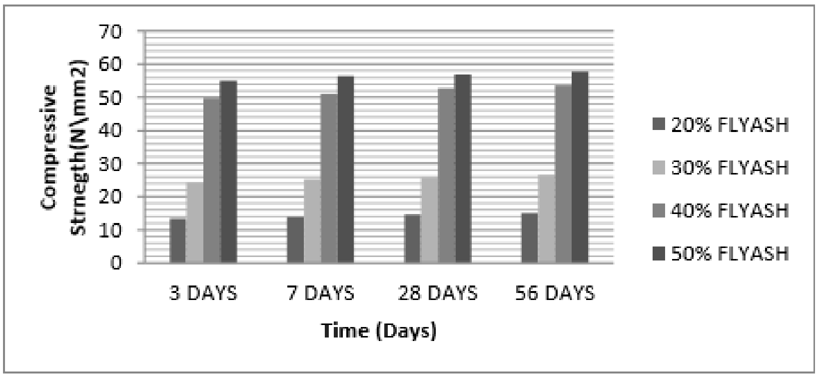
7.Graphs Showing Compressive Strength in N\mm^2 with respect to Time in Days



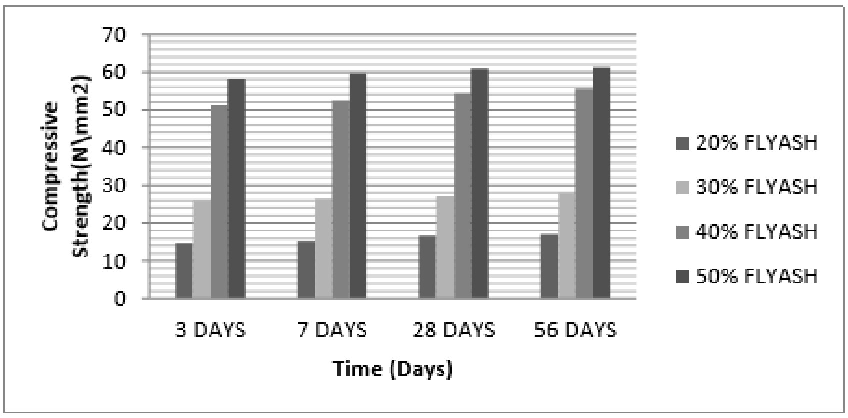
Graph 7.1 Shows Compressive Strength of Normal Concrete Vs Time



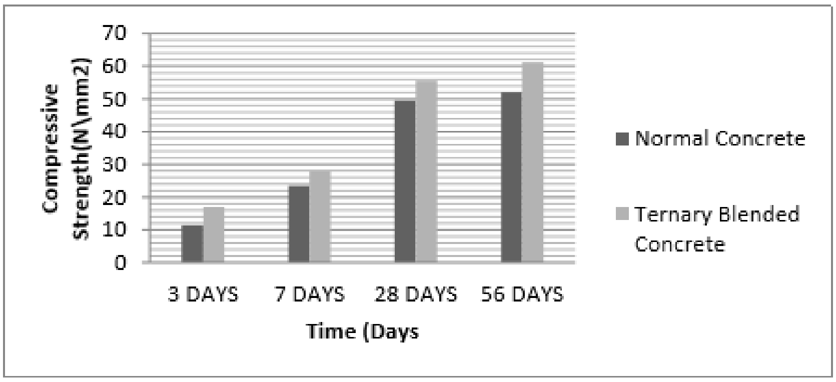
Graph 7.2 Shows Compressive Strength of Ternary Blended Concrete Vs Days for the Combinations (5% METAKAOLINE +20, 30, 40&50%FLYASH)



Graph7.3 Shows Compressive Strength of Ternary Blended Concrete Vs Days for the Combinations (10% METAKAOLINE +20, 30, 40&50%FLYASH)



Graph7.4 Shows Compressive Strength of Ternary Blended Concrete Vs Time for the Combinations (15% METAKAOLINE +20, 30, 40&50%FLYASH)



Graph7.5 Shows Compressive Strength of Normal Concrete and Ternary Blended Concrete (15% METAKAOLINE +50%FLYASH) VsTime

8. CONCLUSIONS:

The following conclusions are drawn from the experimental investigation in present thesis:
1 The combination of (15%METAKAOLINE + 50%FLYASH) performed the best of all the combinations studied at 3days, 7days 28 days and 56 days respectively.
2 The combination of 5% METAKAOLINE with different percentages of FLYASH gave the least Compressive Strength among the ternary mixes at all W/B ratios.
3 The percentage increase in compressive strength of Ternary Blended Concrete is found to be higher at higher ages for all combinations.

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FAILURE OF PAVEMENTS – CAUSES, STEPS TO IMPROVE AND STABILISATION OF ROADS ON POOR SUBGRADE SOILS

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1.0 INTRODUCTION

As soon as a road is constructed and brought into use, it begins to deteriorate, due to the effects of both traffic and weather. A road should be designed for an economic lifespan of 20 years or more. Unfortunately, today, this is not the case. Some of our Roads do not last for up to five years before they begin to show signs of failure. Highway failure occurs when it can no longer perform its traditional function of carrying vehicles and passengers from one location to another in safety and comfort. When such failure occurs before the anticipated design life i.e. either before the substantial completion of the highway project or few years after completion of construction, it is rightfully described as premature. Highway premature failures occur within the highway pavement structure or the road foundation or both. It may also be as a result of problems beside the highway with its consequent effect on the pavement and/or foundation. The result of premature failures of our Highways includes: frequent road accidents with loss of lives and properties, high transportation costs of goods and services, high Government budgeting and spending on road transportation sector, high maintenance cost for vehicles, and discomfort to motorists. It is therefore necessary that occurrence of such failure be avoided as much as possible. In this paper the causes of premature failures on roads, steps to control the failures and stabilization aspects of poor subgrade soils is discussed.

2.0 CAUSES OF PREMATURE FAILURE OF HIGHWAYS

The causes of premature failures on our Highways could be traced immediately to the stages of development of road such as in the Planning, Design and Construction as well as other crucial matters beyond engineering profession or the public service as given below.

(a) Planning and Design

(i) Lack of current traffic studies and Analysis.
(ii) Insufficient hydrological data
(iii) Insufficient geo-technical studies
(iv) Use of inappropriate design

(b) Construction

(i) Choice of Contractors,
(ii) Adequate quality control, Use of substandard, poor or adulterated materials
(iii) Adequate Supervisory/Technical Staff.

(c) Other crucial matters that could cause premature failure

(i) Human and Environmental Factors in project execution,
(ii) Wrong timing of contract awards,
(iii) Interference of Project Supervision and Management,
(iv) Delay in payment to contractors,
(v) Poor maintenance culture,
(vi) Abuse of drains,
(vii) Excessive axle loading,
(viii) Inadequate Training and retraining of Supervisory/Technical Staff and
(xi) Collapse of other modes of transport.

3.0 TYPES OF FAILURES IN FLEXIBLE PAVEMENTS

Different types of failures in flexible pavements and causative factors are discussed in this section.

Pot – holes at isolated locations: Presence of isolated weak spots due to defective materials or improper compaction or segregation of mix during the construction of the surfacing course or any other pavement layer; stagnation of water below the pavement or in any of the pavements layers for prolonged period.

Detachment of thin bituminous surfacing from granular base raveling : (1) Presence of excessive soil / moorum binder on top of WBM base, which has not been brushed and removed properly before the application of prime coat (2) presence of soil / dust on top of granular base (3) directly applying the tack coat over granular base course, without applying prime coat (4) use of inferior or improper binder for prime coat and tack coat (5) improper method of application of prime / tack coats (like the old practice of manually pouring the binder through perforated cans) resulting in non-uniform application of the binder and lack of proper inter face bound (6) delayed laying and / or compaction of hot bituminous mix, at colder temperature than that specified.

Development of fine cracks on bituminous surface course, during or soon after compaction: Fine cracks may be developed in the bituminous surfacing course during or soon after the construction due to : (1) deficiencies in the bituminous binder or mix, such as use of harsh mix with high stability and low flexibility (2) Over heating of bitumen or the aggregates in the mix (3) inadequate quantity / improper quality of filler in the dense bituminous mixes, such as use of crusher dust only without lime filler (4) segregation of mix during laying, resulting in open graded / permeable texture of the surface at some locations (5) use of improper type of roller or adopting improper frequency and amplitude of vibration of the vibratory rollers, with reference to the material in the layer (6) Wrong method of rolling or wrong temperature of rolling in the case of hot bituminous mixes (7) reduction in strength of the pavement due to deficient surface and subsurface drainage and consequent soaking conditions of the pavement layers for prolonged periods or stripping of bitumen in the bituminous mix of the binder course. The fine cracks formed may get healed up under traffic wheel loads during hot weather; however these fine cracks may get widened rapidly resulting in failure, if it rains soon after the construction and water enters the bituminous surfacing course with fine cracks.

Development of Alligator Cracks: Wide alligator cracks are developed on stiff bituminous surfacing courses due to excessive tensile strain under heavy wheel loads in the stiff bituminous layer which may be caused because of : (1) Inadequate thickness of the layer (2) Application of excessive magnitude of wheel loads (3) Hardening of bitumen in the mix due to rapid weathering action (4) Settlement of any of the pavement layers below, as a result of inadequate compaction or other deficiencies in the layer (5) at isolated locations of the pavement, due to the repeated heavy loads presence of weak spots (6) initial fine cracks developed during construction may get widened rapidly under adverse conditions such as the combined action of water and excessive tensile strain in the stiff bituminous surface course results in the initiation of tensile cracks, starting from the bottom of the layer. Repetition of heavy wheel loads under adverse environmental conditions lead to rapid widening of the cracks and development of closely spaced wide alligator cracks on the surface and result in fatigue failure of the layer.

Rutting along the wheel paths: Rutting on pavement surface is the cumulative effect of the permanent deformation in various layers of the flexible pavement, along the wheel paths. Permanent deformation along the wheel path of heavy vehicles take place as a result of cumulative settlement on subgrade and /or other pavement layers due to: (1) inadequate pavement thickness resulting in excessive compressive strain on subgrade under heavy wheel loads (2) inadequate compaction of various layers (3) use of inferior materials in any of the layers or use of improper bituminous mix (4) application of excessive magnitude of wheel loads than those expected at the design stage.

Undulations and unevenness of pavement surface: Undulations on flexible pavement surface are developed due to : (1) improper and non-uniform compaction of subgrade or any of the pavement layers (2) low compaction standards that are often adopted in the design on construction of various layers (3) use of inferior subgrade soil or other pavement materials (4) laying of large boulder stones or blocks or bricks (without proper interlocking) directly over weak / clayey subgrade (5) poor subgrade drainage (6) inferior methods of spreading and compacting the pavement layers.

Bituminous surface too slippery under wet condition ; bleeding of bituminous surface: Smooth and slippery pavement surface is formed on areas with heavy traffic movements due to : (1) the use of aggregates with inadequate hardness or high polishing value (2) use of improper type of bituminous mix in surface course. With low voids content or excessive bitumen content, due to defect in the mix design stage or during actual construction process (3) application of excessive quantity of bituminous tack coat under thin surfacing course.

Edge breaking and edge drop: Edge breaking is caused due to: (1) lack of or inadequate lateral confinement or support of the pavement edge along the shoulders, during or after the construction (2) movement of heavy wheel loads along the edge (3) edge drop caused due to erosion of the soil from the earthen shoulders during heavy rains and passing of heavy wheel loads along the unconfined edges of the pavement.

Total or shear failure at some location along wheel path: Total or shear failure of flexible pavement occurs at location where the total thickness of the pavement or the stability of the pavement is grossly inadequate, with reference to the heavy group of wheel loads that ply. This may occur at some locations where the subgrade and sub base have been rendered very weak due to stagnation of water underneath and / or due to the presence of inferior materials in some locations of the pavement.

Development of large depressions and waves on the road surface: The large depressions and wavy surface may be caused by: (1) non – uniform settlement of the fill, due to variation in the height of fill (2) poor / inferior compaction of various layers during the construction of the fill / embankment and the subgrade (3) non uniform settlement of the embankment foundation in the case of high embankments constructed over compressible soil foundation.

Early failure of thin bituminous surfacing course when stage construction is resorted to: The early failure may be caused due to one or more reasons such as : (1) ineffective sub-surface drainage under or within the pavement (2) stagnation of water on pavement surface due to poor surface drainage system on depressions on the surface (3) wrong choice of bituminous binder and surfacing course materials and their thickness with reference to the pavement design factors (For example, laying of thin, but stiff bituminous surfacing course, such as 25 to 40 mm – thick bituminous concrete surfacing directly over granular base course where heavy wheel loads ply) (3) Improper choice of materials and thickness design of lower layers of the pavement, particularly of the subgrade, sub-base and the base course (4) Open texture with high permeability on the bituminous surfacing course (caused by the segregation of the mix during construction), permitting entry of rain water from the surface and stagnation of the water within / under the pavement layers due to ineffective pavement drainage system.

4.0 STEPS TO IMPROVE ROADS ON POOR SUBGRADE SOILS

Black cotton soils are not so good for laying durable roads. Black cotton soils absorb water heavily, swell, become soft and lose strength. Black cotton soils are easily compressible when wet and possesses a tendency to heave during wet condition. BC soils shrink in volume and develop cracks during summer. They are characterized by extreme hardness and cracks when dry. The stability and performance of the pavements are greatly influenced by the subgrade and embankment as they serve as foundations for pavements. On such soils suitable construction practices and sophisticated methods of design are to be adopted.

Indian Road Congress' code IRC: 37-2001, "Guidelines for design of flexible pavements," suggests the following for pavement on expansive soils.

a. Buffer layer

Providing a non expansive layer of 0.6 to 1m thick prevents ingress of water into expansive soil layer and counteracts swelling. It also reduces the harmful effects of heaving and reduces the stress on expansive layer.

b. Blanket course

A blanket course of at least225 mm thick composed of coarse/medium sand or non plastic moorum with PI less than 6% shall be provided for full width of formation over expansive sub–grade.

c. Drainage

Appropriate surface drainage and sub–surface system to prevent ingress and retention of water in the pavement structure.

d. Thick BT surfacing

Desirably, 40mm thick BT surfacing shall be provided to prevent ingress of water.

e. Adequate arrangements to cater for both surface drainage and sub surface drainage are essential to prevent flooding of roads, weakening of road structure, formation of potholes, stripping of bitumen etc. A drainage layer in the form of coarse graded granular sub base shall be laid for the full width of formation. It also acts as a capillary cut off. At some locations capillary rise of water soaks the sub grade and crust for which GSB as drainage layer is a good remedy.

f. Use of Gravelly Soil in Base and Sub-base

The moorum shall be composed of well graded coarse siliceous and gritty to touch and free from dirt and deleterious matter. Material passing 75 micron shall not exceed 10%. Liquid limit shall not exceed 20% and Plasticity index shall not exceed 6% for sub base, filler material in surface treated WBM roads and backing for revetment. The above values are 33% and 6to 9% respectively for filler material in WBM roads.

g. Screenings

Screenings to fill voids in the coarse aggregate shall generally consist of the same material as the coarse aggregate. However, where permitted, predominantly non-plastic material such as moorum or gravel may be used for this purpose provided liquid limit and plasticity index of such material are below 20 and 6 respectively and fraction passing 75 micron sieve does not exceed 10 percent.

h. Binding material

Binding material to be used for water bound macadam as a filler material meant for preventing raveling, shall comprise a suitable material approved by the Engineer having a Plasticity Index (PI) value of less than 6.

i. Coarse aggregates for BT courses

Coarse aggregates shall consist of crushed rock, crushed gravel or other hard material retained on 2.36mm sieve. They shall be clean, hard, and durable, of cubical shape, free from dust and soft or friable matter, or other deleterious matter. Where crushed gravel is proposed for use as coarse aggregate, not less than 90% by weight of the crushed material retained on the 4.75mm sieve shall have at least two fractured faces. All the above requirements clearly indicate that if the locally available gravel is to be used in sub-base or in base or as coarse aggregates for bituminous surfacing, the gravel must be rocky type material not soil type material.

4.1 Recommendations for Improvement of Roads in Poor Subgrade Soil Areas

1. Drainage layer in the form of Granular Sub Base with crushed aggregates, up to the formation edge shall be provided in all new road constructions. In widening works GSB as a drainage layer shall be extended up to the edge of formation.
2. Base widths shall be more than BT surfacing width by 150mm to 200mm on either side to prevent accumulation of water below BT surfacing and to protect BT edges.
3. Multi layer base constructions shall be restricted to places where required machinery is not available. Wet Mix Macadam or Crusher Run Macadam is cheaper than WBM at many places.
4. Use of natural gravel, which is highly plastic, shall not be allowed in sub base or base layers.

5. Earthwork excavations near the toe of formation shall be prohibited. Only selected earth required for road and bridge works shall be allowed duly ignoring the cost criteria.

6. Shoulder and sub grade are the most neglected items in the road construction. Equal importance shall be given to these items.
7. Necessary steps shall be taken to prevent the unauthorized plying of iron wheeled vehicles and overloaded vehicles.
8. Providing GSB with HBG crushed aggregates as sub base up to the edge of formation.
9. Providing Wet Mix Macadam in base layers.
10. Providing suitable soils for the shoulders.
11. Improving road geometrics wherever necessary.
12. Badly damaged stretches are recycled by fully picking the crust, removing the gravelly soil, sectioning the picked metal, rolling, applying stone dust as screenings and binder. By this way the sunken roads are converted as new WBM surfaced roads. Further, GSB is proposed in low lying stretches and overall WMM followed by BM and SDBC.

5.0 GRANULAR SUB-BASE

This work shall consist of laying and compacting well-graded material on prepared sub-grade in accordance with the requirements of the Specifications. The material shall be laid in one or more layers as sub-base lower sub-base and upper sub-base (termed as sub-base hereinafter) as necessary according to lines, grades and cross-sections.

Materials

The Material to be used for the work shall be natural sand, moorum, gravel, crushed stone, or combination thereof depending upon the grading required. Materials like crushed slag, crushed concrete, brick metal and kankar may be allowed only with the specific approval of the Engineer. The material shall be free from organic or other deleterious constituents and conform to one of the three gradings given in Tables.

While the gradings in Table 1 are in respect of close-graded granular sub-base materials, one each for maximum particle size of 75 mm, 53 mm and 26.5 mm, the corresponding gradings for the coarse-graded materials for each of the three maximum particle sizes are given at Table 2. The grading to be adopted for a project shall be as specified in the contract.

Physical requirements: The Material shall have a 10 per cent fines value. The water absorption value of the coarse aggregate shall be determined as per IS: 2386 (Part 3); if this value is greater than 2 per cent, the soundness test shall be carried out on the material delivered to site as per IS: 383. For Grading II and III materials, the CBR shall be determined at the density and moisture content likely to be developed in equilibrium conditions which shall be taken as being the density relating to a uniform air voids content of 5 per cent.

Table 1. Grading for close-graded granular sub-base materials (MoRTH,2001).

IS Sieve designation (mm)	Per cent by weight passing the IS sieve		
	Grading I	Grading II	Grading III
75	100	----	----
53	80-100	100	----
26.5	55-90	70-100	100
9.5	35-65	50-80	65-95
4.75	25-55	40-65	50-80
2.36	20-40	30-50	40-65
0.425	10-25	15-25	20-35
0.075	3-10	3-10	3-10
Minimum CBR (%)	30	25	20

Table 2. Grading for coarse graded granular sub-base materials (MoRTH,2001).

IS Sieve designation (mm)	Per cent by weight passing the IS sieve		
	Grading I	Grading II	Grading III
75	100	----	----
53		100	----
26.5	55-75	50-80	100
9.5			
4.75	10-30	15-35	25-45
2.36			
0.425			
0.075	< 10	< 10	< 10
Minimum CBR (%)	30	25	20

Note: The Material passing 425 micron (0.425 mm) sieve for all the three gradings when tested according to IS: 2720 (Part 5) shall have liquid limit and plasticity index not more than 25 and 6 per cent respectively.

6.0 SOIL STABILISATION

'Soil stabilization', in the broadest sense, refers to the procedures employed with a view to altering one or more properties of a soil so as to improve its engineering performance. Soil stabilization is only one of several techniques available to the geotechnical engineer and its choice for any situation should be made only after a comparison with other techniques indicates it to be the best solution to the problem. It is a well known fact that, every structure must rest upon soil or be made of soil. It would be ideal to find a soil at a particular site to be satisfactory for the intended use as it exists in nature, but unfortunately, such a thing is of rare occurrence. The alternatives available to a geotechnical engineer, when an unsatisfactory soil is met with, are (*i*) to bypass the bad soil (*e.g.*, use of piles), (*ii*) to remove bad soil and replace with good one (*e.g.*, removal of peat at a site and replacement with selected material), (*iii*) redesign the structure (*e.g.*, floating foundation on a compressible layer), and (*iv*) to treat the soil to improve its properties. The last alternative is termed soil stabilization.

6.1 Stabilization of Soils without Additives

Some kind of treatment is given to the soil in this approach; no additives are used. The treatment may involve a mechanical process like compaction and a change of gradation by addition or removal of soil particles or processes for drainage of soil.

6.1.1 Mechanical Stabilization

'Mechanical stabilization' means improving the soil properties by rearrangement of particles and densification by compaction, or by changing the gradation through addition or removal of soil particles.

Rearrangement of particles—compaction
The process of densification of a soil or 'compaction', as it is called, is the oldest and most important method. In addition to being used alone, compaction constitutes an essential part of a number of other methods of soil stabilization. The important variables involved in compaction are the moisture content, compactive effort or energy and the type of compaction. The most desirable combination of the placement variables depends upon the nature of the soil and the desired properties. Fine-grained soils are more sensitive to placement conditions than coarse-grained soils. Compaction has been shown to affect soil structure, permeability, compressibility characteristics and strength of soil and stress-strain characteristics.

Change of gradation—addition or removal of soil particles
The engineering behavior of a soil depends upon (among other things) the grain-size distribution and the composition of the particles. The properties may be significantly altered by adding soil of some selected grain-sizes, and, or by removing some selected fraction of the soil. In other words, this approach consists in manipulating the soil fractions to obtain a suitable grading, which involves mixing coarse material or gravel (called 'aggregate'), sand, silt and clay in proper proportions so that the mixture when compacted attains maximum density and strength. It may involve blending of two or more naturally available soils in suitable proportions to achieve the desired engineering properties for the mixture after necessary compaction.

Note: 1. Not less than 10% should be retained between each pair of successive sieves specified, excepting the largest pair.

2. Material passing 425 micron I.S. Sieve shall have the following properties:
For base courses: Liquid limit should not be more than 25%
Plasticity Index should not be more than 6%
For surface courses: Liquid limit should not be more than 35%
Plasticity Index : between 4 and 9.

Gravel is used for base courses of pavements and for filter courses. The presence of fines to an extent more than the optimum might make the gravel unsatisfactory. The limit for the fines may be 3 to 7%, depending upon its intended use. The suggested gradings for mechanically stabilized base and surface courses are presented in Table 3.

Table 3. Suggested gradings for mechanically stabilized base and surface courses (MoRTH,2001)

IS Sieve Size (mm)	Per cent passing				Base or surface coarse	
	Base Coarse Max. size		Surface coarse Max. size		Max. size	
	80 mm	40 mm	20 mm	20 mm	10 mm	5 mm
80	100	---	---	---	---	---
40	80-100	100	---	---	---	---
20	60-80	80-100	100	100	---	---
10	45-65	55-80	80-100	80-100	100	---
5	30-50	40-60	50-75	60-85	80-100	100
2.38	---	30-50	35-60	45-70	50-80	80-100
1.18	---	---	---	35-60	40-65	50-80
0.60	10-30	15-30	15-35	---	---	30-60
0.30	---	---	---	20-40	20-40	20-45
0.075	5-15	5-15	5-15	10-25	10-25	10-25

6.1.2 Stabilization by Drainage

Generally speaking, the strength of a soil generally decreases with an increase in pore water and in the pore water pressure. Addition of water to a clay causes a reduction of cohesion by increasing the electric repulsion between particles. The strength of a saturated soil depends directly on the effective or intergranular stress. For a given total stress, an increase in pore water pressure results in a decrease of effective stress and consequent decrease in strength. Thus, drainage of a soil is likely to result in an increase in strength which is one of the primary objectives of soil stabilization. The methods used for drainage for this purpose are:

1. Application of external load to the soil mass,
2. Drainage of pore water by gravity and/or pumping, using well-points, sand-drains, etc.,
3. Application of an electrical gradient or electro-osmosis; and,
4. Application of a thermal gradient.

6.2 Stabilization of Soils with Additives

Stabilization of soil with some kind of additive is very common. The mode and degree of alternation necessary depend on the nature of the soil and its deficiencies. If additional strength is required in the case of cohesionless soil, a cementing or a binding agent may be added and if the soil is cohesive, the strength can be increased by making it moisture-resistant, altering the absorbed water films, increasing cohesion with a cement agent and adding internal friction. Compressibility of a clay soil can be reduced by cementing the grains with a rigid material or by altering the forces of the adsorbed water films on the clay minerals. Swelling and shrinkage may also be reduced by cementing, altering the water adsorbing capacity of the clay mineral and by making it moisture-resistant. Permeability of a cohesionless soil may be reduced by filling the voids with an impervious material or by preventing flocculation by altering the structure of the adsorbed water on the clay mineral; it may be increased by removing the fines or modifying the structure to an aggregated one. A satisfactory additive for soil stabilization must provide the desired qualities and, in addition, must meet the following requirements:

Compatibility with the soil material, permanency, easy handling and processing, and low cost. Many additives have been employed but with varying degrees of success.

6.2.1 Cement Stabilization

Portland cement is one of the most widely used additives for soil stabilization. A mixture of soil and cement is called ‘‘soil-cement’’. If a small percentage of cement is added primarily to reduce the plasticity of fat soils, the mixture is said to be a ‘‘cement-modified soil’’. If the soil cement has enough water which facilitates pouring it as mortar, it is said to be a ‘plastic soil cement’’. It is used in canal linings. The chemical reactions of cement with the siliceous soil in the presence of water are believed to be responsible for the cementing action. Many of the grains of the coarse fraction get cemented together, but the proportion of clay particles cemented is small. Almost any inorganic soil can be successfully stabilized with cement; organic matter may interfere with the cement hydration. Soil-cement has been widely used for low-cost pavements for highways and airfields, and as bases for heavy traffic. Generally, it is not recommended as a wearing coarse in view of its low resistance to abrasion. The important factors which affect the properties of soil-cement are the nature of the soil, cement content, compaction, and the method of mixing.

6.2.2 Bitumen Stabilization

Bituminous materials such as asphalts and tars have been used for soil stabilization. This method is better suited to granular soils and dry climates. ‘Bitumen’s’ are non-aqueous system of hydrocarbons which are completely soluble in ‘Carbon disulphide’. ‘Asphalts’ are natural materials or refined petroleum products, which are bitumen’s. ‘Tars’ are bituminous condensates produced by the destructive distillation of organic materials such as coal, oil, lignite and wood. Most bitumen stabilization has been with asphalt. Asphalt is usually too viscous to be incorporated directly with soil. Hence, it is either heated or emulsified or cut back with a solvent like gasoline, to make it adequately fluid. Tars are not emulsified but are heated or cut back prior to application. Soil-asphalt is used mostly for base courses of roads with light traffic. Bitumen stabilizes soil by one or both of two mechanisms: (*i*) binding soil particles together, and (*ii*) making the soil water-proof and thus protecting it from the deleterious effects of water.

6.2.3 Chemical Stabilization

Chemical stabilization refers to that in which the primary additive is a chemical. Lime and salt have found wide use in the field. Some chemicals are used for stabilizing the moisture in the soil and some for cementation of particles. Certain aggregates and dispersants have also been used.

Lime stabilization

Lime is produced from natural limestone. The hydrated limes, called ‘slaked lines’, are the commonly used form for stabilization. In addition to being used alone, lime is also used in the following admixtures, for soil stabilization: (*i*) Lime-fly ash (4 to 8% of hydrated lime and 8 to 20% of fly-ash) (*ii*) Lime-Portland cement (*iii*) Lime-bitumen. There are two types of chemical reactions that occur when lime is added to wet soil. The first is the alteration of the nature of the adsorbed layer through ion exchange of calcium for the ion naturally carried by the soil, or a change in the double layer on the soil colloids. The second is the cementing action or pozzolanic action which requires a much longer time. This is considered to be a reaction between the calcium with the available reactive alumina or silica from the soil. Lime has the following effects on soil properties: Lime generally increases the plasticity index of low-plasticity soil and decreases that of highly plastic soils; in the latter case, lime tends to make the soil friable and more easily handled in the field. It increases the optimum moisture content and decreases the maximum compacted density; however, there will be an increase in strength. About 2 to 8% of lime may be required for coarse-grained soils, and 5 to 10% for cohesive soils. Certain sodium compounds (*e.g.*, sodium hydroxide and sodium sulphate), as secondary additives, improve the strength of soil stabilized with lime. Lime may be applied in the dry or as slurry. Better penetration is obtained when it is used as a slurry. The construction of lime-stabilized soil is very much similar to that of soil cement. The important difference is that, in this case, no time limitation may be placed on the operations, since the lime-soil reactions are slow. Care should be taken, however, to prevent the carbonation of lime. Lime stabilization has been used for bases of pavements.

7.0 SUMMARY AND CONCLUSION

In order to be abreast with modern approaches and trends in materials testing and road maintenance, there is need for training and re-training of contractors' personnel supervising highway projects. Adequate fund should be voted and released each year for training and curricular development. This training should cover all grade levels and cadres of engineering staff. Weigh bridges should be installed on all roads to check excessive loading of vehicles plying the roads. Apart from imposing fines on the offenders, removal of such excess load must be enforced. Contracts for highway projects should be awarded to the "best responsive bidder" whose answers to the Questionnaire have been found to be very accurate. The lowest bidder need not necessarily be preferred. There is an urgent need for government to expand the Pavement Evaluation Unit, to cover the roads of entire state and its functions widened to include geotechnics investigation.

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STUDY ON OBSERVED VALUES OF SOAKED CBR OF LIME-STABILIZED COPPER SLAG CUSHION LAID OVER EXPANSIVE SOIL BED

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Abstract

Use of waste materials in transportation has been in development all over the world for quite a time and particularly because of the disposal problems associated with it. Copper slag is one of the waste materials that is being used for various applications in civil engineering. Granulated copper slag is more porous and has particle sizes equal to that of medium sand. Also, due to the scarcity of sand, copper slag along with an admixture can be used as an alternative material to sand in road construction. On the other hand, Expansive soils are associated with volume changes when subjected to changes in water content. CNS (cohesive non-swelling) and sand cushion are some of the techniques that have been used to reduce the volume changes and which were originally in practice, bristle with a few disadvantages. The present paper discusses the laboratory test results of soaked CBR (California Bearing Ratio) conducted on a stabilized copper slag cushion-soil system for various thickness ratios ranging from 0.25 to 1.00 when stabilized with various percentages of lime. The results showed that the soaked CBR increases as the ratio of the thickness of the cushion (h_c) to the thickness of the expansive soil bed (h_s) is increased and with the increase in percentage of admixture.

Key words: ***Expansive soil, Copper slag, Lime.***

INTRODUCTION

Expansive clays are related with volume changes that are closely associated with changes in water content. The change in water content is mostly due to seasonal changes. During monsoon, expansive soils imbibe water from outside and during summer, shrinkage occurs due to the evaporation of water. Moisture migration from outside to inside causes uplift of the structure and results in a mound-shaped heave. In pavements, longitudinal cracking may result, due to the migration of moisture from the shoulders to the center. Montmorillonite is the clay mineral having the highest potential to swell by virtue of its structure and illite also swells though not as much as montmorillonite, but it undergoes greater swelling than kaolinite, which is non-swelling in nature. Techniques like CNS (cohesive non-swelling) [1] cushion and sand cushion [2] have been tried to arrest such heave. Copper slag is one of the waste materials being used in Civil Engineering practice. It has particle size equal to that of medium sand. Also, due to the scarcity of sand, copper slag along with an admixture can be used as an alternative material to sand in pavements. If the copper slag is mixed with calcium-based compound like cement or lime in the presence of water, the silica and alumina present in it will react chemically on hydration and the resulting product may be used for the improvement of sub-grades and sub-bases. Metal industry slag, mine stone and mining waste are generally suitable for recycling or reuse and the use of these inorganic wastes as alternative materials in buildings, roads and for other geotechnical applications have been reported [3, 4, 5, 6, 7, 8]. Life-cycle analysis for the use of industrial waste slag in road and earth constructions produced effective results, thus advocating the reuse of waste by-products [9].

By mixing expansive soil with copper slag, it can be used as an effective stabilizing agent for the improvement of problematic soils in highways, embankments, sub-grades and sub-bases. Also, by mixing it with fly ash, it becomes suitable as embankment fill material. Slag, when mixed with fly ash and lime, develops pozzolanic reactions [10]. Fly ash has been widely accepted as embankment and structural fill material [11, 12]. It has been felt that the same copper slag when admixed with lime or cement can be used as a cushion in improving the performance of expansive sub-grades. Similar studies using fly ash and blast furnace slag with lime and cement were carried out earlier [13, 14, 15, 16].
The present paper discusses the results of observed soaked CBR (California Bearing Ratio) values conducted on the cushion-soil system with various percentages of lime as an admixture added to the copper slag. The soaked CBR value with the addition of various percentages of lime (2% to 10%) to the copper slag was from 3% to 25% for the various thickness ratios used, ranging from 0.25 to 1.00.

EXPERIMENTAL STUDY

Expansive soil

Expansive soil used in the present investigation was collected from the Nalgonda district in Andhra Pradesh. The basic properties of soil are presented in Table 1. The plasticity index of the soil is high. It has free swell index of 220% which shows a very high degree of expansiveness.

Table 1 Basic Properties of Soil

Property	**Value**
Grain Size Analysis	
Gravel (%)	4
Sand (%)	33
Silt & Clay (%)	63
Consistency Limits	
Liquid Limit (%)	75
Plastic Limit (%)	35
Plasticity Index (%)	40
IS Classification	CH
Free Swell Index (%)	220
MDD (kN/m^3)	14
OMC (%)	21
CBR (%)	1

Copper Slag

Copper slag was procured from Sterilite Industries, Tuticorin, Tamilnadu. The physical and chemical properties of the slag are presented in Tables 2 and 3 respectively.

Table 2 Physical Properties of Copper Slag

Property	Value
Grain Size Analysis	
Gravel Size (%)	1.00
Sand Size (%)	98.9
Silt & Clay Sizes (%)	0.05
Hardness, Moh's Scale	6.5 – 7.0
Specific Gravity	3.6
Plasticity Index	Non-Plastic
Swelling Index	Non-Swelling
Granule Shape	Angular with sharp edges
MDD (kN/m^3)	23.5
OMC (%)	6
Direct Shear test	
Cohesion (kN/m^2)	0
Angle of internal friction (deg)	40
Permeability(cm/sec)	1.54×10^{-2}
CBR (%)	3.5

(Courtesy: Sterilite Industries Ltd, Tuticorin, Tamilnadu, India)

Table 3 Chemical Composition of Copper Slag

Property	(% wt)
Iron Oxide , Fe_2O_3	55 – 60
Silica, SiO_2	28- 30
Aluminium Oxide, Al_2O_3	1 – 3
Calcium Oxide, CaO	3– 5
Magnesium Oxide, MgO	1.0– 1.5

(Courtesy: Sterilite Industries Ltd, Tuticorin, Tamilnadu, India)

Admixtures

Lime is used as an admixture with the copper slag. Hydrated lime, which consists of 95% of calcium hydroxide was procured from the local market and is used in the present study.

Tests performed

Soaked CBR tests were performed for the copper slag mixed with lime which was laid on the expansive soil bed as a cushion. The copper slag and the admixture were mixed in dry condition and then, water corresponding to the desired percentage of water was added to it. Samples were prepared for different thickness ratios. The ratios of the thickness of the cushion (h_c) to the thickness of the expansive soil bed (h_s) used in the study were 0.00, 0.25, 0.50, 0.75 & 1.00. Laboratory California Bearing Ratio (CBR) tests were conducted on the samples as per IS code procedure (I.S.2720 (Part 16):1987 second revision). The cushion-soil specimen system in the CBR mould consists of expansive soil bed at the bottom and copper slag cushion on its top. This specimen was kept for soaking after placing the surcharge weights and the dial gauge to read the swelling, for 96hrs. The overall thickness of the soil bed and the cushion prepared in the CBR mould for testing was 127 mm and its diameter 150 mm. Both the soil bed and the admixture-mixed copper slag were compacted in the CBR mould at their respective OMC values.

RESULTS AND DISCUSSION

Test Results

Soaked CBR values were determined on the cushion-soil system when admixed with various percentages of lime to the copper slag for various thickness ratios. Soaked CBR for the black cotton soil was less than 1% whereas for the copper slag without any admixtures it was 3.5%. Figure 1 & 2 shows a typical soaked CBR test results of the cushion-soil system with 6% and 10% lime in the copper slag for different (h_c/h_s) ratios. From these curves, it may be noticed that the soaked CBR value increases as the ratio of the thickness of the cushion (h_c) to the thickness of the expansive soil bed (h_s) is increased. Also, an increase in the soaked CBR value corresponding to an increase in percentage of lime added to the copper slag as an admixture. Similar figures were observed with cushions with 2%, 4% and 8% lime also. The range of % increase of the soaked CBR with the addition of lime by varying percentages from 2% to 10% to the copper slag overlying the expansive soil bed for thickness ratios of 0.25, 0.50, 0.75 and 1.00 were from 4.3% to 30%, when compared to that with no cushion.

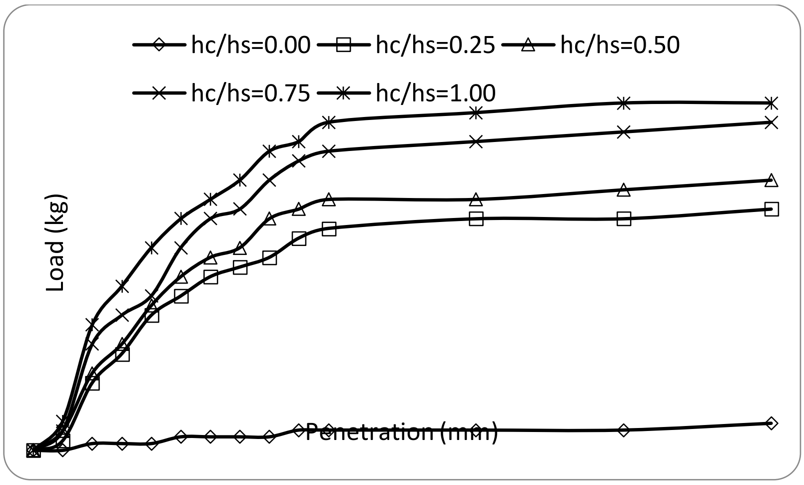

Fig. 1 Load-penetration of cushion-soil system with 6% lime in the cushion after soaking

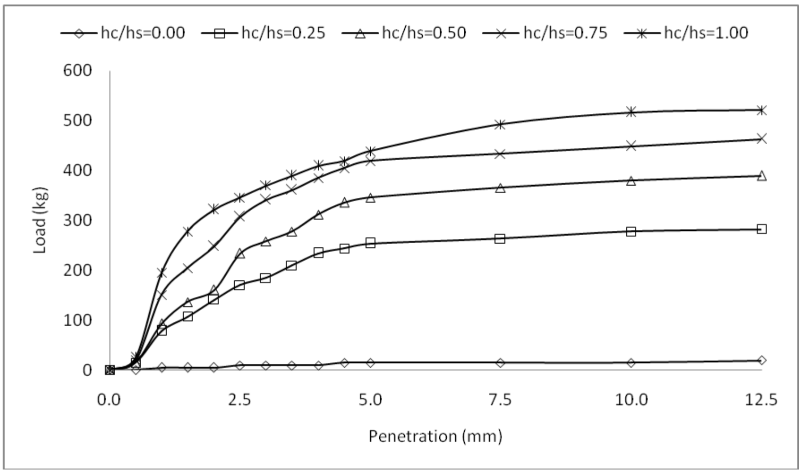

Fig. 2 Load-penetration of cushion-soil system with 10% lime in the cushion after soaking

The results of soaked CBR as given in Table 4 show that, soaked CBR values using lime-stabilized copper slag cushions are more with the increase in cushion thickness.

Table 4 Soaked CBR (%) of cushion-soil system with lime in the copper slag cushion

% Lime	Soaked CBR (%)			
	h_c/h_s=0.25	h_c/h_s=0.50	h_c/h_s=0.75	h_c/h_s=1.00
2	3.05	4.06	4.57	4.57
4	3.91	4.63	4.98	5.34
6	8.12	9.14	10.66	12.18
8	12.18	16.24	21.32	22.84
10	12.45	17.08	22.41	25.26

CONCLUSIONS

From the results, it was noticed that there is a marked improvement in the soaked CBR value of the cushion-expansive soil system when the cushioning material was added with lime. It was noticed that the increase in the soaked CBR with the addition of 2% to 10% lime to the copper slag for the thickness ratios from 0.25 to 1.00 was about 4.3% to 30%, when compared with the expansive soil with no cushion. Studies indicate that with the addition of lime in the copper slag cushion, the value of soaked CBR increases and also with the increase in the thickness ratio of the cushion to that of the expansive soil bed.

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STUDIES ON MECHANICAL PROPERTIES OF HIGH STRENGTH SELF CURING CONCRETE (M70 GRADE)

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Abstract

This study deals with a brief comparison of mechanical properties between a high strength controlled concrete with high strength self-cured concrete. Two different types of self-curing agents were used i.e., Polyethylene Glycol-400 (PEG-400) and Polyvinyl Alcohol (PVA) with different proportions 0.6%, 0.8%, 1%, 1.2%, 1.4% and 1.6% of weight of cement. As it is a high strength concrete a mineral admixture called "micro-silica" or "silica fume" is introduced in the mortar, which alters the properties of the concrete in terms of strength and durability. In order to study the behaviour of self-cured concrete, compressive strength test and split tensile strength tests were conducted on fresh and hardened concrete for a period of 7 days and 28 days, and non-destructive tests were also conducted to know the quality of concrete. After the experimental program it is observed that the highest desired compressive and split tensile strengths were achieved at the proportion of 1.4% of PEG-400 and PVA by weight of cement and with the further increase in the proportion to 1.6% of PEG-400 and PVA by weight of cement, observed a sudden decrement in the strengths. The compressive strengths and split tensile strengths of self-cured concrete were a higher when compared to the controlled concrete.

Keywords: Polyethylene Glycol-400(PEG-400), Polyvinyl Alcohol (PVA), Micro-silica.

I. Introduction

The life of a structure depends on its construction practice i.e., the quality of concrete that been used, the form work provided, steel, placing of concrete, compaction of concrete and ultimately the curing period of the structure. For a concrete structure curing becomes a primary note, in order to gain the strength, with in its lapsed time. Curing is the name given to the procedures used for promoting the heat of hydration of the cement, and consists of a control of temperature and of moisture movement from and into the concrete. Curing allows continuous hydration of cement and consequently continuous gain in the strength, once curing stops strength gain of the concrete also stops. Proper moisture conditions are critical because the hydration of the cement virtually ceases when the relative humidity with in the capillaries drops below 80%. A continuous curing of different structural components for a period of 28 days, can achieve its desired strength, to give life time serviceability. In order to maintain the heat of hydration, which does not alter the properties of concrete and which leads to gaining the strength of concrete, curing is important.

The actual phenomenon of this technology is that, the water which is used during the mixing of concrete itself is used for its curing in order to maintain the heat of hydration for the prescribed duration to maintain the properties of concrete unaltered. This is also known as internal curing(IC), which is achieved by not allowing the water that is used during mixing to evaporate in to the atmosphere. It forms a thin layer on the surface of the concrete, which doesn't allow water to evaporate in to the atmosphere.

II. Design Mix proportion of M70 Grade concrete

For concrete with compressive strength >40 Mpa to 100 Mpa, are considered to be the high strength concrete. Erntroy and Shacklock method is used for the design mix of M70 grade concrete. Erntroy and Shacklock has suggested empirical graphs in design mix procedure. The mix proportion of M70 grade concrete by theoretical procedure of Erntroy and shacklock is:

Table: 1 Mix proportion obtained as per theoretical design mix procedure

	Cement	Fine Aggregate	Coarse Aggregate	Water
Quantity(Kg/m^3)	617.16	373.38	1226.29	185.148
Proportions	1	0.604	1.98	0.30

Trial Mixes have been done by varying the proportions of cement, fine aggregates and coarse aggregates to attain the target mean strength for 28 days. Though the weight of cement obtained per cubic meter of concrete by theoretical procedure cannot be considered at site, as it deviates in economic point of view. A mineral admixture named “micro silica” or “silica fume” is introduced in the mix proportion. Trial mixes have been done by varying micro silica too. The final mix proportion obtained is:

Table: 2 Final design mix proportion after trial mixes

	Cement	Micro Silica	Fine Aggregate	Coarse Aggregate	Water	Super Plasticizers
Quantity(Kg/m^3)	435	42.32	609.9	1188.1	130.8	1.4
Proportions	1	0.097	1.4	2.72	0.3	0.0032

The above mix proportion is also used for self-cured concrete using Polyethylene Glycol-400(PEG-400) and Polyvinyl Alcohol (PVA).

