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EXPERIMENTAL STUDIES ON BEHAVIOUR OF INTERFACE SHEAR STRENGTH OF FLY ASH BASED & GEOSYNTHETICALLY REINFORCED GEOPOLYMER CONCRETE (GSRGPCC)

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ABSTRACT

This paper describes the experimental investigations conducted to study the impact of amount of Total Aggregates (i.e.CA and FA) and impact of incorporation of Geosynthetics fibres on the interface shear strength of GPCC and to compare that with Ordinary Portland Cement Concrete Composites. It has also been proposed to check the suitability of existing empirical formulae used for OPCC for the assessment of interface shear strength capacity of Geosynthetically reinforced GPCC. For this investigation Push- Off specimens were casted and tested to study interface shear behaviour of GPCC. Both reinforced and un-reinforced concrete specimens were prepared for this purpose .It has been observed that the interface shear strength of GPCC is little bit inferior than the corresponding OPCC and the amount of Total Aggregates less than approximately 65% in GPCC may reduces the interface shear strength capacity drastically .Hence while designing GPCC for its interface shear strength capacity , it is proposed to calculate it by adopting a strength reduction factor £ ~ ranging from 30 to 40 % w.r.t .OPCC , if it is to be predicted from the empirical equations that are available for OPCC; provided that the Total Aggregates contents are up to & > 65%.

Keywords: Geosynthetically Reinforced Geopolymer Concrete (GSRGPC/GPCC), Interface Shear Strength, Low calcium Class F Fly Ash(ASTM Type), Alkaline Activated Solution, Super-Plasticizer, Molarity, Aspect ratio, GGR fibres, volume fractions (V_f), Strength Reduction Factor £ , etc..

I INTRODUCTION

Today's world is a concrete world for global construction industries. While in manufacturing process of cement, the Global cement industries contributes more than @ 1.35 billion tonnes of Green House Gas (*GHG)* emissions annually & approx. 7% of total man made *GHG* emitted into the earth atmosphere [1-2].Also during the manufacturing of cement, a very large amount of CO_2 gas is emitted into the atmosphere. This CO_2 emission increases the temperature and resulted in an increase in the overall Global Warming. Therefore in very near future, there will be an adverse impact on both cement productions and construction industries. Although the use of Portland cement is still unavoidable, many efforts are being made in order to reduce the use of OPC in conventional concretes. J.Davidovits [3] has invented a new technology with its chemistry and terminology called "**Geopolymer**", in which cement is 100% replaced by fly ash and activated by alkaline solutions. Geopolymers thus forms the Three-dimensional disordered frameworks of the tecto-aluminosilicate types with the general empirical formula as…

$$\mathbf{M\ n\ [-\ (SiO_2)_Z\ \text{-}AlO_2]\ n.wH_2O.............. \ (1),}$$

where n is the degree of polycondensation, M is predominantly a monovalent cations [like K+, Na] though Ca^{2+} may replace two monovalent cations in the structures[4].Depending on the ratio of Silica to Alumina, there could be geopolymer with either (Si-O-Al) or (Si-O-Si)bond [5-7]. Review of literature shows that Fly ash, metakaolin, rice husk ash, red mud etc...are generally used as alumino-silicate materials and the alkaline solutions include sodium hydroxide, potassium hydroxide, sodium silicate, calcium silicate etc..[8-12].GP concrete is best suited for precast constructions. However, the connective distress found in precast construction, is centered around the shear interfaces (a place where shear stress causes sliding type of failure along a well defined plane called 'shear plane') associated with corbels, bearing shoes, ledger beam bearings, coupled shear walls, wall to foundation, deep beams etc.[13-15]. Study of shear- slippage at the interface of both monolithic and precast construction is very important in such instances. Studies have been conducted in the past to understand the interface shear strength in OPCC. Birkeland et al. [13] proposed a 'shear friction concept' to evaluate the interface shear strength of concrete blocks. Their hypothesis suggests that the external shear load tends to produce slippage along the interface plane and it is resisted by the shear friction and not by bond. They further proposed that, the reinforcement across the interface is stressed in tension and that the dowel action is insignificant. Accordingly, the ultimate shear capacity across the interface of a monolithic concrete with reinforcement across the shear plane has been calculated by ….