III. Experimental Programme

The experimental program consisted of casting and testing specimens for testing the fresh and hardened properties on M70 grade of concrete with and without self-Curing agents. The mix proportion for M70 grade was achieved, by multiple trial mixes of the acquired mix proportion, by altering the proportions of ingredients. A total of 78 no’s of cubes of standard size 150 mm x 150 mm x 150 mm and 78 cylinders of 150 mm diameter and 300 mm height and 39 no’s of cubes of size 100 mm x 100 mm x 100mm, were cast for determining the compressive strength and split tensile strength and Non-destructive test i.e., USPV.

IV. Experimental Results

4.1 Materials:

The different materials that are used in this experimental programme are cement, fine aggregates, coarse aggregates, Silica-fume, super plasticizer, Polyethylene Glycol-400, Polyvinyl alcohol. The physical properties of the materials that are tested in the laboratory are as mentioned in the below tables.

Table: 3 Physical properties of ordinary Portland cement

Sl.No.	Property	Results
1.	Normal Consistency	28.6%
2.	Specific gravity	3.12
3.	Initial setting time	135 minutes
4.	Final setting time	330 Minutes
5.	Fineness	1.0%
6.	Soundness	0.5 mm

Table: 4 Physical properties of fine aggregate and coarse aggregate

Sl.no.	Property	Fine Aggregate	Coarse Aggregate
1.	Specific gravity	2.42	2.65
2.	Bulk Density Loose	1572 kg/m^3	1391 kg/m^3
3.	Bulking	4% w c	-
4.	Fineness Modulus	2.64	6.04%
5.	Flakiness Index	-	8%
6.	Elongation Index	-	0

4.2 Destructive Test Results of fresh and hardened concrete:

4.2.1 Compressive strength: Cube specimens of size 150mmx150mmx150mm have been casted and tested for 7 Days and 28 Days, and the results are furnished in the table 5 and represented in the figure 1 and 2.

Table: 5 Compressive Strength Results

Sl.no.	Type of Concrete	Compressive Strength (N/mm^2)	
		7 Days	28 Days
1	Controlled Concrete	58.15	87.9
2.	Self-Cured Concrete using PEG-400 – 0.6%	42.1	58.7
	0.8%	46.4	69.7
	1%	51.5	76.12
	1.2%	55.43	79.54
	1.4%	**57.4**	**89.3**
	1.6%	53.02	82.1
3.	Self-Cured Concrete using PVA – 0.6%	39.3	48.01
	0.8%	43.1	52.4
	1.0%	47.5	66.12
	1.2%	49.8	79.24
	1.4%	**54.02**	**88.9**
	1.6%	51.4	81.01

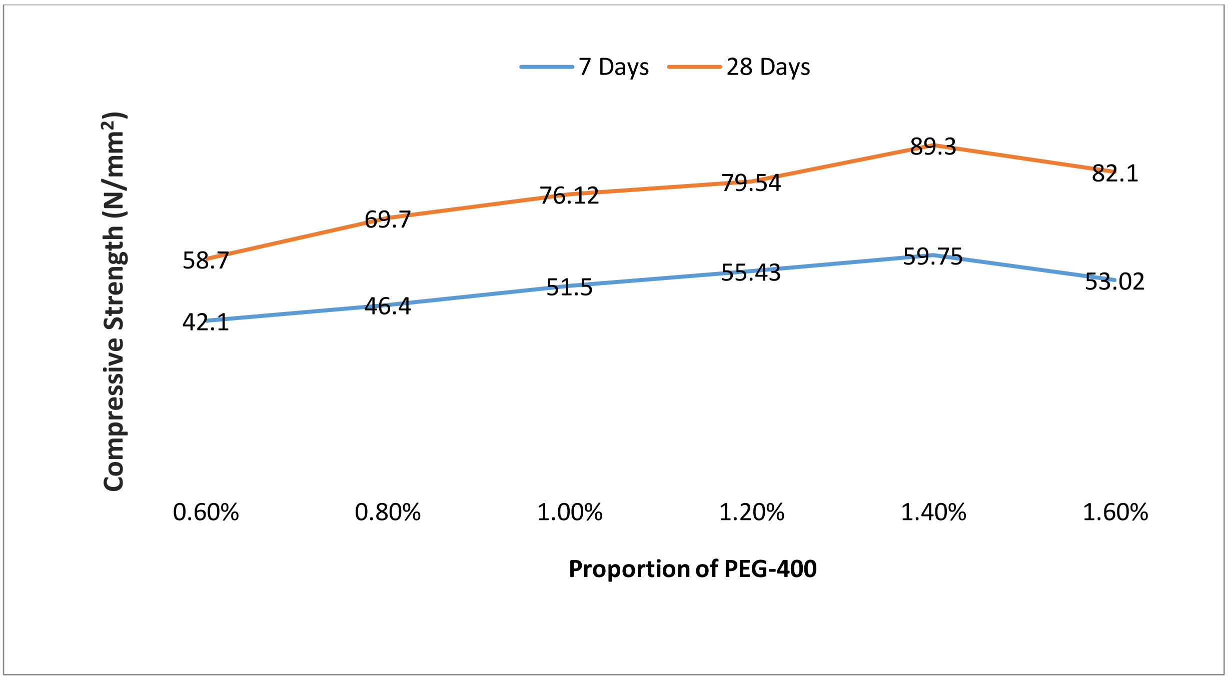

Figure: 1 Polyethylene Glycol - 400 (PEG-400) vs Compressive strength

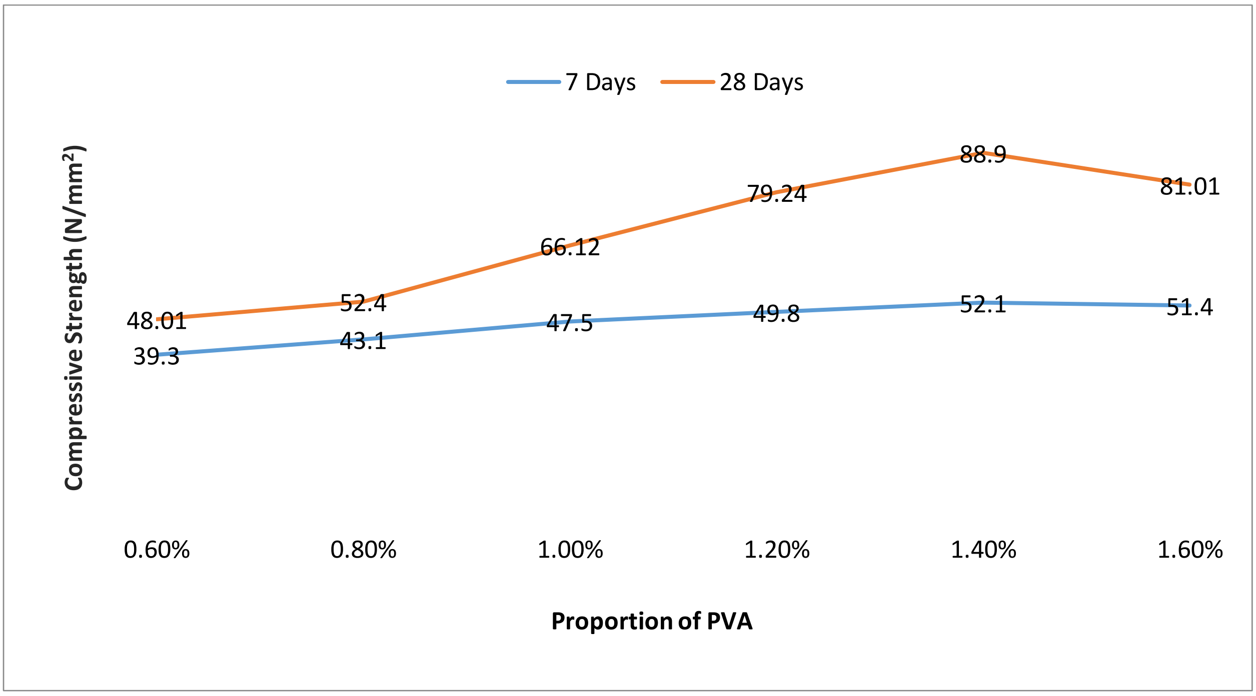

Figure: 2 Polyvinyl Alcohol (PVA) vs Compressive strength

4.2.2 Split Tensile Strength: According to this experimental programme Cylinders of Dia 150mm and Height 300mm have casted to test the split tensile strength and results are presented in the Table 6.

Table: 6 Split Tensile Strength Results

Sl.no.	Type of Concrete	Split Tensile Strength (N/mm^2)	
		7 Days	**28 Days**
1	Controlled Concrete	3.8	6.1
2.	Self-Cured Concrete using PEG-400 – 0.6%	2.21	4.12
	0.8%	2.74	4.87
	1%	2.93	5.02
	1.2%	3.31	5.42
	1.4%	**3.65**	**5.8**
	1.6%	3.4	5.7
3.	Self-Cured Concrete using PVA – 0.6%	2.01	4.07
	0.8%	2.34	4.41
	1.0%	2.85	4.76
	1.2%	3.15	5.1
	1.4%	**3.4**	**5.65**
	1.6%	3.2	5.4

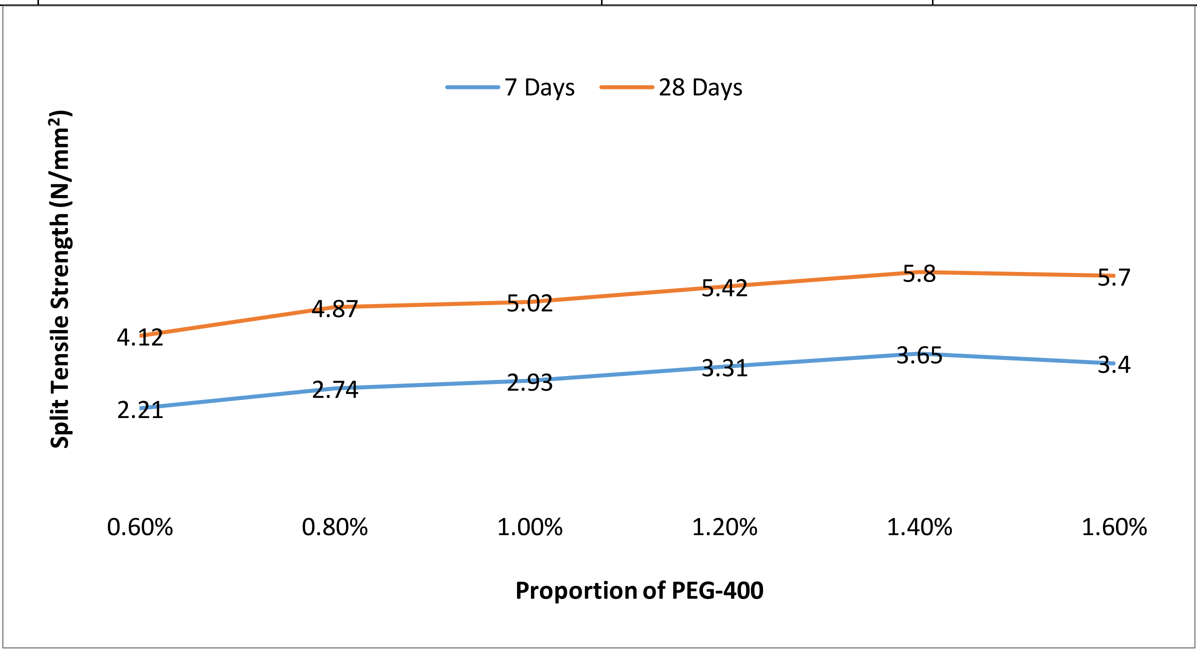

Figure: 3 Polyethylene Glycol - 400 (PEG-400) vs Split Tensile strength

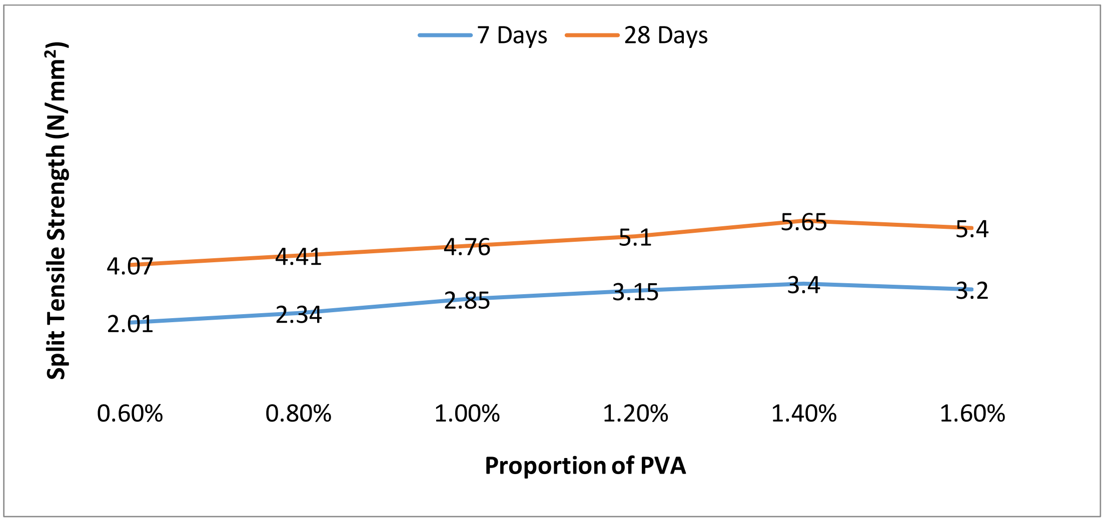

Figure: 2 Polyvinyl Alcohol (PVA) vs Split Tensile strength

4.3 Non-Destructive Test results of fresh and hardened concrete

USPV test are done on conventional concrete and self-cured concrete specimens to assess the quality,. Table: 7 shows USPV values for the different experimental specimens.

Table: 7 Non-destructive test results of M70 grade Conventional and Self-curing concrete

Specimen	Mean pulse velocity(km/sec)	Quality of Concrete
Conventional Concrete	**5.04**	**Very good to Excellent**
Self-Curing Concrete using Polyethylene Glycol - 400 (PEG-400)		
PEG-400 – 0.6%	3.91	Good to Very Good
PEG-400 – 0.8%	3.99	Good to Very Good
PEG-400 – 1.0%	4.35	Very good to Excellent
PEG-400 – 1.2%	4.54	Very good to Excellent
PEG-400 – 1.4%	**4.97**	**Very good to Excellent**
PEG-400 – 1.6%	4.77	Very good to Excellent
Self-Curing Concrete using Polyvinyl Alcohol (PVA)		
PVA – 0.6%	3.51	Good to Very Good
PVA – 0.8%	3.83	Good to Very Good
PVA - 1.0%	3.95	Good to Very Good
PVA - 1.2%	4.27	Good to Very Good
PVA - 1.4%	**4.41**	**Very good to Excellent**
PVA – 1.6%	4.15	Very good to Excellent

V. Discussions on Test Results

From the experimental results it is observed that the compressive strength and split tensile strengths increases gradually with the increase in proportion of self-curing agents up to certain proportions and with further increase in the proportion it is observed that a decrease in the strengths takes place. The compressive strength has been increased from 67% of target mean strength to 102.05% of target mean strength for increase in the proportion of PEG-400 from 0.6% to 1.4% by weight of cement. Further it is observed that compressive strength is decreased from 102.05% of target mean strength to 93.8% of target mean strength, for increase in the proportion of PEG-400 from 1.4% to 1.6% by weight of cement. The split tensile strength have been increased about 0.0168% for increase in the proportion of PEG-400 from 0.6% to 1.4%, by weight of cement further it is observed that split tensile strength is decreased about 0.001% for increase in the proportion of PEG-400 from 1.4% to 1.6%, by weight of cement. Similarly the compressive strength have been increased from 54.86% of target mean strength to 101.60% of target mean strength for increase in the proportion of PVA from 0.6% to 1.4%, by weight of cement further it is observed that compressive strength is decreased from 101.6% of target mean strength to 92.5% of target mean strength for increase in the proportion of PVA from 1.4% to 1.6% by weight of cement. About 0.158% of split tensile strength is increased for increase in the proportion of PVA from 0.6% to 1.4%, by weight of cement, further it is observed that split tensile strength is decreased about 0.0025% for increase in the proportion of PVA from 1.4% to 1.6% by weight of cement. The proportion at which the highest strength is obtained is recorded and the strengths obtained are compared with the conventional tests results and found satisfactory.
Non-Destructive tests have been conducted, and a similar quality of concrete i.e., “Excellent” is observed by using the self-curing agent PEG-400 comparing with the conventional concrete. A “Good” quality of concrete is observed by using the self-curing agent PVA.

VI. Conclusions

6.1 Effect of Poly-ethylene Glycol – 400 (PEG-400) on high strength concrete:

1. The compressive strength results were continuously increasing with the increase in the proportion of self-curing agent from 0.6% to 1.4%. About 34.92% of strength is gained by increasing the proportion of PEG-400 from 0.6% to 1.4% by weight of cement.
2. The sudden decrement of compressive strength of 8.25% is observed with further increase in the proportion of PEG-400 from 1.4% to 1.6% by weight of cement.
3. It is also observed that about 0.0168% of increment in split tensile strength with the increase in the proportion of PEG-400 from 0.6% to 1.4%. With the further increase in the proportion of PEG-400 a sudden decrement of 0.001% in the split tensile strength.
4. The internal curing has been carried out well and good, which remained the properties of concrete unaltered for a proportion of 1.4% by weight of cement of PEG-400.
5. It is observed that on an average of 15% of strength is gaining with the increase in the proportion of PEG-400 from 0.6% to 1.4%.

6.2 **Effect of Polyvinyl Alcohol (PVA) on high strength concrete:**

1. The compressive strength results were continuously increasing with the increase in the proportion of self-curing agent from 0.6% to 1.4%, about 43% of compressive strength is gained with the increase in the proportion of PVA from 0.6% to 1.4% by weight of cement.
2. The sudden decrement of compressive strength of 9.1% is observed with further increase in the proportion of PVA from 1.4% to 1.6% by weight of cement.

3. The split tensile strength has an increment of 0.158%, with the increase in the proportion of PVA, and a sudden decrease of 0.0025% is observed with the further increase in the proportion of PVA i.e., from 1.4% to 1.6%.
4. The internal curing has been carried out well and good throughout the age of 28 days.
5. Non-Destructive tests shown a “very good to excellent quality” of concrete from 1.4% to 1.6% proportions of PVA, by weight of cement, which is equally good when compared with the conventional concrete.

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SEISMIC ANALYSIS OF RCC MULTI-STORIED BUILDING ON A SLOPING GROUND

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Abstract

Always the construction of buildings is not possible on plain ground when the construction takes place on sloping ground .so many factors has to be considered. There are different types of structures constructed on sloped area especially on hilly areas why because of the increasing population, economic development and the land value, even hilly areas undergoing rapid changes which influence the structure in terms of material, shape and method of construction. Many buildings on hilly slopes are supported by columns of different height and they are highly irregular and asymmetric in plan or elevation and they are subjected to not only axial loads but also subjected severe torsion in addition to lateral forces under the action of earthquakes.In this project an asymmetrical RCC building of 25 storey's resting on flat and sloping ground by considering different slopes of ground angles using Staad Pro under the wind and earthquake forces. Analyses is carried out for the models by considering zone II using response spectrum method and results like displacements, story drifts , overturning effect , base shear induced in columns and torsion etc. are studied and compared with flat and sloping ground conditions.
Keywords: *STADD Pro, RCC Designs, IS: 1893(Part-1), Seismic Analysis.IS800-2007IS1893-2002.*

1. INTRODUCTION

Seismology is the study of vibrations of earth mainly caused by earthquakes. The study of these vibrations by various techniques, understanding the nature and various physical processes that generate them from the major part of the seismology

Elastic rebound theory is one such theory, which was able to describe the phenomenon of earthquake occurring along the fault lines. Seismology as such is still a much unknown field of study where a lot of things are yet to be discovered.

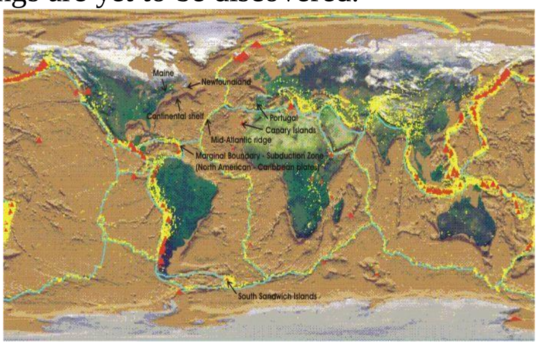


Fig. 1.1 Seismicity of World

The above Picture is showing the fault lines and we can see that epicenters are all concentrated all along the fault lines. The reason for seismic activities occurring at places other than the fault lines are still a big question mark. Also the forecasting of earthquake has not been done yet and would be a landmark if done so.

There is general saying that it's not the earthquake which kills people but its the bad engineering which kills people. With industrialization came the demand of high rise building and came dangers with that.

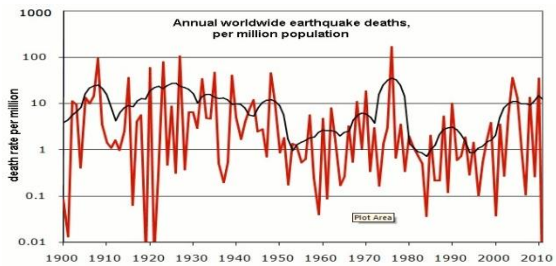


Fig. 1.2 Annual world earth quake deaths

A seismic design of high rise buildings has assumed considerable importance in recent times. In traditional methods adopted based on fundamental mode of the structure and distribution of earthquake forces as static forces at various stories may be adequate for structures of small height subjected to earthquake of very low intensity but as the number of stories increases the seismic design demands more rigorous.[1]*

During past earthquakes, reinforced concrete (RC) frame buildings that have columns of different heights within one storey, suffered more damage in the shorter columns as compared to taller columns in the same storey. Two examples of buildings with short columns in buildings on a sloping ground and buildings with a mezzanine floor can be seen in the figure given below. [1]*

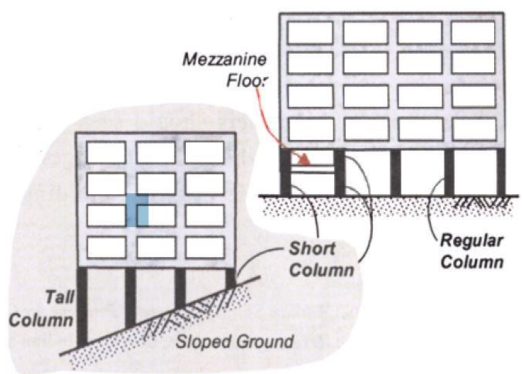


Fig. 1.3 Types of Columns

However, the short column is stiffer as compared to the tall column, and it attracts larger earthquake force. Stiffness of a column means resistance to deformation- the larger is the stiffness, larger is the force required to deform it.

If a short column is not adequately designed for such a large force, it can suffer significant damage during an earthquake. This behavior is called **Short Column Effect**. The damage in these short columns is often in the form of X-shaped cracking - this type of damage of columns is due to **shear failure.**

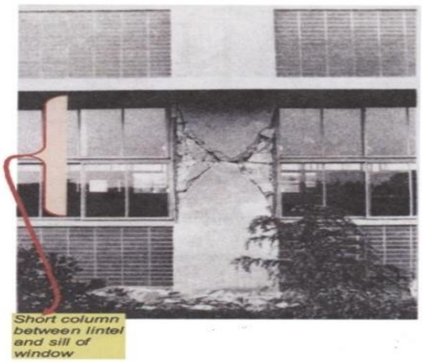


Fig 1.5 Short column Effect in Buildings

Many situations with short column effect arise in buildings. When a building is rested on sloped ground, during earthquake shaking all columns move horizontally by the same amount along with the floor slab at a particular level (this is called **rigid floor diaphragm action**). If short and tall columns exist within the same storey level, then the short columns attract several times larger earthquake force and suffer more damage as compared to taller ones. The short column effect also occurs in columns that support mezzanine floors or loft slabs that are added in between two regular floors.

Scope and objective of investigation

The scope of the present study pertaining to building and loading, modeling and analysis method, and different parametric studies are as follows:

Building and Loading The study is carried out by considering a RC framed residential building resting on isolated footing. Seismic force is applied considering parabolic load pattern.

To analyze and compare the physical constraints mainly storey displacement for building on sloping ground with the building on flat ground in ZONE 3.

To analyze and compare the base shear for both the buildings i.e. building rested on sloping surface and on flat surface in ZONE 3. To study and compare the effect of earthquake on both the buildings in ZONE 3

2. LITERATURE REVIEW

2.1 Overview

In this review, characteristics of the structures due to the variation of the slope angle are explained. Then the effect of the irregular configurations on vulnerability due to seismic forces is discussed. There are very few researchers who explained the effect of change of sloping angle.

No research work is done based on experimental investigation of the structures on sloping ground.

2.2 Seismic Behavior of Irregular Buildings on slopes in India

2.2.1 Ravikumar et al. (2012) studied two kinds of irregularities in building model namely the plan irregularity with geometric and diaphragm discontinuity and vertical irregularity with setback and sloping ground. Pushover analysis was performed taking different lateral load cases in all three directions to identify the seismic demands. All the buildings considered are three storied with different plan and elevation irregularities pattern. Plan irregular models give more deformation for fewer amounts of forces where the vulnerability of the sloping model was found remarkable. The performances of all the models except sloping models lie between life safety and collapse prevention. Hence it can be concluded that buildings resting on sloping ground are more prone to damage than on buildings resting on flat ground even with plan irregularities.

2.2.2 Sreerama and Ramancharla (2013) observed that recent earthquakes like Bihar-Nepal (1980), Shillong Plateau and the Kangra earthquake killed more than 375,000 people and over 100,000 of the buildings got collapsed. Dynamic characteristics of the buildings on flat ground differ to that of buildings on slope ground as the geometrical configurations of the building differ horizontally as well as vertically. Due to this irregularity the centre of mass and the centre of stiffness do not coincide to each other and it results in torsional response. The stiffness and mass of the column vary within the storey's that result in increase of lateral forces on column on uphill side and vulnerable to damage. In their analysis they took five G+3 buildings of varying slope angles of 0, 15, 30, 45, 60° which were designed and analyzed using IS-456 and SAP2000 and further the building is subjected and analyzed for earthquake load i.e., N90E with PGA of 0.565g and magnitude of M6.7. They found that short column attract more forces due to the increased stiffness. The base reaction for the shorter column increases as the slope angle increases while for other columns it decreases and then increases.

2.2.3 Patel et al. (2014) studied 3D analytical model of eight storied building was analyzed using analysis tool ETabs with symmetric and asymmetric model to study the effect of variation of height of column due to sloping ground and the effect of concrete shear wall at different locations during earthquake. In the present study lateral load analysis as per seismic code was done to study the effect of seismic load and assess the seismic vulnerability by performing pushover analysis. It was observed that vulnerability of buildings on sloping ground increases due to formation of plastic hinges on columns in each base level and on beams at each storey level at performance point. The number of plastic hinges is more in the direction in which building is more asymmetric. Buildings on sloping ground have more storey displacement as compared to that of buildings on flat ground and without having shear wall. Presence of shear wall considerably reduces the base shear and lateral displacement.

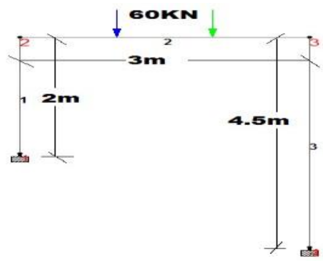
3. EXPERIMENTAL MODELLING

PHASE 1

Under the 2^{nd} chapter we took up a simple 2 dimensional frame subjected to concentrated loads with un-equal supporting columns. This is for the analogy of the actual problem statement of design of a multi-storied building on a sloping ground.

4.Problem statement- and Model Calculations

Analysis of a portal frame as shown in the figure:



Solution: we first calculated the distribution factors as given in the table I

(a) Non-sway analysis Fixed end moments

M 1-2=M 2-1=M3-4=M 4-3=0

M 2-3= - [60/3²] (1 x 2²+ 2 x 1²) =40 kN-m

M 3-2= +40 kN-m

	3/5	2/5	3/5	2/5	
0	0	-40	40	0	0
	24	16	24	-16	
12		-12	8		-8
	7.2	4.80	-4.8	-3.2	
3.6		-2.40	2.4		-1.60
	1.44	0.96	-1.44	-0.96	
0.72		-0.72	0.48		-0.48
	0.43	0.29	-0.29	-0.19	
0.22		-0.15	0.15		-0.10
	0.09	0.06	-0.09	-0.06	
0.05		-0.05	0.03		-0.03
	0.03	0.02	-o.02	-0.01	
16.59	33.19	-33.19	+20.42	-20.42	-10.21

So horizontal reaction at 1= 16.59+33.19/2=24.98 kN. Horizontal reaction at 4= -20.42-10.21/4.5= -6.8067 kN

Therefore net sway force = 24.89-6.8067= 18.0833 kN.

(b) Sway analysis:-

Now the frame will be analyzed for a sway force of 18.0833 kN

The ratio of the initial equivalent moments of the column is given by

= (I1/ l1²) : (I2/l2²) = 20.25: 4

Now the moment distribution is performed choosing the above fixed end moments

	3/5	2/5	3/5	2/5	
-20.25	-20.25	0	0	-4	-4
	12.15	8.10	2.4	1.6	
6.08		1.20	4.05		0.80
	-0.72	-0.48	-2.43	-1.62	
-0.36		-1.22	-0.24		-0.81
	0.73	0.49	0.14	0.10	
+0.36		-1.22	0.25		0.05
	-0.04	0.49	-0.15	-0.10	
-0.02		0.07	-0.02		-0.05
	+0.05	-0.03	-0.01	0.01	
0.02		0	0.02		0
	0	0	-0.01	-0.01	
A= -14.17	-8.08	8.08	4.02	-4.02	-4.01

Horizontal reaction at 1= -14.17-8.08/2= -11.125 kN
Horizontal reaction at 4= -4.02-4.01/4.5= - 1.7844 kN
Resolving we have total S=11.125+1.7844 =12.9094 kN
Now the actual sway force of 18.0833 kN, the sway moments will be (18.0833/12.904) x A as in above table.

(c) Reactions

H1=9.31 kN
H4=-9.31 kN V4=61.39 kN
V1=120-61.39=58.61KN

3.3 RESULTS OBTAIND FROM STAAD Pro.

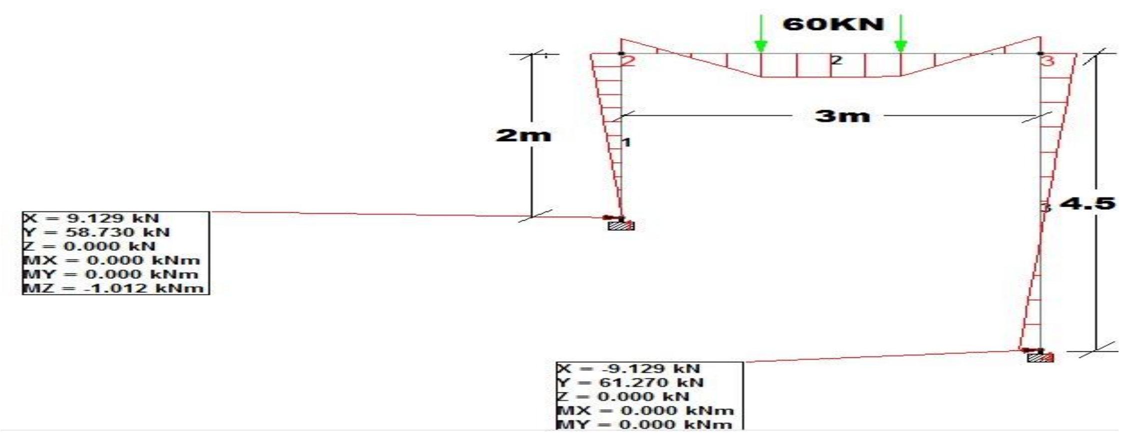
Reactions

Node	L/C	Horizontal FX (kN)	Vertical FY (kN)	Horizontal FZ (kN)	Moment MX (kNm)	MY (kNm)	MZ (kNm)
1	1:concentrated	9.129	58.730	0.000	0.000	0.000	-1.012
4	1:concentrated	-9.129	61.270	0.000	0.000	0.000	15.462

Beam End Forces

Sign convention is as the action of the joint on the beam.

Beam	Node	L/C	Axial Fx (kN)	Shear Fy (kN)	Fz (kN)	Torsion Mx (kNm)	Bending My (kNm)	Mz (kNm)
1	1	1:CONCENTR	58.864	9.036	0.000	0.000	0.000	-3.959
	2	1:CONCENTR	-58.864	-9.036	0.000	0.000	0.000	22.031
2	2	1:CONCENTR	9.048	58.864	0.000	-100.000	0.000	22.031
	3	1:CONCENTR	-9.048	61.136	0.000	-80.000	0.000	-25.439
3	3	1:CONCENTR	61.136	-9.054	0.000	0.000	0.000	-25.439
	4	1:CONCENTR	-61.136	9.054	0.000	0.000	0.000	-15.304



PROBLEM STATEMENT:-

MEMBER PROPERTIES

- All Beams: Rectangular, 400 mm width X 500 mm depth
- All Columns: Rectangular, 400 mm width X 500 mm depth

MEMBER ORIENTATION

- All members : Default

MATERIAL CONSTANTS

- Modulus of Elasticity: 22 KN/sq.mm
- Density : 25 kn/cu.m
- Poisson's Ratio : 0.17

SUPPORTS

Base of all columns : Fixed

LOADS Load case 1: Earth Quake Load

- Zone- III(Z= 0.16)
- Special revisiting moment frame(RF= 5)
- Importance factor = 1
- Soil type medium

- RC frame
- Damping ratio=5
- Self -weight of the structure.
- 1893 load in global x direction Load case 2: Dead Load
- Self -weight of the structure.
- Beams : 30 kN/m in global Y downward
- Load case 3: Live Load
- Beams : 200 kN/m in global Y downward
- Load Case 4: DEAD + LIVE
- L2 X 1.5 + L3 X 1.5
- Load Case 5: DEAD +LIVE+EARTH QUAKE
- L1 X 1.2 + L2 X 1.2+ L3X 1.2
- Load Case 6: DEAD +LIVE-EARTH QUAKE
- -L1 X 1.2 + L2 X 1.2+ L3X 1.2
- Load Case 7: DEAD + EARTH QUAKE
- L1 X 1.5 + L2 X 1.5
- Load Case 8: DEAD -EARTH QUAKE
- -L1 X 1.5 + L2 X 1.5
- Load Case 9: DEAD +EARTH QUAKE
- -L1 X 1.5 + L2 X 0.9
- Load Case 10: DEAD -EARTH QUAKE
- -L1 X 1.5 + L2 X 0.9

ANALYSIS TYPE: P-DELTA

CONCRETE DESIGN:

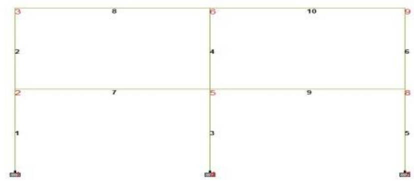
Consider all the load cases.

- Parameters: ultimate tensile strength of steel-415 N/sq.mm
- Concrete strength: 30 N/sq.mm
- Clear cover: 30 mm.
- Centre to centre distance of each beam- 4 m
- Height of each storey
- (a)First the structure is on level ground all the supporting columns being of 4 m height.
- (b)For the second case the web design the frame for same loading combinations but on a sloping ground of I in 5.
- Each beam length = 5m

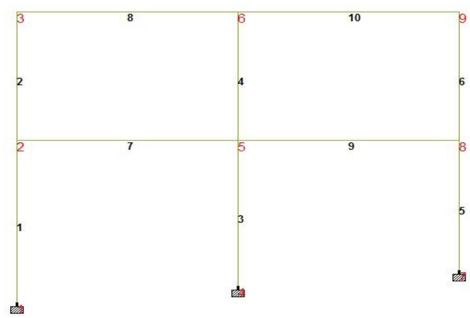
So for this the dimensions of the supporting column are 4m, 4.5 m,5m, 5.5m and 6m.

5.1TWO STOREY (DOUBLE BAY)

PLANE GROUND



SLOPING GROUND

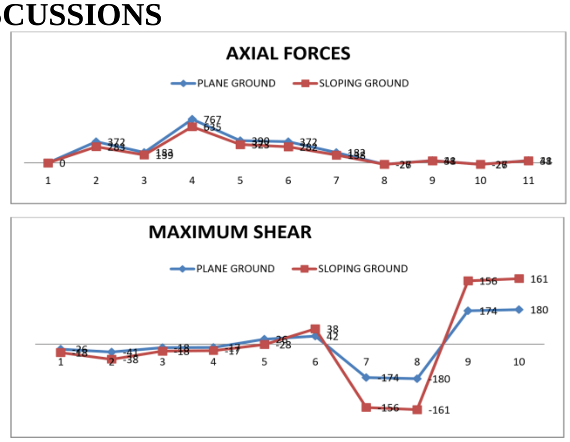


4.3.2 Comparison of Results

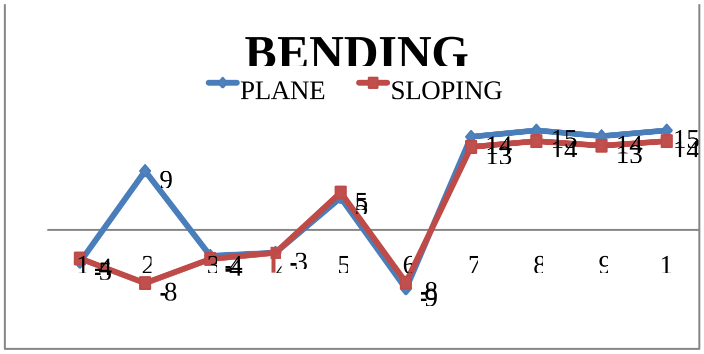
CONCLUSION AT THE END OF PHASE 1-

The results are almost similar and matching with the manual calculations.

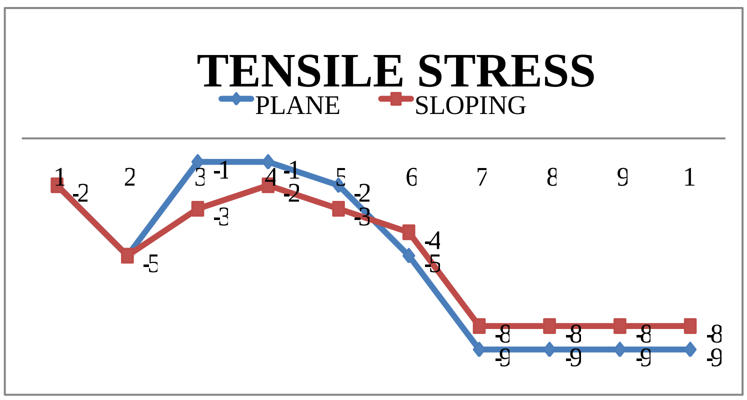
4.0. RESULTS AND DISCUSSIONS



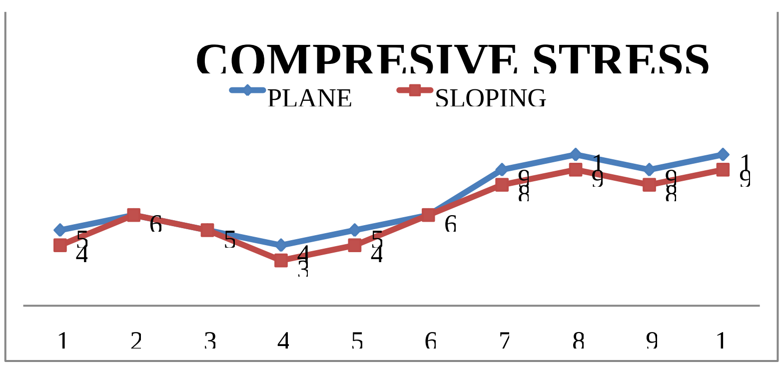
4.1 Axial and shear force variations at different node



4.2 Bending moment variation at different nodes



4.3 Tensile stress variations at different nodes



4.4 Compresive stres variations at different nodes

DUCTILITY DESIGN AND DETAILING

A detailed design of a frame has been carried out with the design aid of IS456 and IS 13920:1993
This phase can be again broadly divided into following:-

6.1Design of a flexural member
6.2Design of an exterior column
6.3 Design of An interior column

To illustrate the design of a sub-frame a flexural member with maximum bending moment has been carried out.
General specification

- The member is designed according to IS 456:2000
- Building > 3 storey height, minimum grade of concrete M20 – we used M30Steel reinforcement of FE 415 used.

6.1Design of the flexural member

General

- Factored axial stress less than 0.1 f_{ck}
- The member should preferably have a width to depth ratio of more 0.3
- Width/depth=400/500=0.8 > 0.3, hence ok
- Width should not be less then 200mm. But we provided width of 400 mm which is ok.
- Depth should not be greater than 0.25(clear span) i.e.(5000- 400)= 4600mm.

6.1.1Longitudinal reinforcement (node 16)

We will find the reinforcement due to a sagging moment of 455kN-m.
Assuming 25 mm dia bars with 25 mm clear cover
Effective depth(d)= 500-25-(25/2)=465mm
From table D of SP 16 :1980 $M_{u,lim}/bd^2$ = 4.14 (for M-30 and FE 415)
$M_{u,lim}$= 4.14 x 400 x 465^2 = 358.1 kN-m.
Actual moment 455 kN-m is greater than 358.1 kN-m ,so we go for the doubly reinforced section.
Reinforcement from table 50 of SP 16: 1890 M_u/bd^2 = 5.26 d^1/d = 0.08
P_t (bottom) = 1.7028
P_c (top) = 0.784

Reinforcement due to the sagging moment

328.98kN-m is the design hogging moment which is not greater than 358.1kN-m ,so we go for the singly reinforced section.
M_u/bd^2 = 3.80
From table 4 we have, P_t (top)=1.282 now required reinforcement is the maximum of 0.784 and 1.282,so finally we have

Reinforcement due to the hogging moment

P_t (top) = 1.282
P_c (bottom) = 1.7028
Reinforcement at top (At)=1.282x 400x465= 2384 mm^2.
Reinforcement at the bottom=1.7028x400x465 =3167.20 mm^2.

Checks

1) The top and bottom reinforcements should atleast contain 2 bars which is the case here.
2) Tension steel ratio p min≤ 0.24 (fck/fy) 1/2=0.258 but we have 1.7028. Hence ok

6.1.2 Longitudinal reinforcement (node 34)

We will find the reinforcement due to a sagging moment of 235 kN-m**.**
Assuming 25 mm dia bars with 25 mm clear cover
Effective depth(d)= 500-25-(25/2)=465mm
From table D of SP 16:1980 $M_{u,lim}/bd^2$ = 4.14(for M-30 and FE 415)
$M_{u,lim}$= 4.14 x 400 x 465^2 =358.1 KN-m.
Actual moment 235 KN-m is less than 358.1 KN-m ,so we go for the singly reinforced section.
Reinforcement from table 4 SP 16: 1890
M_u/bd^2 =2.717, d^1/d=0.08
P_t (bottom) = 0.80

Reinforcement due to the sagging moment

349 KN-m is the design hogging moment which is not greater than 358.1kN-m ,so we go for the singly reinforced section.
M_u/bd^2 = 4.03
From table 4 of SP 16, we have P_t(top)=1.391 so finally we have P_t (top) = 1.391
P_c (bottom) = 0.80
Reinforcement at top =1.391 x 400 x 465= 2587 mm^2.
Reinforcement at the bottom=0.808x400x465 =1503 mm^2.

Checks

1) The top and bottom reinforcements should at least contain 2 bars which is the case here.
2)Tension steel ratio p min≤ 0.24 (fck/fy) 1/2=0.258 but we have 0.808 hence ok

6.1.3 Shear reinforcement requirement

Shear force under consideration will be the maximum of the:-

- **Calculated shear force (** V = **375)**
- **Shear force sue to the formation of the plastic hinges. At both the ends of the beam.**

At node no 16
P_t =3216/(400x465)=2.31% (at top)
$M_{u,lim}/bd^2$ = 6.9 (P_t =2.31,d'/d = 0.08)
$M_{u,lim}$ = 6.9 x 400 x 465^2 = 596 kN-m (maximum hogging moment)
P_t = 2450/(400x465) = 1.32% (at bottom)
P_t = 8.1 (1.32% , d'/d= 0.08)
$M_{u,lim}$ = 8.1 x 400 x 465^2 = 700.56 kN-m(maximum sagging moment)

At node 34

P_t = 1960/(400x465) = 1.05% (at bottom) $M_{u,lim}/bd^2$ =7.3(P_t = 1.05%, d'/d = 0.08)

$M_{u,lim}$ = 7.3 x 400 x 465 ^{2} = 631 kN-m (maximum sagging moment)
P_t =2613/(400x465)=1.40%(at top)
$M_{u,lim}$ /bd^2 = 4.15 (P_t =1.05%, d'/d=0.08)
$M_{u,lim}$ = 4.15 x 400 x 465^2 = 358kN-m(maximum hogging moment)
$V_{34}^{D+L} = V_{16}^{D+L}$ = 1.2 x (30 + 20) = 60

for sway to right:

$$V_{u,a} = V_a^{D+L} - 1{\cdot}4 \left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}} \right]$$

and $$V_{u,b} = V_b^{D+L} + 1{\cdot}4 \left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}} \right]$$

for sway to left:

$$V_{u,a} = V_a^{D+L} + 1{\cdot}4 \left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}} \right]$$

and $$V_{u,b} = V_b^{D+L} - 1{\cdot}4 \left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}} \right]$$

For sway to right
$V_{u,34}$=60-1.4[631+596]/4.6 = - 313kN
$V_{u,16}$ =60+1.4[631+596]/4.6 = 433
For sway to left
$V_{u,34}$=60+1.4[358+700.56]/4.6 = -382 kN.
$V_{u,16}$ =60-1.4[358+700.56]/4.6 = -262 kN
The minimum percentage of steel used is = 1.05% τ_c = 0.66 N/ mm^2. τ_v = 433/(400x 465) =2.32 N/ mm^2. τ 1c,max for M 30 = 3.5 N/ mm^2.
V_{us}= V_u - $\tau_c$bd = 433 - 122= 310 N/ mm^2
We adopt 8 mm two legged stirrups
A_{sv}= 100.52 mm^2 S_{max} is minimum of

a) d/4 = 465/4= 116
b) 8 d_{min}= 8 x 25 = 200
c) S=.87 x 415 x 100.5 x 465/(310 x 1000)= 54.42 = 60 mm.

So we provide stirrups @ 60 mm c/c.
6.2 Design of exterior column [2]
In this example the columns of the ground floor are designed for illustrations. The exterior columns no 1 is designed for the forces based on maximum interaction ration (1 in this case)

- We have size of the column 400mm x 500 mm
- Concrete mix M 30
- Vertical reinforcement Fe 415
- Axial load 1160 KN
- Moment from load 229 KN

The general requirement of the column for the ductility will follow IS 13920:1993 and vertical reinforcement of the column is designed according to IS 456:2000. The transverse and the special confinement reinforcement will be d by following the IS 13920:1993 and IS 456:2000.

General (Column subjected to bending and axial load)
IS 13920:1993 will be applicable if the axial stress > 0.1 f 1ck.
1160 x 1000/ (400 x 500) = 5.8 > 0.1 f 1ck = .
Minimum dimension of the member ≥ 250 and we have taken 400 which is ok.
Shortest cross section dimension / perpendicular dimension ≥ 0.4

Conclusion

The tasks of providing full seismic safety for the residents inhabiting the most earth quake prone regions are far from being solved. However in present time we have new regulations in place for construction that greatly contribute to earthquake disaster mitigation and are being in applied in accordance with world practice.
In the regulations adopted for implementation in India the following factors have been found to be critically important in the design and construction of seismic resistant buildings:

- Sites selection for construction that are the most favourable in terms of the frequency of occurrence and the likely severity of ground shaking and ground failure;
- High quality of construction to be provided conforming to related IS codes such as IS 1893 , IS 13920 to ensure good performance during future earthquakes.
- To implement the design of building elements and joints between them in accordance with analysis .i.e. ductility design should be done.
- Structural-spatial solutions should be applied that provide symmetry and regularity in the distribution of mass and stiffness in plan and in elevation.
- Whereas such the situations demands irregularity maximum effort should be given to done away with the harmful effects like that of "SHORT COLUMN EFFECT"

Researchers indicate that compliance with the above-mentioned requirements will contribute significantly to disaster mitigation, regardless of the intensity of the seismic loads and specific features of the earthquakes. These modifications in construction and design can be introduced which as a result has increase seismic reliability of the buildings and seismic safety for human life.

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STUDIES ON ANALYSIS OF RAINFALL AND RUNOFF IN TELANGANA REGION OF INDIA

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Abstract

Thirty one year rainfall data of Telangana region from 1984-2014 were obtained from the digital records of Government of India (www.data.gov.in). The thirty one year data includes monthly, seasonal and annual rainfall for Telangana region. The average, standard deviation, variance, return period, probability of exceedance are calculated for all monthly, seasonal and annual rainfall values using MS EXCEL. Twenty year rainfall and runoff data in Dindi medium irrigation project of Telangana from 1968- 1988 are obtained from the records and Multiple regression analysis is done using MS EXCEL analysis tool pack. The relationship obtained is in the form Y= 0.2366X - 89.838 where Y is runoff values in mm and X is rainfall values in mm. The correlation coefficient obtained is 0.76.

Keywords: *Rainfall Analysis,Multiple regression, MS EXCEL*

1.0 INTRODUCTION

Accurate and current rainfall characterization is an important rule for water-related system design and management. Updated rainfall intensity-duration-frequency (IDF) relationships in Peninsular India were developed; impacts on runoff and groundwater recharge attributable to changes in rainfall characteristics are discussed. Two data sets were used from gauge in Hyderabad city: hourly rainfall data for the 19 years from 1993-2011 and daily rainfall data for the 30 years from 1982-2011. Hourly data were used to develop updated rainfall IDF relationships; Daily data were used for trend analysis of threshold-basedrainfall events. Greater intensity storms may reduce groundwater recharge and increase runoff, making the surface storage of runoff increasingly important to enhance recharge and reduce flooding risks. (Daniel Dourte et al. 2013).There is mounting evidence from global and regional studies that precipitation patterns are shifting toward more common higher intensity storms and fewer light and moderate events (Kunkel et al. 1999) Effective storm water management plans depend on reliable rainfall intensity-duration-frequency (IDF) relationships. Due to the perception of high-intensity rainfall events as occurring more frequently than expected, the Michigan Dept. of Transportation (MDOT) commissioned a study to update rainfall IDF estimates for each of seven durations 1, 2, 3, 6, 12, 18, and 24 h and six recurrence intervals 2, 5, 10, 25, 50, and 100 years. In contrast to a traditional at-site frequency analysis using method of moments estimators, this study applied a regional frequency analysis approach based on L-moments. Data were compiled from 76 hourly recording stations and 152 daily recording stations, and trend and outlier analyses were conducted on both annual maximum series (AMS) and partial duration series (PDS) data. With the entire state considered a homogeneous region, two regional index flood models were applied: a generalized Pareto distribution (GPD)fit to PDS data PDS/GPD model, and a generalized extreme value distribution(GEV) fit to AMS data AMS/GEV model . Verification of results indicated that the revised rainfall IDF estimates provide more reliable values than those previously used (Christopher M. Trefry et. al 2005). Frequency Analysis of Extreme events was explained (Stedinger et al. 1993)..Hydrologic Frequency Analysis was described (R.A.Wurbs and W.P.James,2009).Probability, Risk and Uncertainty Analysis for Hydrologic and Hydraulic Design was described by Larry W.Mays (2004).

2.0 METHODOLOGIES ADOPTED

(a)DESCRIPTION OF STUDY AREA

Telangana is one of the 29 states in India, located in southern India. Formed in June 2014, it is the youngest state in India.
Its major cities include Hyderabad, Warangal, Khammam, Karimnagar, Nizamabad and Nalgonda. Telangana is bordered by the states of Maharashtra to the north and north west, Chhattisgarh to the north, Karnataka to the west and Andhra Pradesh to the east and south.

Fig 1

Fig 2

RAINFALL

One of the major things that has a great impact to all is change in climate. Telangana generally has three seasons. They are:

1. Summer

Like most countries in South Asia, summer in Telangana generally begins during March and ends somewhere in end-may or Mid-June. During this time, the temperature in the region can reach 40 degrees Celsius especially in May, although the average temperature is normally 25 degrees.

2. Monsoon.

Immediately after summer, from June to about October or so, the monsoon season sets in. During this time, the heat can become very much bearable, warm and humid. Monsoon in Telangana is pretty much marked by plenty of rainfall, with a high level of humidity accompanying it; more than 75% of the rainfall that the state receives happens during this season. July is usually the month where there are more rainy days but September is when the rains are heaviest.

3. Winter.

From November to February, Telangana experiences the winter season. Unlike the northern parts of India, winter in the state isn't overly chilly; it's actually pretty pleasant. During winter, the average minimum temperature hovers around the 13 degree-Celsius range, although it can rise up to the 28-degree range. Humidity in the morning is usually very high, especially during the monsoon season, when it can exceed 80%. During the summer months, humidity can drop to an average of 25 to 30%. The focus of this report will mainly look into the extent of rainfall over the years of 1984-2014 in Telangana, the frequency, probability of exceedance and analysis of the output. The reason is basically linked to the fact that understanding rainfall and its trends is crucial for knowing about the climatic changes, growth of plant forms, everyday usage etc.

DINDI PROJECT

Dindi Project is an existing Medium Irrigation Project constructed across River Dindi a tributary of Krishna River near Dindi (V) & (M), Nalgonda District. This reservoir is a medium water reservoir across Dindi tributary of River Krishna located near Dindi, Mahabubnagar town in Telangana, It is part of Srisailam Left Bank Canal. This medium reservoir has 59 million cubic meters gross storage capacity. It is close to Nagarjunsagar-Srisailam Tiger Reserve, around 95 kilometers from Hyderabad. The Project was commenced during the year 1940 and completed in the year 1943 at a cost of Rs.34.36 Lakhs to irrigate an Ayacut of 12,835 acres. It is located mid way between Hyderabad to Srisailam on Hyderabad Srisailam Highway. This Project contemplates to provide Irrigation facilities to an extent of 12,835 acres.There are no Inter-State problems for this Project. This report also focuses on the rainfall and runoff between the years 1968-1988 of the Dindi Project. The reason is to analyse the data by preparing a multiple regression equation using MS EXCEL. The details of the project are taken from the National Water Management Project Report of Dindi Project (1990).

(b)DATA ACQUISITION
The data are acquired from the web site www.data.gov.in of Government of India.

MONTHLY RAINFALL FROM 1984-2014(All Dimensions in mm)

NAME	Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec	Annual
Telangana	1984	4.2	4.9	3.8	15.9	4.1	91.4	248.1	102	113.3	84.9	4.5	0.3	677.4
Telangana	1985	6.6	0	6.5	13.8	16.2	159.4	208.7	151.9	64	109.7	2.5	4.8	744.1
Telangana	1986	29.2	44.4	1.4	18.1	12.4	105.9	208	352.5	73.2	30.3	26.6	14.6	916.6
Telangana	1987	12.5	1	14.4	9.7	25.7	101.1	218.1	200.4	59.4	112.1	141.8	1.6	897.8
Telangana	1988	0	4.3	1.7	23	7.9	108.3	536.2	294.5	278.9	37.9	0.9	4.9	1298.5
Telangana	1989	0	0	39.6	2.7	7.1	207	422.4	248	170.3	20.8	4.2	9.3	1131.4
Telangana	1990	2.8	3.9	18.3	5.4	172.6	208.4	159.2	397.6	111.9	188.1	7.1	0.1	1275.4
Telangana	1991	16.8	1.9	6	26.6	31.4	196.8	241.7	130.6	141.9	32.9	40.5	3	870.1
Telangana	1992	4.9	0.1	0.6	10.7	25.4	137.2	156.6	244.7	92.3	57.3	43.3	0	773.1
Telangana	1993	1	1.2	15.3	17.5	37.1	76.3	256.2	133.4	152.3	112	2.2	15.5	820
Telangana	1994	9.4	11.9	0.2	24	19.9	85.9	257.7	221.7	45.5	176.1	34.7	0	887
Telangana	1995	65.7	1.1	16.1	16.2	45.8	115.7	227.2	178.9	124.4	285.3	12.4	0	1088.8
Telangana	1996	0.1	5.3	2.2	28.2	12.1	112.9	213.4	265.7	150.3	98.8	18.9	4.1	912
Telangana	1997	24.3	0.5	19.2	47.5	19.3	65.2	166.5	131	166.6	80.5	44	39.3	803.9
Telangana	1998	3.1	20.9	9.6	19.2	25.3	135	226	246.8	185.8	82.6	19.6	0	973.9
Telangana	1999	0	9.9	3.6	1.9	63.6	106.1	194.5	178.1	153.5	44.1	2.3	0	757.6
Telangana	2000	0.5	15.3	1	14.4	49.4	229.6	196.4	389.4	63.6	25.4	4.2	6	995.2
Telangana	2001	5.2	0	17.5	32.5	7.4	183.8	122.3	240.6	144	139.8	2.9	0	896
Telangana	2002	13.3	1.8	1.3	6.3	33.3	136.9	88.9	289.6	86.5	108.7	1.1	0	767.7
Telangana	2003	0	2.7	19.8	11.4	0.6	119.6	317.7	297.2	111.2	108.8	0.4	17.3	1006.7
Telangana	2004	24.1	8.8	6.5	27.4	35.1	61	234.7	149.5	118.4	89.7	6.5	0	761.7
Telangana	2005	36.5	7.3	22	10.5	11.1	99.6	397.6	145.8	315.5	180.8	2.3	1.8	1230.8
Telangana	2006	0	0	47.2	47.6	49.5	103.6	146.5	290.8	289.1	31.6	38.7	0	1044.6
Telangana	2007	0	2.7	0.1	13.3	16	170.9	135.8	212.8	281.3	54.2	15.5	0	902.6
Telangana	2008	0	20	113.2	16	8.4	124.1	173.7	354.1	145.6	28.8	13.8	0.3	998
Telangana	2009	2.2	0	5.3	6.2	26.9	65.5	112.2	165.4	154.8	94.3	31.9	1.1	665.8
Telangana	2010	11.2	7.2	0.9	10.9	40.6	102.3	389.5	278.9	242.3	101.7	45.7	17.7	1248.9
Telangana	2011	0	9.7	2.3	33.1	20.3	80.5	261.6	230.3	84.4	15.7	1.7	0	739.6
Telangana	2012	8.1	0	0.3	18.3	13.7	116.5	263.9	224	182.8	83.8	61.3	0	972.7
Telangana	2013	3.9	30.2	0.1	25.1	12.5	185.6	386.2	212.3	165.1	232.6	17.3	1.1	1021
Telangana	2014	0.1	1.9	47.7	10	70.5	55.4	142.9	173.4	126.8	43.3	12	1.5	685.6

Table 1

Source- www.data.gov.in

SEASONAL RAINFALL FROM 1984-2014 (All dimensions in mm.)

Name	Year	Jan-Feb	Mar-May	Jun-Sep	Oct-Dec
Telangana	1984	9.1	23.8	554.8	89.7
Telangana	1985	6.6	36.5	584	117
Telangana	1986	73.6	31.9	739.6	71.5
Telangana	1987	13.5	49.8	579	255.5
Telangana	1988	4.3	32.6	1217.9	43.7
Telangana	1989	0	49.4	1047.7	34.3
Telangana	1990	6.7	196.3	877.1	195.3
Telangana	1991	18.7	64	711	76.4
Telangana	1992	5	36.7	630.8	100.6
Telangana	1993	2.2	69.9	618.2	129.7
Telangana	1994	21.3	44.1	610.8	210.8
Telangana	1995	66.8	78.1	646.2	297.7
Telangana	1996	5.4	42.5	742.3	121.8
Telangana	1997	24.8	86	529.3	163.8
Telangana	1998	24	54.1	793.6	102.2
Telangana	1999	9.9	69.1	632.2	46.4
Telangana	2000	15.8	64.8	879	35.6
Telangana	2001	5.2	57.4	690.7	142.7
Telangana	2002	15.1	40.9	601.9	109.8
Telangana	2003	2.7	31.8	845.7	126.5
Telangana	2004	32.9	69	563.6	96.2
Telangana	2005	43.8	43.6	958.5	184.9
Telangana	2006	0	144.3	830	70.3
Telangana	2007	2.7	29.4	800.8	69.7
Telangana	2008	20	137.6	797.5	42.9
Telangana	2009	2.2	38.4	497.9	127.3
Telangana	2010	18.4	52.4	1013	165.1
Telangana	2011	9.7	55.7	656.8	17.4
Telangana	2012	8.1	32.3	787.2	145.1
Telangana	2013	34.1	37.7	949.2	251
Telangana	2014	2	128.3	498.5	56.8

Table 2

Source- www.data.gov.in

RAINFALL AND RUNOFF PARTICULARS OF DINDI PROJECT

Year	Rainfall (mm)	Runoff(mm)
1968-69	555.74	15.96
1969-70	579.08	27.15
1970-71	486.06	50.69
1971-72	478.79	19.63
1972-73	271.25	2.03
1973-74	518.44	21.52
1974-75	678.72	47.29
1975-76	1012.61	162.79
1976-77	682.61	91.61
1977-78	583.69	49.18
1978-79	994.58	201.65
1979-80	487.33	76.52
1980-81	459.81	15.03
1981-82	570.07	61.04
1982-83	536.81	10.53
1983-84	634.13	119.9
1984-85	538.81	17.83
1985-86	478.13	9.47
1986-87	489.53	0
1987-88	830.72	11.05

Source : National Water Management Project Report.

3.0 RESULTS AND DISCUSSION

The average, standard deviation, variance, return period, probability of exceedance are calculated for all monthly, seasonal and annual rainfall values using MS EXCEL

ANALYSIS OF RAINFALL DATA

Notation: Probability of Exceedence (Pro)

YEAR	Annual rainfall in mm	Annual rainfall in descending order	Ranking	Return period=(n+1/m)	Pro=(1/return period)
1984	677.4	1298.5	1	100	3.125
1985	744.1	1275.4	2	50	6.25
1986	916.6	1248.9	3	33.33333333	9.375
1987	897.8	1230.8	4	25	12.5
1988	1298.5	1131.4	5	20	15.625
1989	1131.4	1088.8	6	16.66666667	18.75
1990	1275.4	1044.6	7	14.28571429	21.875
1991	870.1	1021	8	12.5	25
1992	773.1	1006.7	9	11.11111111	28.125
1993	820	998	10	10	31.25
1994	887	995.2	11	9.090909091	34.375
1995	1088.8	973.9	12	8.333333333	37.5
1996	912	972.7	13	7.692307692	40.625
1997	803.9	916.6	14	7.142857143	43.75
1998	973.9	912	15	6.666666667	46.875
1999	757.6	902.6	16	6.25	50
2000	995.2	897.8	17	5.882352941	53.125
2001	896	896	18	5.555555556	56.25
2002	767.7	887	19	5.263157895	59.375
2003	1006.7	870.1	20	5	62.5
2004	761.7	820	21	4.761904762	65.625
2005	1230.8	803.9	22	4.545454545	68.75
2006	1044.6	773.1	23	4.347826087	71.875
2007	902.6	767.7	24	4.166666667	75
2008	998	761.7	25	4	78.125
2009	665.8	757.6	26	3.846153846	81.25
2010	1248.9	744.1	27	3.703703704	84.375
2011	739.6	739.6	28	3.571428571	87.5
2012	972.7	685.6	29	3.448275862	90.625
2013	1021	677.4	30	3.333333333	93.75
2014	685.6	665.8	31	3.225806452	96.875

The twenty year rainfall particulars and runoff particulars at Dindi Medium Irrigation Project in Telangana (table 5) are fitted for regression relationship and Regression equation obtained is Y=0.2366X-89.838

Where Y-Runoff value in 'mm',
X-Rainfall value in 'mm'.

The correlation coefficient (r) is 0.762, coefficient of determination (r-square) is 0.58. The standard error computed is 36.86. (Fig 9). The coefficient of determination (r-square) is the ratio between the Regression sum of the square to the total sum of squares (or) the ratio of explained variance to total variance. The present simple regression model has the coefficient of determination 0.58 which indicates that 42% of the total variance is unexplained. The correlation coefficient (r) is the positive square root of the coefficient of the determination. It is the correlation between the response and the observed values. It's value is 0.76. The correlation coefficient being high, there is reason to believe that there is strong linear dependence between the annual and runoff. The standard error of estimate which is the square root of the sum of the squares of the errors. (i.e., errors deviations from mean) divided by the degrees of freedom comes to 36.86. This being not so high is reasonably acceptable. The student's t-test distribution determines the importance of independent variables. The t- statistics for x variable is 4.994. F-test determines the regression model adequacy. The F-test determine whether or not a dependent variable is significantly related to independent variable. The computed F value for the regression is 24.947.The unexplained variance in the Rainfall-Runoff relationship can be reduced by including other relevant parameters, from the data about land and water resources of India.

4.0 CONCLUSIONS AND RECOMMENDATIONS

Reservoir operations based on regression relationship between Rainfall and Runoff at dam sites contributes to environmental sustainability.The regression relationship to be morerealistic must take into account to the parameters such as Catchment area, Interceptionstorage,Slope,etc.
The calculated values of the average,standard deviation,variance,return period andprobability of exceedance for all monthly,seasonal,and annual rainfall for thirty one years from 1984-2014 Indicate the patterns of rainfall inTelangana region.

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SAFETY AT SIGNALIZED INTERSECTIONS – A CASE STUDY IN KOLKATA CITY

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Abstract

Even though intersections constitute a very less area in the overall roadway system, they are the major conflicting areas, as different streams of traffic occupy the same space at the same time. So, a well-engineered geometrical design and optimized traffic control are vital to provide a safe road environment. In this study, existing Indian Road Safety Audits and other relevant guidelines for signalized intersections are reviewed and a new comprehensive format is developed to assist the engineer in formulating a checklist of performance measures. In order to know the safety lacunas at existing signalized intersections, a methodology is formulated to check the operating road conditions for strict conformance to standards. An urban corridor in Kolkata city is selected to implement the methodology. A survey is done at nine signalized intersections in the study corridor using modified checklist and IRC standards are followed to check against operating conditions. Observed lacunas in the design are addressed with possible treatments to improve the road user group safety

Keywords: *Signalized Intersections, Performance Measures, Road Safety Audits, User group safety.*

INTRODUCTION

Even though technology is playing its vital role in the field of transportation, the occurrence of accidents is becoming inevitable. This provoked Engineers to implement better designing procedures and Safety enhancement programs like Road Safety Audit Survey. According to world statistics, intersection related crashes constitute more than 20 percent of fatal crashes, even though intersections constitute very less area of the overall highway system. It is not unusual that crashes are concentrated at intersections, because intersections are the only areas where vehicles moving in different directions want to occupy same space at the same time.

Traffic signals are a common form of traffic control used by state and local agencies to address roadway operations. They allow the shared use of road space by separating conflicting movements in time and allocating delay. Good geometric design in combination with optimized traffic control would ensure safe and efficient operation through intersections. So, it is necessary to check the accuracy of existing road facilities to make sure of safe road environment at signalized intersections.

BACKGROUND

The detailed study of intersections and signal control gives a fair idea of what all need to be looked for in order to assess safety and what can be done to enhance safety. The engineer should be aware of road users and their needs and limitations with regard to signalized intersections. Information displayed in advance of and at the intersection needs to be consistent, timely, legible, and relevant.

Awareness of how human factors play a role in the task of using the intersection will go a long way toward reducing error and the collisions this may cause ("FHWA-Signalized Intersections-Informational guide",).

A facility designed for a different purpose other than intended purpose will not fetch good results in terms of both safety and efficiency. Geometric design often involves a broad range of aspects, which need to be taken care in order to assure safety. Among all, visibility is the primary consideration at an intersection as it is vital to provide any sort of information. All the sight triangles should be free of obstructions. In addition to Visibility, Turning radii, Channelization, Auxiliary lanes and Accessibility also contributes for safe road operations. Channelization requires careful attention as it is the means of separating conflicts in space. (NCHRP Report-500, Volume 12). All the intersection movements should be obvious to the drivers. (IRC: SP-41 1994).Reduce frequency of intersection conflicts through traffic control and operational improvements. An approaching driver should be able to see signals and signs well in time to react. Sequence of signals, location of Signal& Sign posts, Visibility condition of sign posts and markings and also night time lighting should be in conformance with the local regulations and standards. ("IRC-Road safety audits").

OBJECTIVE

The broad objective of this study is to formulate a list of performance measures and identify lacunas in terms of safety of different user groups at signalized intersections and suggesting improvement measures. The measures considered go across the full range of Engineering.

METHODOLOGY

Formulation of Performance Measures: In this division of work, Existing road safety audits and other guidelines for signalized intersections are reviewed and inferred a comprehensive format, which details all measures that need to be looked for in order to ensure safety at signalized intersections.

Selection of Study Corridor: To signify the importance of design and operation of a facility in conformance with the standards, an urban corridor in the Kolkata city is selected which is one among the busy corridors. It starts from Birla Planetarium Intersection to Tollygunge Square Intersection, covering a total of seven major signalized intersections and two minor signalized intersections.

Data Collection and Audit Findings: The checklist survey is done at all signalized intersections in the study corridor. All the operational measures are checked against standards and identified where the deficiencies are prominent.

Table1: Curbs without ramp extensions

Curb Ramps
 
Issue: Curbs connected to cross walks are not provided with ramp extensions. **Justification:** Disabled road users will not be able to access the roads safely unless the ramps are provided to access curbs.

Table 2 : Excessive Advertising Posts

Advertising Posts

Issue:

More advertising posts (commercial) are there near signal and sign heads.

Justification:

Traffic control measures are intended to demand driver's attention well in time to take proper action. But more advertising posts near controlling measures will divert driver's attention and eventually leads to the improper response of the driver.

Table 3 : Buses Halting in the Intersection Area

Bus Stops

Issue:

Buses are halting near intersection obstructing the other vehicular movements.

Justification:

Buses are halting near intersection (2-4 buses per signal cycle during Peak hours), which are creating the major conflict prone zone at the approaches. In order to eliminate rear-end & right angle collisions, buses should be prohibited from halting at intersection area.

All Red Time

Issue:

All red time is not included in the signal cycle.

Justification:

All the intersections in the study area are wide and allowing high traffic volumes. So, in order to assist driver to leave the intersection safely it is required to provide all red time in the signal cycle.

Conclusions & Recommendations:

This study is aimed at developing safety measures for different user groups at signalized intersections. All the measures formulated are grouped under four divisions, first division deals with general layout, second division considers visibility related aspects, third division go across traffic control and management issues and the fourth division takes all the other aspects which also plays major role in ensuring safety at intersections. These all measures are checked for strict conformance to standards at all the signalized intersections in the study corridor and it is found that few deficiencies like absence of curb ramp extensions, hazardous bus halts in the intersection area and excessive existence of advertising posts near traffic informative bodies are common to all. In addition to these common deficiencies, each intersection is found to have specific lacunas like insufficient pedestrian refuge islands, improper location of signal and sign posts and absence of road markings and delineators. All these safety lacunas are addressed with possible improvement measures. These recommendations are listed below,

- All curbs are to be provided with ramp extensions to facilitate the safe movement of disabled road users. And pedestrian waiting area has to be enhanced either by providing refuge islands or delineators.
- Sight triangles at all approaches should be made free of tree branches and vegetation. Re-stripe the road markings and re-install the Sign posts, if they are faded.
- Install pedestrian signals where ever it is necessary. Optimize signal cycles and clearance intervals where the existing cycle lengths are insufficient. And re-locate the signal posts and sign posts according to the relevant standards.

- Eliminate on-street business on the carriageway and sidewalks. Prior intimation about existence of intersection and route directions should be done in the form of informatory boards.
- Wherever the violation of traffic rules (halts beyond stop line & parking violations) are there, serious action should be taken by local enforcement authority.

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DURABILITY STUDIES OF SFR-SCC SUBJECTED TO ELEVATED TEMPERATURES

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Abstract
An attempt was made to develop the concrete with high performance in addition to flow ability. To obtain these properties we require high measure of cementitious substance, however, the main consequence is the concrete will become more brittle in nature, to avoid this steel fibers are introduced in the mix. The concrete having the ability to flow under its own weight even in difficult conditions and in congested reinforcement sections is known as Flowing Concrete or Self Compacting Concrete (SCC). Scarcely any explores are done to consider the properties of SCC consideration of steel fibers. From previous studies two optimum mixes(PF1.12 ,s/a 0.5and PF 1.14,s/a 0.57) were arrived by studying Plain SCC(PSCC) of medium and high strengths(MS&HS) for varying Packing Factors (PF) and s/a(0.5&0.57). In this paper the mechanical characteristics of SFR-SCC(Steel Fibre Reinforced-SCC) containing straight steel fibres (0% -1.2%) of different volume fraction are studied. Maximum compressive strength was observed for a concrete with 1% steel fibre. Hereafter the effect of high temperature (100,200,400,600ºC) on optimum mix was studied as the properties of the materials significantly varies with temperature due to its high compactness. In the present investigations, the specimens are subjected to required temperature for 60min and let to cool down in the furnace itself. The hardened properties measured after fire exposure are compressive strength, split tensile strength, upv, rebound and resistivity. Spalling was noticed when specimens of high strength are exposed to high temperature.

Keywords: Packing Factor), Steel Fibre Reinforced Concrete, Elevated temperature

1. Introduction

Concrete became one of the major construction material in all branches of modern constructions In the present scenario, concrete is the most common and widely used structural material in construction field, apart from steel. The social problem of the poor durability performances of Japanese concrete in structures due to inadequate consolidation of the concrete in the casting operations in 1983 and reduction in the number of skilled workers in Japan had led to the development of SCC. The development of Self-Compacting Concrete (SCC) is a successful achievement in the construction industry. By 1988, the concept was developed and made ready for the first real-scale tests and at the same time the first prototype of self-compacting concrete was completed using materials already on the market. The prototype performed satisfactorily with regard to drying and hardening shrinkage, heat of hydration, denseness after hardening, and other properties and was named “High Performance Concrete.” At almost the same time, “High Performance Concrete” was defined as a concrete with high durability due to low water-cement ratio by professor Aitcin (Ouchi et al., 1996). Since then, the term high performance concrete has been used around the world to refer to high durability concrete. Due to its high fluidity and resistance to segregation it can be pumped longer distances. The concept of SCC was proposed in 1986 by Professor Hajime Okaruma, but the prototype was first developed in 1988 in Japan, by

Professor Ozawa (1989) at the University of Tokyo.[5] Therefore, Okamura (1997) has changed the term for the proposed concrete to "Self-Compacting High Performance Concrete. During their studies, they found the employment of self-compacting concrete had reduced the construction period, eliminated the noise due to vibration and assured compaction in confined zones where compaction is difficult. The cost of SCC was compensated by the elimination of vibrating compaction and work done to level the surface of the normal concrete (Khayat et al., 1997). Once this obstacle has been eliminated, concrete construction could be rationalized and a new construction system, including formwork, reinforcement, support and structural design, could be developed. There is weakness due to the presence of micro cracks in the mortar-aggregate interface. This weakness can be removed or can be made negligible by the inclusion of steel fibres in the mixture. Different types of fibers such as polymer, glass, etc., can also be used in composite materials which can be introduced into the concrete mixture to increase its toughness, or ability to resist crack growth. The self-concrete in which the fibres help to transfer loads at the internal micro cracks is called a Fibre-reinforced Self Compacting Concrete (FRSCC).

2. **Experimental work**:

Objective of work – Considering SFRSCC is a newer type of concrete compared to conventional concrete and PSCC, the research performed on fire exposure is limited. So an attempt was made to study the behaviour of SFRSCC when exposed to fire exposure after studying the mechanical charateristics of SFRSCC containing fibres of 0%,0.4%,0.8%,1%,1.2% volume fraction of straight steel fibres.

2.1 **Materials**

CEMENT: Cement used in the present study is Ordinary Portland cement of 53 grade had a specific gravity 3.2.
AGGREGATE : Coarse aggregate obtained from a local source had a specific gravity 2.64 of 10mm,12.5 and 20mm and river sand with specific gravity 2.6 passing through 4.75mm sieve are used in order attain flowability of concrete.
WATER: Ordinary potable water of normally pH 7 is used for mixing and curing the concrete specimen.
CHEMICAL ADMIXTURES: An accelerator retarding super plasticizer having poly carboxylic ether and water reducing admixture VMA is used. based upon the requirement
MINERAL ADMIXTURE: Fly Ash a supplementary cementious material obtained from thermal power station is used as filler material.
FIBRES : Straight Steel fibres of circular c/s with a dia of 0.5mm and aspect ratio 26 were utilized.

2.2 Mix Design to casting of test specimens : Both medium and high strength SCC for varying PF and s/a were designed and developed by using method suggested by Nan-Su et al (2001) and SV Rao et. al (2010). In this present investigation only fly ash was used as a mineral admixture. The chemical admixtures like Super Plasticizers (SP) and Viscosity Modifying agents (VMA) are used to improve the workability. A total number of 10 mixes are designed in which two are PSCC mixes of grades M40 and M80. Apart from these, fours mixes (M40 - 0.4, 0.8, 1, 1.2) are designed to define the optimize percentage of the steel fiber content. After defining the optimum steel fiber content four SFR-SCC mixes (1.12-0.5 & 1.14-0.57) of different grades are developed to study the durability properties.

2.2.1 Mixing

First 50% of water mixed with75% dosage of super plasticizer is introduced in the mixer and then coarse aggregateis added and agitated for two minutes, then like the dry ingradients cement flyash sand are added to the mix and mixed gradually for 3 min..The remaining 25% of water was mixed with the superplasticizer and VMA and was poured into the mixer and mixed for five minutes.Later required quantities of Steel fibres were sprinkled over the concrete mix and mixed for one minute to get a uniform mix. After the mixing was completed ,tests like V-funnel,J-Ring,L-Box were perfomed and checked accordingly as instructed in EFNARC specifications.

2.2.2 casting

A total number of six 100mm cube specimens, 4 number of 100x200mm cylinders are casted for each mix are prepared to access mechanical properties after 28 days of moist curing. At the age of 28 days the specimens which meant to be exposed to high temperature are heated in electric furnace fig1 Four temperatures are examined to determine the specimens fire resistance ie 100,200,400and 600^{0}C The heating rate is applied at 10^{0}C/min until the target strength is reached and maintained for one hour and the specimens are allowed to cool down and tested to determine properties such as compressive strength, splitting tensile strength, upv, rebound hammer and resistivity.

2.3 Test Procedure:

2.3.1.Fresh or workability tests

In order to characterise the flow and workability properties of the mix slump flow,L-box,U-box were used to measure flowing capacity,filling ability and passing ability of the mix.The obtained results are compared with EFNARC specifications(2).

2.3.2 Compressive strength : The original compressive strength of various SFRSCC mixes containing fibres of 0%,0.4%,0.8%,1%,1.2% volume fraction of straight steel fibres apart ffrom the PSCC are measured on 100mm cubes at the age of 28 days. Residual compressive strength is measured on 100mm cubes after subjecting the cubes to various temperatures and noting the weight loss.

Split tensile strength : The tesile sprength of various mixes is determined on 100x200mm cylinders.

Pulse velocity : Pulse velocity is measured on 100mm cubes according to the procedure described by EN 12504-4 after 28 days of curing using a PUNDIT upv testing devise.

The rebound number is measured using Rebound hammer.test conducted on 100mm cubes on a smooth surface in horizontal position which indirectly gives the compressive strength by giving rebound number.

Resistivity : It is measured on 100mm cubes using electric resistivity meter.by a direct two probe technique. In order to ensure uniform current density brass plate electrodes of the same size and shape of the specimen were used.The brass plate electrodes are kept in good contact with the concrete specimen through a wet cloth.

The resistivity of the concrete was measured by a direct two probe technique on 10 cm cubes. The details of specimen preparation, number of specimens prepared in each grade and the number of specimens exposed in each exposure conditions were given in Chapter-3. In order to ensure a uniform current density, while measuring resistance, brass plate electrodes of the same size and shape as the end surfaces of the specimen were used. The test set up was illustrated in the Fig. 7.1. It can be seen that the brass plate electrodes are kept in good contact with the concrete specimen through a wet cloth. The plate electrodes are firmly clamped on the specimen through a `C' clamp and steel plates which are separated from the electrical circuit through a Neoprene rubber pad. Before starting the experiment the surfaces of the cubes were cleaned to remove any dust or loose material. The resistances were measured immediately on wet surfaces of the cubes taken from normal water. The resistivity is calculated as follows.

$$\rho = RA/L$$

where,
ρ = Resistivity in kohm-cm
R = Resistance measured in kohms
A = Area of the contact surface in cm^2
L = length between two electrodes in cm.

Table 1 -Concrete Mix Proportions

<table>
<tr><th>Trial</th><th>s/a</th><th>PF</th><th>Cement (Kg)</th><th>Flyash (Kg)</th><th>CA (kg)</th><th>FA (Kg)</th><th>Steel fibres (kg)</th><th>Water (ml)</th><th>SP (ml)</th><th>VMA (ml)</th></tr>
<tr><td>HS-PSCC</td><td>o.5</td><td>1.12</td><td>21.5</td><td>0.75</td><td>21.5</td><td>20.3</td><td>Nil</td><td>4200</td><td>283</td><td>30</td></tr>
<tr><td>MS-PSCC</td><td>0.5</td><td>1.12</td><td rowspan="5">29.98</td><td rowspan="5">4.48</td><td rowspan="5">29.98</td><td rowspan="5">27.6</td><td>Nil</td><td rowspan="5">5200</td><td rowspan="5">150</td><td rowspan="5">30</td></tr>
<tr><td rowspan="4">MS-FR-SCC</td><td>0.5</td><td>1.12</td><td>0.82</td></tr>
<tr><td>0.5</td><td>1.12</td><td>1.64</td></tr>
<tr><td>0.5</td><td>1.12</td><td>2.05</td></tr>
<tr><td>0.5</td><td>1.12</td><td>2.46</td></tr>
<tr><td>HS-FR-SCC</td><td>0.5</td><td>1.12</td><td>21.5</td><td>0.75</td><td>21.5</td><td>20.3</td><td>2.05</td><td>4200</td><td>283</td><td>30</td></tr>
<tr><td>HS-FR-SCC</td><td>0.57</td><td>1.14</td><td>19</td><td>0.75</td><td>19</td><td>23.59</td><td>2.05</td><td>4200</td><td>225</td><td>30</td></tr>
<tr><td>MS-FR-SCC</td><td>0.5</td><td>1.12</td><td>29.98</td><td>4.48</td><td>29.98</td><td>27.6</td><td>2.7</td><td>5200</td><td>150</td><td>30</td></tr>
<tr><td>MS-FR-SCC</td><td>0.5</td><td>1.14</td><td>25.08</td><td>4.6</td><td>25.08</td><td>31.19</td><td>2.7</td><td>5000</td><td>150</td><td>30</td></tr>
</table>

Table2 -Fresh properties

Trial	s/a	PF	Flow test(mm)	U-box(mm)	V-box(sec	V-t15(sec)	L-box
M80	0.5	1.12	satisified	28	12	20	0.8
M80	0.57	1.14	satisified	22	30	9.5	0.9
M40	0.5	1.12	satisified	25	10	29	0.9
M40	0.5	1.14	satisified	10	8.2	4	1

3.Results and discussions : The original compressive strength at the age of 28 days containing fibres of 0%,0.4%,0.8%,1%,1.2% volume fraction of straight steel fibres are presented in Table 3 and in graph 1 which shows that SCC develops higher values when compared with PSCC. Also SFRSCC containing 0.4%-1% fibres shows increase in compressive atrength and at 1.2% decrease in strength is seen.
Residual compressive strength ,weight loss upv,rebound number and resistivity after subjecting the cubes to various temperatures are shown in Tables 3-7 for PSCC,MS-SFR-SCC , HS-SFR-SCC for two optimum mixes(PF 1.12 and s/a 0.5 and PF 1.14 and s/a 0.57)which were obtained in previous studies The loss in weight is around 5% for PSCC and 16% for SFR-SCC after subjecting to elevated temperatures for both mixes.It is also observed that for both mixes there is a reduction in rebound number upv after subjecting the specimens to elevated temperatures may be due to crack propagation. An increase in compressive strength when the cubes are subjeted to 100 and 200^{0}C and at 400^{0}C there is a loss of 10-15% strength and at 600^{0}C around 30-35% loss is observed.

Hardened properties

Table 3-Test results of variation of fibre consistency

Description	% of SF	Compressive strength(N/mm2)	Split tensile strength(n/mm2)
PSCC	0	38.26	4.23
M40 SFRSCC	0.4	47.09	4.67
	0.8	50.45	4.92
	1	64.97	5.56
	1.2	56.23	5.19

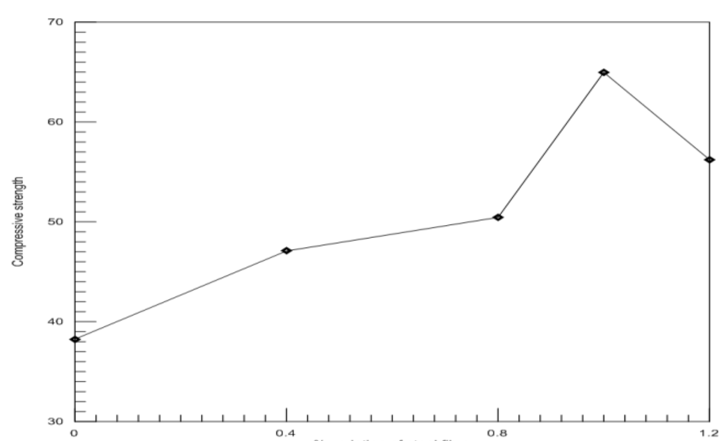


Table 4-Hardened properties of MS-SFR-SCC&HS-SFR-SCC

Sample ID	s/a	PF	UPV		Rebound No		Comp strength(N/mm2		Flexure
	-	-	28days	90 days	28 days	90 days	28days	90 days	90 days
PSCC	o.5	1.12	5090	5090	29	31	86.09	96	
	0.57	1.14	4963	5090	32	35	95.6	77.25	
	0.5	1.12	4963	4787	39	37	55.28	90.31	
	0.57	1.14	5955	5090	32	36	69	61	
HSSFRSCC	o.5	1.12	5017	4673	49	43	93	88	0.72
	0.57	1.14	5127	5025	41	39	96	96	1.07
MSSFRSCC	0.5	1.12	4095	4673	30	37	56	60	0.773
	0.57	1.14	4106	4785	32	35	64	66	0.878

Elevated Temperatures:

The Pulse Velocity at Elevated Temperature i.e.100degrees, 200degrees, 400degrees and 600 degrees of both M40 & M80 Mixes is found out which is illustrated in the tables below.