$$\mathbf{Ast.fy.tan\text{ø}.....\ (1)}$$

where Ast and fy are the total cross sectional area of the reinforcement across the shear plane and yield strength of reinforcement respectively. The angle of internal friction **ø** varies with the nature of interface and is to be determined by tests. They have suggested a value of 1.7 for 'tan ø'. Mast [14], based on the experimental study on monolithic concrete and concrete having crack at the interface, has suggested that the value of 'tan ø' ranges between 1.4 and 1.7. He proposed a lower bound value of 1.4 for design purposes. Hofberck [15] reported a study on the shear strength of reinforced concrete with and without a crack existing along the shear plane of push-off specimens and

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concluded that shear transfer stress depends on initial crack condition, product of reinforcement ratio and yield strength of shear reinforcement. It is suggested that, the dowel action of reinforcing bars crossing the shear plane is insignificant in initially uncracked concrete, but it is substantial in concrete with a pre-existing crack along the shear plane.

The shear-friction design proposed by ACI suggested the value of coefficient of friction (μ or tanø) for monolithically placed concrete as 1.4 λ, where the value of λ for normal weight concrete is one [15]. The value of λ depends on the type of concrete; namely normal weight (λ =1), sand light weight (λ = 0.85) and all lightweight (λ = 0.75).On the basis of experimental investigations using push-off specimen, Mattock [16] proposed an alternate equation for predicting the ultimate interface shear capacity**,** given by….

Vu = 0.8 (Ast x fy) + (Ac x 400 psi)……….(2).