Table5&6 -Elevated temperature Results for MS-SCC

Mix	PF	s/a	Wt (kg)	%loss in upv	UPV(m/s)	%loss in upv	Resistivity(Kohms)	%loss in resistivity	Comp(Mpa)	% loss in strength
M40			Initial		Initial		Initial		Initial	
100	1.12	0.5	2.48	0	5025	2.45	1.16	25.86	49.5	1.93
200	1.12	0.5	2.27	0.529	5155	4.91	2.85	58.94	49.5	2.46
400	1.12	0.5	2.53	4.116	5291	9.56	2.21	39.82	49.5	23.56
600	1.12	0.5	2.31	4.671	5155	22.2	2.21	39.82	49.5	55.95

Mix	PF	s/a	Wt(kg)	%loss in upv	UPV(m/s)	%loss in upv	Resistivity(Kohms)	%loss in resistivity	Comp(Mpa)	% loss in strength
M40			Initial		Initial		Initial		Initial	
100	1.14	0.57	2.41	0	4785	4.58	1.23	-13.8	45	0
200	1.14	0.57	2.206	9.25	4785	6.71	1.29	13.18	45	2.22
400	1.14	0.57	2.237	3.97	4902	8.94	1.31	25.19	45	20
600	1.14	0.57	2.411	4.23	5025	42.1	1.31	25.19	45	43

Table7&8 -Elevated temperature values for MS-SFR-SCC

Mix	PF	s/a	Wt(kg)	%loss in upv	UPV(m/s)	%loss in upv	Resistivity(Kohms)	%loss in resistivity	Comp(Mpa)	% loss in strength
M40			Initial		Initial		Initial		Initial	
100	1.12	0.5	2.76	10.14	4902	2.39	2.6	-5.769	49.5	-19.43
200	1.12	0.5	2.77	11.48	4902	4.67	3.3	-15.15	49.5	-18.42
400	1.12	0.5	2.8	12.07	5025	23.8	3.02	-39.07	49.5	18.73
600	1.12	0.5	2.77	16.25	4785	68.5	2.93	-43.34	49.5	32.32

Mix	PF	s/a	Wt(kg)	%loss in upv	UPV(m/s)	%loss in upv	Resistivity (Kohms)	%loss in resistivity	Comp (Mpa)	% loss in strength
M40			Initial		Initial		Initial		Initial	
100	1.14	0.57	2.68	10.02	4902	4.67	2.76	-4.348	45	-15.82
200	1.14	0.57	2.65	11.13	4902	4.67	2.59	9	45	-13.58
400	1.14	0.57	2.7	13.69	4673	28.4	2.6	-38.46	45	11.56
600	1.14	0.57	2.65	16.6	4785	51.1	2.69	-53.16	45	31.16

Table 9&10 -Elevated temperature values for HS-SCC

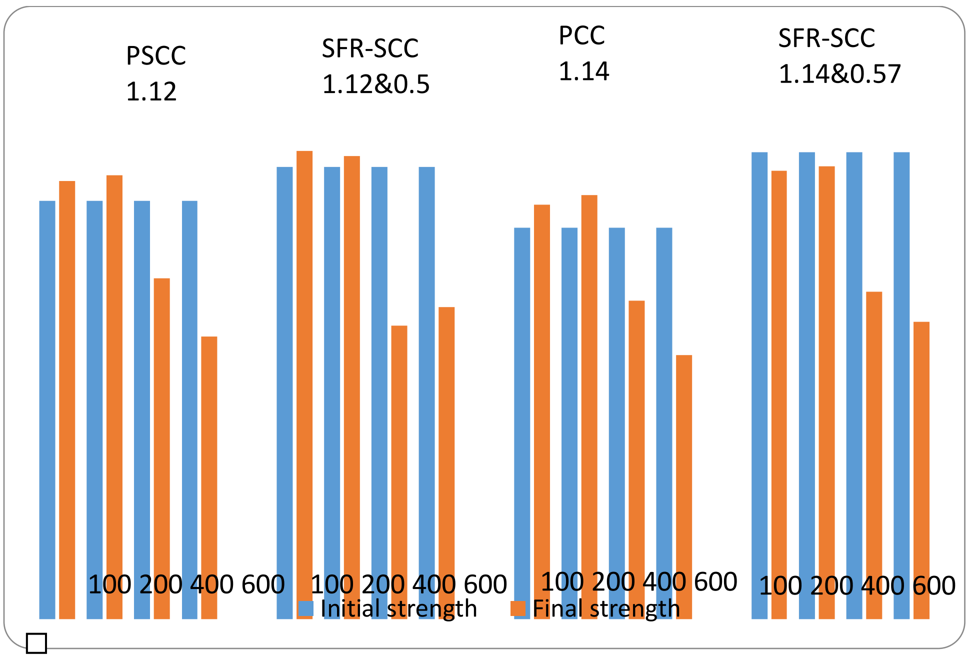
Mix	PF	s/a	Wt(kg)	%loss in upv	UPV(m/s)	%loss in upv	Resistivity(Kohms)	%loss in resistivity	Comp(Mpa)	% loss in strength
M80			Initial		Initial		Initial		Initial	
100	1.12	0.5	2.45	0	5291	2.57	1.57	6.37	86.09	-4.68
200	1.12	0.5	2.41	0.91	5155	9.35	1.37	11.7	86.09	-5.97
400	1.12	0.5	2.5	8.11	5025	13.09	3.23	67.8	86.09	18.52
600	1.12	0.5	2.31	4.94	4902	31.31	3.23	67.8	86.09	32.44

Mix/Temp	PF	s/a	Wt(kg)	%loss in Wt	UPV(m/s)	%loss in upv	Resistivity(Kohms)	%loss in resistivity	Comp(Mpa)	% loss in strength
M80			Initial		Initial		Initial		Initial	
100	1.14	0.57	2.43	-1.65	5291	2.57	1.44	4.86	80.56	-5.79
200	1.14	0.57	2.39	0.83	5155	7.18	1.53	30.1	80.56	-8.27
400	1.14	0.57	2.49	7.80	5155	15.2	2.31	58	80.56	18.74
600	1.14	0.57	2.35	5.53	5025	26.6	2.31	58	80.56	32.57

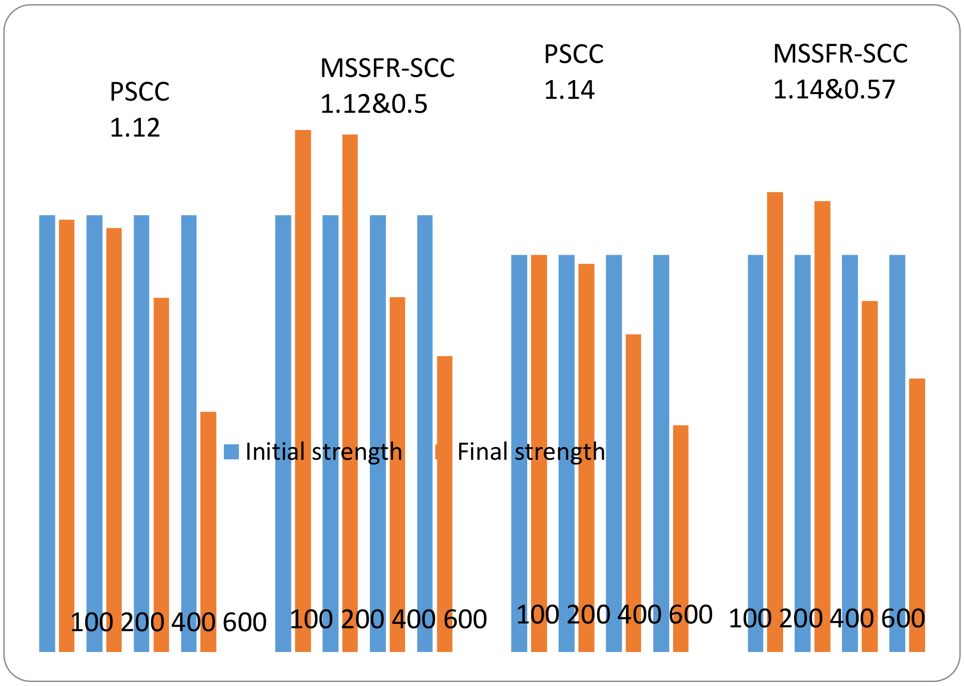
Table 11&12 -elevated temperature values for HS-SFR-SCC

Mix/Temp	PF	s/a	Wt(kg)	%loss in weght	UPV(m/s)	%loss in upv	Resistivity(Kohms)	%loss in resistivity	Comp(Mpa)	% loss in strength
M80			Initial		Initial		Initial		Initial	
100	1.12	0.5	2.695	-0.9	5155	2.522	2.02	4.95	86.09	-11.77
200	1.12	0.5	2.665	0.56	5025	7.005	2.55	-60	86.09	-10.61
400	1.12	0.5	2.69	7.06	4902	28.17	2.68	-17.2	86.09	29.95
600	1.12	0.5	2.685	4.36	4785	26.42	2.88	-11.1	86.09	25.51

Mix	PF	s/a	Wt(kg)	%loss in upv	UPV(m/s)	%loss in upv	Resistivity(K ohms)	%loss in resistivity	Comp(Mpa)	% loss in strength
M80			Initi al		Initial		Initial		Initial	
100	1.14	0.57	2.74	1.46	5155	4.908	2.04	-49.5	80.56	-14.46
200	1.14	0.57	2.75	2.55	4902	6.854	2.35	-94	80.56	-15.57
400	1.14	0.57	2.72	5.66	5155	41.57	2.74	-17.9	80.56	16.45
600	1.14	0.57	2.78	4	4902	37.98	2.97	-9.43	80.56	24.01



Graph1 : Loss in strength for elevated temperatures of different degree centigade



Graph2 : Loss in strength for elevated temperatures of different degree

Conclusions

1. It was observed from the previous experimental results that concrete with PF 1.12 and s/a=0.50 has more flow-ability compared to other packing factors as it contains more paste content. Interestingly, for PF 1.14 and s/a 0.57, material seems to have optimum packing density which reflects on the strength .
2. It was further perceived that from the experiment conducted addition of steel fibers affects slump flow and the maximum strength was observed at 1% addition of steel fibres which clearly depicts from graph 1.
3. The compressive strength of SCC has a pessimistic effect on elevated temperature, as the strength increases more thermal stress are developed for higher strengths compared to medium strength concrete due to its less void content.
4. For temperatures 100 and 200 significant variation was not observed for upv and resistivity. However for temperatures 400 and 600 upv and resistivity decreased by 10%.
5. From elevated temperature studies at 100 and 200 degree centigrade it is observed that there is increase in strength due to fulfilment of hydration in concrete, and at 400 and 600 degree centigrade the strength decreased due to development of thermal stresses.

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STUDIES ON HYBRID FIBRE REINFORCED SELF-COMPACTING CONCRETE MADE WITH STEEL AND POLYPROPYLENE FIBERS

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Abstract

In the present paper, the behaviour of fibre reinforced self-compacting concrete has been studied. Fibres used are Polypropylene Fibres, Steel Fibres and Hybrid Fibres (combination of Polypropylene and Steel Fibres). In the present work, SCC mix of M30 grade was developed without fibres and with Poly-propylene, Steel and Hybrid Fibres. The mechanical properties like compressive strength, tensile strength and flexural strength were studied. For developing PPFRSCC, high dispersion polypropylene fibres 12 mm filament length (0.042% percentage per 1 cu.m. of concrete) were used and for SFRSCC, steel fibres of 30 aspect ratio and 0.4mm diameter (1.3% percentage per 1 cu.m. of concrete recommended was 1.3%) were used. The HFRSCC was obtained by combining 0.042% (by weight of concrete) poly-propylene fibres and 1.3% (by weight of concrete) steel fibres to the plain SCC mix satisfying the fresh concrete properties. All the mixes developed satisfied the fresh and hardened properties of SCC. It was observed that the 28days compressive strength of plain SCC mix is 35.31 MPa. When poly-propylene fibers are added, the strength observed is 38.11 MPa, that is, an increase of 7.93%. When steel fibers were added, the compressive strength observed is 38.20 MPa, that is, an increase of 8.18%. This was found to be 40.44 MPa when hybrid fibres are used with a percentage increase of 14.52.The test results show that use of fibre in SCC, resulting product called fibre reinforced concrete (FRC), improves compressive strength, split tensile strength and flexural performance compared to conventional SCC during loading.

Keywords- *polypropylene fibre, steel fibre, hybrid fibre, SCC, self-compacting concrete*

1.0 Introduction

The development of Self Compacting Concrete (SCC) by Professor Hajme Okamura in 1986 has made a remarkable impact on the construction industry by overcoming some of the problems associated with fresh concrete. Fiber is a small piece of reinforcing material possessing certain characteristics properties. They can be circular or flat. The fiber is often described by a convenient parameter called "aspect ratio". The aspect ratio of the fiber is the ratio of its length to its diameter. Typical aspect ratio ranges from 30 to 150. Fiber reinforced concrete (FRC) is concrete containing fibrous material which increases its structural integrity. It contains short discrete fibers that are uniformly distributed and randomly oriented. Fibers include steel fibers, glass fibers, synthetic fibers and natural fibers. Within these different fibers that character of fiber reinforced concrete changes with varying concretes, fiber materials, geometries, distribution, orientation and densities. Fibres are usually used in concrete to control plastic shrinkage cracking and drying shrinkage cracking. They also lower the permeability of concrete and thus reduce bleeding of water. Some types of fibres produce greater impact, abrasion and shatter resistance in concrete. Generally fibres do not increase the flexural strength of concrete, so it cannot replace moment resisting or structural steel reinforcement. Some fibres reduce the strength of concrete. The amount of fibres added to a concrete mix is measured as a percentage of the total volume of the composite (concrete and fibres) termed volume fraction (V_f).

2.0 Poly-propylene Fibres (Recron 3s Fibre)

Polypropylene is one of the lightest of all thermoplastics (0.9 g/cc). Steel fibre is one of the most commonly used fibre which improves flexural impact and fatigue strength of concrete. Polypropylene is good and increasing impact strength unlike that of flexural strength. Fig 1 shows the Recron 3s fibres.

Fig 1: Recron 3s fibre

3.0 Steel Fibers

Steel fibres are proved to be very effective in conventional and SCC concrete mixes for enhancing their properties. Steel fibres of different diameters, aspect ratios and shapes have been successfully used. Slurry Infiltrated Fibre Concrete (SIFCON) is one type of special concrete with high performance characteristics. Studies on the incorporation of steel fibre in SCC, that is, Steel Fibre Reinforced SCC (SFRSCC) provided feasible and attractive solution to some problems posed by SCC.

4.0 Hybrid Fibre Reinforcement

It has been reported that hybridization of fibres further enhances the efficiency of fibre reinforced concrete. As individual studies on PPFRSCC and SFRSCC have shown, both types of fibres enhance the efficiency of SCC in terms of mechanical properties and durability. A combination of poly-propylene and steel fibres are used in the present studies to investigate the behaviour of HFRSCC. The proportion of poly-propylene fibres to steel fibres is obtained from trial mixes to satisfy fresh and hardened properties.

The fibres used in the present research are Poly-propylene Fibres, Steel Fibres and Hybrid Fibres obtained by mixing poly-propylene and steel fibres in suitable proportion for maintaining the fresh properties of SCC mix.

5.0 Objectives of Present Investigation

Fibre Reinforced Concrete is another special concrete which can be considered for concrete walls. The present investigations are aimed at producing standard grade (M30) FRSCC with Poly-propylene Fibers, Steel Fibers and Hybrid Fibers.

5.1 Phases of Present Investigation

The different phases of the present research work are as follows:

Phase I: Development of M30 grade SCC and obtaining its fresh and hardened properties.

Phase II: Development of

1. Polypropylene Fibre Reinforced SCC (PPFRSCC),
2. Steel Fibre Reinforced SCC (SFRSCC),
3. Hybrid Fibre Reinforced SCC (HFRSCC), and study of fresh and hardened properties.

Phase III: Evaluation of strength properties of PPFRSCC, SFRSCC and HFRSCC.

6.0 Experimental Investigations

In this chapter, the mechanical behaviour of Fibre Reinforced Self-Compacting Concrete of M30 grade made with Steel Fibres, Polypropylene Fibres and Hybrid Fibres (with a mixture of polypropylene and steel fibres) is examined. For this, experimental investigations were carried out on the fresh and hardened properties of fibre reinforced self-compacting concrete of M30 grade. The experimental programme was taken up in various steps to achieve the following objectives:

1. To develop plain SCC of M30 grade and obtain its fresh and hardened properties.
2. To develop Steel and Polypropylene Fibre Reinforced SCC of M30 grade separately and study their fresh and hardened properties.
3. To develop Hybrid Fibre Reinforced SCC with a combination of polypropylene and steel fibres and study its fresh and hardened properties.

6.1 Materials Used

1. Cement

Ordinary Portland cement of 53 grade available in the local market was used in the present investigations. Tests were conducted to establish various properties in accordance with IS: 4031–1988 and found to conform to IS: 12269–1987.

2. Coarse Aggregate

Crushed angular granite available from a local market was used in the investigations. The aggregate was tested as per IS: 2386–1963 and found to be conforming to the specifications.

3. Fine Aggregate

River sand available in the local market was used as fine aggregate. The fine aggregate was tested for its properties as per IS: 2386–1963 and found to be conforming to specifications.

4. Fly Ash

Fly ash from a thermal power station in Andhra Pradesh was used in the investigations. The physical and chemical properties of the fly ash as used in the investigations conform to grade I fly ash of IS 3812–2003.

5. Super Plasticizer

Super plasticizer with Sulphonated Naphthalene based Formaldehyde (SNF) conforming to IS: 9103–1999 was used in the present investigations.

6. Viscosity Modifying Admixture (VMA)

Viscosity modifying agent from a standard agency conforming to standard specifications was also used.

7. Water

Potable water conforming to IS: 3025–1986 part 22 & 23 and IS:456–2000 was used in the investigations.

8. Polypropylene Fibres

Recron 3s fibre (CT-2024) is a fibre developed after extensive research at Reliance Technology Centre.CT-2024 is monofilament fibre designed specially to provide integral secondary reinforcement of concrete. Recron 3s fibres are Polyester staple fibres mainly used for mixing in concrete and mortar for improving certain properties of the concrete and mortar. Recron 3s fibres are availab le in 6mm and 12mm length. In our research work fibre dosage was fixed as 0.5% per bag of cement.

Fig. 2. Polypropylene fibre

9. Steel Fibres

Plain steel fibres of 0.4 mm dia and Aspect Ratio of 30, cut from steel wire were used in the investigations.

Fig. 3: Steel Fibres

7.0 PHASE I: Development of Plain SCC and Investigations on its Fresh and Hardened Properties

In this phase of investigations, M30 grade SCC mix is developed using mineral and chemical admixtures to study its fresh and hardened properties. For developing SCC of M30 grade, the mix proportions were designed based on the method suggested by Nan-Su et al(2001) using fly ash as mineral admixture and chemical admixtures like Super Plasticizers (SP) and Viscosity Modifying Agents (VMA). Finally, SCC mixes which have given required compressive strength with satisfactory fresh properties were taken for the next phase of investigations.

7.1 Mix Design and Trial Mix Proportions of Self Compacting Concrete

An SCC mix of M30 standard grade was aimed and the initial mix proportion was obtained using the mix design methods as mentioned above. The mix proportion thus obtained was fine-tuned by incorporating different guidelines available and making various trial mixes to obtain the mix which satisfies the required fresh and hardened properties.

The final mass of ingredients for 1m^3 of SCC are as follows:

Mass of Cement	=	330.0 kg
Mass of filler (Fly Ash)	=	150.0 kg
Mass of water	=	186.0 kg
Mass of Coarse Aggregate (CA)	=	794.4 kg
Mass of Fine Aggregate(FA)	=	860.6 kg
Super plasticizer dosage	=	1.5% by weight of cement (bwc)
VMA dosage	=	0.6% by weight of cement (bwc)

7.2 Testing of Hardened SCC

A proper time schedule for testing of hardened SCC specimens was maintained in order to ensure proper testing on the due date. The specimens were tested using standard testing procedures as per IS : 516 – 1959.

7.2.1 Compression Test

After the required curing period, the SCC cubes were taken out of the curing tanks and the moisture was wiped off to make the surface dry. They were placed in the Compression Testing Machine (CTM) in such a way that the face perpendicular to the direction of pouring of SCC mix was on the bearing surfaces and the load was applied centrally at a uniform rate of 140 Kg./sq.cm./minute until the failure of the specimens, in accordance with IS 516–1959. The testing was done on a 3000 kN capacity Compression Testing Machine.

7.2.2 Split Tension Test

The split tension test was carried out on a cylindrical specimen of diameter 150mm and 300mm long as per IS specifications.
The horizontal stress = $2P / \pi LD$
Where P = compressive load applied on the cylinder, L = length and D = diameter of the cylinder.
In order to reduce the concentration of high compressive stress near the points of application of the load, narrow strips of suitable packing material, such as plywood, are placed in between the cylindrical specimen and the loading platens of the testing machine.

7.2.3 Flexural strength test

The flexural strength test of beam, a specimen of size 500*100*100mm is placed over two point loading arrangement and the stresses produced during breakage of specimen. The flexural strength is reported as Modulus of Rupture f_t (N/mm2) and is calculated as $f_t = PL/bd^2$
where P = Load at which the beam specimen fails (in KN)
L = effective length of the beam specimen (in mm)
b = width of the beam specimen (in mm)
d = depth of the beam specimen (in mm).

8.0 PHASE II: Development of FRSCC and Studies on Fresh and Hardened Properties

8.1 Addition of Polypropylene Fibres to SCC Mixes

Anti-crack high dispersion poly-propylene fibres were added in different dosages to the selected SCC mixes in the first batch of investigation and Polypropylene Fibre Reinforced Self-Compacting Concrete (PPFRSCC) was developed. After adding poly-propylene fibres to SCC mixes, its influence on fresh and hardened states was observed by conducting tests on fresh and hardened PPFRSCC. The tests on fresh and hardened PPFRSCC were conducted in the same way as they were conducted for SCC.

8.2 Development of Steel Fibre Reinforced Self-Compacting Concrete and Hybrid Fibre Reinforced Self-Compacting Concrete Mix Proportions

The PPFRSCC mix with an optimum dosage of poly-propylene fibres, satisfying the fresh and the hardened properties, was considered for the next phase of investigation. Similarly, the dosages of steel fibres in Steel Fibre Reinforced Self-Compacting Concrete (SFRSCC) with a fixed aspect ratio and Hybrid Fibre Reinforced Self-Compacting Concrete (HFRSCC) consisting of a mixture of poly-propylene and steel fibres, were developed by trial mixes. The ratio of steel fibres to poly-propylene fibres was determined in the laboratory by trial mixes satisfying the fresh and the hardened properties.

9.0 Test Results

The results of experimental investigations carried out in different phases are presented below

9.1 PHASE I: Development of SCC and Studies on Fresh and Hardened Properties

The first phase of investigations was carried out to develop SCC mix of a minimum strength M30 grade using fly ash and chemical admixtures, and to study its fresh and hardened properties. For developing SCC of strength M30 grade, the mix was designed based on Nan-Su method of SCC mix design using fly ash as the mineral admixture. Finally, SCC mixes which yielded satisfactory fresh properties and required compressive strengths, were selected and taken for further investigations.

9.2 Determination of Optimum Dosage of Polypropylene Fibres and Steel Fibres

Based on the results given in Table 1, literature and trial mixes, the minimum optimum dosage of polypropylene fibre and steel fibre in SCC was selected as 1.0 kilograms per cubic metre and 31.42 kilograms per cubic metre of SCC, respectively. The PPFRSCC, SFRSCC and HFRSCC mixes with optimum dosage of polypropylene fibres and steel fibres satisfying fresh and hardened properties were considered for the next phase investigations. The mix proportions are shown in Tables 1 and 2.

Table 1 Mix Proportions of SCC and FRSCC

S. No	Cement kg	Fly Ash kg	CA kg	FA kg	Water kg	SP % bwp	VMA % bwp	Polypropylene Fibres % of 1 m^3 concrete	Steel Fibres % of 1 m^3 concrete	Design-ation
1	330	150	794.4	860.6	186	1.2	0.06	-	-	SCCP
2	330	150	794.4	860.6	186	1.2	0.06	0.042	-	PPFRSCC
3	330	150	794.4	860.6	186	1.4	0.06	-	1.3	SFRSCC
4	330	150	794.4	860.6	186	1.5	0.06	0.042	1.3	HFRSCC

bwp -by weight of cement and fly ash

Table 2 Fibre Content of FRSCC

S.No	Designation	Mix proportion
1	PPFRSCC	SCCP + 1.0 kg/m^3 of HD Polypropylene Fibre
2	SFRSCC	SCCP + 31.42 kg/m^3 of Steel fibre
3	HFRSCC	SCCP + 1.0 kg/m^3 of HD Polypropylene Fibre + 31.42 Kg/m^3 of Steel Fibre

Table 3 Compressive and split tensile strength properties of SCC and FRSCC at 28 days

S.No	Designation	Cube Compressive Strength MPa	% Increase	Cylinder Compressive Strength MPa	% Increase	Split-Tensile Strength MPa	% Increase
1	SCCP	35.31	-	26.02	-	5.89	-
2	PPFRSCC	38.11	(+) 7.93	27.66	(+) 6.30	6.07	(+) 3.05
3	SFRSCC	38.20	(+) 8.18	28.52	(+) 9.61	6.65	(+)12.90
4	HFRSCC	40.44	(+)14.52	32.16	(+)23.59	6.90	(+)17.15

Table 4 Flexural strength properties of SCC and FRSCC at 28 days

S.No	Designation	Flexural Strength MPa	% Increase
1	SCCP	6.34	
2	PPFRSCC	7.19	(+) 13.40
3	SFRSCC	7.83	(+)23.50
4	HFRSCC	8.52	(+) 34.38

The main thrust of the present investigations is to develop M30 grade SCC and to study its behaviour when different types of fibres are introduced into it. For this purpose, M30 grade plain SCC was developed using fly ash as an ingredient with super plasticizer and viscosity modifying agent as a chemical admixture. The method of mix design suggested by Nan-Su(2001) is adopted. The number of trial mixes were developed in the laboratory and a mix satisfying the guidelines given by EFNARC in fresh state and compressive strength in hardened state, was finalized. The cement content used is 330 kg/m^3 with 150kg of fly ash and 182 litres of water giving a water/binder ratio of 0.3875.

In the present work, SCC with three different types of fibers, namely poly-propylene fibres, steel fibres and hybrid fibres, consisting a mixture of polypropylene and steel fibres satisfying strength and fresh properties, were developed. Here, the basic proportion of mix is not altered but the volume and aspect ratio of fibres were so chosen as to satisfy the fresh properties. For developing PPFRSCC, high dispersion poly-propylene fibres with 12mm filament length (0.042% percentage per 1 cu.m. of concrete) were used and For SFRSCC, steel fibres of 30 aspect ratio and 0.4mm diameter (1.3% percentage per 1 cu.m. of concrete recommended was 1.3%) were used. The HFRSCC was obtained by combining 0.042% of poly-propylene fibres and 1.3% steel fibres to the plain SCC mix satisfying the fresh concrete properties.

The hardened properties of different SCC mixes such as cube and cylinder compressive strengths and split tensile strengths, were obtained by testing the specimens of standard size as given by BIS specifications. Cubes and cylinders of standard sizes were cast, cured and tested as per IS 516-1959.

From the compressive cube strength results, it can be seen that the 28days compressive strength of plain SCC mix is 35.31 MPa. When polypropylene fibers are added, the strength observed is 38.11 MPa, that is, an increase of 7.93%. When steel fibers were added, the compressive strength observed is 38.20 MPa, that is, an increase of 8.18%. This was found to be 40.44 MPa when hybrid fibres are used with a percentage increase of 14.52. The above results clearly show that the addition of fibres has enhanced the compressive strength which is due to the holding of the concrete that is, confining the concrete. However, the effect is different in different types of fibers, and hybridization of fibers enhanced the confining effect partly due to the presence of high dispersion polypropylene fibers holding the concrete at micro-crack level and steel fibers at a later stage.

It can be seen that the split tensile strength of plain SCC is 5.89 MPa, that is, 16.68% of compressive strength, and it was enhanced with the addition of different fibers. The percentage enhancement of split tensile strength for PPFRSCC over plain SCC is 3.05 %, for SFRSCC 12.90%, and for HFRSCC 17.15%. The increase is due to the presence of fibers.

It can be seen that the flexural strength of plain SCC is 6.34 MPa, and it was enhanced with the addition of different fibers. The percentage enhancement of flexural strength for PPFRSCC over plain SCC is 13.40 %, for SFRSCC 23.50%, and for HFRSCC 34.38 %.

10.0 Conclusions

Based on the investigations carried out on Fibre Reinforced Self-Compacting Concrete Mixes the following conclusions are drawn.

1. In the case of high dispersion of polypropylene fibres, a dosage of 1 kg of fibres/m^3 of concrete (0.042% by weight of concrete) is used as optimum dosage by suitably adjusting the dosage of admixtures.
2. The aspect ratio and volume of steel fibres are selected satisfying the fresh and hardened properties of self-compacting concrete by suitably adjusting the dosage of admixtures. In the case of steel fibres, a dosage of 31.42 kg of fibres/m^3 of concrete (1.3% by weight of concrete) is used as optimum dosage by suitably adjusting the dosage of admixtures.
3. The compressive strengths of the FRSCC design mixes are found to be increased by the addition of fibres.
4. The addition of polypropylene fibres and steel fibres has shown improved compressive strengths. The increase in compressive strength in SFRSCC was found to be higher than that of PPFRSCC.
5. In the case of HFRSCC, the compressive strengths were found to be further enhanced due to the combined action of polypropylene and steel fibres, and the increase in compressive strength is 14.52% over plain SCC.
6. The addition of fibres improved the split tensile strength which is found to be maximum in HFRSCC. Hence, it is concluded that the hybridization of poly-propylene and steel fibres is useful in improving the strength properties of FRSCC.
7. The addition of polypropylene fibres and steel fibres has shown significantly improved flexural strengths. The increase in flexural strength in SFRSCC was found to be higher than that of PPFRSCC.

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STUDIES ON FRESH AND HARDENED PROPERTIES OF SELF-COMPACTING CONCRETE WITH GGBS, SILICA FUME AND METAKAOLIN AS MINERAL ADMIXTURE (M30 GRADE)

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Abstract

This Experimental studies demonstrates that fresh and hardened properties of self-compacting concrete with Ground granulated blast furnace slag, silica fume and Metakoalin as mineral admixture (M30 grade). based on IS 10262-2009 mix design for self-compacting concrete, Slump-flow, V-funnel, L-box and U-box tests have been carried out on silica fume, GGBS and Metakoalin. The compressive strength of the specimens have been analyzed for 7 days and 28 days respectively. The percentages of cement are replaced by Silica fume of 3%,6%,9%,12%, and GGBS of 15%,20%,25%,30% and Metakoalin of 15% and 20% for different water-cement ratio like 0.40 and 0.45 are taken respectively. From the Experimental investigations, The Compressive strength of silica fume is higher at 6% i.e., 36.83 N/mm^2 in case of 0.40 water-cement ratio compare to 35.33 N/mm^2 (0.45 w/c). Beyond 6% of silica fume, the compressive strength decreases with increase in various percentages. In case of GGBS compressive strength is higher at 25% i.e., 43.66 N/mm^2 in case of 0.40 water-cement ratio compare to 42 N/mm^2 (0.45 w/c). Beyond 25% of GGBS the compressive strength started decreasing with increase in various percentages. In case of Metakoalin compressive strength decreases from 15 to 20% in case of both water cement ratio i.e.,38.50 N/mm^2 (0.40 w/c) and 37.66 N/mm^2 (0.45 w/c).The compressive strength of GGBS in all various percentages is higher than silica fume and Metakoalin properties of self-compacting concrete.

Keywords: *Silica fume, GGBS, Metakoalin, Fresh and Hardened properties, Compressive strength.*

1. INTRODUCTION

In general, a newly placed concrete is compacted by vibrating equipment to remove the entrapped air, thus making it dense and homogeneous; this is referred to as normally vibrated concrete (NVC) in this thesis. Compaction is the key for producing good concrete with optimum strength and durability (The Concrete Society and BRE, 2005). However, in Japan in the early 1980's, because of the increasing reinforcement volumes with smaller bar diameters and a reduction in skilled construction workers, full compaction was difficult to obtain or judge, leading to poor quality concrete (Okamura and Ouchi, 1999). Professor Okamura therefore proposed a concept for a design of concrete independent of the need for compaction. Ozawa and Maekawa produced the first prototype of SCC at the University of Tokyo in 1988 (Ozawa et al., 1989; RILEM TC 174 SCC, 2000). Since that time SCC has gone from a laboratory novelty to practical applications all over the world. The increasing numbers of papers published every year that deal with all aspects of SCC, e.g. mix design, rheological and physical properties and applications in practice indicate research on this technology is thriving. Recommendations on the design and applications of SCC in construction have now been developed by many professional societies, including the American Concrete Institute (ACI), the American Society for Testing and Materials (ASTM), Center for Advanced Cement-Based Materials (ACBM), Precast Consulting Services (PCS) and Reunion International des Laboratories et Experts des Matériaux, systems de construction et ouvrages (RILEM) etc. Symposiums and workshops on this topic have been organized by these societies and several test methods have been or are in the process of standardization. The self-compacting concrete, which is rich in fines content, is proved to be more durable. The usage of mineral admixtures in the production of SCC not only provides economical benefits but also reduces heat of hydration (EFNARC guidelines 2002). It is also known that some mineral admixtures may improve rheological properties and reduce thermally-induced cracking of concrete due to the reduction in the overall heat of hydration, and increase of the workability and long-term properties of concrete (Recommendation for Construction of Self Compacting Concrete 1998). There is no standardized mix proportion for designing SCC, hence in this work the IS 10262:2009 design mix is adopted by GGBS, Silica fume and metakoalin as mineral admixtures with replacement of cement. Further, a comparison of the self-compatibility properties, and hardened properties like Compressive Strength for GGBS based SCC, and SF based SCC and Metakoalin based SCC is made.

2. DESIGN MIX PROCEDURE

2.1 GENERAL

There is no standardized mix proportion for designing SCC, hence in this project work the IS 10262:2009 design mix is adopted by GGBS, Silica fume and metakoalin as mineral admixtures with replacement of cement.

TABLE: 1 Total mix quantities proportions

MIX QUANTITIES:

MATERIAL	CEMENT (Kg/m^3)	FA (Kg/m^3)	CA (Kg/m^3)	SUPER PLASTICIZER (litres)	WATER (litres)
Silica fume	18.825	68.4	47.36	0.320	9.072
GGBS	23.231	86.0	59.60	0.425	11.345
Metakoalin	19.37	34.4	23.78	0.170	4.536

3. DISCUSSION ON EXPERIMENTAL RESULTS

3.1 MATERIALS

The materials used in the experimental investigation of SCC were
1. Ordinary Portland cement-53 grade
2. Coarse Aggregate of size 10mm
3. River sand
4.Water
5. Admixture a. Mineral Admixtures (Silica fume, GGBS, Metakoalin) b. Chemical Admixtures (B233)

TABLE: 2 Physical properties of Ordinary Portland Cement

S. No	Property	Test Method	Test Results	IS Standard
1.	Normal Consistency	Vicat Apparatus (IS: 4031 Part - 4)	35%	
2.	Specific gravity	Sp. Gr bottle (IS: 4031 Part - 4)	3.15	
3.	Initial setting time Final setting time	Vicat Apparatus (IS: 4031 Part - 4)	90 minutes 207 minutes	Not less than 30 minutes Not less than
4.	Fineness	Sieve test on sieve no.9 (IS: 4031 Part – 1)	2%	10%
5.	Soundness	Le-Chatlier method (IS: 4031 Part – 3)	2 mm	Not more than 10 mm

TABLE: 3 Properties of fine and coarse aggregate

Sl. No.	Property	Fine Aggregate	Coarse Aggregate
1	Specific gravity	2.80	2.80
2	Loose Density	1545.69 kg/m^3	1310 kg/m^3

TABLE: 4 properties of super platicizer: GLENIUM B233

Sl. No	Property	Result
1	Form or State	Liquid
2	Color	Brown
3	Specific gravity	1.22. to 1.225 at 30^0C
4	Dosage	0.5 to 2.5 liters per 100 kg of cement

TABLE: 5 physical properties of metakoalin

S.No	Description of physical properties	Units	Results
1	Color		1Close To Std
2	Appearance		1 OFF white Powder
3	Bulk Density	Gm/cc	0.50
4	Oil Absorption	Ml/100gm	64.00
5	Moisture (EX-Work)	%	0.22
6	PH (10% A2 Slurry)		6.22
7	RESIDUE on 325 Mesh	%	0.13
8	PSD –D(50)- 50% particles	μ	1.68
9	Ret.on 500 mesh		0.05%
10	Water absorption	Ml/100gm	66.80
11	Whiteness		95.80%
12	Brightness		93.50%
13	Specific gravity		2.63

TABLE: 6 Physical properties of silica fume

1	State	Amorphous Sub micron Powder
2	Colour	Grey
3	Specific gravity	2.1 to 2.4
4	Solubility	Insoluble
5	Specific gravity(a)Densified (b)Un Densified	608 to 720 kg/m3 192 to 320 kg/m3

TABLE: 7 Physical and Chemical properties of GGBS

S.NO	CHARACTERISTICS	REQURIMENT AS PER BS:6699	TEST RESULT
1	**Fineness (M/KG)**	275(MIN)	390
2	Specific gravity		2.85
3	Particle size (cumulative %)	45 MICRON	97.10
4	Insoluble residue (%)	1.5 (MAX)	0.49
5	Magnesia,content(%)	14.0(MAX)	7.73

6	Sulphide sulphur	2.00(MAX)	0.50
7	Sulphite content (%)	2.50(MAX)	0.38
8	Loss on ignition (%)	3.00(MAX)	0.26
9	Manganese content (%)	2.00(MAX)	0.12
10	Chloride content (%)	0.10(MAX)	0.009
11	Glass content (%)	67(MIN)	91
12	Moisture content (%)	1.00(MAX)	0.10

3.2 FRESH PROPERTIES

TABLE: 8 Fresh properties of M30 SCC (W/C-0.40 variable)

W/C-0.40	**SILICA FUME**				**GGBS**				**METAKOALIN**	
REPLACEMENT	3%	6%	9%	12%	15%	20%	25%	30%	15%	20%
Slump flow (mm)	639	636	635	638	653	656	656	655	650	653
T 50(sec)	4	5	6	5	6	5	4	5	4	5
V-funnel(sec)	6	7	8	7	9	7	7	8	7	8
V-funnel T5min(sec)	10	11	9	10	12	9	12	11	11	14
L-box(h2/h1)	0.944	0.948	0.942	0.921	0.849	0.913	0.937	0.921	0.931	0.939
U-box (mm)	6	9	10	7	12	11	13	8	9	9

EFFECT OF SILICA FUME ON FRESH PROPERTIES: For 0.40 w/c ratio as the silica fume increases from 3% to 6% Slump value decreases by 0.469 %, T500 value increases by 25.00%, V funnel value increases by 16.666%, V funnel at T5 minutes increases by 10.00 %,L box value increases by 0.424 %, U box value decreases by 50.00%. For 0.40 w/c ratio as the silica fume increases from 3% to 9% Slump value decreases by 0.626 %, T500 value increases by 50.00 %, V funnel value increases by 33.333 %, V funnel at T5 minutes decreases by 10.00 %, L box value decreases by 0.211 %, U box value increases by 66.667 %.For 0.40 w/c ratio as the silica fume increases from 3% to 12% Slump value decreases by 0.156 %, T500 value increases by 25.00%, V funnel value increases by 16.667%, V funnel at T5 minutes remains the same,L box value decreases by 2.436 %, U box value increases by 16.667 %.

EFFECT OF GGBS ON FRESH PROPERTIES: For 0.40 w/c ratio as the GGBS increases from 15% to 20% Slump value increases by 0.459 %, T500 value decreases by 16.667%, V funnel value decreases by 22.222%, V funnel at T5 minutes decreases by 25.00 %,L box value increases by 7.538 %, U box value decreases by 8.333%.For 0.40 w/c ratio as the GGBS increases from 15% to 25% Slump value increases by 0.459 %, T500 value decreases by 33.333 %, V funnel value decreases by 22.222 %, V funnel at T5 minutes remains the same, L box value increases by 10.365 %, U box value increases by 8.333 %.For 0.40 w/c ratio as the GGBS increases from 15% to 30% Slump value increases by 0.306 %, T500 value decreases by 16.666%, V funnel value decreases by 11.111 %, V funnel at T5 minutes decreases by 8.333%, L box value increases by 8.481 %, U box value decreases by 33.333 %.

EFFECT OF METAKOALIN ON FRESH PROPERTIES: For 0.40 w/c ratio as the metakoalin increases from 15% to 20% Slump value increases by 0.462 %, T500 value decreases by 25.00%, V funnel value increases by 14.286%, V funnel at T5 minutes increases by 27.273 %,L box value increases by 0.859 %, U box value remains the same.

TABLE: 9 Fresh properties of M30 SCC (W/C-0.45 variable)

W/C-0.45	**SILICA FUME**				**GGBS**				**METAKOALIN**	
REPLACEMENT	3%	6%	9%	12%	15%	20%	25%	30%	15%	20%
Slump flow (mm)	632	637	645	649	653	654	645	655	653	651
T 50(sec)	3	5	5	6	4	8	4	5	4	5
V-funnel(sec)	6	7	8	7	9	7	7	8	7	7
V-funnel T5min(sec)	10	10	11	12	11	12	10	11	11	11
L-box(h2/h1)	0.947	0.937	0.952	0.941	0.919	0.939	0.946	0.951	0.939	0.947
U-box (mm)	11	9	10	12	11	10	8	13	9	11

EFFECT OF SILICA FUME ON FRESH PROPERTIES: For 0.45 w/c ratio as the silica fume increases from 3% to 6% Slump value increases by 0.791 %, T500 value increases by 66.666%, V funnel value decreases by 12.50%, V funnel at T5 minutes remains the same,L box value decreases by 1.056 %, U box value decreases by 18.18%.For 0.45 w/c ratio as the silica fume increases from 3% to 9% Slump value increases by 2.057 %, T500 value increases by 66.667 %, V funnel value remains the same, V funnel at T5 minutes increases by 10.00 %, L box value increases by 0.528 %, U box value decreases by 9.090 %.For 0.45 w/c ratio as the silica fume increases from 3% to 12% Slump value increases by 2.618 %, T500 value increases by 100.00%, V funnel value increases by 12.50%, V funnel at T5 minutes increases by 20.00 %,L box value decreases by 0.633 %, U box value increases by 9.091 %.

EFFECT OF GGBS ON FRESH PROPERTIES: For 0.45 w/c ratio as the GGBS increases from 15% to 20% Slump value increases by 0.153 %, T500 value decreases by 25.00%, V funnel value decreases by 11.111 %, V funnel at T5 minutes increases by 9.091%, L box value increases by 2.176 %, U box value decreases by 9.090%.For 0.45 w/c ratio as the GGBS increases from 15% to 25% Slump value decreases by 1.225 %, T500 value remains the same, V funnel value decreases by 22.222 %, V funnel at T5 minutes decreases by 9.090 %, L box value increases by 2.938 %, U box value decreases by 27.272 %.For 0.45 w/c ratio as the GGBS increases from 15% to 30% Slump value increases by 0.306 %, T500 value decreases by 25.00%, V funnel value decreases by 11.111%, V funnel at T5 minutes remain the same, L box value increases by 3.482 %, U box value increases by 18.182 %.

EFFECT OF METAKOALIN ON FRESH PROPERTIES: For 0.45 w/c ratio as the metakoalin increases from 15% to 20% Slump value decreases by 0.306 %, T500 value decreases by 25.00%, V funnel value remains the same, V funnel at T5 minutes remains the same, L box value increases by 0.852 %, U box value decreases by 22.222%.

3.3 HARDENED PROPERTIES

TABLE: 10 Mechanical properties of M30 SCC(0.40 w/c)

N/mm^2	W/C-0.40										
	CC	**SILICA FUME**				**GGBS**				**METAKOALIN**	
		3%	**6%**	**9%**	**12%**	**15%**	**20%**	**25%**	**30%**	**15%**	**20%**
7 days	28.00	22.00	24.50	24.00	21.50	26.00	27.50	28.50	28.50	26.00	24.50
	29.00	23.00	27.50	26.50	23.00	25.50	26.50	27.00	26.00	27.00	25.00
	30.00	25.00	22.00	22.00	24.00	25.00	25.00	26.50	25.50	25.50	25.50
AVERAGE	**29.00**	**23.33**	**24.83**	**24.16**	**22.83**	**25.50**	**26.33**	**27.33**	**26.66**	**26.16**	**25.00**
28 days	39.50	29.50	38.00	35.00	22.00	38.50	41.00	43.00	43.00	39.00	38.00
	41.00	39.50	36.50	36.50	38.50	41.00	41.50	45.50	42.50	39.50	36.00
	43.00	32.50	36.00	36.00	33.00	40.00	42.50	42.50	42.00	41.50	38.50
AVERAGE	**41.16**	**33.83**	**36.83**	**35.83**	**33.50**	**39.80**	**41.50**	**43.66**	**42.50**	**39.85**	**37.50**

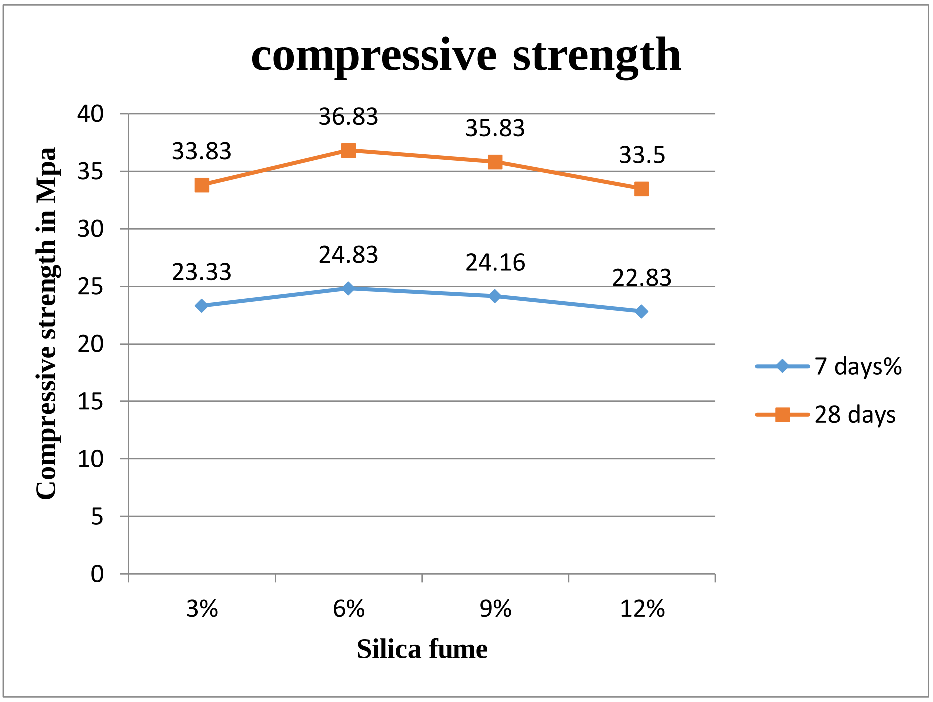

FIGURE:1 Silica fume vs compressive strength(w/c-0.40)

EFFECT OF SILICA FUME ON THE HARDENED PROPERTIES: For 0.40 w/c ratio as the silica fume increases from 3% to 6% Compressive Strength increases by 8.86%.For 0.40 w/c ratio as the silica fume increases from 3% to 9% Compressive Strength increases by 5.912 %.For 0.40 w/c ratio as the silica fume increases from 3% to 12% Compressive Strength decreases by 0.975 %.

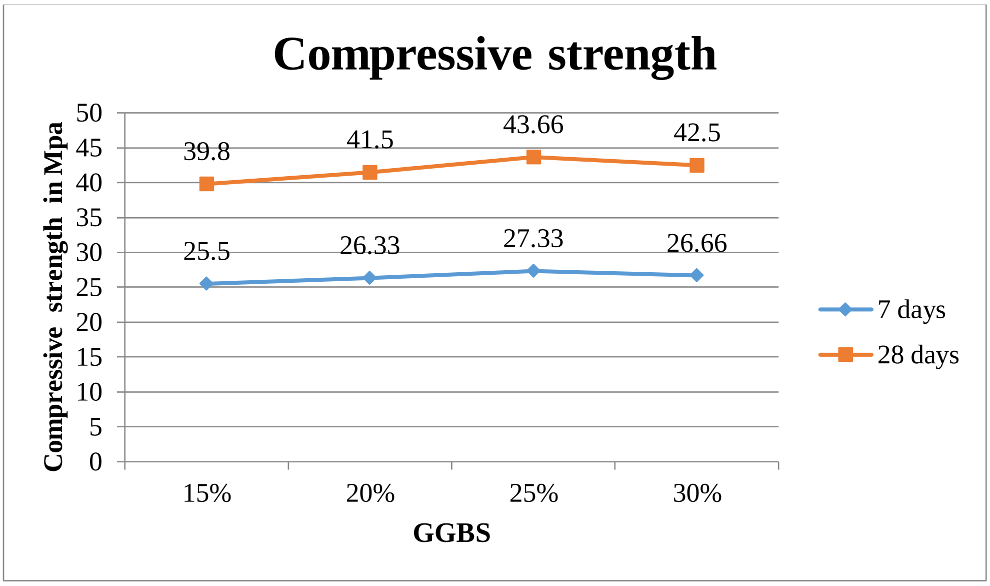

FIGURE:2 GGBS vs compressive strength(w/c-0.40)

EFFECT OF GGBS ON THE HARDENED PROPERTIES: For 0.40 w/c ratio as the GGBS increases from 15% to 20% Compressive Strength increases by 4.271 %.For 0.40 w/c ratio as the GGBS increases from 15% to 25% Compressive Strength increases by 9.698 %.For 0.40 w/c ratio as the GGBS increases from 15% to 30% Compressive Strength increases by 6.784 %.

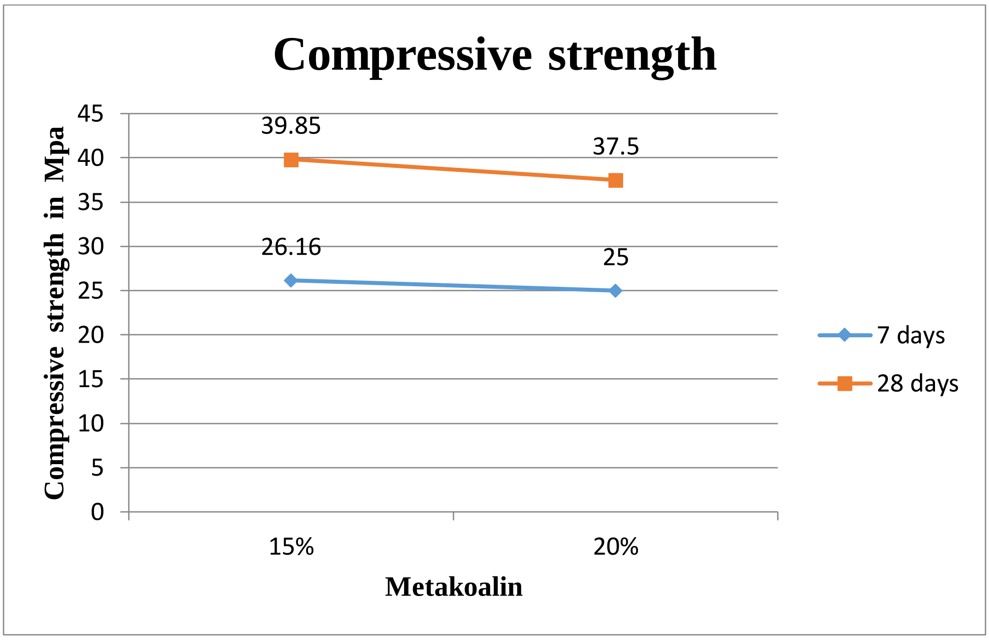

FIGURE:3 Metakoalin Vs Compressive strength(w/c-0.40)

EFFECT OF METAKOALIN ON THE HARDENED PROPERTIES : For 0.40 w/c ratio as the metakoalin increases from 15% to 20% Compressive Strength decreases by 5.897%.

TABLE: 11 Mechanical properties of M30 SCC(0.45 w/c)

N/mm^2	W/C-0.45											
	CC	SILICA FUME				GGBS				METAKOALIN	
		3%	6%	9%	12%	15%	20%	25%	30%	15%	20%
7 days	27.50	21.00	23.00	22.00	21.00	25.00	26.00	26.00	26.50	23.50	22.50
	28.50	22.50	25.50	24.50	22.00	24.16	24.50	25.50	24.50	26.00	23.00
	29.00	24.50	21.00	21.00	23.00	23.00	23.50	25.00	24.00	25.00	24.00
AVERAGE	**28.33**	**19.00**	**23.16**	**22.50**	**22.00**	**24.16**	**24.66**	**25.50**	**25.00**	**24.83**	**23.16**
28 days	39.00	27.50	36.00	34.00	31.00	36.50	38.50	41.00	42.00	38.50	37.00
	40.00	37.00	34.50	35.00	36.50	39.00	39.00	43.50	41.50	37.00	35.50
	41.50	31.50	35.50	35.50	34.00	38.50	39.50	41.50	41.50	39.00	37.50
AVERAGE	**40.16**	**32.00**	**35.33**	**34.80**	**33.80**	**38.00**	**39.00**	**42.00**	**41.66**	**38.16**	**36.66**

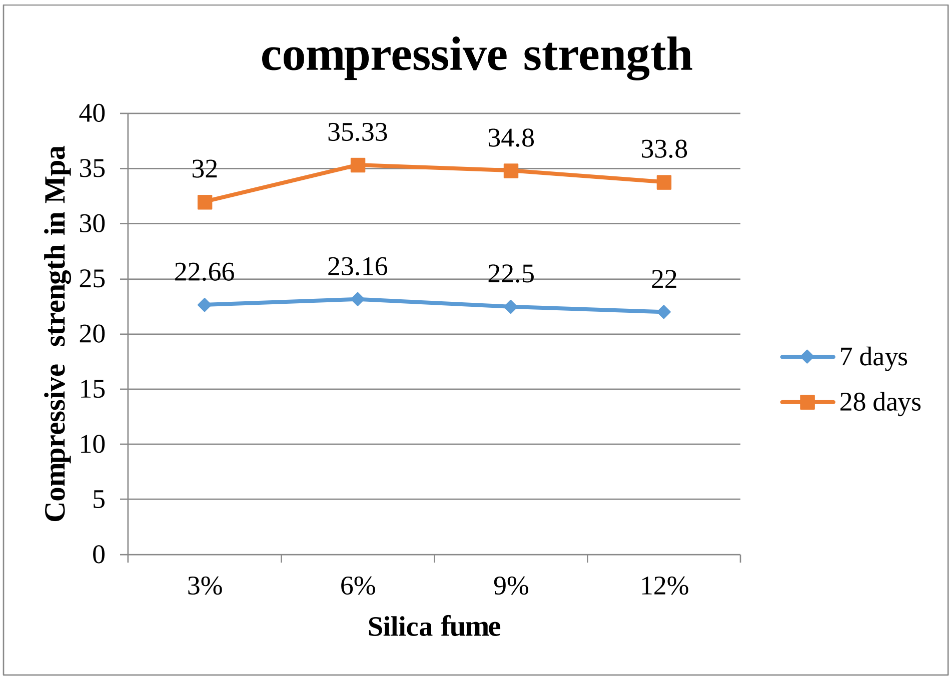

FIGURE:4 Silica fume Vs Compressive strength(w/c-0.45)

EFFECT OF SILICA FUME ON THE HARDENED PROPERTIES: For 0.45 w/c ratio as the silica fume increases from 3% to 6% Compressive Strength increases by 10.406 %. For 0.45 w/c ratio as the silica fume increases from 3% to 9% Compressive Strength increases by 0.875 %.For 0.45 w/c ratio as the silica fume increases from 3% to 12% Compressive Strength decreases by 0.562 %.

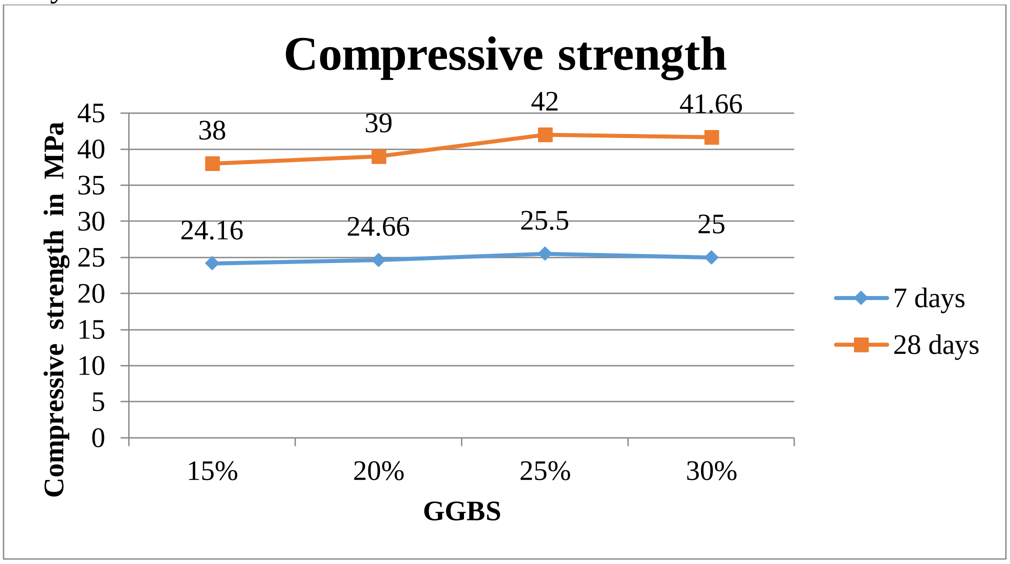

FIGURE:5 GGBS Vs Compressive strength(w/c-0.45)

EFFECT OF GGBS ON THE HARDENED PROPERTIES: For 0.45 w/c ratio as the GGBS increases from 15% to 20% Compressive Strength increases by 2.632 %.For 0.45 w/c ratio as the GGBS increases from 15% to 25% Compressive Strength increases by 10.526 %.For 0.45 w/c ratio as the GGBS increases from 15% to 30% Compressive Strength decreases by 9.632 %

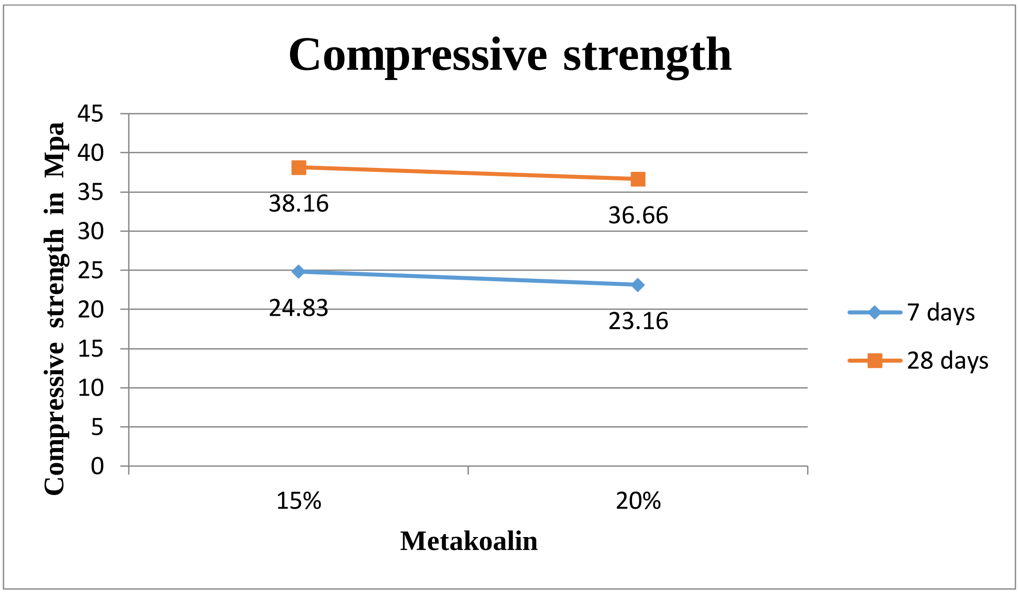

FIGURE:6 Metakoalin Vs Compressive strength(w/c-0.45)

EFFECT OF METAKOALIN ON THE HARDENED PROPERTIES : For 0.45 w/c ratio as the metakoalin increases from 15% to 20% Compressive Strength decreases by 3.930 %.

5.CONCLUSIONS

- The change in the percentage variation of flow values decreases as the water cement ratio increases from 0.40 to 0.45, flow values decreases in slump flow, T50, v-funnel, T5 min, L box and U box
- The compressive strength decreased with the increase in the water cement ratio.
- Compressive strength of silica fume is higher at 6% i.e., 36.83 N/mm^2 in case of 0.40 water-cement ratio compare to 35.33 N/mm^2 (0.45 w/c). Beyond 6% of silica fume, the compressive strength decreases with increase in various percentages.
- In case of ggbs compressive strength is higher at 25% i.e., 43.66 N/mm^2 in case of 0.40 water-cement ratio compare to 42 N/mm^2 (0.45 w/c). Beyond 25% of ggbs the compressive strength started decreasing with increase in various percentages.
- In case of metakoalin compressive strength decreases from 15% to 20% in case of both water cement ratio i.e.,38.50 N/mm^2 (0.40 w/c) and 37.66 N/mm^2 (0.45 w/c).Further percentages are not been take into consideration due to decrease in compressive strength.
- The compressive strength of ggbs in all various percentages is higher than silica fume and metakoalin properties of self compacting concrete.

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A BRIEF REVIEW ON STRENGTH AND DURABILITY PROPERTIES OF FLY ASH AND SLAG BASED GEOPOLYMER CONCRETE

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Abstract

Concrete is the world's most versatile, durable and reliable construction material. Next to water, concrete is the most used material, which required large quantities of Portland cement. Ordinary Portland cement production is the second only to the automobile as the major generator of carbon dioxide, which polluted the atmosphere. In addition to that large amount energy was also consumed for the cement production. Hence, it is inevitable to find an alternative material to the existing most expensive, most resource consuming Portland cement. Geopolymer concrete is an innovative construction material which shall be produced by the chemical action of inorganic molecules. Fly Ash, a by- product of coal obtained from the thermal power plant is plenty available worldwide. Flyash is rich in silica and alumina reacted with alkaline solution produced aluminosilicate gel that acted as the binding material for the concrete. It is an excellent alternative construction material to the existing plain cement concrete. Geopolymer concrete shall be produced without using any amount of ordinary Portland cement. This paper briefly reviews the constituents of geopolymer concrete, its strength and potential applications.

Keywords: *Geopolymer Concrete, Fly Ash, GGBS, Strength, Durability.*

1.0 Introduction

Concrete is one of the most widely used construction material in the world. Ordinary Portland cement (OPC) is conventionally used as the primary binder to produce concrete. Production of Portland cement is currently exceeding 2.6 billion tons per year worldwide and growing at 5 percent annually which generates nearly 7% of atmospheric carbon-dioxide contributing largely to the global warming. Cement manufacturing is power intensive and about 120 kWh of power is required to produce one tons of cement resulting into consumption of nearly 200 kg of coal. On the other hand, a huge volume of fly ash is generated around the world. Most of the fly ash is not effectively used, and a large part of it is disposed in landfills which affects aquifers and surface bodies of fresh water. Hence, it is the need of hour to find an alternative material to the existing most expensive cement-concrete. Several studies have been carried out to reduce the use of Portland cement in concrete to address the global warming issues. These include the utilization of supplementary cementing materials such as fly ash, silica fume, granulated blast furnace slag, rice husk ash and metakaolin and the development of alternative binders to Portland cement.

Davidovits [1988] proposed that an alkaline liquid could be used to react with the silicon (Si) and the aluminium (Al) in a source material of geological origin or in by-product materials such as fly ash, GGBS and rice husk ash to produce binders. Because the chemical reaction that takes place in this case is a polymerization process, he coined the term "Geopolymer" to represent these binders. Geopolymer concrete is concrete which does not utilize any Portland cement in its production. Geopolymer concrete is being studied extensively and shows promise as a substitute to Portland cement concrete. Research is shifting from the chemistry domain to engineering applications and commercial production of geopolymer concrete.

There are two main constituents of geopolymers, namely the source materials and the alkaline liquids. The source materials for geopolymers based on alumina-silicate should be rich in silicon (Si) and aluminium (Al). These could be natural minerals such as kaolinite, clays, etc. Alternatively, by-product materials such as fly ash, silica fume, slag, rice-husk ash, red mud, etc could be used as source materials. The choice of the source materials for making geopolymers depends on factors such as availability, cost, type of application, and specific demand of the end users. The alkaline liquids are from soluble alkali metals that are usually sodium or potassium based. The most common alkaline liquid used in geopolymerisation is a combination of sodium hydroxide (NaOH) or potassium hydroxide (KOH) and sodium silicate or potassium silicate.

This paper is devoted to heat-cured low-calcium fly ash-based geopolymer concrete. Low-calcium (ASTM Class F) fly ash is preferred as a source material than high-calcium (ASTM Class C) fly ash. The presence of calcium in high amounts may interfere with the polymerization process and alter the microstructure [Gourley and Johnson, 2005].

II. Geopolymer Concrete Materials

Fly ash: A by-product from thermal power stations which is found to have rich in silica and alumina is used in geopolymer concrete which further helps in reducing global warming.

GGBS: Ground-granulated blast-furnace slag which is a by-product of iron and steel-making industry obtained from a blast furnace and is a fine powder. GGBS is a glassy, granular, non-metallic material consisting essentially of silicates and aluminates of calcium and other bases.

Aggregates: Fine and Coarse Aggregates are used in Geopolymer concrete.

Alkaline Solutions: The most common alkaline liquid used in geopolymerisation is a combination of sodium hydroxide (NaOH) or potassium hydroxide (KOH) and sodium silicate or potassium silicate.

Super Plasticizer: High range water reducing (naphthalene sulphonate-based) super plasticizer was used in the mixtures at the rate 1.5% of fly ash to improve the workability of the fresh geopolymer concrete.

III Geopolymer Production

The primary difference between geopolymer concrete and Portland cement concrete is the binder. The silicon and aluminium oxides in the low-calcium fly ash and GGBS reacts with the alkaline liquid to form the geopolymer paste that binds the loose coarse aggregates, fine aggregates and other un-reacted materials together to form the geopolymer concrete. As in the case of Portland cement concrete, the coarse and fine aggregates occupy about 75 to 80% of the mass of geopolymer concrete. The influence of aggregates, such as grading, angularity and strength, are considered to be the same as in the case of Portland cement concrete [Lloyd and Rangan, 2009]. Therefore, this component of geopolymer concrete mixtures can be designed using the tools currently available for Portland cement concrete.

Studies have been carried out on fly ash-based geopolymer concrete. The compressive strength and the workability of geopolymer concrete are influenced by the proportions and properties of the constituent materials that make the geopolymer paste. Research results [Hardjito and Rangan, 2005] have shown the following:

- Higher concentration (in terms of molar) of sodium hydroxide solution results in higher compressive strength of geopolymer concrete.
- Higher ratio of sodium silicate solution-to-sodium hydroxide solution ratio by mass, results in higher compressive strength of geopolymer concrete.
- The slump value of the fresh geopolymer concrete increases when the water content of the mixture increases. Superplasticizers may assist in improving workability.
- As the H_2O-to-Na_2O molar ratio increases, the compressive strength of geopolymer concrete decreases.

As can be seen from the above, the interaction of various parameters on the compressive strength and the workability of geopolymer concrete is complex. In order to assist the design of low-calcium fly ash-based geopolymer concrete mixtures, a single parameter called "water-to-geopolymer solids ratio" by mass was devised. In this parameter, the total mass of water is the sum of the mass of water contained in the sodium silicate solution, the mass of water used in the making of the sodium hydroxide solution, and the mass of extra water, if any, present in the mixture. The mass of geopolymer solids is the sum of the mass of fly ash, the mass of sodium hydroxide solids used to make the sodium hydroxide solution, and the mass of solids in the sodium silicate solution (i.e. the mass of Na_2O and SiO_2). The results showed that the compressive strength of geopolymer concrete decreases as the water-to-geopolymer solids ratio by mass increases [Hardjito and Rangan, 2005]. This test trend is analogous to the well-known effect of water-to-cement ratio on the compressive strength of Portland cement concrete. Obviously, as the water-to-geopolymer solids ratio increased, the workability increased as the mixtures contained more water

IV Design of Geopolymer Concrete Mixtures

Concrete mixture design process is vast and generally based on performance criteria, some simple guidelines for the design of heat-cured low-calcium fly ash-based geopolymer concrete have been proposed [Hardjito et al, 2004; Rangan, 2008; Sumajouw, 2007]. The performance criteria of a geopolymer concrete mixture depend on the application. For simplicity, the compressive strength of hardened concrete and the workability of fresh concrete are selected as the performance criteria. In order to meet these performance criteria, the alkaline liquid-to-fly ash ratio by mass, water-to-geopolymer solids ratio by mass, the wet-mixing time, the heat-curing temperature, and the heat-curing time are selected as parameters.

The mixture design process is illustrated by the following Example: Mixture proportion of heat-cured low-calcium fly ash-based geopolymer concrete with design compressive strength of 45 MPa is needed for precast concrete products.
Assume that normal-density aggregates in SSD condition are to be used and the unit-weight of concrete is 2400 kg/m3. Take the mass of combined aggregates as 77% of the mass of concrete, i.e. 0.77x2400= 1848 kg/m3. The combined aggregates may be selected to match the standard grading curves used in the design of Portland cement concrete mixtures. For instance, the aggregates may comprise 277 kg/m3 (15%) of 20mm aggregates, 370 kg/m3 (20%) of 14 mm aggregates, 647 kg/m3 (35%) of 7 mm aggregates, and 554 kg/m3 (30%) of fine sand to meet the requirements of standard grading curves. The fineness modulus of the combined aggregates is approximately 5.0.

The mass of low-calcium fly ash and the alkaline liquid = 2400 – 1848 = 552 kg/m3. Take the alkaline liquid-to-fly ash ratio by mass as 0.35; the mass of fly ash = 552/ (1+0.35) = 408 kg/m3 and the mass of alkaline liquid = 552 – 408 = 144 kg/m3. Take the ratio of sodium silicate solution-to-sodium hydroxide solution by mass as 2.5; the mass of sodium hydroxide solution = 144/ (1+2.5) = 41 kg/m3; the mass of sodium silicate solution = 144 – 41 =103 kg/m3.

Therefore, the trial mixture proportion is as follow: combined aggregates = 1848 kg/m3, low-calcium fly ash = 408 kg/m3, sodium silicate solution = 103 kg /m3, and sodium hydroxide solution = 41 kg/m3. To manufacture the geopolymer concrete mixture, commercially available sodium silicate solution A53 with SiO2-to-Na2O ratio by mass of approximately 2, i.e., Na2O = 14.7%, SiO2 = 29.4%, and water = 55.9% by mass, is selected. The sodium hydroxide solids (NaOH) with 97-98% purity is purchased from commercial sources, and mixed with water to make a solution with a concentration of 8 Molar. This solution comprises 26% of NaOH solids and 74% water, by mass.

For the trial mixture, water-to-geopolymer solids ratio by mass is calculated as follows: In sodium silicate solution, water = 0.559x103 = 58 kg, and solids = 103 – 58 = 45 kg. In sodium hydroxide solution, solids = 0.26x41 = 11 kg, and water = 41 – 11 = 30 kg. Therefore, total mass of water = 58+30 = 88 kg, and the mass of geopolymer solids = 408 (i.e. mass of fly ash) +45+11 = 464 kg. Hence the water-to-geopolymer solids ratio by mass = 88/464 = 0.19. Using the data given in Table 2, for water-to-geopolymer solids ratio by mass of 0.19, the design compressive strength is approximately 45 MPa, as needed. The geopolymer concrete mixture proportion is therefore as follows:

20 mm aggregates = 277 kg/m3, 14 mm aggregates = 370 kg/m3, 7 mm aggregates = 647 kg/m3, fine sand = 554 kg/m3, low-calcium fly ash (ASTM Class F) = 408 kg/m3, sodium silicate solution (Na2O = 14.7%, SiO2 = 29.4%, and water = 55.9% by mass) = 103 kg/m3, and sodium hydroxide solution (8 Molar) = 41 kg/m3 (Note that the 8 Molar sodium hydroxide solution is made by mixing 11 kg of sodium hydroxide solids with 97-98% purity in 30 kg of water).

The geopolymer concrete must be wet-mixed at least for four minutes and steam-cured at 60oC for 24 hours after casting. The workability of fresh geopolymer concrete is expected to be moderate. If needed, commercially available super plasticizer of about 1.5% of mass of fly ash, i.e. 408x (1.5/100) = 6 kg/m3 may be added to the mixture to facilitate ease of placement of fresh concrete.

V Geopolymer Concrete Properties

A. Compressive Strength

Compressive strength is one of the most essential properties of concrete. Anuar et. al, (2011) explained that the higher concentration of sodium hydroxide solution inside the Geopolymer concrete will produce higher compressive strength of ; because NaOH will make the good bonding between aggregate and paste of the concrete.

B.Vijya Rangan et. al, (2004) stated that the compressive strength of Geopolymer concrete is very high compared to the ordinary Portland cement concrete. The compressive strength of Geopolymer concrete is about 1.5 times more than that of the compressive strength with the ordinary Portland cement concrete, for the same mix. Similarly the Geopolymer Concrete showed good workability as of the ordinary Portland Cement Concrete.

C.K. Madheswaran et. Al. (2013) concluded from their experimental study that increasing the molar ratio of NaOH in GPC from 8M to 16M increases the compressive strength.

B. Durability

Rangan, B.V. (2008) stated that Geopolymer concrete is more resistant to heat, sulphate attack, water ingress & alkali-aggregate reaction. The role of calcium in Geopolymer concrete made up of fly ash is very prominent since it may cause flash setting.

Wallah et. al, (2006) Explained that, heat-cured fly ash-based Geopolymer concrete undergoes low creep and very little drying shrinkage in the order of about 100 micro strains after one year. And it has an excellent resistance to sulphate attack. Chanh et al., (2008) stated that fly ash-basedGeopolymer had been proved to provide better resistance against aggressive environment. As such, this advantage can be used to construct structure that exposed to marine environment. Sathia et al., (2008) explained that the exposure of Geopolymer in acid solution shows that the weight loss due to the exposure is only 0.5% compared to normal concrete when immersed in 3% sulphuric acid.N A Lloyd and B V Rangan (2010) concluded that heat-cured, low-calcium fly ash-based Geopolymer concrete is estimated to be about 10 to 30 percent cheaper than that of Portland cement concrete. In addition, the appropriate usage of one tonnne of fly ash earns approximately one carbon-credit which in terms of ecological aspect makes it more economical. One tonnne of low-calcium fly ash can manufacture approximately three cubic meters of high quality fly ash-based Geopolymer concrete. Furthermore, the very little drying shrinkage, the low creep, the excellent resistance to sulfate attack, and good acid resistance offered by the heat-cured low-calcium fly ash-based Geopolymer concrete may yield additional economic benefits when it is utilized in infrastructure applications. Geopolymer concrete has significant advantages over standard concretes. It is much more durable than standard concrete and requires little repair, thus saving huge amounts of money to be spent on repairing and maintaining concrete based infrastructure.

Jamdade P.K et.al (2014) promoted the use of industrial waste fly ash as the replacement for cement. Researchers done experiments on curing time, curing temperature of geopolymer concrete. The compressive strength rises from 12 hrs to 24 hrs at 60°c. The compressive strength is considerably achieved but for the polymerization the temperature is not sufficient. The study shows that, for polymerization the temperature 90°c is quite sufficient. Geopolymer concrete gives more strength than normal concrete in minimum period of curing. Geopolymer concrete has larger compressive strength with higher curing temperature. Increase in the curing temperature beyond 60° c did not increase the compressive strength substantially. As the curing time is increased, it will improve the polymerization and increase the compressive strength.

Krishnan L et.al (2014) conducted studies and concluded that the geopolymer technology is suitable for application in concrete industry as an alternative binder to the Portland cement. Geopolymer binder is prepared using fly ash and GGBS (ground granulated blast furnace slag) with alkaline liquids sodium hydroxide and sodium silicate. Cruz and Gillen reported that the Portland cement paste follows a similar trend expanding to temperatures up to 93 °C and continually contracts thereafter. The thermal incompatibility induces stresses and hence cracking in both geopolymer and OPC concretes damaging the bond between the aggregate and the paste in the concrete. The initial loss in compressive strength of both concretes from ambient temperature to 400 °C is also attributed to the thermal gradient between the surface and core temperatures of concrete cylinder. Kristensen and Hansen reported cracking in concrete due to thermal gradient between 20 and 30 °C over 50mm length. The thermal gradient observed in the cylinders at 200 °C is about 89.5 °C over 20mm. This might generate stresses within the specimens and lead micro cracking within the concrete past research also reported similar reduction in compressive strength of geopolymer paste beyond 520 °C, which is very consistent to that observed in this study.

Rickard et al. proposed that after 600 °C, the shrinkage of paste increases and is believed to be contributed to the reduction of compressive strength of geopolymer concretes at 800 °C. The shrinkage/densification of geopolymer paste is due to viscous sintering of geopolymer matrix filling the voids in the material

VI Economic Benefits of Geopolymer Concrete

Heat-cured low-calcium fly ash-based geopolymer concrete offers several economic benefits over Portland cement concrete. The price of one ton of fly ash is only a small fraction of the price of one ton of Portland cement. Therefore, after allowing for the price of alkaline liquids needed to the make the geopolymer concrete, the price of fly ash-based geopolymer concrete is estimated to be about 10 to 30 percent cheaper than that of Portland cement concrete. In addition, the appropriate usage of one ton of fly ash earns approximately one carbon-credit that has a significant redemption value. One ton low-calcium fly ash can be utilized to manufacture approximately three cubic meters of high quality fly ash-based geopolymer concrete, and hence earn monetary benefits through carbon-credit trade. Furthermore, the very little drying shrinkage, the low creep, the excellent resistance to sulfate attack, and good acid resistance offered by the heat-cured low-calcium fly ash-based geopolymer concrete may yield additional economic benefits when it is utilized in infrastructure applications.

CONCLUDING REMARKS

The paper presented brief details of fly ash-based geopolymer concrete. A simple method to design geopolymer concrete mixtures has been described and illustrated by an example. Geopolymer concrete has excellent properties and is well-suited to manufacture precast concrete products that are needed in rehabilitation and retrofitting of structures after a disaster. The economic benefits and contributions of geopolymer concrete to sustainable development have also outlined. Study shows that Geopolymer concrete is more resistant to corrosion and fire, has high compressive and tensile strengths, and it gains its full strength quickly (cures fully faster). It also shrinks less than standard concrete. Thus, owing to these structural advantages it may be concluded that in near future Geopolymer concrete may find an effective alternate to standard cement concrete. To ensure further uptake of geopolymer technology within the concrete industry, research is needed in the critical area of durability. Current research is focusing on the durability of geopolymer concrete.

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AN EXPERIMENTAL INVESTIGATION ON MECHANICAL PROPERTIES OF SELF-CURING CONCRETE (M20 GRADE)

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Abstract
The aim of this investigation is to study the mechanical properties of concrete using water-soluble Polyethylene Glycol-400(PEG-400) and Polyvinyl Alcohol (PVA) as self-curing agents. The function of self-curing agent is to reduce the water evaporation from concrete, and hence they increase the water retention capacity of concrete compared to the controlled concrete. The use of self-curing agents is very important from the point of view that saving of water is a necessity every day. The benefit of self -curing admixtures is more significant in desert areas where water is not adequately available. In this investigation two different types of self-curing agents were used i.e., Polyethylene Glycol-400and Polyvinyl Alcohol with different proportions 0.6%, 0.8%, 1%, 1.2% and 1.4% by weight of cement. The use of Polyethylene Glycol-400 and Polyvinyl Alcohol in concrete as an admixture helps better hydration and hence the strength of concrete increases. They trap the moisture within the structure and prevent it from evaporation which normally occurs due to the hydration process. In order to study the behaviour of self-cured concrete, compressive strength test and split tensile strength tests were conducted on fresh and hardened concrete at the age of 7 days and 28 days, and non-destructive tests were also conducted to know the quality of concrete. From the experimental results it is observed that the compressive strength and split tensile strengths gradually increases with the increase in proportion of self-curing agents, optimum strengths were achieved at the proportion of 1% for Polyethylene Glycol-400 and 0.8% for Polyvinyl Alcohol by weight of cement. Further increase in the proportion of PEG-400 to 1.2% by weight of cement and PVA to 1% by weight of cement, it is observed that a decrease in the strengths takes place. The compressive strengths and split tensile strengths of self-cured concrete were higher than those achieved by the controlled concrete.
Keywords: *Self–Curing, Polyethylene Glycol-400, Polyvinyl Alcohol*

1. INTRODUCTION

Concrete is the basic engineering material used in most of the civil engineering structures. Its popularity as basic building material in construction is because of its economy of use, good durability and ease with which it can be manufactured at site. Concrete like other engineering materials needs to be designed for properties like strength, durability, workability. With advent of new generation admixtures, it is possible to achieve higher grades of concrete with high workability levels economically. Curing is the maintaining of a satisfactory moisture content and temperature in concrete during its early stages so that desired properties (of concrete) may develop.
The concept of self-curing agents is to reduce the water evaporation from concrete and to increase the water retention capacity of the concrete compared to conventional concrete. It was found that water soluble polymers can be used as self-curing agents in concrete.
The actual phenomenon of this technology is that, the water which is used during the mixing of concrete itself is used for its curing in order to maintain the heat of hydration for the prescribed duration to maintain the properties of concrete unaltered. This is also known as internal curing(IC). Self-curing or internal curing is a technique that can be used to provide additional moisture in concrete for more effective hydration of cement and reduced self-desiccation. When concrete is exposed to the environment evaporation of water takes place

and loss of moisture will reduce the initial water cement ratio which will result in the incomplete hydration of the cement and hence lowering the quality of the concrete.

2. SCOPE AND OBJECTIVE

Scope of the study is to identify the effect of Polyethylene Glycol - 400 (PEG) and Polyvinyl Alcohol (PVA) on strength characteristics of self-curing concrete and also to evaluate influence of polyethylene glycol on mechanical properties which are experimentally investigated.
The main objective of the present work is to study the influence of self-curing concrete when incorporated with PEG400 and PVA by testing mechanical characteristic of concrete i.e., compressive strength and split tensile strength by varying the percentage of PEG400 and PVA from 0.6% to 1.4% by weight of cement for M20 grade concrete.

3. DESIGN MIX PROPORTION OF M20 GRADE CONCRETE

The mix was designed according to the code IS-10262:2009 “Guide lines for Concrete Mix Proportioning”. The mix proportion of M20 grade concrete designed is:

Table 1: Mix Proportion obtained as per design mix procedure

WATER	CEMENT	FINE AGGREGATE	COARSE AGGREGATE
210L	467kg/m^3	532kg/m^3	1337kg/m^3
0.45	1	1.13	2.86

Table 2: Final design mix proportion obtained after the trial mixes

WATER	CEMENT	FINE AGGREGATE	COARSE AGGREGATE
160L	355kg/m^3	606kg/m^3	1521kg/m^3
0.45	1	1.70	4.28

The above mix proportion is used for both controlled concrete and self-cured concrete using Polyethylene Glycol-400 (PEG-400) and Polyvinyl Alcohol (PVA).

4. EXPERIMENTAL PROGRAMME

The experimental program consisted of casting and testing specimens for testing the fresh and hardened properties on M20 grade of concrete with PEG-400, PVA as chemical admixture. Indian standard concrete IS 102362:2009 mix proportioning guidelines are adopted for mix design to achieve suitable mix proportions. The mix proportion for M20 grade was achieved, taking the different w/c ratio into consideration. A total of 54 cubes of standard size 150 mm x

150 mm x 150 mm and 54 cylinders of 150 mm diameter and 300 mm height were casted for determining the compressive strength and split tensile strength respectively.

5. MATERIALS USED

The different materials used in this investigation are:
1. ***Cement:*** Cement used in this investigation was 53 grade Ordinary Portland Cement confirming IS-12269:1987
2. ***Fine Aggregate:*** The fine aggregate used was obtained from a nearby river source confirming to IS-383:1970
3. ***Coarse Aggregate:*** The coarse aggregate according to IS-383:1970 was used. Maximum size of coarse aggregate used is 12.5mm.
4. ***Water:*** Portable water used in the experimental work for both mixing and curing purposes.
5. ***Polyethylene Glycol – 400:*** Polyethylene glycol is a condensation polymer of ethylene oxide and water with general formula $H(OCH_2CH_2)n(OH)$, where n is the average number of repeating ox ethylene groups.
6. ***Polyvinyl Alcohol:*** Polyvinyl Alcohol (PVOH, PVA, or PVA l) is a water-soluble synthetic polymer. It has the idealized formula $[CH_2CH(OH)]_n$.

6. EXPERIMENTAL RESULTS

6.1 The physical properties of the materials that are tested in the laboratory are as mentioned in the below tables:

Table 3: Physical properties of Ordinary Portland Cement

S. No	Property	Test Method	Test Results	IS Standard
1.	Normal Consistency	Vicat Apparatus (IS: 4031 Part - 4)	31%	26-33%
2.	Specific gravity	Sp.gr bottle (IS:2720 Part – 3)	3.12	3.15
3.	Initial setting time Final setting time	Vicat Apparatus (IS: 4031 Part - 4)	45 minutes 182 Minutes	Not less than 30 minutes Not more than 10 hours
4.	Fineness	Sieve test on sieve no.9 (IS: 4031 Part – 1)	1.3%	10%
5.	Soundness	Le-Chatlier method (IS: 4031 Part – 3)	2 mm	Not more than 10 mm
6.	Compressive Strength	Compressive Testing Machine IS:4031 (Part 6) : 1988	55.3 N/mm^2	53.0N/mm^2

Table 4: Physical Properties of Fine aggregate and Coarse Aggregate

S. No	Property	Method	Fine Aggregate	Coarse Aggregate
1.	Specific gravity	Pycnometer IS:2386 Part 3-1986	2.60	3.21
2.	Bulk Density Loose	IS:2386 Part 3-1986	1545.69 kg/m^3	1400 kg/m^3
3.	Bulking	IS:2386 Part 3-1986	4% w c	-
4.	Fineness Modulus	Sieve Analysis (IS:2386 Part 2-1963)	2.64	6.04

Table 5: Physical Properties of Polyethylene Glycol-400

S.NO	DESCRIPTION	PROPERTIES
1	Molecular Weight	400
2	Appearance	Clear fluid
3	Moisture	0.2%
4	P^H	6
5	Specific Gravity	1.12

Table 6: Physical Properties of Polyvinyl Alcohol

S.NO	DESCRIPTION	PROPERTIES
1	Molecular Weight	13000-23000g/mol
2	Appearance	Crystalline powder
3	Melting Point	200ºC
4	Boiling Point	228ºC
5	Density	1.19-1.31g/cm^3

6.2 Destructive Test Results of fresh and hardened concrete:

6.2.1Compressive Strength: The cube specimens of size 150mmx150mmx150mm has been casted and tested for 7 Days and 28 Days, and the results are furnished in the table 7 and represented in the figure 1 and 2.

Table 7: Compressive Strength results

S.NO	DAYS	CONTROLLED CONCRETE (N/mm^2)	SELF-CURING CONCRETE PEG-400 (N/mm^2)				
			0.6%	0.8%	1%	1.2%	1.4%
1	7	19	20	21.66	**22.89**	20.74	18
2	28	27	26.30	27.67	**29.56**	28.20	24

S.NO	DAYS	CONTROLLED CONCRETE (N/mm^2)	SELF-CURING CONCRETE PVA (N/mm^2)		
			0.6%	0.8%	1%
1	7	19	19.72	**20.5**	19.2
2	28	27	25.65	**28**	27

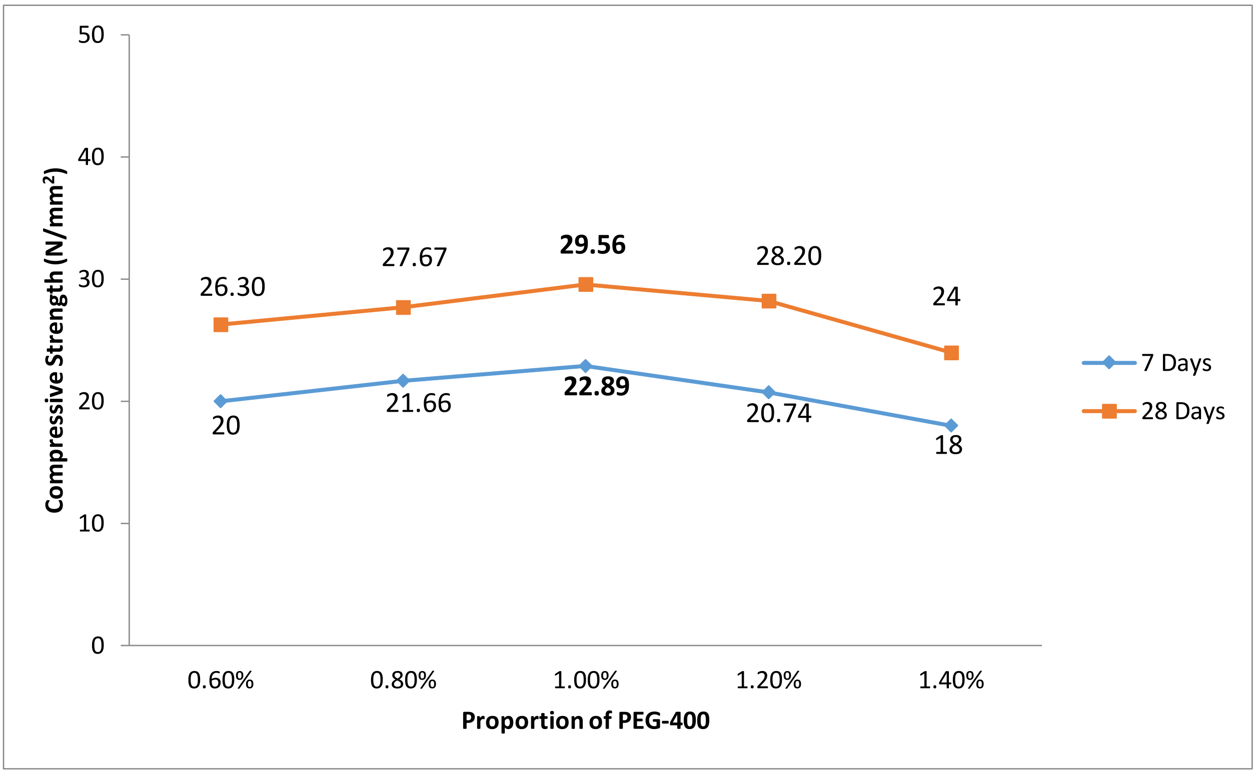

Figure 1: Compressive strength of PEG-400 for 7days & 28days

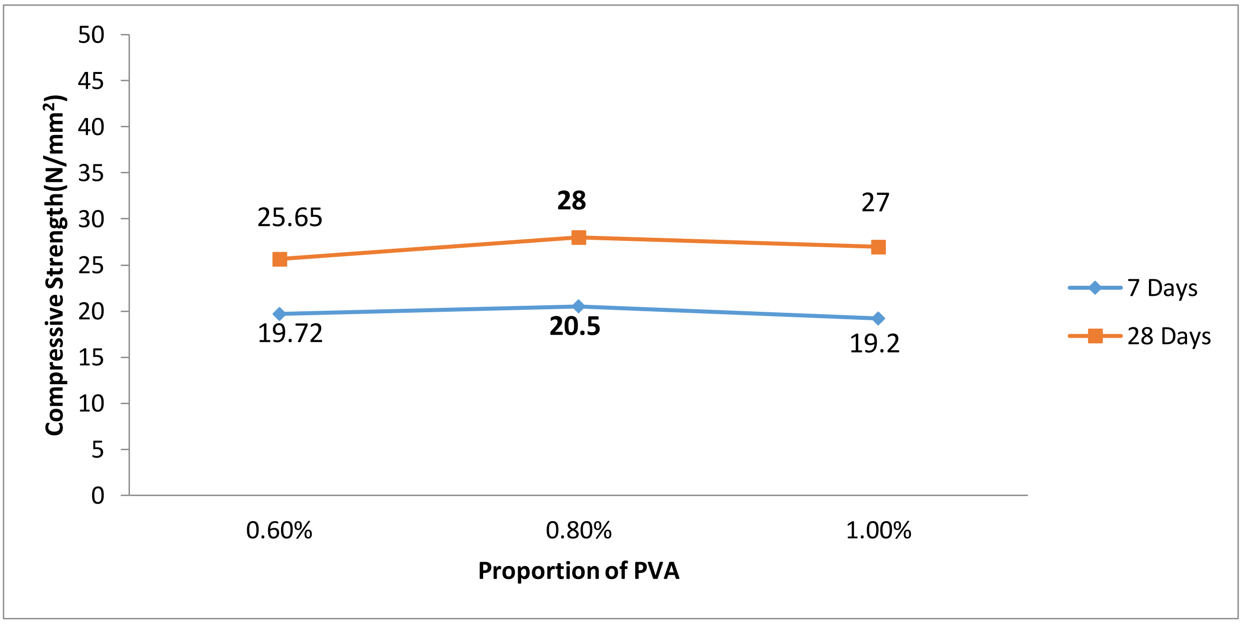

Figure 2: Compressive strength of PVA for 7days & 28days

6.2.2 Split Tensile Strength: The cylinders of size 150mm diameter and 300mm height has been casted and tested for 7 Days and 28 Days, and the results are furnished in the table 8 and represented in the figure 3 and 4.