Mattock [17] and Lawrence [18] have conducted experimental research and proposed modification to the ACI equation [15] to predict the interface shear strength of high strength concrete. Alan H Mattock and Zuhua Wang [19] studied the shear carrying capacity of RC beams subjected to axial force. A total of 38 beams were tested in this study with axial force varying up to $0.7f_c'$. These methods of calculation of strength were found to be conservative for all the members tested and very conservative in some cases. Modifications to the ACI Building Code procedures for the shear design of members subject to axial force are proposed to reduce unnecessary conservatism and to safeguard against shear failure primarily due to inclined compression failure of the web concrete. Sarkar S, Adwan O and Bose B [20], studied the strength and the failure modes of normal reinforced high strength concrete shear critical beams. Based on the test results of six RC beams of dimension 150×250×1505 mm (effective span) with varying concrete strength between 40 MPa and 110 MPa, an empirical model given was proposed for predicting the shear strength of concrete beams. It was pointed out that the dowel action of the reinforcement contributes significantly along with the aggregate interlock mechanism to resist shear in the reinforced concrete beams. Paul Y. L. Kong and Vijaya Rangan B.[21], carried out an experimental investigation on the shear performance of RC beams with transverse reinforcement. The test results of 48 high strength RC beams were presented in this report. Based on the test results, it was concluded that the beam strength does not increase with higher values of (a/d) ratio > 2.5); while a significant increase in beam strength can be observed for a shear span ratio (a/d) < 2.5. The test data showed a large scatter from the predicted shear strength based on the code provisions. This indicated that the code provisions needs to amendment to accommodate the test data of high strength concrete beams. ASCE-ACI Committee 445 [22], reviewed the literature on the concepts and development of various approaches proposed for evaluating the shear capacity of the structural concrete member. The concepts and methodology of compression field approach, truss-model approach and various empirical methods have been described in this literature. Realistic models for predicting the shear strength of corbels, and deep beams have been illustrated in this report. The details of 175 published literatures have been referred in this report. AS 3600-1994 [23] yield reasonable shear strength prediction for high strength concrete beams. Eric J. Tompos and Robert J. Frosch [24]**,** tested six RC beams under shear loading to study the influence of beam size, longitudinal steel reinforcement ratio and the effectiveness of the stirrups. The test results indicated that the current ACI shear design provisions "**ACI 318- 1999 [25] "**provided un-conservative results for large beams and beams with the low levels of longitudinal steel reinforcements. The close stirrup's provided required development length and hence beams with closed stirrup's provided required development length and hence beams with closed stirrups showed higher shear strength when compared with the beams with U-shaped stirrups. Satish B. Desai [26] investigated the influence of additives such as lime stone filler, pulverized fuel ash, ground granulated blast furnace slag and its combinations on the shear behavior of RC beams. The test results indicated that the code provisions of shear strength computation ACI 318-2002 [27] of RC beams and slabs yield overlay conservative results for modified concrete beams. It was pointed out that, it is appropriate to evaluate the nominal shear strength of concrete in terms of its splitting tensile strength, which will accommodate the variations in the concrete mixture to some extent. Benny Joseph & George Mathew [28] studied the interface shear strength of low calcium Fly Ash based Geopolymer Concrete, based on the their experimental studies, they have concluded that the interface shear strength of GPCC is inferior to that with the corresponding OPC concrete and also that the amount of Aggregates less than approximately 65% in GPCC; reduces the interface shear strength capacity drastically. The calculations of shear strength have been mostly on TVC. In this regard, the interface shear strength of concrete depends on various parameters such as type of concrete, type of aggregate, cohesive strength of concrete, percentage of reinforcement across the shear plane, etc...However, the study on shear transfer strength of geopolymer concrete has not been reported in literature. Hence, it has been proposed to carry out an experimental investigation to study the interface shear behavior of Geopolymer Concrete with Geosynthetically reinforced Composites so as to enhance the ductility and flexibility of Geopolymer Concrete Composites.

II EXPERIMENTAL PROGRAMME

2.1 Materials

Cement:-53 Grade Ordinary Portland Cement conforming to BIS: 269[36] was used in the present study.

Fly ash:- Low calcium fly ash (ASTM Class F), having a specific gravity of 2.3, was used as the alumino-silicate as a source material for making geopolymer binder. The chemical composition of fly ash, as determined by XRF analysis, is shown in **Table 1.**The important characteristics of fly ash, which helps to improve the flowability and reduces the water demand is its shape and size.. The fly ash used in this experimentation is obtained from the silos of Thermal Power Station situated at Parali -Vaijanath, Dist. Beed, Maharashtra State, India; which is of low calcium, Class F-ASTM type. The chemical composition of fly ash is shown in Table 1, obtained from test reports of CSRL Structwel Lab. Pune Pvt. Ltd., Maharashtra State, India.

Table 1: Chemical Composition of Fly Ash

Sr. No	Chemical constituents in % or Type of Test	Percentages As per Test Results	Requirement as per IS:3812-2003 [32]			
			Part-I		Part-II	
			Siliceous fly ash	Calcareous fly ash	Siliceous fly ash	Calcareous fly ash
1	Silica content SiO_2	61.49	35.0 min.	25.0min.	35.0 min.	25.0min.
2	Alumina content(Al_2O_3)	31.34	----	----	----	----
	Ferric Oxide(Fe_2O_3)		----	----	----	----
3	Silica+ Alumina +Ferric Oxide	92.83	70 min.	50min.	70min.	50min.
4	Calcium Oxide(CaO)	1.92	----	----	----	----
5	Magnesium Oxide(MgO)	0.56	5max.	5max.	5max.	5max.
6	Sulphur Tri –Oxide(SO_3)	0.51	3max.	3max.	5max.	5max.
7	Loss on Ignition(LOI)	0.49	5max.	5max.	7max.	7max.
8	Chloride	----	0.05max.	0.05max.	0.05max.	0.05max.