Table 8: Split Tensile Strength results

S.NO	**DAYS**	**CONTROLLED CONCRETE (N/mm^2)**	**SELF-CURING CONCRETE PEG-400 (N/mm^2)**				
			0.6%	0.8%	1%	1.2%	1.4%
1	7	3.05	3.13	3.25	**3.34**	3.18	2.96
2	28	3.63	3.58	3.68	**3.80**	3.71	3.42

S.NO	**DAYS**	**CONTROLLED CONCRETE (N/mm^2)**	**SELF-CURING CONCRETE PVA (N/mm^2)**		
			0.6%	0.8%	1%
1	7	3.05	3.10	**3.16**	3.06
2	28	3.63	3.54	**3.70**	3.63

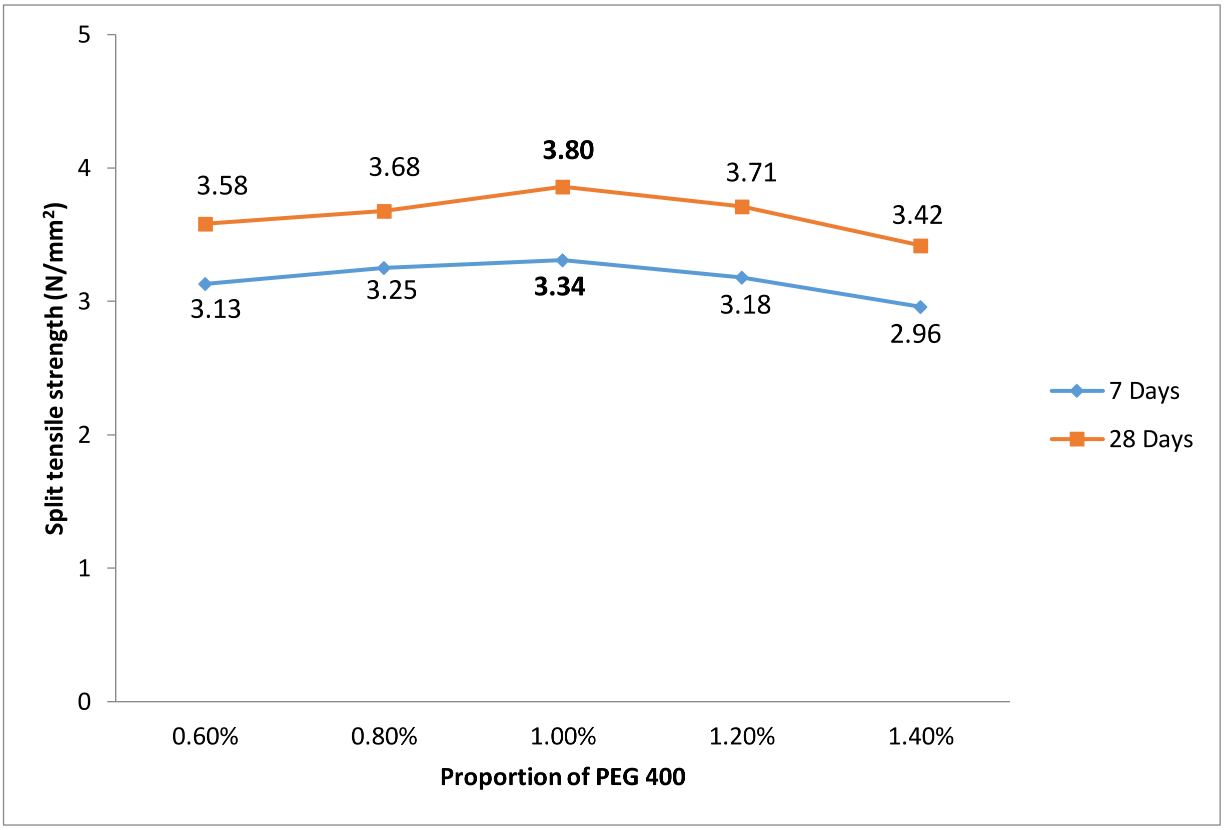

Figure 3: Split Tensile strength of PEG-400 for 7days & 28days

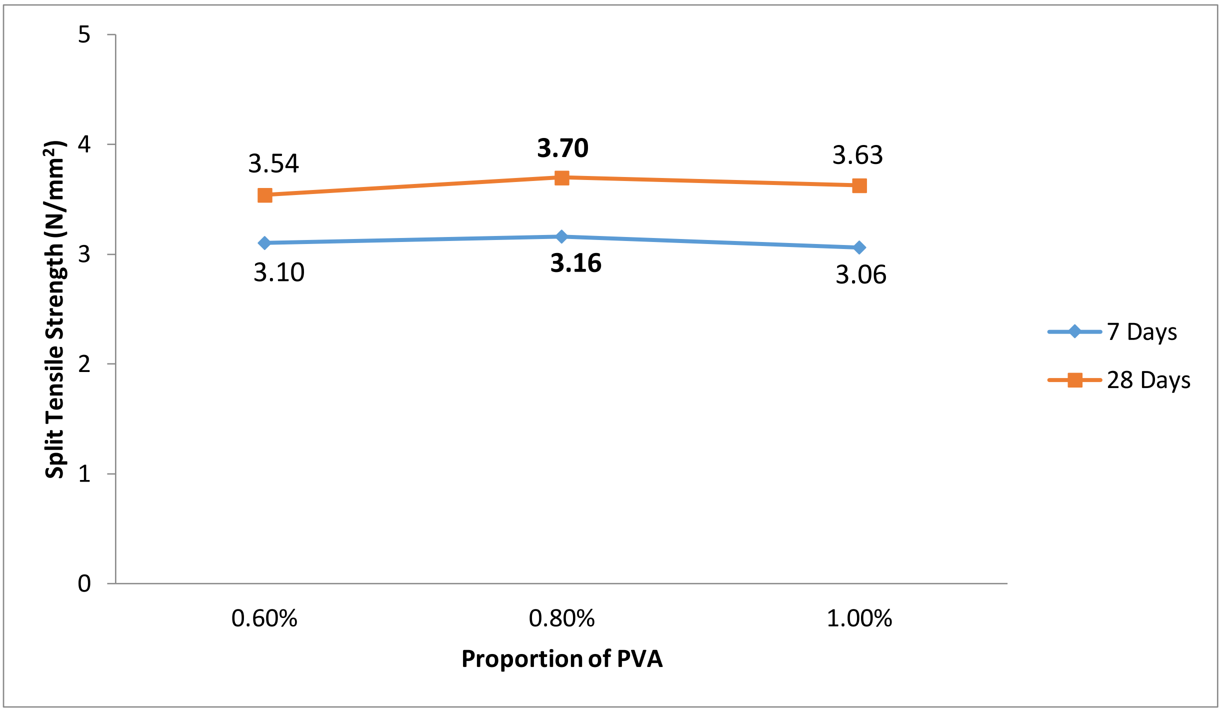

Figure 4: Split Tensile strength of PVA for 7days & 28days

6.3 Non-Destructive Test results of fresh and hardened concrete:

The Rebound Hammer and Ultra Sonic Pulse Velocity tests are done to check the compressive Strength and quality of concrete.

Rebound Hammer Test:

Table 9 Showing Rebound Hammer test results for various specimens

SPECIMENS	REBOUND NUMBER	COMPRESSIVE STRENGTH
Controlled Concrete	33	28.5
PEG-400	35	31.2
PVA	34	30

Ultra Sonic Pulse Velocity Test:

SPECIMENS	MEAN PULSE VELOCITY (km/sec)	QUALITY OF CONCRETE
Controlled Concrete	3.89	Good to very good
PEG-400	4.38	Very good to excellent
PVA	3.90	Good to very good

7. DISCUSSIONS ON TEST RESULTS

From the experimental results it is observed that, initially with the usage of Poly ethylene glycol-400 (PEG-400) the compressive strength has increased from 98.87% of target mean strength to 104% of target mean strength with the increase in the proportion of PEG-400 from 0.6% to 0.8% by weight of cement for 28days, 104% of target mean strength to 111.12% of target mean strength of increment in the compressive strength with the increment in the proportion of PEG-400 from 0.8% to 1% by weight of cement for 28days, and a sudden decrease in the compressive strength from 111.12% of target mean strength to 106% of target mean strength with the increase in the proportion of PEG-400 from 1% to 1.2% and there is a further decrement in strength 106% of target mean strength to 90.22% of target mean strength with further increase in the PEG-400 proportion i.e., from 1.2% to 1.4% by weight of cement for 28days. The desired strength of PEG-400 is achieved at 1 % by weight of cement. Further increase in the dosage of PEG-400 is leading to the decrease in the strength.

The split tensile strength has been increased about 0.0017% for increase in the proportion of PEG-400 from 0.6% to 1%, further it is observed that split tensile strength is decreased about 0.0038% for increase in the proportion of PEG-400 from 1% to 1.4%. The sudden decrement in the compressive strength and split tensile strength is due to the increase in the workability of concrete, which is due to the increase in the proportion of PEG-400. Comparing to the varying percentages of PEG-400 the compressive and split tensile strength test result shows that the optimum strength of self-curing concrete attained at 1% of PEG-400 by weight of cement. Further increase in the dosage is leading to the decrease in the strengths. With this comparison it is clear that the internal curing is been carried out and heat of hydration is maintained continuously, which didn't altered the properties of the self-curing concrete when compared with the normal curing concrete.

In the case of Poly vinyl Alcohol (PVA), a similar trend of behavior has been observed in the experiments that are conducted on fresh and hardened concrete. The compressive strength is increased from 96.42% of target mean strength to 105.26% of target mean strength with the increase in the proportion of PVA from 0.6% to 0.8% by weight of cement for 28days, and a sudden decrease in strength from 105.26% of target mean strength to 101.50% of target mean strength with further increase in the PVA proportion i.e., from 0.8% to 1% by weight of cement for 28 days. The desired strength of PVA is achieved at 0.8 % by weight of cement. Further increase in the dosage of PVA is leading to the decrease in the strength.

The split tensile strength has been increased about 0.0007% for increase in the proportion of PVA from 0.6% to 0.8% by weight of cement for 28days, further it is observed that split tensile strength is decreased about 0.0007% for increase in the proportion of PEG-400 from 0.8% to 1% by weight of cement for 28days. The desired strength of PVA is achieved at 0.8 % by weight of cement. Further increase in the dosage of PVA is leading to the decrease in the strength. The sudden decrease in compressive strength and split tensile strength is due to the increase in the workability of concrete, which is due to the increase in the proportion of self-curing agent PVA. It is clear that the heat of hydration and internal curing is carried which helped in achieving the strengths at respective age at a particular proportion of self-curing agent and further increase in the dosage is leading to the decrease in the strengths.

8. CONCLUSIONS

1. The self-curing agent Polyethylene Glycol-400 is used at different percentages (0.6, 0.8, 1, 1.2 and 1.4) with M20 grade concrete mix. It shows that the compressive strength of concrete is found to be increased by 12.25% by increasing the proportion of Polyethylene Glycol-400 from 0.6% to 1% by weight of cement for 28 days. And there is decrease in compressive strength by 5.12% by increasing the proportion of Polyethylene Glycol-400 from 1% to 1.2% by weight of cement for 28 days.
2. The split tensile strength of concrete is found to be increased by 0.0022% by increasing the proportion of Polyethylene Glycol-400 from 0.6% to 1% by weight of cement for 28days. And there is decrease in split tensile strength by 0.0009% by increasing the proportion of Polyethylene Glycol-400 from 1% to 1.2% by weight of cement for 28 days.
3. The self-curing agent Polyvinyl Alcohol is used at different percentages (0.6, 0.8, 1, 1.2 and 1.4) with M20 grade concrete mix. It shows that the compressive strength of concrete is found to be increased by 8.84% by increasing the proportion of Polyethylene Glycol-400 from 0.6% to 0.8% by weight of cement for 28days. And there is decrease compressive strength by 3.76% by increasing the proportion of Polyvinyl Alcohol from 0.8% to 1% by weight of cement for 28 days.
4. The split tensile strength of concrete is found to be increased by 0.0016% by increasing the proportion of Polyvinyl Alcohol from 0.6% to 0.8% by weight of cement for 28days. And there is decrease in split tensile strength by 0.0007% by increasing the proportion of Polyvinyl Alcohol from 0.8% to 1% by weight of cement for 28 days.
5. The Non-Destructive tests were done and the quality of concrete based on average Rebound Hammer is found to be a “Good layer”. The quality of concrete based on Ultra Sonic Pulse Velocity is found to be a “Very good concrete”.
6. It has been observed that cubes and cylinders casted with Self-cured concrete containing Polyethylene Glycol-400 and Polyvinyl Alcohol shows less cracks when compared to controlled concrete.

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STUDIES ON PERMEABILITY AND SORPTIVITY OF LOW CALCIUM FLYASH AND SLAG BASED GEOPOLYMER CONCRETE

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Abstract

Geopolymer concrete is an environment friendly concrete which has lower carbon footprint as compared to that of conventional concrete. The objective of the present work is to find the durability in terms of permeability and sorptivity of the low calcium flyash and slag based geopolymer concrete. Alkaline solution of sodium silicate (Na_2Sio_3) is added which is pre-mixed with NaoH cristals for polymerisation process. The specimens were tested for permeability, sorptivity and also compared the results with controlled concrete of respective grades. The satisfactory results have come, since there was slightly reduction in permeability and sorptivity coefficients in case of geopolymer concrete, when compared with the respective grades of controlled concrete.

***Keywords**: Geopolymer concrete, Sorptivity, Permeability, Percolation, capillarity.*

1.0 Introduction

Concrete is one of the most widely used construction materials; it is usually associated with Portland cement as the main component for making concrete. The demand for concrete as a construction material is on the increase. On the other hand, the climate change due to global warming, one of the greatest environmental issues has become a major concern during the last decade. The global warming is caused by the emission of greenhouse gases, such as CO_2, to the atmosphere by human activities. Among the greenhouse gases, CO_2 contributes about 65% of global warming. The cement industry is responsible for about 6% of all CO_2 emissions, because the production of one ton of Portland cement emits approximately one ton of CO_2 into the atmosphere. Although the use of Portland cement is still unavoidable until the foreseeable future, many efforts are being made to reduce the use of Portland cement in concrete. These efforts include the utilisation of supplementary cementing materials such as fly ash, silica fume, granulated blast furnace slag, rice-husk ash and metakaolin, and finding alternative binders to Portland cement.

2.0 Project Significance

Geopolymer concrete is an environment friendly concrete which has lower carbon footprint as compared to that of conventional concrete. Fly ash (Class F) is a byproduct of coal obtained from thermal power plant. It is also rich in silica and alumina. In this paper, fly ash of F class is used to produce a geopolymer concrete. Geopolymer is a material resulting from the reaction of a source material that is rich in silica and alumina with alkaline solution. Geopolymer concrete is totally cement free concrete. Fly-ash and alkaline activator undergo geo-polymerization process to produce alumino silicate gel.

3.0 Objectives of the present work

The main objective of the present work is to find the durability in terms of permeability and sorptivity of the low calcium flyash and slag based geopolymer concrete of G30 & G50 grades and which are to be compared with M30 & M50 grades of controlled concrete.

4.0 Experimental investigations

Experimental investigations are carried out as follows:

4.1 To determine the concrete water permeability as per IS 3085:1965.
4.2 To find out the water absorption rate (sorptivity test) of geopolymer concrete specimens to estimate the volume of permeable voids and their interconnected pore space distribution.

5.0 Materials and Mix Proportions

The materials used for concrete are briefly reviewed in the following sections:

5.1 Ordinary Portland cement of 53 grade, confirming to IS: 12269-1987 used in this investigation.
5.2 Locally available clean, well-graded, natural river sand having fineness modulus of 2.65 conforming to Zone II of IS 383-1970 was used as fine aggregate. Crushed granite angular aggregate of size 20 mm nominal size from local source with specific gravity of 2.71 was used as coarse aggregate.
5.3 Class F-fly ash is used, which is obtained from Vijayawada thermal power station in Andhra Pradesh conforming to IS: 3812-part 1 2003.
5.4 Ground Granulated Blast Furnace Slag (GGBS) conforming to IS : 12089- 1987
5.5 High range water reducing Master Glenium Sky 8233 (Formerly Glenium B233) super plasticizer was used for concrete conforming IS: 9103-1999.
5.6 Water used for mixing and curing is fresh potable water, confirming to IS: 3025 – 1986 and IS: 456 – 2000.
5.7 Alkaline liquids such as NaOH and Na_2SiO_3 are used.
5.8 The grades of concrete used in the present investigation are standard grades (M30 & M50) of controlled concrete and equivalent grades (G30 & G50) of geopolymer concrete. The M30 and M50 mix proportions are designed using BIS method where as G30 and G50 mix proportions are developed by using the same proportions of controlled concrete by replacing 100% cement with 85% fly ash and 15% with GGBS and in addition to these alkaline liquids have been added based on trial and error to develop the same strength as that of controlled concrete.

The mix proportions are as follows:

Standard grade (M30) of Controlled Concrete	1:1.89:3.27:0.45
Standard grade (G30) of Geopolymer Concrete	1:1.89:3.27:0.45
Standard grade (M50) of Controlled Concrete	1:1.35:3.16:0.4
Standard grade (G50) of Geopolymer Concrete	1:1.35:3.16:0.4

6.0 Test Results and Discussions

Results of the experimental investigations conducted are presented below:

6.1 Studies on Water Permeability

The tables 1 and 2 and figure 1 and 2 presents the coefficients of permeability values determined as per IS 3085 for both grades of controlled and geopolymer concrete specimens of age 28 and 90 days.

Table 1: Coefficients of Permeability for Controlled and Geopolymer Concrete Specimens of Age 28 days

Grade of Concrete	Type of specimen	Pressure head H (m)	Quantity of water collected (ml)	Coefficient of permeability x 10^{-9} m/sec	% Decreased
M 30	Controlled	100	8688	2.04	-
G 30	Geopolymer	100	8266	1.95	4.86
M 50	Controlled	100	5574	1.31	-
G 50	Geopolymer	100	5356	1.26	3.91

Table 2: Coefficients of Permeability for Controlled and Geopolymer Concrete Specimens of Age 90 days

Grade of Concrete	Type of specimen	Pressure head H (m)	Quantity of water collected (ml)	Coefficient of permeability x 10^{-9} m/sec	% Decreased
M 30	Controlled	100	8336	1.97	-
G 30	Geopolymer	100	8078	1.90	3.09
M 50	Controlled	100	5448	1.28	-
G 50	Geopolymer	100	5274	1.24	3.19

Table 3: Coefficient of Water Permeability Ranges as per IS: 3085-1965

Water Permeability	Very Low	Low	Medium	High
Coefficient of permeability (x 10^{-9} m/sec)	< 0.5	0.5-1.0	1.0-2.0	>2.0

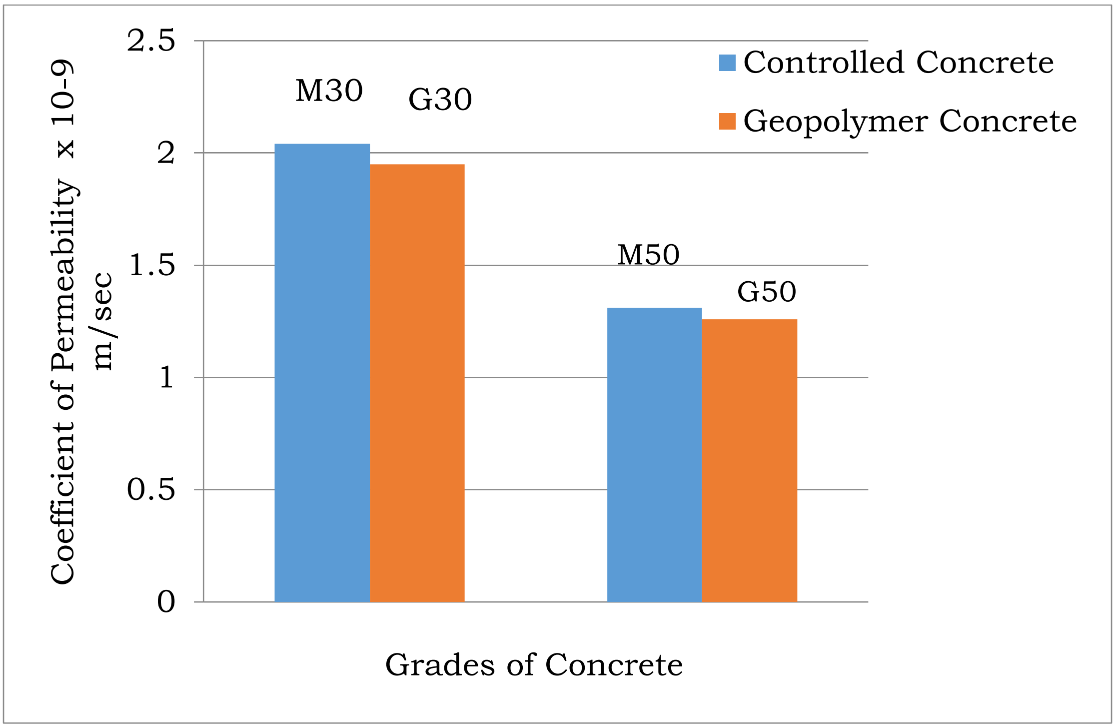

Fig 1: Variation of Coefficients of Permeability for Controlled and Geopolymer Concrete Specimens of Age 28 days

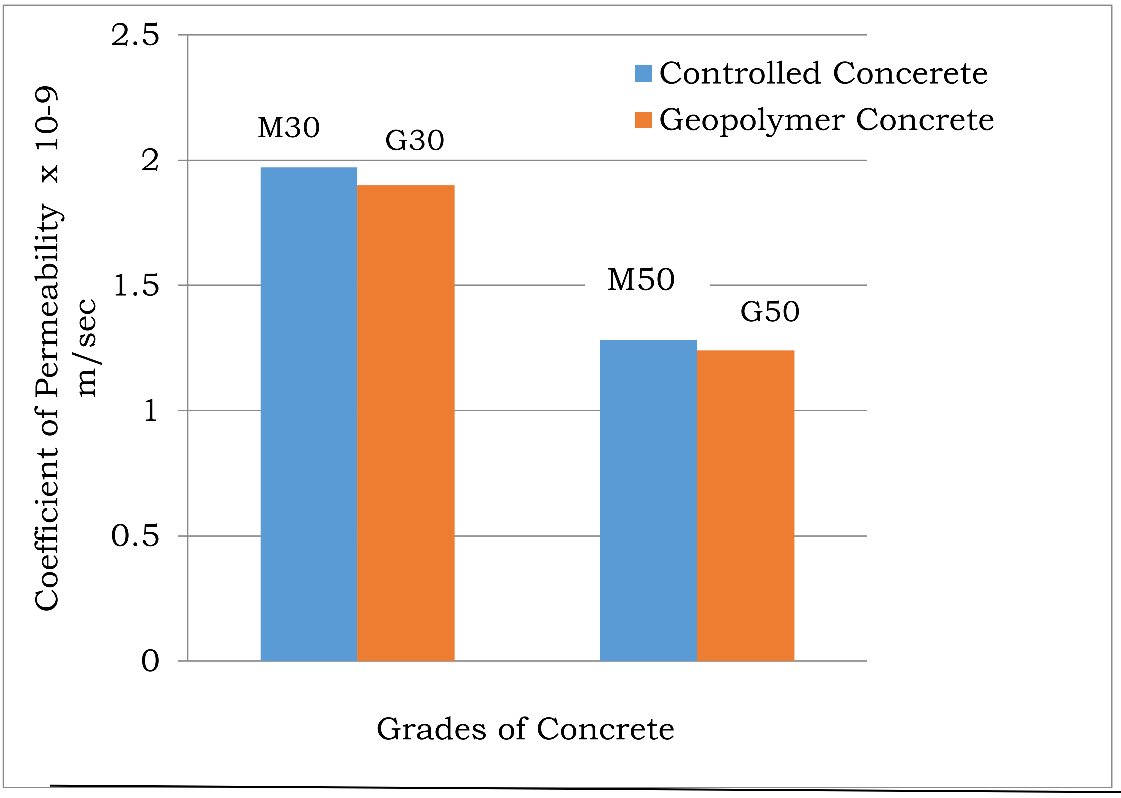

Fig 2: Variation of Coefficients of Permeability for Controlled and Geopolymer Concrete Specimens of Age 90 days

From the presented test results, it was observed that negligible decrease in water permeability in geopolymer specimens than controlled specimens under 100m water head. Results showed that the geopolymer concrete resulted in little lower coefficient of permeability of range 1.26 – 1.95 x 10^{-9} m/sec in comparison to controlled concrete which has coefficient of permeability of 1.28 – 2.04 x 10^{-9} m/sec for both grades of concretes. Decrease in water permeability of specimens in geopolymer is nearly 4.86% and 3.91% in G30 and G50 grade concretes respectively of age 28days. Similarly decrease in water permeability of specimens in geopolymer concrete is nearly 3.09% and 3.19% in G30 and G50 grades of concrete respectively at 90days age. It shows that both grades of geopolymer concretes are little less permeable than the controlled concretes.

6.2 Studies on Sorptivity

The table 4 illustrates the gain in mass per unit area over the density of water 'I' (m) at regular intervals of time 't' (min) and figure 3 shows plot between the gain in mass per unit area over the density of water (I) and the square root of the elapsed time (√t). The slope of the line of best fit of these points is reported as the sorptivity coefficient (k). Table 5 and Figure 4 shows the sorptivity coefficient (k) of controlled (M30 & M50) and geopolymer (G30 & G50) concretes.

Table 4: The Gain in Mass per Unit Area over the Density of Water 'I' (m) at Regular Intervals of Time 't' (min)

Controlled concrete			
M30		M50	
I x 10^{-3} (m)	t (min)	I x 10^{-3} (m)	t (min)
0	0	0	0
0.00045	15	0.00035	15
0.0006	30	0.00045	30
0.00085	60	0.0006	60
0.0012	120	0.0009	120
0.00165	240	0.00125	240
0.0021	360	0.0015	360
0.0041	1440	0.0031	1440
0.0058	2880	0.00435	2880
0.0071	4320	0.00515	4320

Geopolymer Concrete			
G30		G50	
I x 10^{-3} (m)	t (min)	I x 10^{-3} (m)	t(min)
0	0	0	0
0.00035	15	0.00025	15
0.00045	30	0.00035	30
0.00065	60	0.0005	60
0.0009	120	0.00075	120
0.00125	240	0.00105	240
0.00155	360	0.00125	360
0.00315	1440	0.0025	1440
0.00435	2880	0.0035	2880
0.00555	4320	0.0044	4320

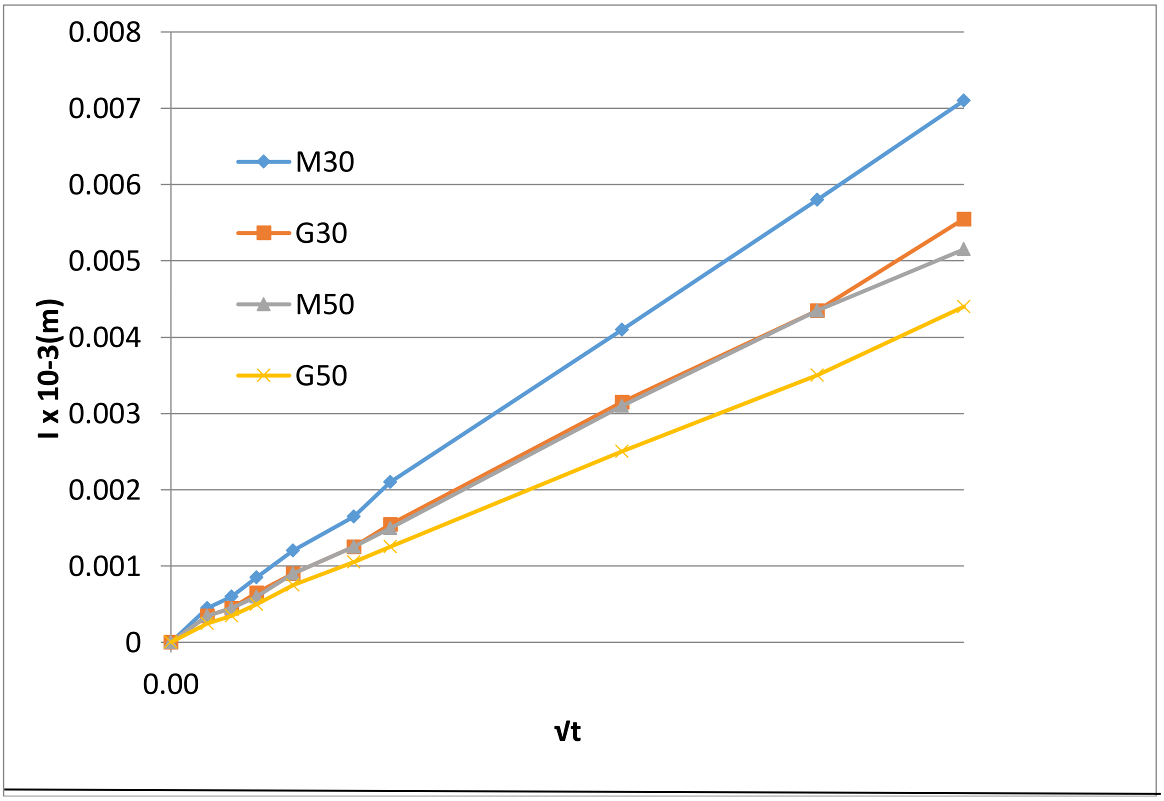

Fig 3: Plot between I- √t to calculate Sorptivity Coefficient (k)

Table 5: Sorptivity Coefficients of both grades of Controlled and Geopolymer Concrete Specimens

	Grade of the Concrete	Sorptivity Coefficient (k) x 10^{-3} m/min$^{0.5}$	Percentage Decreased
Controlled Concrete	M30	0.108	-
	M50	0.081	-
Geopolymer Concrete	G30	0.081	25
	G50	0.067	17.28

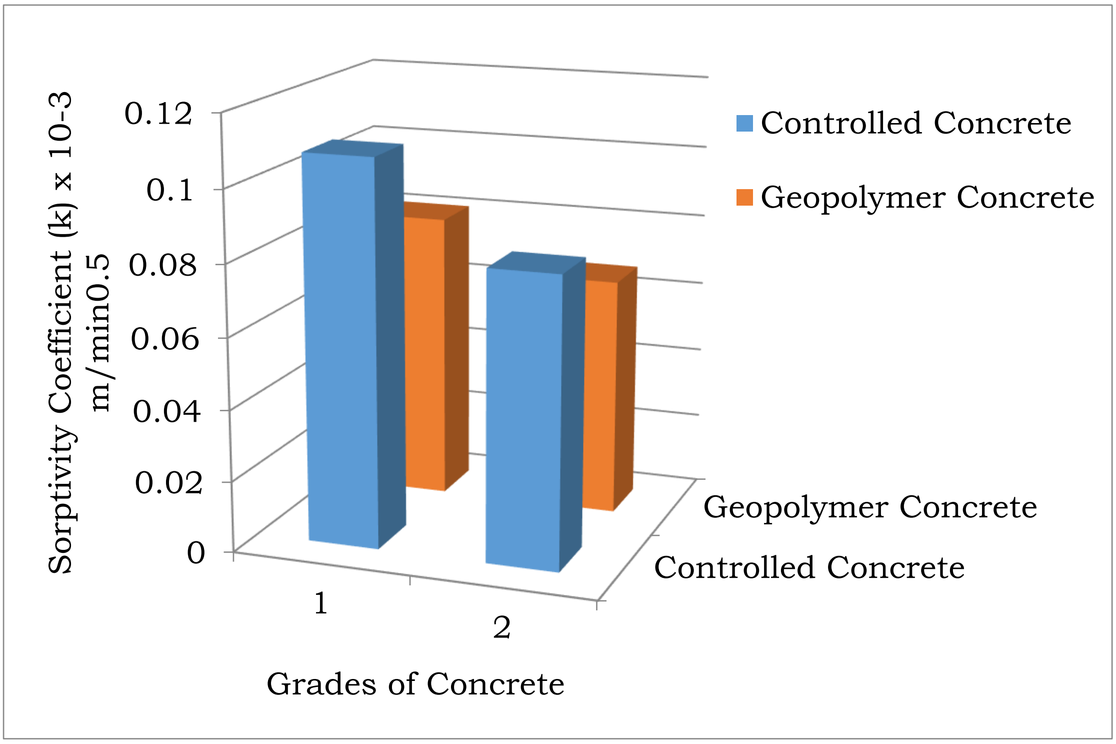

Fig 4: Variation of Sorptivity of both grades of Controlled and Geopolymer Concrete

From table 4 it is apparent that sorptivity decreases systematically for geopolymer concrete specimens for both the grades. The sorptivity coefficients of geopolymer concrete specimens are low for both grades when compared with corresponding grades of controlled concrete specimens because the pores in the bulk paste or in the interfaces between aggregate and cement paste is filled by the dense structure hence, the capillary pores are reduced. Sorptivity values for geopolymer concrete were in the range of 0.081 to 0.067 mm/min$^{0.5}$ for the grades of G30 & G50 and for controlled concretes its value are in the range of 0.108 to 0.081 mm/min$^{0.5}$ of the equivalent grades respectively. The capillary absorption coefficient (k) is greatly influenced by the addition of microorganisms to the concrete. The reduction in capillary in geopolymer concrete specimens is from 17 to 25 %. The water absorption, capillary and porosity characteristics indirectly reflect the durability performance of the geopolymer concrete.

7.0 Conclusions

Based on the results reported in this research work and key findings during the experimental investigations, the following conclusions can be drawn:

1) It is observed that the permeability is slightly less in case of geopolymer concrete when it is compared with conventional concrete for both the grades. So, geopolymer concrete is more preferred to conventional concrete for particular area where low permeability is more essential.
2) The sorptivity of geopolymer concrete is slightly less than conventional concrete for G30 and G50 grades when it is compared with corresponding grades (M30 & M50), because the pore sizes in geopolymer concrete is less due dense structure.
3) The attainment of early strengths are possible in an oven curing at $60^{O}C$ in case of geopolymer concrete when compared to conventional concrete, so it can be used wherever early strengths are required.

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A CASE STUDY PARTIAL REPLACEMENT OF CEMENT WITH MARBLE POWDER IN CONCRETE MIX

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ABSTRACT

The waste materials to the environment directly can cause environmental Hazardous. Hence the reuse of waste material has been emphasized. Waste can be used to produce new products or can be used as admixtures so that natural resources are used more efficiently and the environment is protected from waste deposits. Marble stone industry generates both solid waste and stone slurry. Whereas solid waste results from the rejects at the mine sites or at the processing units, stone slurry is a semi liquid substance consisting of particles originating from the sawing and the polishing processes and water used to cool and lubricate the sawing and polishing machines. Presently large amount of marble dust is generated in stone processing plant. So this project aims to test the strength by replacing it with cement in concrete.

1.0 GENERAL: The waste materials are gaining attention to use the materials as a substitute to natural getting them utilized in cement, concrete, and other construction materials, it helps in reducing the cost of cement

.**1.1 CEMENT:** It is the most important and costliest ingredient of concrete. The mix-design of concrete indirectly means optimising the use of cement for obtaining the desired properties of concrete in green as well as hardened state. Ordinary portland cement (opc) OPC category we have three grades of cement available in Indian market Grade 33,Grade 43,Grade 53

1.2 AGGREGATES: Aggregates are the important constituents in concrete. They give body to the concrete, reduce shrinkage and effect economy. the aggregates occupy 70–80 per cent of the volume of concrete, their impact on concrete is undoubtedly considerable.

1.2.1 CLASSIFICATION: Aggregates can be classified as (i) Normal weight aggregates, (ii) Light weight aggregates and (iii) Heavy weight aggregates. The size of aggregate bigger than 4.75 mm is considered as coarse aggregate and aggregate whose size is 4.75 mm and less is considered as fine aggregate.

1.3 WATER: It is indispensable because it is required for reaction of hydration. But its use should be restricted to minimum as possible considering the requirement for chemical reaction with cement and workability only.

1.4 About Marble Powder

Recycling of industrial wastes has actually environmental, economical and technical benefits. For the producer, the environmental benefit can be attained as far as the waste is recycled. It is estimated that 1 tone clinker production releases 1 tone CO_2. Mixing of clinker to supplementary materials called blending is considered as a very effective way to reduce CO_2 emission. It is estimated that the Rajasthan Marble Processing Enterprise produces $1800m^3$ (4500 tons) marble waste annually

3.0 Literature Review

Er. Raj P. Singh Kushwah , Prof. (Dr.), Ishwar Chand Sharma, Prof (Dr.) PBL Chaurasia(2015) presented in his paper that the marble can be utilized in concrete mix by replacement of fine aggregates. **Bahar Demirel (2010)** presented the use of marble dust as in place of fine aggregate in concrete ix and check the mechanical properties of mix. **Baboo Hassan A. Mohamadien (2012)** investigates the effect of marble powder and silica fume of different percentages as partial replacement for cement on mortar. **Noha M. Soliman (2013)** presented the effect of using Marble Powder in Concrete Mixes and also its effect on the strength of R.C. Slabs**V. M. Sounthararajan and A. Sivakumar (2013)** add lime content in marble powder and check its effects on concrete mix.**Animesh Mishra, Abhishek Pandey, Prateek Maheshwari, Abhishek Chouhan, S. Suresh, Shaktinath Das (2013)** uses marble dust in green cement for sustainable concrete. The compressive strength and mechanical properties were investigated in this study. **Prof. Veena G. Pathan, Prof. Md. Gulfam Pathan** studied feasibility of marble powder as the constituent of concrete mix.

4.0 OBJECTIVE

- Main objective is to study the influence of partial replacement of cement with marble powder,
- Compare it with the compressive and tensile strength of ordinary M20 concrete.
- Find the percentage of marble powder replaced in concrete
- Reduces the cement content which results in cost reduction.
- TO improving durability concrete
- To attaining and maintaining strength at specific days of concrete

5.0 Advantages Of Marble Powder

• It Intensifies The Properties Of Mortar And Concrete.
• Huge Amount Of Marble Waste Is Generated And So Cost Of Procuring Of This Waste Is Low.

5.1 Disadvantages Of Marble Powder

• Marble does not take part in the hydration process.
• Marble waste is not available in all regions and so additional transportation cost will be added for its procurement

6.0 Methodology

Cement was replaced with replacements levels of cement (0%, 5%, 10%, 15% and 20%). Six numbers of cubes of 150 X 150 X 150 and Four number of cylinders 150 mm diameter and 300 mm length were casted for each %age replacement. Hence 50 numbers of cubes were casted for each compressive and split tensile strength. The compressive strength and split tensile strength of concrete of all mixes was determined at the ages of 7 and 28 days of curing for the various replacement level of cement and addition of cement and addition of waste marble powder (0%,5%, 10%, 15%, 20%) at the end of different curing periods.
Slump and air content of fresh concrete and absorption and compressive strength of hardened concrete were also investigated. Test results show that this industrial bi product is capable of improving hardened concrete performance up to 10%, Enhancing fresh concrete behavior and can be used in architectural concrete mixtures containing white cement

Figure 1 Marble Dust Powder

7.0 TESTING ON PHYSICAL PROPERTIES OF MATERIALS

Initial setting time of Cement= 84 minutes
Fineness Modulus of Cement= 6%
Specific gravity of Cement = 3.14 (USING LE – CHATLIER FLASK)
SPECIFIC GRAVITY OF FINE AGGREGATE-

Table: 1. specific gravity of sand

SL NO	Determination	Gm
1	Pycnometer (M1)	458.10
2	Pycnometer + sand (half of bottle) (M2)	676
3	Pycnometer + sand + full of water (M3)	1390
4	Pycnometer + full of water (M4)	1253

8.0 CALCULATIONS

Specific gravity = [(M2-M1)]/[(M2-M1)-(M3-M4)]
a). (676-458.10)/ [(676-458.10)-(1390-1253)] = 2.693
b) (697-451)/ [(697-451)-(1405-1258)] = 2.49
Therefore, specific gravity of fine aggregate = 2.59

Table 2 Specific Gravity Of Course Aggregate

SL NO	Determination	Gm
1	Pycnometer (M1)	458
2	Pycnometer + aggregate (M2)	706
3	Pycnometer + aggregate + full of water (M3)	1405
4	Pycnometer + full of water (M4)	1258

8.1 CALCULATIONS

Specific gravity = (M2-M1)/[(M2-M1)-(M3-M4)]
(a) (706-458)/ [(706-458)-(1405-1258)] = 2.45
(b) (666-463) /[(706-458)-(1405-1258)]= 2.859
Mean of these = 2.66
Therefore, specific gravity of coarse aggregate = 2.66

8.2 SIEVE ANALYSIS

(A) SAND

Quantity of sand = 1 Kg Time of sieving = 15 minutes

Sieve analysis of fine aggregate

Fineness modulus = 304.2 / 100 = 3.042%

COARSE AGGREGATE

Quantity of materials = 4 kg
Time of sieving = 15 minutes

Sieve analysis of coarse aggregate

Fineness modulus = 710.77/100 = 7.10%

Table 3 Sieve analysis of fine aggregate

Sl No	Sieve Size	Weight Retained(Gm)	% Of Weight Retained	Cumulative % Retained	Cumulative % Passing
1	40 mm	0	0	0	100
	20 mm	585.0	14.625	14.65	85.375
3	10 mm	3260	81.5	96.12	3.88
4	4.75 mm	155	3.875	100	0
5	2.36 mm	0	9.8	100	0
6	1.18 mm	0	6.0	100	0
7	600 micron	0	18.8	100	0
8	300 micron	0	39.7	100	0
9	150 micron	0	22.1	100	0

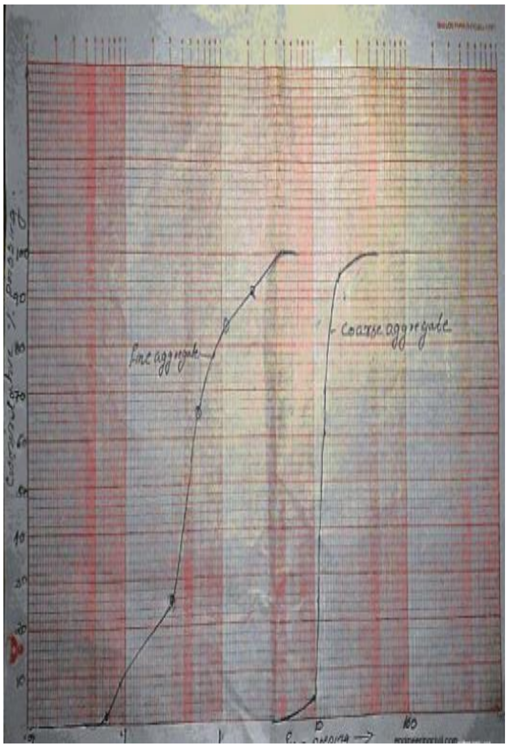

Fig 3.2.1.6 Graph of Sieve

9.0 MIX DESIGN Mix design for concrete was made using the properties of constituents of concrete. Grade of concrete was taken as M20 and the mix design was done as per IS:10262-1982 and IS:456-2000. The water cement ratio was taken as 0.5 which should be the maximum for M20 grade under mild exposure condition.

Mix design of	:	M20
characteristic compressive strength at 28 days	:	20 N/mm^2
maximum size of aggregates	:	20mm
degree of quality of control	:	good
Type of exposure	:	Mild

TEST DATA FOR MATERIAL :

Specific gravity of cement :3.14
Specific gravity of fine aggregate :2.59
Specific gravity of coarse aggregate :2.66

TARGET MEAN STRENGTH FOR MIX DESIGN

fck =fck +1.65s (s = standard deviation)
fck =20+1.65×4 = 26.6N/mm^2

Table 4 Chemical Composition of Cement

SL NO	OXIDE	PERCENT CONTENT (%)
1	CaO	60 – 67
2	SiO_2	17 -25
3	Al_2O_3	3 – 8
4	Fe_2O_3	0.5-6
5	MgO	0.1 – 4
6	K_2O,Na_2O	0.4 – 1.3

Table 5 Chemical Composition of Marble Powder

SL NO	TEST PARAMETERS	RESULTS (% by mass)
1	Silica as SiO_2	11.06
2	Aluminum as Al_2O_3	2.59
3	Iron as Fe_2O_3	0.85
4	Calcium as CaO	34.02
5	Magnesium as MgO	23.91
6	Sodium as NaO	< 0.01
7	Potassium as K_2O	< 0.01
8	Titanium as TiO_2	< 0.01
9	Phosphorous as P_2O_5	< 0.07
10	Sulphur as SO_4	< 0.01
11	Loss on Ignition (LoI)	27.09

Table 6 Design stipulation

SL NO	DESIGN STIPULATIONS	QUANTITY
1	Characteristic compressive strength in the field at 28 days	20 N/mm2
2	Maximum size of aggregates	20mm
3	Degree of quality of control	Good
4	Type of exposure	Mild

As per IS:10262-1982,
Water cement ratio = 0.5
water cement ratio and aggregate size as table 2 of IS: 10262 -2009 .
Water content = 186 kg/m3
Sand content = 35%
No adjustments are made.

DETERMINATION OF CEMENT CONTENT

Water cement ratio = 0.50
Water = 186
Cement = 186/0.50 = 370 kg/m3
This cement content is adequate for mild exposure condition, according to Appendix A of IS : 456-1978.

9.1 DETERMINATION OF COARSE AND FINE AGGREGATE CONTENT

From Table 3, for the specified maximum size of 20 mm , the amount of entrapped air in the wet concrete is 2%.
Taking this into account and applying equations from 3.5.1 of IS: 10262 -1982.
Therefore
For fine aggregate :
0.98 = [186 + (370/3.14)+(1/0.35) x (fa / 2.59)] x (1/1000)
fa = 606.06 kg/m3
For coarse aggregate :
0.98 = [185 + (370/3.14) + (1/(1-0.35) x (Ca / 2.66)] x (1/1000)
Ca = 1171.42 kg/m3
The mix proportions then becomes
Water: Cement : Fine aggregate : Coarse aggregate
186: 370 kg: 606.06 kg: 1171.42 kg
or
1: 1.638: 3.16

9.2 VOLUMES

Volume of cube = 15 x 15 x 15 =3375cm3
Volume of cylinder = 176.714 x 30 =5301.44 cm3
Total volume = 8676.44 cm3
Add 10% extra volume = 9544.084 cm3

9.3 FOR 30 CUBE SPECIMEN

Weight of cement = 45 kg
Weight of fine aggregate = 81.9 kg
Weight of coarse aggregate = 158 kg
Required water = 25 liters

9.4 FOR 20 CYLINDER SPECIMEN

Weight of cement = 47.187 kg
Weight of fine aggregate = 85.73 kg
Weight of coarse aggregate = 165.39 kg
Required water = 26.17 liters

9.5 MIX PROPORTIONS

Five concrete mixes with stone dust were produced, replacing 0%(reference mixture), 5%,10%,15%,and
20% Cement, in terms of weight. The concrete mix proportion for M20 grade was designed in accordance with I.S. code.

9.6 QUANTITY OF MARBLE POWDER FOR 30 CUBES;

5% of cement replaced by marble powder = 500gm
10% of marble powder = 1000 gm
15% of marble powder = 1500 gm
20 %of marble powder = 2000 gm

9.7 QUANTITY OF MARBLE POWDER FOR 20 CYLINDERS;

5% of cement replaced by marble powder = 523gm
10% of marble powder = 1047 gm
15% of marble powder = 1570 gm
20 %of marble powder = 2093 gm

10 Experimental conditions:

Compressive strength of concrete was undertaken on 15 cm cubic specimens. at 7 days and 28 days of age. Regarding splitting tensile strength, cylinders with 30 cm of height and 15 cm of diameter were casted and tested at 28 days of age. All specimens were removed 24 hours after casting, and then transferred to regular conditions (interior of the laboratory) till testing.

11 PREPARATION AND CURING OF SPECIMEN: Standard cubic specimens of 150 mm size were cast. Concrete cubes were cast for compressive strength. The standard cylindrical specimen of 100mm diameter and 300mm height cylindrical specimens were caste for tensile strength.

Fig 2 Applying oil to moulds

Fig 3 weighed Aggregate and Cement

Fig 4 Mixing in Drum roller **Fig 5 Compaction of concrete using vibrator**

Fig 6 Casted cubes for 7 days and 28 days test

12. Test Programme

12.1 Compressive Strength

Compressive strength of concrete can be defined as maximum resistance of concrete to axial loading. The specimens used for compression test were cubes of size 150x 150x150 mm size. Compressive strength was determined for 7 and 28 days respectively. The test was conducted in the laboratory on the compression testing machine of 3000 KN capacity and the reading at the time of failure of specimen was taken. The results are tabulated in Table7.1(a) to 7.1(b).

Table 7.1(a) Compressive strength at 7 days

% of marble powder replaced	Cube 1 (KN)	Cube 2 (KN)	Cube 3 (KN)	Average (KN)	Compressive strength (MPa)
0%	410	390	430	410	18.22
5%	420	430	400	416.6	18.51
10%	480	430	450	453.3	20.14
15%	380	370	400	383.3	17.03
20%	300	280	330	303.3	13.45

Table 7.1(b) Compressive strength at 28 days

% of marble powder replaced	Cube 1	Cube 2	Cube 3	Average	Compressive strength (MPa)
0%	413	394	430	412.33	18.27
5%	426	433	400	419.66	19.61
10%	486	434	450	456.66	21.14
15%	383	374	400	385.66	17.63
20%	302	286	330	306	14.45

Table 8.1 Tensile Strength – 7 Days

% of marble powder replaced	Cylinder 1 (KN)	Cylinder 2 (KN)	Average (KN)	Tensile strength (MPa)
0%	160	140	150	2.11
5%	150	160	155	2.18
10%	190	170	180	2.53
15%	160	170	165	2.32
20%	120	110	115	1.61

Table 8.2 Tensile Strength – 28 Days

% of marble powder replaced	Cylinder 1	Cylinder 2	Average	Tensile strength (MPa)
0%	160	140	150	2.22
5%	150	160	155	2.32
10%	190	170	180	2.55
15%	160	170	165	2.37
20%	123	120	121	2.22

13.0 RESULTS AND DISCUSSION

13.1 Compression and Tensile Strength Test

Mechanical behavior of concrete cubes prepared without chemical admixtures was studied by compressive tests (Grade M20) curing time of 7 days and 28 days. It can be noticed that 5% replacement of cement with marble dust in mild condition and 10% replacement of cement with marble dust in mild condition, are showing increase in compressive strength and Tensile strength.

14 .0 CONCLUSION

Due to marble dust, it proved to be very effective in assuring very good cohesiveness of mortar and concrete. From the above study, it is concluded that the marble dust can be used as a replacement material for cement and 10% replacement of marble dust gives an excellent result in strength aspect and quality aspect and it is better than the control concrete. The results showed that the substitution of 10% of the cement content by marble stone dust induced higher compressive strength, higher splitting tensile strength, and improvement of properties related to durability. Test results show that this industrial waste is capable of improving hardened concrete performance up to 15%, enhancing fresh concrete behaviour and can be used in plain concrete.

15.0 REFERENCES

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[2] IS:10262-1982 Recommended Guidelines for Concrete Mix Design—Bureau of Indian Standards, New Delhi.
[3] IS: 456-2000, Plain and Reinforced Concrete—Code of Practice—Bureau of Indian Standards, New Delhi.
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[5] IS: 8112-1989, 53 Grade Ordinary Portland cement—Specification, Bureau of Indian Standards, New Delhi.

STUDIES ON STRENGTH CHARACTERISTICS OF SELF-CURING CONCRETE

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Abstract

Today concrete is most widely used construction material due to its good compressive strength and durability. Depending upon the nature of work the cement, fine aggregate, coarse aggregate and water are mixed in specific proportions to produce plain concrete. Plain concrete needs congenial atmosphere by providing moisture for a minimum period of 28 days for good hydration and to attain desired strength. Any laxity in curing will badly affect the strength and durability of concrete. Self-curing concrete is one of the special concretes in mitigating insufficient curing due to human negligence paucity of water in arid areas, inaccessibility of structures in difficult terrains and in areas where the presence of fluorides in water will badly affect the characteristics of concrete. The present study involves the use of shrinkage reducing admixture polyethylene glycol (PEG 400) in concrete which helps in self-curing and helps in better hydration and hence strength. In the present study, the effect of admixture (PEG 400) on compressive strength, split tensile strength and modulus of rupture by varying the percentage of PEG by weight of cement from 0% to 2% were studied both for M20 and M40 mixes. It was found that PEG 400 could help in self-curing by giving strength on par with conventional curing. It was also found that 1% of PEG 400 by weight of cement was optimum for M20, while 0.5 % was optimum for M40 grade concretes for achieving maximum strength without compromising workability.

Index Terms: *Self-curing concrete; Water retention; Relative humidity; Hydration; Absorption; Permeable pores; Sorptivity; Water permeability*

1. INTRODUCTION

Proper curing of concrete structures is important to meet performance and durability requirements. In conventional curing this is achieved by external curing applied after mixing, placing and finishing. Self-curing or internal curing is a technique that can be used to provide additional moisture in concrete for more effective hydration of cement and reduced self-desiccation.

1.1 Methods of self curing

Currently, there are two major methods available for internal curing of concrete. The first method uses saturated porous lightweight aggregate (LWA) in order to supply an internal source of water, which can replace the water consumed by chemical shrinkage during cement hydration. The second method uses poly-ethylene glycol (PEG) which reduces the evaporation of water from the surface of concrete and also helps in water retention.

1.2 Mechanism of Internal Curing

Continuous evaporation of moisture takes place from an exposed surface due to the difference in chemical potentials (free energy) between the vapour and liquid phases. The polymers added in the mix mainly form hydrogen bonds with water molecules and reduce the chemical potential of the molecules which in turn reduces the vapour pressure, thus reducing the rate of evaporation from the surface.

1.3 Significance of Self–curing

When the mineral admixtures react completely in a blended cement system, their demand for curing water (external or internal) can be much greater than that in a conventional ordinary Portland cement concrete. When this water is not readily available, significant autogenous deformation and (early-age) cracking may result. Due to the chemical shrinkage occurring during cement hydration, empty pores are created within the cement paste, leading to a reduction in its internal relative humidity and also to shrinkage which may cause early-age cracking.

1.4 Potential Materials for Internal Curing (IC)

The following materials can provide internal water reservoirs:

- Lightweight Aggregate (natural and synthetic, expanded shale)
- Super-absorbent Polymers (SAP) (60-300 nm size)
- SRA (Shrinkage Reducing Admixture) (propylene glycol type i.e. polyethylene-glycol)

1.5 Advantages of Internal Curing

- Internal curing (IC) is a method to provide the water to hydrate all the cement, accomplishing what the mixing water alone cannot do.
- Provides water to keep the relative humidity (RH) high, keeping self-desiccation from occurring.
- Eliminates largely autogenous shrinkage.
- Maintains the strengths of mortar/concrete at the early age (12 to 72 hrs.) above the level where internally & externally induced strains can cause cracking.
- Can make up for some of the deficiencies of external curing, both human related (critical period when curing is required in the first 12 to 72 hours) and hydration.

1.6 Polyethylene Glycol

Polyethylene glycol is a condensation polymer of ethylene oxide and water with the general formula $H(OCH_2CH_2)_nOH$, where n is the average number of repeating oxyethylene groups typically from 4 to about 180. The abbreviation (PEG) is termed in combination with a numeric suffix which indicates the average molecular weights. One common feature of PEG appears to be the water-soluble nature. Polyethylene glycol is non-toxic, odorless, neutral, lubricating, non-volatile and non-irritating and is used in a variety of pharmaceuticals. The behaviour of Polyethylene glycol is shown in Fig 1.

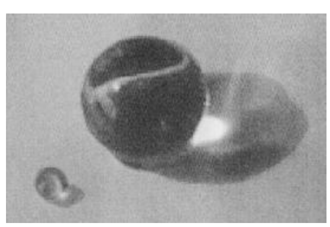


Fig. 1. Behaviour of Polyethylene glycol

2. LITERATURE REVIEW

Ole and Hansen describe a new concept for the prevention of self-desiccation in hardening cement-based materials using fine, super absorbent polymer (SAP) particles as a concrete admixture. The SAP will absorb water and form macro inclusions and this leads to water entrainment, i.e. the formation of water-filled macro pore inclusions in the fresh concrete. Consequently, the pore structure is actively designed to control self-desiccation. In this work, self-desiccation and water entrainment are described and discussed.

Roland Tak Yong Liang, Robert Keith Sun carried work on internal curing composition for concrete which includes a glycol and a wax. The invention provides for the first time an internal curing composition which, when added to concrete or other cementitious mixes meets the required standards of curing as per Australian Standard AS 3799.

Wen-Chen Jau stated that self curing concrete is provided to absorb water from moisture from air to achieve better hydration of cement in concrete. It solves the problem when the degree of cement hydration is lowered due to no curing or improper curing by using self curing agent like poly-acrylic acid which has strong capability of absorbing moisture from atmosphere and providing water required for curing concrete.

A.S. El-Dieb investigated water retention of concrete using water-soluble polymericglycol as self-curing agent. Concrete weight loss and internal relative humidity measurements with time were carried out, in order to evaluate the water retention of self-curing concrete. Water transport through concrete is evaluated by measuring absorption%, permeable voids%, water sorptivity and water permeability. The water transport through self-curing concrete is evaluated with age.The effect of the concrete mix proportions on the performance of self-curing concrete were investigated, such as, cement content and water/cement ratio.

PietroLura The main aim of his study was to reach a better comprehension of autogenous shrinkage in order to be able to model it and possibly reduce it. Once the important role of self-desiccation shrinkage in autogenous shrinkage is shown, the benefits of avoiding self-desiccation through internal curing become apparent.

3. SCOPE AND OBJECTIVE

- The scope of the paper is to study the effect of polyethylene glycol (PEG 400) on strength characteristics of Self-curing concrete
- The objective is study the mechanical characteristics of concrete such as compressive strength, split tensile strength and modulus of rupture by varying the percentage of PEG from 0% to 2% by weight of cement for both M20 and M40 grades of concrete.

4. EXPERIMENTAL PROGRAMME

The experimental program was designed to investigate the strength of self curing concrete by adding poly ethylene glycol PEG400 @ 0.5%, 1%, 1.5% and 2% by weight of cement to the concrete. The experimental program was aimed to study the workability, compressive strength, split tensile strength and modulus of rupture. To study the above properties mixes M20 and M40 were considered. The scheme of experimental program is given in Table No.1

SL. No	Nature	M20			M40		
		Cube *	Cylinder #	Prism $	Cube *	Cylinder #	Prism $
1	Plain	3	3	3	3	3	3
2	0.5%	3	3	3	3	3	3
3	1%	3	3	3	3	3	3
4	1.5%	3	3	3	3	3	3
5	2%	3	3	3	3	3	3

Table 1: Details of specimens cast.

*The size of each cube is 150 x150 x150 mm.
The size of each cylinder is 150 mm in dia and 300 mm in height.
$ The size of each prism is 100 x100 x400 mm

5. MATERIALS USED

The different materials used in this investigation are

5.1 Cement: Cement used in the investigation was 53 grade
ordinary Portland cement confirming IS: 12269: 1987.

5.2 Fine aggregate: The fine aggregate used was obtained from a near byriver source. The fine aggregate conforming to zone III according to IS: 383-1970 was used.

5.3 Coarse aggregate: Crushed granite was used as coarse aggregate. The coarse aggregate according to IS: 383-1970 was used. Maximum coarse aggregate size used is 20 mm.

5.4 Polyethylene Glycol-400: Polyethylene glycol is a condensation polymer of ethylene oxide and water with the general formula $H(OCH_2CH_2)_nOH$, where n is the average number of repeating oxyethylenegroups typically from 4 to about 180. The abbreviation (PEG) is termed in combination with anumeric suffix which indicates the average molecular weight. One common feature of PEG appears to be the water-soluble nature. The PEG-400 use in the investigation have Molecular Weight 400, Appearance Clear liquid, pH 5-7, Specific Gravity 1.126

5.5 Water: Potable water was used in the experimental work for both mixing and curing purposes.

6. CASTING PROGRAMME:

Casting of the specimens were done as per IS:10086-1982, preparation of materials, weighing of materials and casting of cubes, cylinders, beams. The mixing, compacting and curing of concrete are done according to IS 516: 1959. The plain samples of cubes, cylinders and prisms were cured for 28 days in water pond and the specimens with PEG400 were cured for 28 days at room temperature by placing them in shade. The M20 and M40 grades of concrete are designed and the material required per cubic meter of concrete is shown in Table 2.

SL. No	Mix	Cement (kg)	Fine Aggregate (kg)	Coarse Aggregate (kg)	Water (kg)
1	M20	340	610	1300	187
2	M40	440	520	1220	154

Table 2: Materials required per cubic meter of concrete

7. TESTING

7.1 Slump Test & Compaction Factor.

Slump test is the most commonly used method of measuring consistency of concrete which can be employed either in laboratory or at site of work. It does not measure all factors contributing to workability. However, it is used conveniently as a control test and gives an indication of the uniformity of concrete from batch to batch. The compacting factor test is designed primarily for use in the laboratory but it can also be used in the field. It is more precise and sensitive than the slump test and particularly useful for concrete mixes of very low workability as are normally used when concrete is to be compacted by vibration. Such dry concretes are insensitive to slump test.

7.2 Compressive strength:

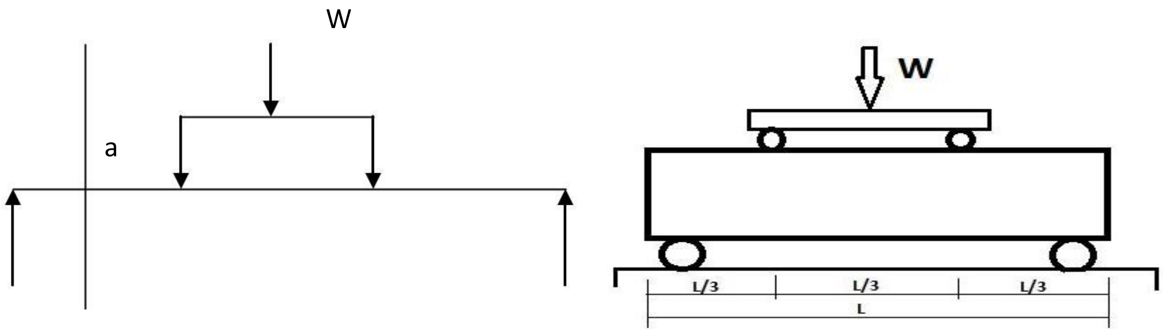
The cube specimens were tested on compression testing machine of capacity 3000KN.The bearing surface of machine was wiped off clean and sand or other material removed from the surface of the specimen. The specimen was placed in machine in such a manner that the load was applied to opposite sides of the cubes as casted that is, not top and bottom. The axis of the specimen was carefully aligned at the centre of loading frame. The load applied was increased continuously at a constant rate until the resistance of the specimen to the increasing load breaks down and no longer can be sustained. The maximum load applied on specimen was recorded.
$f_c = P/A$, where, P is load & A is area

7.3 Split tensile strength:

The cylinder specimens were tested on compression testing machine of capacity 3000KN.The bearing surface of machine was wiped off clean and looses other sand or other material removed from the surface of the specimen. The load applied was increased continuously at a constant rate until the resistance of the specimen to the increasing load breaks down and no longer can be sustained. The maximum load applied on specimen was recorded.

$f_{split} = 2\,P/\pi DL$, where P=load, D= diameter of cylinder, L=length of the cylinder

7.4 Modulus of rupture:



The beam specimens were tested on universal testing machine for two-point loading to create a pure bending. The bearing surface of machine was wiped off clean and sand or other material is removed from the surface of the specimen. The two point bending load applied was increased continuously at a constant rate until the specimen breaks down and no longer can be sustained. The maximum load applied on specimen was recorded. The test set–up is shown in Fig. 2. The modulus of rupture depends on where the specimen breaks along the span. The specimens while testing compressive strength, split tensile and Modulus of rupture is shown in Fig. If the specimen breaks at the middle third of the span then the modulus of rupture is given by

$f_{rup.} = (WL)/(bd^2)$

If the specimen breaks at a distance of 'a' from any of the supports then the modulus of rupture is given by

$f_{rup.} = (3Wa)/(bd^2)$, where W = load at failure, L = length of specimen (400mm)

b = width of specimen (10mm),

d=depth of specimen (100mm)

Fig 3: Specimens while testing

8. RESULTS & DISCUSSION

8.1 Slump and Compaction factor test:

The results of the Slump & Compaction factor test were represented in Table 3. The graphical representation of the Slump & Compaction factor results is shown in Fig 4 and Fig 5 respectively.

As the % of PEG400 is increased the slump and compaction factor is found to increase. But, the rate of increase of slump & compaction factor for M40 concrete is less than that of M20 plain concrete.

Sl. No	PEG 400	Slump (mm)		Compaction Factor	
		M20	M40	M20	M40
1	Plain	80	45	0.88	0.85
2	0.50%	92	65	0.90	0.87
3	1.00%	112	95	0.91	0.90
4	1.50%	140	130	0.93	0.91
5	2.00%	175	160	0.96	0.94

Table 3: Results of Workability

8.2 Compressive Strength:

The results of the compressive strength are represented in Table 4 and the graphical representation is shown in Fig 6. The compressive strength was found to increase up to 1% PEG400 and then decreased for M20 grade. In the case of M40 compressive strength increased up to 0.5% and then decreased. The increase in compressive strength was 7.23% at 1% of PEG 400 compared to conventional concrete for M20, while the increase is 1.24% at 0.5% of PEG400 in case of M40 grade of concrete.

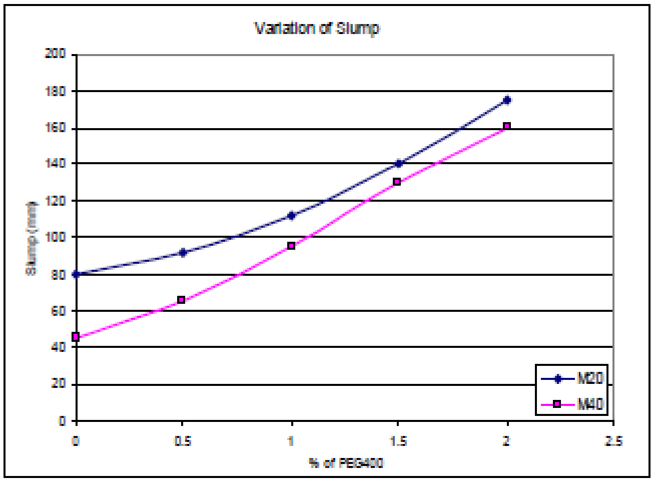


Fig 4. Variation of Slump

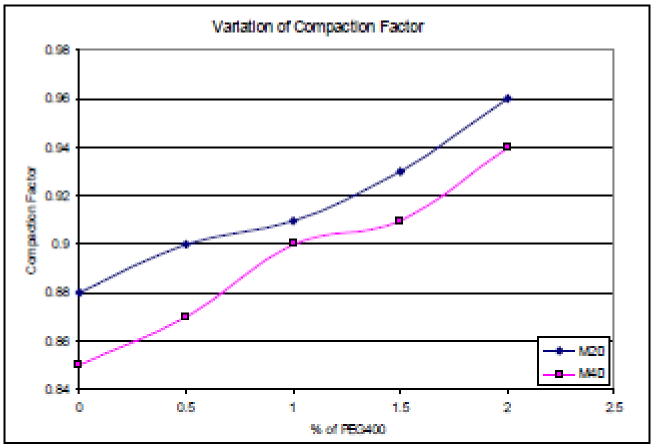


Fig 5. Variation of Compaction Factor

Sl.	PEG	f_c (N/mm^2)		f_{split} (N/mm^2)		f_{rup} (N/mm^2)	
No		M20	M40	M20	M40	M20	M40
1	Plain	26.60	46.65	1.81	2.42	3.50	4.62
2	0.50%	27.61	47.23	1.96	2.50	3.75	4.75
3	1.00%	28.49	45.93	2.02	2.45	3.80	4.64
4	1.50%	26.74	44.62	1.92	2.34	3.68	4.53
5	2.00%	25.03	42.44	1.85	2.25	3.55	4.46

Table 4: Mechanical Properties

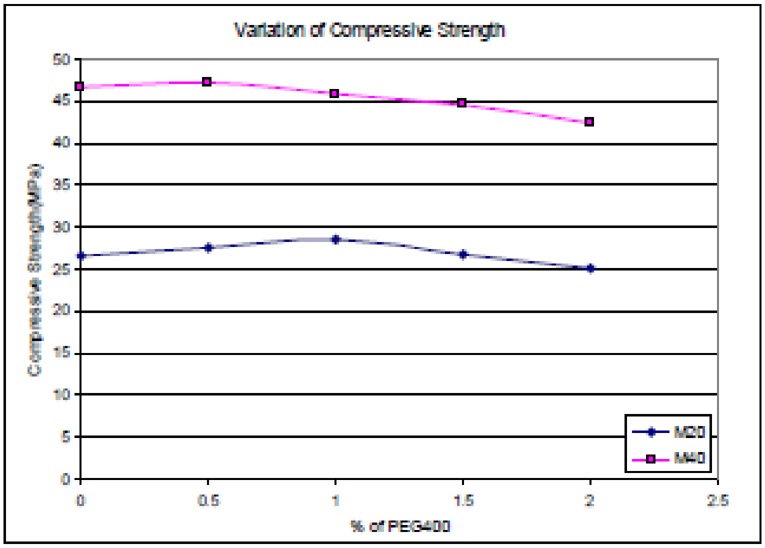


Fig 6. Variation of Compressive Strength

8.3 Split Tensile Strength:

The results of the split tensile strength are represented in Table 4 and the graphical representation is shown in Fig 7. The split tensile strength was found to increase up to 1% PEG400 and then decreased for M20 grade. In the case of M40 split tensile strength increased up to 0.5% and then decreased. The increase in split tensile strength was 11.60% at 1% of PEG400 compared to conventional concrete for M20, while the increase is 3.30% at 0.5% of PEG400 in case of M40 grade of concrete.

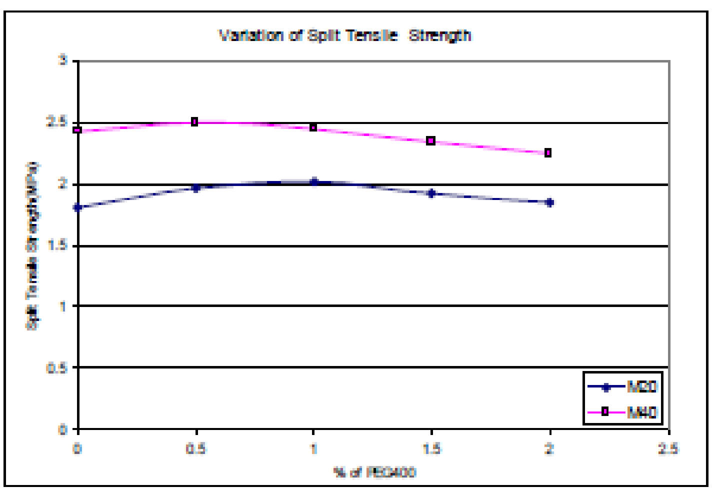


Fig 7. Variation of Split Tensile Strength

8.4 Modulus of rupture:

The results of the modulus of rupture are represented in Table 4 and the graphical representation is shown in Fig 8. The modulus of rupture was found to increase up to 1% PEG400 and then decreased for M20 grade. In the case of M40 modulus of rupture increased up to 0.5% and then decreased. The increase in modulus of rupture was 8.57% at 1% of PEG 400 compared to conventional concrete for M20, while the increase is 2.81% at 0.5% of PEG400 in case of M40 grade of concrete.

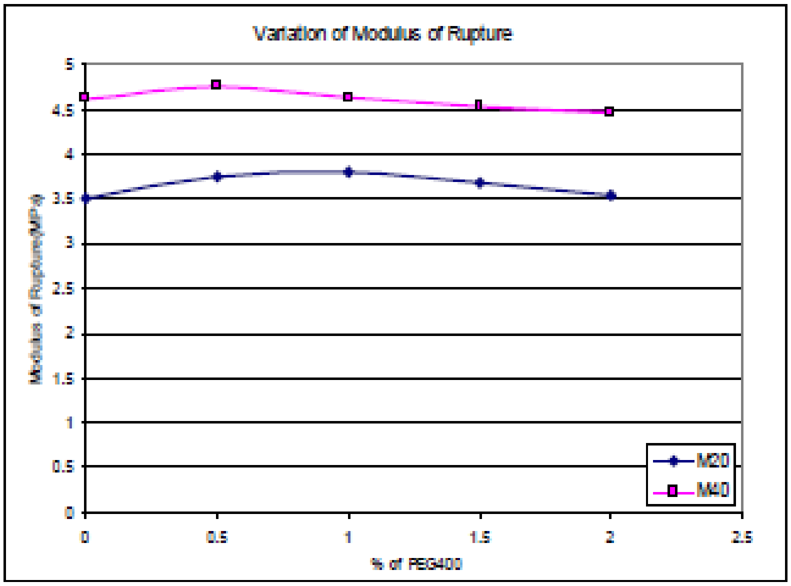


Fig 8. Variation of Modulus of Rupture

9. CONCLUSIONS:

1. The optimum dosage of PEG400 for maximum strengths (compressive, tensile and modulus of rupture) was found to be 1% for M20 and 0.5% for M40 grades of concrete.
2. As percentage of PEG400 increased slump increased for both M20 and M40 grades of concrete.
3. Strength of self curing concrete is on par with conventional concrete.
4. Self curing concrete is the answer to many problems faced due to lack of proper curing.

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STRENGTH APPRAISAL OF CONCRETE BY USING RECYCLED AGGREGATES

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Abstract
Aggregates are one of the main ingredients in producing concrete. It covers 75% of the total for any concrete mix. The construction industry is increasingly making higher demands of this material and is feared to accommodate the many requests at one time. Hence need for an alternative coarse aggregate arises. Recycled aggregates are the resultant aggregates which are left out after the demolition of a building. They are predominantly used now a days in place of coarse aggregates concrete.

The experimental investigation were carried out using detailed strength related tests such as compressive strength, Flexural strength and split tensile strength are performed for both normal and recycled aggregates.. The aim for this project was to determine the strength characteristics of structural concrete by replacing coarse aggregates with Recycled aggregates, which will give a better understanding on the properties of concrete with these aggregates. The test results of the recycled aggregates compared to the normal aggregates are lesser but are within the range of normal aggregate concrete limits. From the experimental investigation it was found that Recycled aggregates can be used as coarse aggregates. However further investigations have to be made to study long term effects concrete.

Keywords: RA-Recycled Aggregate, RAC- Recycled Aggregate Concrete

I. INTRODUCTION

Recycled aggregates consists of natural aggregate coated with cement paste residue, pieces of natural aggregate, or just cement paste and some impurities. The recycled aggregates are collected after the demolition of a building or any civil engineering infrastructure. There is a general consensus that the amount of cement paste has a significant influence on the quality and the physical, mechanical and chemical properties of the aggregates and as such as potential influence on the properties of RC concrete. Concrete has variety of Strength characteristics such as compressive strength, split tensile strength and flexural strength, although these properties play a part in defining the strength of concrete the most important parameter of concrete is its compressive strength values and all other values are related to it.

Aggregates are most widely used construction material in the world. This project focuses on the importance and strength variation in the concrete by using the recycled aggregates. The recycled aggregate concrete and the normal aggregate concrete strengths are compared by performing various Concrete tests

Recycled aggregates Collected Site and Information

The aggregates are collected after the demolition of a 20 year old building which was constructed in 1996 at Gandhi nagar, bank of baroda colony,HYD.

The aggregates are handpicked and sieved with IS sieve analysis. The aggregates passing through 25mm and retained in 20mm are used for the project.

II. LITERATURE REVIEW

Ravindraraja demonstrated that the average value of water absorption in recycled aggregate was 6.35%, where as in natural aggregate it was 0.9%. "*Properties of Concrete made with CrushedConcrete as Coarse Aggregate*"(2000)

M. C. Limbachiya, T. Leelawat and R. K. Dhir studied about the use of recycled aggregate in high strength concrete. The effects of coarse Recycled concrete aggregate on the ceiling strength, bulk engineering and durability properties of such concretes are established. The results showed that 30% coarse recycled concrete aggregate had no effect on the concrete strength but there after there was gradual reduction as the recycled concrete aggregate content increased."*Use of recycled concrete aggregate in high strength concrete"(2000)*

Gomez showed that the porosity increases considerably when natural aggregate is replaced by recycled coarse aggregate."*porosity of recycled concrete with substitution of recycled concrete aggregate"*(2002)

Gonzalez concluded that recycled aggregate concrete shows more water absorption than conventional concrete."*Concretes with aggregates from demolition waste"(2008)*

Hansen et al investigated that the specific gravity decreases from 4.5 to7.6% when compared with specific gravity of natural aggregate. "*Strength of recycled aggregate concrete made from crushed concrete coarse aggregate.Concrete"* (1983)

Hansen concluded that the density changes with the size of the aggregate and the amount of adhered mortar to the aggregate, when the concrete is grinded with the same type of the machine and the same energy applied."*Recycled aggregates and recycled aggregates concrete"*(1985)

III. DESIGN MIX FOR CONCRETE

M_{25} Grade of concrete Mix design for the the results are as per IS-10262-2009 code book. The design ratio is as follows for the M25 grade of concrete

Cement/Cube	w/c ratio	Coarse aggregate(kg)	Fine aggregate(kg)
1.08	0.45	2.64	3.4

The cubes, cylinders and beams are casted based on the grade of mix and the number of cubes required. The cubes are kept in water for the period of time which is required to test the strength for the following specimen. The test results of 7 and 28 days are taken into consideration for the following tests.

IV.TESTS PERFORMED ON NORMAL AGGREGATES AND RECYCLED AGGREGATES

A) Impact Strength Test
B) Aggregate Crushing Strength Test.
C) Water Absorption Test
D) Compressive Strength Test
E) Split tensile Strength Test
F) Flexural Strength Test

A. *Impact Strength Test Results*

The Impact Strength test results for the normal aggregates and recycled aggregates are determined by taking 3 samples and the average value of the 3 sample is the strength percentage for the Aggregate Impact Test.

Impact Strength Test Results for normal aggregates and recycled aggregates are as follows

Normal Aggregates	Recycled Aggregates
13.85%	15.84%

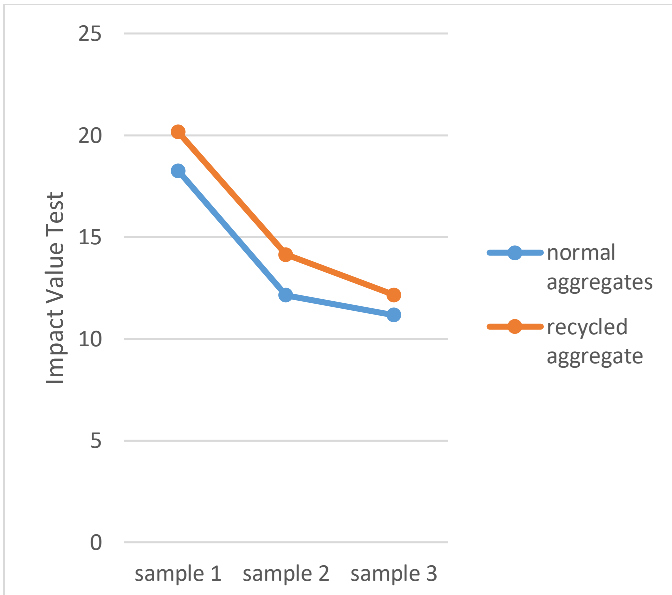

Fig 1: Comparison of impact value test % of normal aggregates with recycled aggregates

The impact value strength of the recycled aggregates is higher than the normal aggregates

B. *Aggregate Crushing Strength Test*

The Aggregate crushing strength test results for the normal aggregates and recycled aggregates are as follows

Normal Aggregates	Recycled Aggregates
31.39%	33.41%

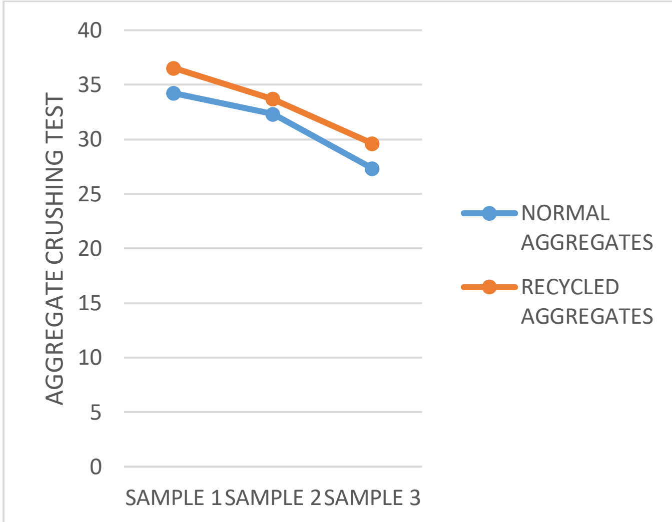

Fig 2: Comparing the Aggregate crushing strength of normal aggregates with the recycled aggregates

The mean average strength of the recycled aggregates is higher than the normal aggregates.

C. *Water Absorption Test Results*

The water absorption test results of normal aggregate and the recycled aggregates are as follows

Normal Aggregates	Recycled Aggregates
1.89%	3.59%

The durability is high for the recycled aggregates when compared to the normal aggregates. The durability of the aggregate depends on its water absorbing nature.

D. *Compression Strength Test Results*

The compression strength test results for the normal aggregates of M_{25} Grade of Concrete are as follows

w/c ratio	**7 days(N/mm^2)**	**28 days(N/mm^2)**
0.45	**22.50**	**33.4**

The compression Strength test results for the recycled aggregates are as follows

w/c ratio	**7 days(N/mm^2)**	**28 days(N/mm^2)**
0.45	21.35	32.54

The graphical representation of the test results

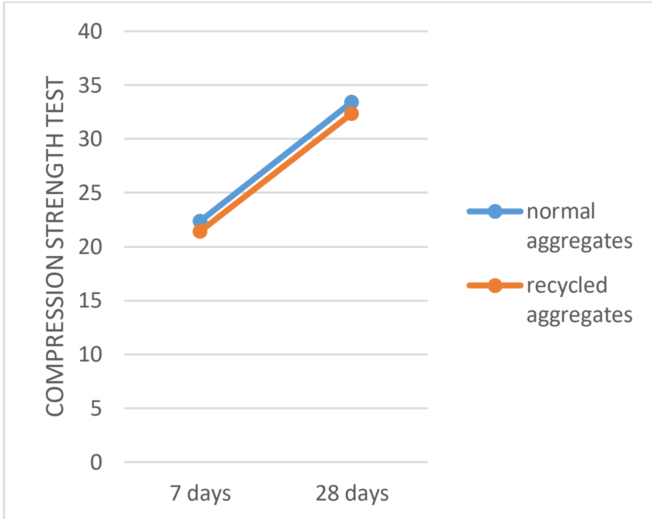

Fig3: Comparing the Compression strength of normal aggregate concrete with recycled aggregate concrete.

*E. **Split tensile Strength Test Results***

The split tensile strength results for the M_{25} Grade of concrete are as follows
The Split tensile strength results for Normal Aggregates

w/c ratio	7 days(N/mm^2)	28 days(N/mm^2)
0.45	2.33	3.42

The split tensile strength results for Recycled Aggregates

w/c ratio	7 days(N/mm^2)	28 days(N/mm^2)
0.45	2.06	3.04

The split tensile strength is performed by casting cylinders and the values of the test results for 7 days and 28 days show that the strength of the normal aggregate concrete is higher compared to the recycled aggregate concrete. The 7day split tensile strength difference between the recycled aggregate concrete and normal aggregate concrete is around 11.58% but within the limits of the normal aggregate concrete. The 28 day split tensile strength difference between the normal aggregate concrete and the recycled aggregate concrete is around 11.11% but within the range of limits of the normal aggregate concrete strength

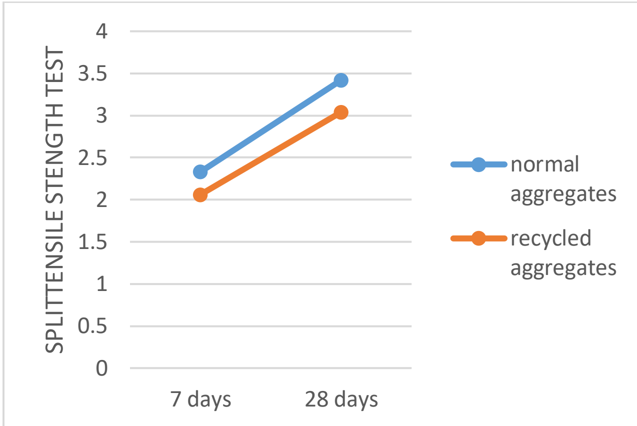

Fig 4: Comparing the split tensile strength test results of normal aggregate concrete with recycled aggregate concrete

F. ***Flexural Strength Test Results***

The Flexural Strength Test Results For Normal Aggregates of M_{25} Grade of concrete are as follows

w/c ratio	7 days(N/mm^2)	28 days(N/mm^2)
0.45	6.93	7.95

The flexural Strength Test Results for Recycled Aggregates of M_{25} Grade of concrete as follows

w/c ratio	7 days(N/mm^2)	28 days(N/mm^2)
0.45	6.25	6.92

The graphical representation of the test results are as follows

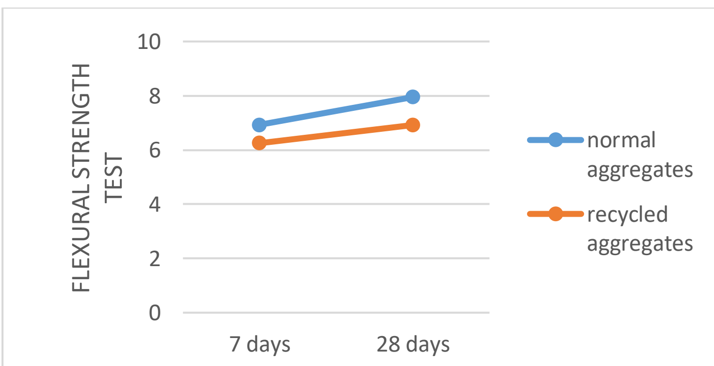

Fig 5: Comparison of flexural strength of normal aggregate concrete with recycled aggregate concrete

The flexural strength of concrete is performed by casting prisms and the values of the test results for 7 day and 28 day strength indicate that the normal aggregate concrete strength is higher than the recycled aggregate concrete and the difference in the 7 day strength of flexural strength of recycled aggregate concrete and normal aggregate concrete is 9.81%.For the 28 days flexural strength of recycled aggregate concrete is lower compared to the normal aggregate concrete, the difference in the strength is around 12.95%.The flexural strength is within the limits of the normal aggregate concrete strength.

Experimental Study

From the experiment results it is observed that the property values of the recycled aggregate concrete are within the limits of the normal aggregate concrete. The strength of the recycled aggregate concrete gradually increases from 7 days to 28 days, the change in 7 day results for both the Recycled aggregate concrete and normal aggregates concrete are nearly 6 to 7% depending up on the test. The 28th day strength of the recycled aggregate concrete is less than the normal aggregate concrete with a difference of 11-12%.The compression test values indicate that the recycled aggregate concrete also provide the strength within its range of limits. The difference in the 7 day compression strength test for recycled aggregates compared to normal aggregates is 4.2%.The 28 day strength difference is around 3.17% but the strength limit is within the range of normal aggregate concrete.

The water absorption properties of Recycled aggregates are higher than the normal aggregate concrete as they have good water absorption behaviour compared to the freshly used normal aggregates. The water absorption test proves that the durability of the recycled aggregates is higher than the normal aggregates

The flexural strength of concrete is performed by casting prisms and the values of the test results for 7 day and 28 day strength indicate that the normal aggregate concrete strength is higher than the recycled aggregate concrete and the difference in the 7 day strength of flexural strength of recycled aggregate concrete and normal aggregate concrete is 9.81%. For the 28 days flexural strength of recycled aggregate concrete is lower compared to the normal aggregate concrete, the difference in the strength is around 12.95%.The strength is within the limits of the normal aggregate concrete strength.

The split tensile strength is performed by casting cylinders and the values of the test results for 7 days and 28 days show that the strength of the normal aggregate concrete is higher compared to the recycled aggregate concrete. The 7day split tensile strength difference between the recycled aggregate concrete and normal aggregate concrete is around 11.58% but within the limits of the normal aggregate concrete. The 28 day split tensile strength difference between the normal aggregate concrete and the recycled aggregate concrete is around 11.11% but within the range of limits of the normal aggregate concrete strength

CONCLUSIONS:

1. By the experimental results we can conclude that the strength of the recycled aggregate concrete varies but it is within the limits of the normal aggregate concrete strength.
2. Split tensile strength of recycled aggregate concrete is 11.11% lower than the normal aggregate concrete but within the range of limits of the normal aggregate
3. flexural strength of recycled aggregate concrete is 12.95% lower compared to the normal aggregate concrete

4. As the usage of recycled aggregates increases the cost productivity for the normal aggregates decreases.
5. The durability factor plays a key role in determining the water absorption of the aggregates, the recycled aggregates possess more durability compared to the normal aggregates .The water absorbing nature is higher than the normal aggregates.
6. The impact test strength results of the recycled aggregates are higher than the normal aggregates. The recycled aggregate strength is 12% more than the normal aggregates
7. The aggregate crushing strength of the recycled aggregates is higher than the normal aggregates. The recycled aggregates strength is 6.04% more than the normal aggregates
8. The test results of ages 7 and 28 days for the following tests indicate the strength of the recycled aggregate concrete is lower than the normal concrete aggregate but within the limits of the concrete strength.

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ASSESMENT OF CHLORIDE ION PENETRATION OF BACTERIAL CONCRETE

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Abstract
In Reinforced concrete structures chloride ion penetration is considered to be a major cause of corrosion of reinforcing bars. Conventional concretes fail to prevent the intrusion of moisture and aggressive ions adequately. The deterioration of reinforced concrete structures usually involves the transport of aggressive substances from the surrounding environment followed by physical and chemical actions in its internal structure. The transport of aggressive gases and/or liquids into concrete depends on its permeation characteristics. Therefore, permeation of concrete is one of the most critical parameters in the determination of concrete durability in aggressive environments. The use of microorganisms has been reported to increase the resistance of concrete to deterioration by aggressive chemicals such as chlorides. In this investigation bacteria Bacillus subtilus JC3 is incorporated into concrete during mixing. Microbiologically induced calcite precipitation (MICP), a highly impermeable calcite layer formed over the surface of an already existing concrete layer, due to microbial activities of the bacteria Bacillus subtilis JC3 (cultured at JNTU) seals the cracks in the concrete structure and also has excellent resistance to corrosion therefore increase the strength and durability of concrete structures. To establish the resistance of bacterial concrete to chloride ion penetration, ASTM C1202 chloride permeability test was conducted and experimental investigations are presented.

Keywords: *Bacterial concrete, Bacillus subtilus JC3, Chloride Ion Penetration, durability.*

1. Introduction: Permeation property of concrete is one of the most critical parameters in the determination of durability of concrete in aggressive environments. Permeation, is dictated by the microstructure of concrete, controls the ingress of moisture, ionic and gaseous species into concrete. Chemical degradation, e.g. sulphate attack, carbonation, alkali-aggregate reaction and corrosion of steel reinforcement, as a result of reaction between an external agent and the ingredients of concrete, and some physical effects, such as frost attack, can be greatly reduced by reducing the permeability of concrete. As the permeation of concrete decreases, its durability, in terms of physico-chemical degradation, increases. Reinforced concrete structures exposed to the environment, chloride ion penetration is considered to be a major cause of corrosion of reinforcing bars. Conventional concretes fail to prevent the intrusion of moisture and aggressive ions adequately; therefore, special concretes with low permeability or novel techniques which can reduce the permeation capacity are needed. The use microorganisms in concrete have been reported to increase the resistance of concrete to deterioration by aggressive chemicals such as chlorides. The potential synergy between specific type of bacteria and its cell concentration needs to be investigated in the context of achieving an optimum balance for the development of high performance and chloride ingress resistant concrete. Since high resistance to chloride penetration can be directly related to low permeability that dominates the deterioration process in concrete structures, the resistance to chloride penetration is one of the simplest measures to determine the durability of concrete. Flow of chloride ions driven through concrete induced by an electrical potential, is referred to as chloride ion permeability. Therefore, in this study, the rapid chloride permeability test method designated in ASTM C 1202(1997) is adopted. This test, originally developed by Whiting [1981], is commonly referred to as the "Rapid Chloride Permeability Test" (RCPT). This name is inaccurate as it is not the permeability that is being measured but ionic movement. In addition, the movement of *all* ions, not just chloride ions, affects the test result (the total charge passed).The advantage of adopting this rapid chloride permeability test (RPCT) test is direct cost savings could be quantified when compared to other tests and the brief procedural steps involved significantly reduce the technician time necessary to evaluate a particular concrete.

There have been a number of criticisms of this technique, although this test has been adopted as a standard test, is widely used in the literature [Saito and Ishimoiri, 1995; Goodspeed at al., 1995; Thomas and Jones, 1996; Samaha and Hover, 1996].The main criticisms are: (i) the current passed is related to all ions in the pore solution not just chloride ions, (ii) the measurements are made before steady-state migration is achieved, and (iii) the high voltage applied leads to an increase in temperature, especially for low quality concretes, which further increases the charge passed [Andrade, 1993; Zhang and Gjorv, 1991; Malek and Roy, 1996; Roy, 1989; Geiker, et al., 1990]. Lower quality concretes heat more as the temperature rise is related to the product of the current and the voltage. The lower the quality of concrete, the greater the current at a given voltage and thus the greater heat energy produced. This heating leads to a further increase in the charge passed, over what would be experienced if the temperature remained constant. Thus, poor quality concrete looks even worse than it would otherwise.

2.Biocalcification: Microbiologically induced calcite precipitation (MICP) using environment friendly bacteria to precipitate calcite ($CaCo_3$) during its microbial activities under prevailing Indian conditions is investigated to formulate a strategy to present Bacterial Concrete as best innovative self crack healing method in Concrete structures. During the process of biocalcification, the enzymatic hydrolysis of urea takes place forming ammonia and corbondioxide. Urease which is provided by bacteria deposits $CaCO_{3,}$ a highly impermeable calcite layer, over the surface of an already existing concrete layer which is relatively dense and can block cracks and thus hamper ingress of water efficiently increasing corrosion resistance and consequently increasing the strength and durability of concrete structures. MICP is a complex mechanism and is a function of cell concentration, ionic strength, nutrient and pH of the medium. Modern techniques such as XRD & SEM analysis can be used to quantify the study of stages of calcite deposition on the surface and in cracks.

3. Experimental Programme:

Materials

Ordinary Portland cement of 53 grade confirming to IS 12269 was used in this investigation. The specific gravity of the cement was 3.06. Locally available river sand passing through 4.75 mm IS sieve was used. The specific gravity of the sand is found to be 2.68. Crushed granite aggregate of size 20 mm available from local sources has been used. The specific gravity of coarse aggregate is 2.75. Bacteria cultured distilled water was used for mixing. The mineral admixture Silica fume is used for development of high grade concrete.

Microorganisms

Aerobic alkaliphilic microorganisms Bacillus subtilis strain JC3 cultured and grown at JNTUH Biotech Laboratory was used in this study. The medium composition required for growth of culture is Peptone: 5 g/lt., NaCl: 5 g/lt. and Yeast extract: 3 g/lt. supplemented with urea buffer.

4. Test Methods:

Rapid Chloride Ion Penetration Test

The test was performed according to ASTM C 1202-97 “Standard Test Method for Electrical Indication of Concrete’s Ability to Resist Chloride Ion Penetration”. In the AASHTO T277 (ASTM C1202) test, a water-saturated, 50-mm thick, 100-mm diameter concrete specimen is subjected to a 60 V applied DC voltage for 6 hours. Components of the test arrangement as marked in Fig 1 along with the details of the cell. The prepared samples were fitted between the two compartments of the cell, and then the cell was tightened on all four corners with the help of long screws and nuts. After tightening the cell, the edges of the sample and the compartment were completely sealed with silicon rubber to prevent leakage. Once all cells were ready the test was commenced by connecting each cell to a power supply.

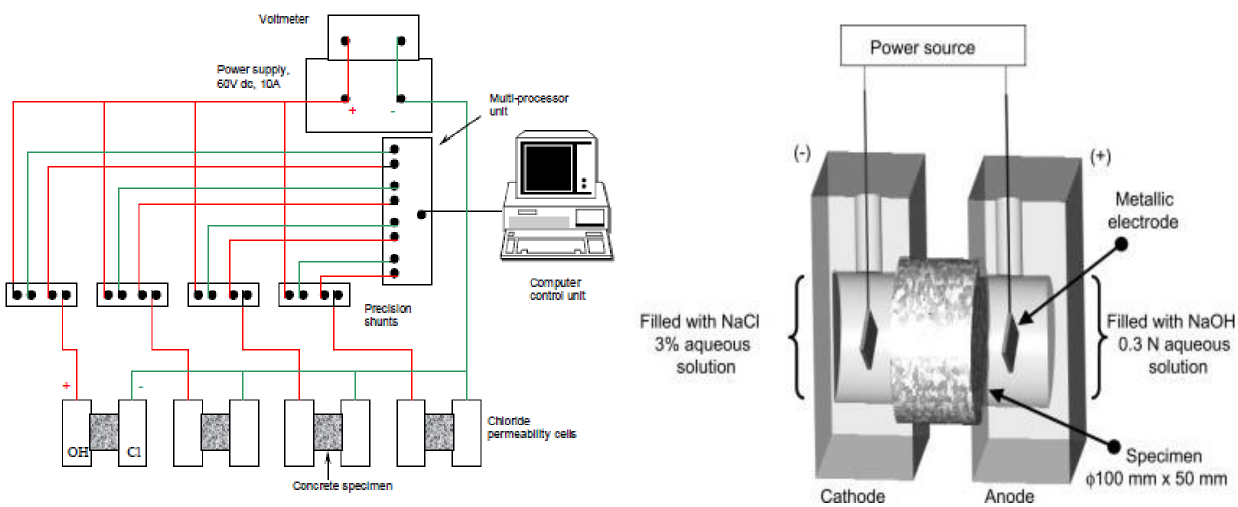

Fig 1: Schematic Diagram of Rapid Chloride Permeability Test Setup

Preparation of solution: Two solutions are prepared in distilled water, 3% (by weight) of sodium chloride (NaCI) solution (30 ml for 1 litre of distilled water or 30 grams of NaCl powder for 1 litre of distilled water) is filled in the left compartment of the cell, which served as cathode(-ve) where current flows in. The other compartment is filled with 0.3 N sodium hydroxide (NaOH) solution (12 gms of NaOH pellets for 1 litre of distilled water) and served as anode (+ve) where current flows out. These concentrations give the equal electrical conductivity of both the solutions.

$$N=\frac{\begin{array}{c}\text{(Weight of NaOH to be added}\\ \text{for 1 litre of solution)}\end{array}}{\begin{array}{c}\text{(Gram equivalent weight}\\ \text{of NaOH)}\end{array}}\times\frac{1000}{V}$$

Where N= Normality, a measure of concentration, is equal to the gram equivalent weight per liter of solution. Gram equivalent weight is a measure of the reactive capability of a molecule. V is the volume of solution, in this case 1 litre (or 1000 ml)

$$0.3\ N=\frac{\begin{array}{c}\text{(Weight of NaOH in grams}\\ \text{to be added for 1 litre of solution)}\end{array}}{40}\times\frac{1000}{1000}$$

$$\begin{array}{l}\text{Weight of NaOH in grams}\\ \text{to be added for 1 litre of solution}\end{array}=12\text{ grams}$$

In some literature instead of 0.3N NaOH, 0.3M NaOH is stated. M stands for morality and N stands for normality. Both morality and normality are measures of concentration. One is a measure of the number of moles per liter of solution and the other changes depending on the solution's role in the reaction. The solution's role in the reaction determines the solution's normality. Molarity is the most commonly used measure of concentration. A 1 M solution of H_2SO_4 contains 1 mole of H_2SO_4 per liter of solution.H_2SO_4 dissociates into H^+ and SO_4^- ions in water. For every mole of H_2SO_4 that dissociates in solution, 2 moles of H^+ and 1 mole of SO_4^- ions are formed. This is where normality is generally used. For acid reactions, a 1 M H_2SO_4 solution will have normality (N) of 2 N because 2 moles of H^+ ions are present per liter of solution. For sulfide precipitation reactions, where the SO_4^- ion is the important part, the same 1 M H_2SO_4 solution will have a normality of 1 N.

In the present case 1M NaOH solution will have normality of 1N

$$0.3\ M\ NaOH \equiv 0.3\ N\ NaOH$$

Connection: All the permeability cells holding the test specimens are connected to the power supply in such a way that the left compartment bearing the NaCI solution must be connected to the positive terminal of the voltage. The external voltage cell is always maintained at 60V of power supply. Chamber containing NaOH solution is connected to the negative terminal of the external DC voltage cell. The concentrations of these two solutions provide equal conductivity. The permeability cell, which is made of Perspex and consists of two parts each with a reservoir being capable of holding 250 ml of chemical solution and copper mesh of 100 mm diameter to act as an electrode. An external voltage cell is used to apply a voltage difference of 60V between the electrodes. The electrochemical cell, constituted by this assembly, results in the rapid migration of chloride ions from the sodium chloride solution to the sodium hydroxide solution, via the pore network offered by the concrete disc shaped specimen. The movement of chloride ions is proportional to the intensity of electric current as measured by an Ammeter in the power source.

Thermocouples: Thermocouples are immersed in NaOH solution for monitoring the temperature during the test. The test is carried out for duration of 6 hours and the current is measured at 30 min intervals. The chloride ion permeability is computed as the total charge passed through by using the formula given below.

Chloride ion permeability:

Coulombs = $(I_0 + I_1 + I_2 + I_3 + I_4 + I_5 + I_6)$ mAh

$\approx$ I mAh = I x 0.001A x 3600 s

Where I_0, I_6 are the initial and final currents and I_1, I_2, I_3, I_4, I_5, are the intermediate currents. I is the total current at the end of the test. The test determines the electrical conductance of the test specimen, expressed as the total electrical charge passed through the specimen, in coulomb. The total charge passed is determined and this is used to rate the quality of the concrete according to the criteria rating. Rapid chloride ion penetrability tests were conducted on controlled and bacterial specimens. Usually chlorides penetrate in concrete by diffusion along water paths or open pores. Some of these chlorides can react with the cement compounds, mainly tricalcium-aluminates (C_3A), forming stable chloro complexes. The excess of chloride, which is free, leads to the initiation of the corrosion process. In addition resistivity or conductivity can also be determined from the Final current reading, since the resistance of the specimen can be calculated immediately from Ohm's law: R=V/I

Where R is resistance, V is voltage, and I is current.

The resistivity is determined from:

$$\text{Resistivity} = R \times (A/L)$$

Where A is area of the specimen, and L is thickness of the specimen.

In the present investigation chloride ion permeability in concrete is studied. A total of 8 cylindrical specimens of 100 x 50 mm size of Ordinary Grade (M20), standard Grade (M40) and High Strength Grade Concrete (M60) are prepared. The chloride permeability values of Bacterial Concrete were compared to controlled concrete as per ASTM C1202 criteria.

Table 1: RCPT ratings (per ASTM C1202)

Charge Passed(coulombs)	Chloride Ion Permeability
>4000	High
2000-4000	Moderate
1000-2000	Low
100-1000	Very Low
<100	Negligible

Specimen Preparation:

Preparation of Test Solutions

NaCl : 30 grams of NaCl salt is mixed with 1000 ml of distilled water to get the sodium chloride solution of 3% strength.

NaOH : 12 grams of NaOH salt is mixed with 1000 ml of distilled water to obtain the sodium hydroxide solution of 0.3N strength.

The test is carried out on cylindrical specimens of diameter 100 mm and height 50 mm.

Preparation of Concrete Specimens

Cylindrical specimens of 100 x 50 mm size are used.

5. Test Results and Discussion:

Permeability values reduce due to formation of calcite crystal layers on the surface and in the pores of concrete. In case of high grade concretes, with bacteria induced in it, chloride ion permeability is reduced to very low to negligible levels.(in accordance with the rating) whilst this was moderate in ordinary grade concretes. The results of this testing verified that the use of bacteria greatly reduced the relative chloride ion penetration of the concrete. However, regardless of the type of concrete, the ability of concrete to resist the penetration of chloride ions is also dependent on the soundness of the concrete. The presence or development of cracks in the concrete structures provides a direct path for chloride ions to reach the reinforcing steel, reducing any benefits the concrete may provide in resisting the chloride penetration.

Results of the total charge passed on the concrete specimens of 100 mm diameter and 50 mm thick during 6 hours at an applied voltage of 60volts indicates that bacteria induced concrete has shown between 85% to 90 % higher resistance against the chloride movements in bacterial concrete as compared to the chloride movements in normal concrete.

The percentage decrease in chloride ion permeability is in the range of 85 to 90 % in all grades of concrete for all ages considered for study. With the age increasing, the pores become less well connected and therefore more resistant to the passage of electrical current are recorded.

6. Conclusions: The results indicate that permeability values reduced gradually with increased curing age; beyond 180 days the reductions were almost negligible in high strength grade concrete. Significant reductions in the permeability index with values dropping to 93 Coulombs (i.e. “negligible”). As SF was incorporated at 10%, in high strength grade concrete a significant reduction in permeability was exhibited and the whole range of results was shifted towards smaller values.

Bacterial concrete is less permeable than controlled concrete. The reason may be attributed due to the fact that presence of calcium carbonate crystals reduces the porosity by plugging the pores. Bacteria heal the cracks causing interconnecting voids to be minimum. Decrease in chloride ion permeability is more pronounced in high strength grades of concrete. The same trend is observed at all ages of concrete. The permeability of concrete depends on the pore structure of concrete, while electrical conductivity or resistivity of concrete is determined by both pore structure and the chemistry of pore solution. Factors, that have little to do with the transport of chloride, can have great effects on electrical conductivity of concrete. Thus, the electrical conductivity or resistivity of concretes cannot be used as an indication of their permeability. However, it can be used as a quality control indicator when the concretes have the same components and mixing proportions.

Table 2(a): Chloride Ion Permeability at 28 days

Grade of Concrete	Current (m.A)	Charge Passed (Coulombs)	Chloride Permeability as per ASTM C 1202
Concrete without Bacteria			
M20	112	2419	Moderate
M40	93	2008	Moderate
M60	47	1022	Low
Concrete with Bacteria			
M20	17	367	Very Low
M40	11	238	Very Low
M60	8	173	Very Low

Table 2(b): Chloride Ion Permeability at 60 days

Grade of Concrete	Current (m.A)	Charge Passed (Coulombs)	Chloride Permeability as per ASTM C 1202
Concrete without Bacteria			
M20	109	2213	Moderate
M40	90	1991	Low
M60	45	997	Low
Concrete with Bacteria			
M20	16	351	Very Low
M40	10	222	Very Low
M60	6	159	Very Low

Table 2(c): Chloride Ion Permeability at 90days

Grade of Concrete	Current (m.A)	Charge Passed (Coulombs)	Chloride Permeability as per ASTM C 1202
Concrete without Bacteria			
M20	99	2100	Moderate
M40	83	1817	Low
M60	39	943	Low
Concrete with Bacteria			
M20	15	327	Very Low
M40	8	202	Very Low
M60	5	96	Very Low

Table 2(d): Chloride Ion Permeability at 180 days

Grade of Concrete	Current (m.A)	Charge Passed (Coulombs)	Chloride Permeability as per ASTM C 1202
Concrete without Bacteria			
M20	99	2100	Moderate
M40	83	1817	Low
M60	39	943	Low
Concrete with Bacteria			
M20	15	327	Very Low
M40	8	202	Very Low
M60	5	93	Very Low

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LATERAL RESPONSE OF STEEL BRACED REINFORCED CONCRETE STRUCTURE OF UNSYMMETRICAL HIGH RISE BUILDINGS

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Abstract

Steel bracings are lateral load resisting structural members which helps in increasing the strength of RC structures. They also serve both architectural and structural requirements considering gravity and lateral loads. Present analytical seismic study deals with optimum location of internal steel bracing system in unsymmetrical high rise building of G+30 storeys in Zone 5 following IS 1893(part-1):2002 considering also wind effect by ETABS-2013 software. An unsymmetrical building plan of T-Shape is considered with different steel bracing system such as X, Knee, Single Diagonal, Inverted V bracings are assigned in various locations of building. Gravity, wind and seismic forces are acted to the structure. Response of the structure is compared with bracings and without bracings in Equivalent static analysis and Response spectrum analysis. Braces are connected within the storey and more than one storey considering five different types. The results are compared with bare frame and five types and optimum location of bracings are compared. The response of the structure is compared considering lateral displacement, story drift, story stiffness, base shear, story torsion, eccentricity and time period.

Keywords*: Unsymmetrical Building Plan, steel bracing, RC Frame, lateral loading, Response Spectrum Analysis, Ductility.*

1.0 Introduction

Functionality of all kinds of structural forms is to transfer gravity load effectively and structure should resist lateral load caused by wind and seismic forces. Lateral loads influences structure in developing high stresses and more sway. Therefore structure needs to have sufficient strength against vertical loads and adequate stiffness to resist lateral forces and resisting deformation without any collapse. For overcoming this problem lateral force resisting systems such as bracings, shear wall increase the strength of RC framed structures and helps for rehabilitation of structural damage caused by earthquake or strengthening of an undamaged structure focusing stability and stiffness.

2.0 STRUCTURAL FORMS

The determination of structural form of a tall building or high rise building would perfectly involves arrangement of the major structural elements to resist various combinations of lateral loads and gravity loads. Structural considerations strongly influence the selection of structural form and resist bending or flexure, shear and axial tension or compression.

The ability of the structural system and material to deform and absorb energy without collapse or fracture is termed as ductility. Selection of structural forms strongly influences following factors that has to be taken into account:

1. The internal planning
2. The material and the method of construction
3. The nature and magnitude of the horizontal loading
4. The external architectural treatment
5. The height and proportions of the building and
6. The planned location and routing of the service systems

3.0 Objectives of the present work

Earthquakes are occurring very frequently now-a-days. The seismic analysis and design of buildings has traditionally focused on reducing the risk and loss of life. To reduce the effects caused by these earthquakes and wind loads different lateral loading systems are introduced in the structures. Steel bracings are one of the lateral loading systems commonly constructed in high rise buildings.

In symmetrical buildings centre of mass and centre of rigidity coincides but in the case of unsymmetrical buildings the centre of mass and centre of rigidity does not coincide. Position of steel bracing in unsymmetrical building is a challenge considering stability, stiffness and reducing torsional effect. So, it is very necessary to locate position of steel bracing in unsymmetrical buildings to minimize the torsion effect. The position and type of steel bracing effects the building in aspects of displacement, base shear, and storey drifts etc. In the present work the following tasks are carried out,

1 To study the behavior of high rise building of G+ 30 storeys's RCC structure considering T –shape plan providing steel bracings at various exterior locations of structure with seismic, wind loading and different types of steel bracings.
2 Both Equivalent static analysis and Response spectrum analysis are carried out.
3 The variation of minimum storey shear, storey torsion, and maximum storey stiffness of the models has been studied.
4 The variation of storey drifts, time period and displacements of the models has been studied.
5 Considering all aspects comparing all models optimum location of steel bracing focusing ductility, control drifts and stiffness has been studied.

4.0 BUILDING CONFIGURATION:

Table 4.1: Modeling Information

Software	ETABS-2013
No. of stories	G + 30
Height of Building	105.6 m
Length of building	40 m(unsymmetrical along X Direction)
	40 m(symmetrical along Y Direction)
Bay Dimensions	8 m (both directions)
Material Properties	
Grade of concrete	M40
Grade of steel	415
Density of reinforced concrete	25 KN/m 3
Member Properties	
Thickness of slab	0.200 m
Beam Cross Section	0.3m X 0.9 m
Plinth Beam Cross Section	0.3m X 0.75m
Column Cross Section	1.2m X 1.2m (Up to story 10)
	1.0m X 1.0m (11 to 20 story)
	0.8m X 0.8m
Steel Bracing Section	ISA 200X200X25

Table 4.2: Loading Information

Live Load	3 KN/m2
Floor Finish	1.5 KN/m2
Wall Loading	9.2 KN/m
Wall Loading(parapet)	4.37 KN/m
Seismic Loading	**Conditions**
Zone	V
Soil Type	Medium
Importance Factor	1
Response reduction	5
Wind Loading Conditions	
Wind Speed	47m/s
Terrain Category	1
Structure Class	C
Topography factor (k3)	1.2

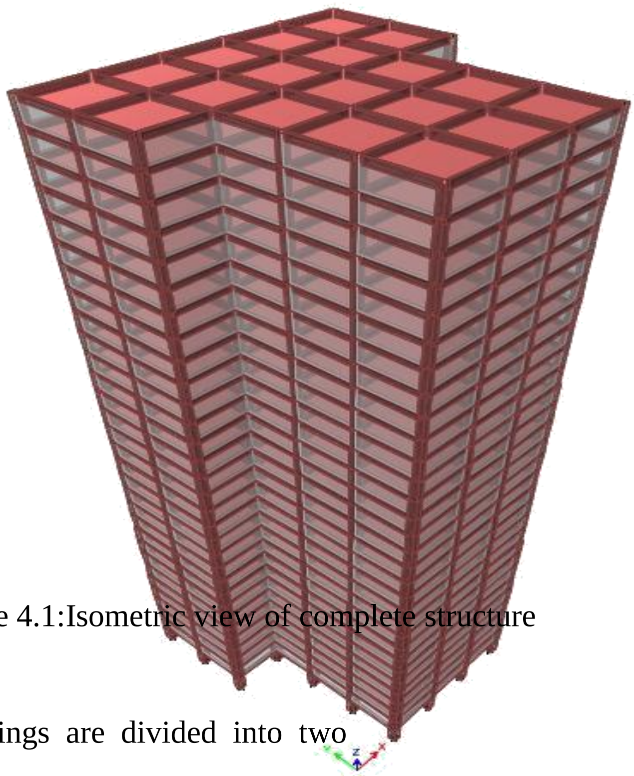
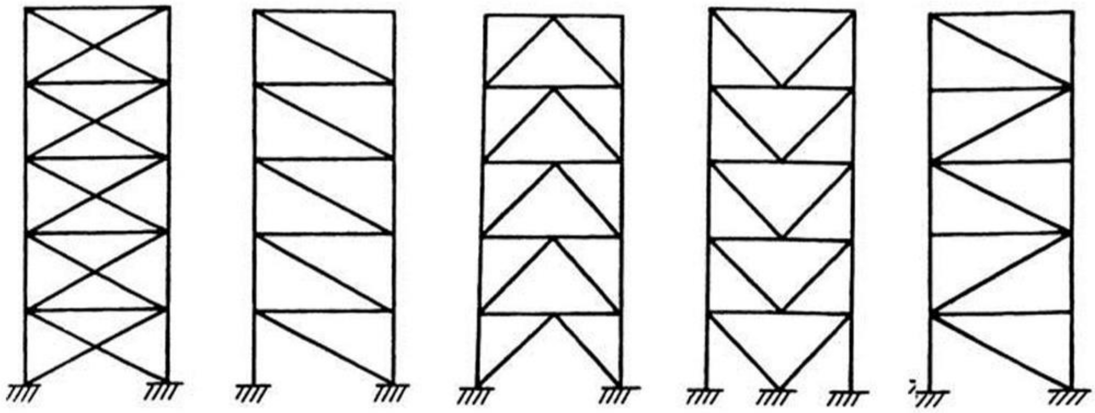


Figure 4.1:Isometric view of complete structure

5.0 Types of bracings:

5.1 Geometrical View: In geometrical view, bracings are divided into two categories- concentric bracings and eccentric bracings.

5.1.1 Concentric bracings: In concentric braced frames columns, beams, and braces intersect at a common point such that the member forces are axial. They accommodates fully triangulated vertical truss with combination of beams, braces and columns. These bracings are generally used where no obstructions in serviceability (considering architectural view) of building and they are most efficient systems. The vertical truss system restricts in bending and suitable for gravity and lateral loading. Various types are X, Chevron (V-shape and Inverted V-shape), Single diagonal bracings.



(a) (b) (c) (d) (e)

Figure 1.4: Concentric bracings (a) X bracing, (b) Single diagonal bracing, (c), (d) Chevron bracing, (e)

Single diagonal in alternate directions

5.1.2 Eccentric bracings: Eccentric bracings are not fully triangulated vertical truss and they are generally used where are more obstructions in serviceability, generally door, window openings and other obstructions. They are not efficient of lateral resisting system when compared to concentric braced frames. Types of bracings are Knee bracing and bracing layout without forming complete triangular truss.

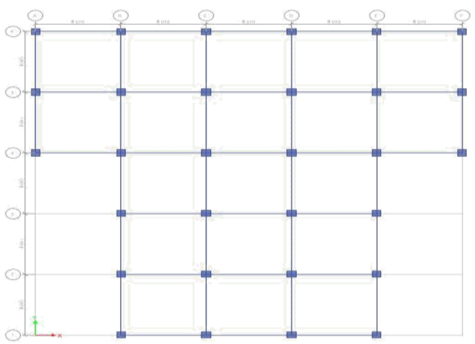


Figure 4.1: Top view of Model 1(Without steel bracing)

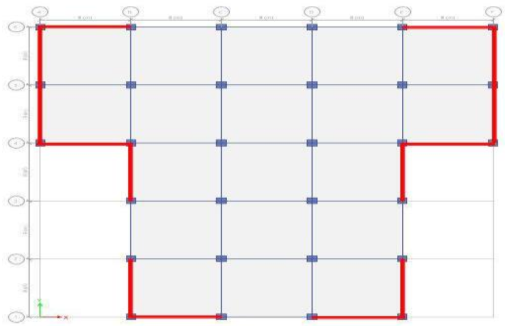


Figure 4.2: Top view of Model 2 (X bracing), Model 3 (Inverted V bracing) and Model 4 (Single Diagonal bracing) in highlighted part

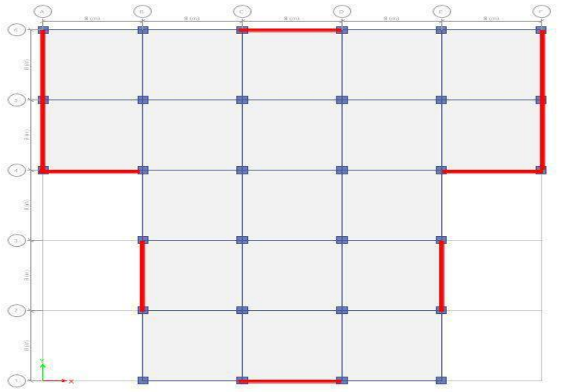


Figure 4.3: Top view of Model 5 (X bracing), Model 6 (Inverted V bracing) and Model 7 (Single Diagonal bracing) in highlighted part

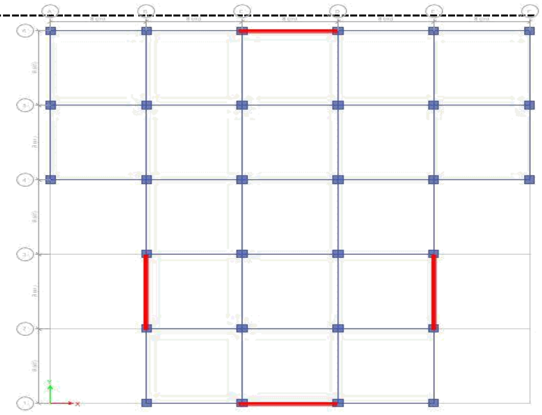


Figure 4.4: Top view of Model 8 (X bracing), Model 9 (Inverted V bracing) and Model 10 (Single Diagonal bracing) in highlighted part

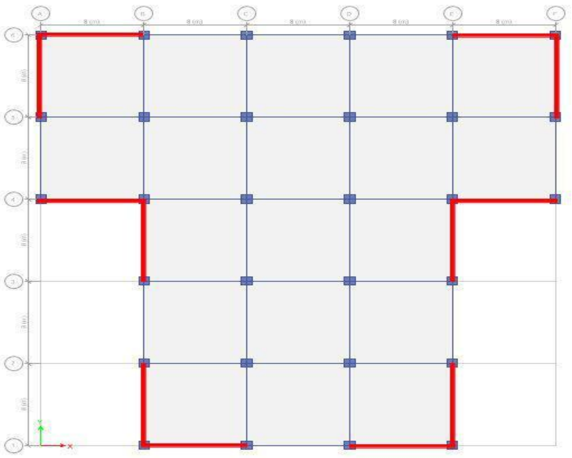


Figure 4.5: Top view of Model 11 (X bracing), Model 12 (Inverted V bracing), Model 13 (Single Diagonal bracing) and Model 14 (Knee bracing) in highlighted part

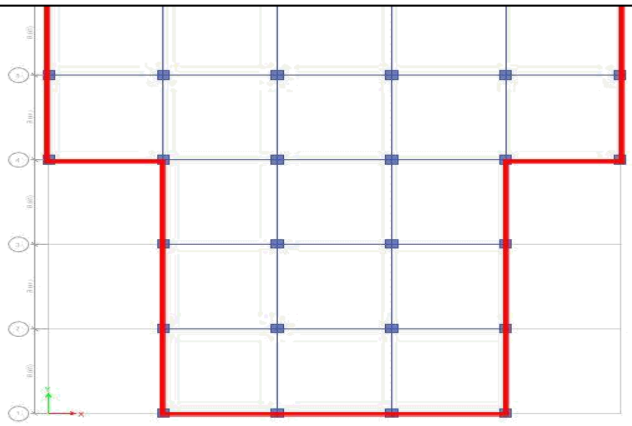
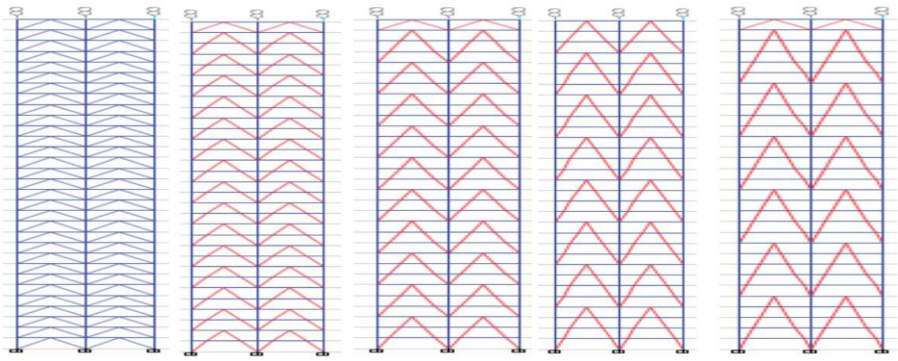


Figure 4.6: Top view of Model 15 (X bracing), Model 16 (Inverted V bracing), Model 17 (Single Diagonal bracing) and Model 18 (Knee bracing) in highlighted part



(a) ***(b)*** ***(c)*** ***(d)*** **(e)**

Figure 4.7: Elevation view of five types of models (a) Type-1, (b) Type-2, (c) Type-3, (d) Type-4,(e) Type-5.

6.0 Test Results and Discussions

Results of the experimental investigations conducted are presented below:

6.1. Response Spectrum Analysis:

6.1.1. Base Shear:(kN)

Table 6.1: Comparison of all models considering reduction (%) comparing bare frame along Y direction.

MODEL NO.	TYPE-1	TYPE-2	TYPE-3	TYPE 4	TYPE 5
(1)BARE FRAME					
1	-	-	-	-	-
(2)ADEQUATE STIFFNESS					
2	-20.73	-23.57	-18.76	-12.89	-7.2
3	-32.58	-13.53	-5.2	-3.61	-2.7
4	-6.38	-8.2	-5.38	-4.04	-3.12
5	-11.68	-13.99	-10.17	-6.38	-4.78
6	-14.64	-6.48	-3.92	-2.72	-2
7	-4.7	-5.03	-4.04	-1.09	-2.37
8	-2.95	-3.11	-2.6	-2.1	-1.64
9	-3.15	-2.14	-1.36	-0.98	-0.75
10	-1.63	-1.72	-1.39	-1.09	-0.85
11	-13.48	-6.33	-3.93	-2.76	-2.07
12	-10.74	-12.81	-9.28	-6.19	-4.71
13	-4.68	-4.91	-3.97	-3.07	-2.36
(3)FULL STIFFNESS					
14	-33.02	-50.84	-32.07	-22.91	-15.55
15	-37.67	-35	-10.1	-4.67	-3.56
16	-11.97	-27.77	-11.57	-5.47	-3.91

From above comparative table we can observe that:

Base shear is maximum in type-1 group compared to model 1 and comparatively decreases from type-2 and reaches minimum in type-5.

Providing more stiffness leads to maximum of base shear in type-2 group respect to X and single diagonal bracing system.

Base shear decreases maximum in type-5 of -1.64% (X-bracing-model-8), -0.85% (single diagonal bracing-model-10) and -0.75% (inverted V bracing-model-9).

6.1.2. Storey Torsion:(kN-m)

Table 6.2: Comparison of all models considering reduction (%) comparing bare frame along Y direction at the base.

MODEL NO.	TYPE 1	TYPE 2	TYPE 3	TYPE 4	TYPE 5
(1)BARE FRAME					
1	-	-	-	-	-
(2)ADEQUATE STIFFNESS					
2	-20.73	-23.57	-18.76	-12.89	-7.2
3	-32.58	-13.53	-5.2	-3.61	-2.7
4	-5.93	-7.41	-4.34	-2.77	-1.69
5	-11.68	-13.99	-10.17	-6.38	-4.78
6	-14.64	-6.48	-3.92	-2.71	-2
7	-4.31	-4.22	-3.05	-0.12	-1.33
8	-2.95	-3.11	-2.6	-2.1	-1.64
9	-3.15	-2.14	-1.36	-0.98	-0.75
10	-1.32	-1.06	-0.51	-0.12	0.13
11	-13.48	-6.33	-3.93	-2.76	-2.07
12	-10.74	-12.81	-9.28	-6.19	-4.71
13	-4.17	-3.89	-2.6	-1.5	-0.72
(3)FULL STIFFNESS					
14	-43.99	-65.92	-32.07	-22.91	-15.55
15	-49.29	-48.5	-10.11	-4.67	-3.56
16	-19.56	-40.55	-9.1	-2.76	-1.09

From above comparative table we can observe that:

Providing more stiffness leads to more torsional effect and torsional effect decreases from type-3 considering X and inverted V bracing.

Storey torsion decreases maximum in type-5 of -1.64% (X-bracing-model-8), 0.13% (single diagonal bracing-model-10) and -2.64% (inverted V bracing-model-9).

6.1.3. Top storey Displacements:(mm)

Table 6.3: Comparison of all models considering reduction (%) comparing bare frame along Y direction (DL+EQY).

MODEL No	TYPE 1	TYPE 2	TYPE 3	TYPE 4	TYPE 5
(1)BARE FRAME					
1	-	-	-	-	-
(2)ADEQUATE STIFFNESS					
2	39.29	40.97	38.64	35.66	32.96
3	38.92	36.03	29.52	21.79	16.39
4	28.77	26.26	21.14	15.27	10.52
5	38.18	39.39	37.62	34.45	29.61
6	39.76	35.29	26.16	19.27	14.62
7	25.61	24.67	19.27	4.56	10.15
8	14.43	15.46	14.06	12.2	10.52
9	16.01	12.85	9.31	6.8	5.03
10	8.85	8.47	6.42	4.56	3.07
11	35.1	31.47	23.74	17.6	13.31
12	33.89	34.73	33.52	30.63	26.63
13	22.91	21.88	17.04	12.48	8.75
(3)FULL STIFFNESS					
14	43.48	45.53	43.02	40.04	36.5
15	45.81	40.22	34.08	27.56	21.32
16	30.91	27.75	22.53	18.72	13.22

From above comparative table we can observe that:

Displacement decreases maximum in type-2 of 45.53% (X-bracing-model-14), type-1 of 30.91% (single diagonal bracing-model-16) and 45.81% (inverted V bracing-model-15). Considering adequate stiffness displacement decreases maximum in type-2 of 40.97% (X-bracing-model-2), type-1 of 28.77% (single diagonal bracing-model-4) and 38.92% (inverted V bracing-model-3).

Table 6. 4: Comparison of all models considering reduction (%) comparing bare frame along Y direction (DL-WLY).

MODEL No	TYPE 1	TYPE 2	TYPE 3	TYPE 4	TYPE 5
(1)BARE FRAME					
1	-	-	-	-	-
(2)ADEQUATE STIFFNESS					
2	53.45	48.57	44.93	39.66	34.31
3	50.58	40.43	30.36	22.54	17.2
4	30.13	29.2	23.39	17.66	12.94
5	40.28	42.37	39.12	34.39	29.74
6	43.14	35.32	26.34	19.52	14.87
7	26.18	25.56	20.6	5.5	11.62
8	15.18	16.11	14.72	12.78	11
9	16.65	13.4	9.76	7.2	5.34
10	9.6	9.37	7.36	5.5	4.03
11	38.73	32.15	24.4	18.2	13.94
12	36.17	37.96	35.24	31.29	27.34
13	24.01	23.39	18.82	14.33	10.61
(3)FULL STIFFNESS					
14	55.23	58.02	54.14	48.41	42.45
15	58.4	48.88	37.57	28.58	22.23
16	36.95	35.71	28.89	22.08	16.42

From above comparative table we can observe that:

Displacement decreases maximum in type-2 of 58.02% (X-bracing-model-14), type-1 of 36.95% (single diagonal bracing-model-15) and 58.4% (inverted V bracing-model-16).Considering adequate stiffness displacement decreases maximum in type-1 of 53.45% (X-bracing-model-2), 30.13% (single diagonal bracing-model-4) and 50.58% (inverted V bracing-model-3).

6.1.4. Maximum storey drift:(m)

Table 6.5: Comparison of all models considering reduction (%) comparing bare frame along Y direction (DL+EQY).

MODEL NO	TYPE 1	TYPE 2	TYPE 3	TYPE 4	TYPE 5
(1)BARE FRAME					
1	-	-	-	-	-
(2)ADEQUATE STIFFNESS					
2	43.63	45.43	42.12	37.8	33.98
3	43.63	39.16	32.33	23.18	16.41
4	31.46	28.37	22.75	14.9	9.29
5	41.68	43.27	40.6	36.14	30.09
6	43.48	37.44	27.57	19.22	13.82
7	27.72	26.49	20.73	4.68	9.07
8	17.93	19.29	17.93	15.19	11.88
9	19.8	15.33	11.09	7.7	5.18
10	10.37	9.65	7.27	4.68	2.16
11	39.67	34.49	26.35	18.72	13.1
12	38.08	39.24	37.15	33.05	27.79
13	25.13	23.47	18.72	11.59	7.06
(3)FULL STIFFNESS					
14	48.02	50.18	46.51	41.83	36.65
15	50.47	43.27	36.07	27.72	20.73
16	33.26	28.65	21.17	14.97	8.42

From above comparative table we can observe that:

Storey drift decreases maximum in type-2 of 50.18% (X-bracing-model-14), type-1 of 33.26% (single diagonal bracing-model-16) and 50.47% (inverted V bracing-model-15).Considering adequate stiffness storey drift decreases maximum in type-2 of 45.43% (X-bracing-model-2), type-1 of 31.46% (single diagonal bracing-model-4) and type-2 of 43.63% (inverted V bracing-model-3).

Table 6.6: Comparison of all models considering reduction (%) comparing bare frame along Y direction (DL-WLY).

MODEL NO	TYPE 1	TYPE 2	TYPE 3	TYPE 4	TYPE 5
(1)BARE FRAME					
1	-	-	-	-	-
(2)ADEQUATE STIFFNESS					
2	59.07	55.2	50.83	44.36	36.78
3	57.13	45.91	34.4	24.28	18.14
4	34.96	34.18	27.54	19.91	14.44
5	46.07	48.62	44.41	38.11	31.19
6	49.39	39.49	29.09	19.86	14.71
7	29.65	29.31	23.84	6.36	12.17
8	20.96	22.51	20.02	16.65	13.05
9	23.17	17.53	12	7.85	5.64
10	12.17	12.11	9.51	6.36	4.37
11	46.68	38.16	28.21	19.47	14.38
12	43.69	46.02	42.2	36.67	30.03
13	28.37	27.82	22.62	15.82	11.28
(3)FULL STIFFNESS					
14	60.56	63.27	58.74	51.6	43.58
15	63.77	53.04	40.21	29.31	22.4
16	40.71	39.16	30.59	22.01	15.6

From above comparative table we can observe that:

Storey drift decreases maximum in type-2 of 63.27% (X-bracing-model-14), type-1 of 40.71% (single diagonal bracing-model-16) and 63.77% (inverted V bracing-model-15).Considering adequate stiffness storey drift decreases maximum in type-1 of 59.07% (X-bracing-model-2), 34.96% (single diagonal bracing-model- 4) and 57.13% (inverted V bracing-model-3).

6.1.5. Maximum storey stiffness:(kN/m)

Table 6.20: Comparison of all models considering increase (%) comparing bare frame along Y direction.

MODEL NO	TYPE 1	TYPE 2	TYPE 3	TYPE 4	TYPE 5
(1)BARE FRAME					
1	-	-	-	-	-
(2)ADEQUATE STIFFNESS					
2	74.94	84.49	79.28	61.69	49.08
3	88.25	60.64	37.62	24.04	16.07
4	41.06	47.39	41.87	33.78	26.53
5	57.5	64.8	60.43	47.13	37.49
6	66.8	46.42	28.72	18.36	12.27
7	31.09	35.66	31.3	8.81	19.79
8	19.97	22.52	20.66	16.31	13.06
9	23.5	16.19	10.02	6.41	4.27
10	10.85	12.44	10.89	8.81	6.95
11	65.82	46.12	28.66	18.36	12.27
12	56.26	63.44	59.38	46.69	37.25
13	30.84	35.54	31.35	25.36	19.96
(3)FULL STIFFNESS					
14	94.9	106.67	100.23	76.96	60.87
15	108.37	75.28	46.5	29.71	19.86
16	51.72	59.9	52.93	42.62	33.29

From above comparative table we can observe that:

Storey stiffness increases maximum in type-2 of 106.67% (X-bracing-model-14), 59.9% (single diagonal bracing-model-16) and type-1 of 108.37% (inverted V bracing-model-15).Considering adequate stiffness storey drift decreases maximum in type-2 of 84.49% (X-bracing-model-2), 47.39% (single diagonal bracing-model- 13) and type-1 of 88.25% (inverted V bracing-model-3).

7.0 Conclusions

1. Providing bracing influences global performance of the structure. Providing more stiffness by bracings leads to reduces ductile nature, time period and increases torsional effect. Providing adequate stiffness and continuous load path provides better results with good ductility and reduces torsion.
2. Base shear is maximum by connecting bracing within the storey (type 1) and reduces when connecting more than one storey. More the connectivity between storeys, storey shear reduces and ductility increases. Single diagonal bracing controls maximum storey shear and helps increase of ductility.
3. Storey displacement reduces maximum in type 1 along x-direction and type 2 along Y-direction. Displacement increases when connecting of bracing more than one storey

along x direction and more than two storeys along y-direction. Storey displacements are controlled by X bracing both along X and Y direction.

4. Storey drift reduces maximum in type 1 along x-direction and type 2 along Y-direction. Drift control decreases when connecting of bracing more than one storey along x direction and more than two storeys along y-direction. Maximum storey drift are controlled by X bracing both along X and Y direction.
5. Storey stiffness is maximum in type 1 and minimum in type 5 compared to bare frame. If connecting of bracing between the storey increases stiffness decreases. Inverted V bracing provides maximum stiffness in type 1 and rest X bracing provides maximum results.
6. Eccentricity is maximum in type 1 and minimum in type 5. If connecting of bracing between the storey increases eccentricity decreases and torsion effect also decreases. Single diagonal bracing controls maximum eccentricity in type 5 group.
7. Storey torsion is maximum when bracing is connected within the storey (type 1) and reduces when connecting increases more than one storey. Single diagonal bracing provides good results in control of torsion.
8. Time period is maximum in type 5 and minimum in type 1 because when storey shear increases time period decreases and storey shear is maximum in type 1.
9. Providing bracing where the structure component is weak (such as tension capacity is less at re-entrant corners) provides good stiffness and should be careful with maintaining good ductile nature. In the present models, type-5 provides maximum ductile nature but reduces stiffness compared to type-1 group. So, providing adequate stiffness and considering ductile nature is most important aspect.

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