Alkaline Activated Solution (AAS):- A mixture of NaOH and Na_2SiO_3 solution (SiO_2 = 34.31%, Na_2O= 16.37%, water 49.32%) was used as AAS having details as follow--

1] **Sodium Hydroxide (NaOH)-** (Laboratory Grade):- in flake or pellets/solid form and its cost is mainly varied according to its purity and its main function is to activate the Sodium Silicate Solution ;hence it is better to use it within economical purity i.e. up to the purity range from 94% to 96%.The concentrations of NaOH used was of 13 Molar as per previous research[36].

2] S**odium silicate (NA_2SiO_3) solution-** in gel form (known as water glass) with a ratio of oxides between SiO_2 to Na_2O = 2 was used ;which were silicates & which are supplied to the detergent company and textile industries as bonding agent; same type of silicates were used for this investigation for making Geopolymer concrete. The Chemical **C**ompositions of both NaOH and Na_2SiO_3 are presented in Table 2. The specific gravity of the made up solution was 1.54.

Table 2: Chemical Compositions of NaOH and Na_2SiO_3

Chemical Compositions of NaOH (Min. Assay)	**Percentages 97%**		**Chemical compositions of Sodium Silicate (Na_2SiO_3) Solution Colour-light yellow liquid**	**Percentages**
Carbonate(Na_2O_3)	2		Na_2O (%)	16.37%
Chloride(Cl)	0.01		SiO_2 (%)	34.31
Sulphate(SO_2)	0.05		Ratio of Na_2O: SiO_2	1:2.09
Potassium(K)	0.1		Total Solids (%)	50.68
Silicate(Si_2O_3)	0.05		Water content H_2O (%)	49.32
Lead(Pb)	0.001		Appearance	Liquid(Gel)
Zink(Zn)	0.02		Boiling point	102^0C for 40% aq.soln
Specific gravity	1.16		Specific gravity	1.57
Colour	Colourless		Molecular Weight	184.04

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Aggregates:-F.A and C.A:- Locally available Godavari (Paithan) river sand as fine Aggregates and Crushed Basaltic stones (from stone crushers near outskirts of Aurangabad city of Waluj premises) were used as F.A and C.A.respectively with C.A. of NMS as 20 mm and 12.5 mm having their physical properties as shown in Table 3 and their gradings are shown in Table 4 and 5 respectively.

Table 3: Physical Properties of Aggregates (FA &CA) [33]

Physical Properties	Coarse Aggregates(CA)		Fine Aggregates(FA)
	C.A-I	C.A-II	F.A (Sand)
Type	crushed	Crushed	Godavari sand
Maximum size	20mm	12.5mm	4.75mm
Specific Gravity	2.641	2.639	2.563
Water Absorption	0.58%	0.84%	1.58%
Moisture content	Nil	Nil	Nil

Table 4: Gradings of CA {in Saturated Surface Dry- conditions}

Sr. No.	IS Sieve sizes in mm	Cumulative percentage Passing for			
		CA-I 20 mm	CA-II 12.5 mm	Combined Aggregates CA-I CA-II 65:35	Required grading as per BIS 383:1970 [33]
1	40	100	100	100	100
2	25	100	100	100	---
3	20	87.50	100	89.90	90-100
4	16	6.80	100	39.42	--
5	12.5	0.40	96.5	35.04	--
6	10	0.00	76.40	27.75	25-35
7	4.75	0.00	0.96	0.35	0-10
8	2.36	0.00	0.00	0.00	---

Table 5: Grading of FA {Saturated Surface Dry- conditions}[33]

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Sr.No.	IS Sieve size designation mm	Cumulative percentage Passing for Godavari river sand	Remarks
1	10	100	FM = 3.74, Confirming to Zone II, IS 383-1970 [33] Specific Gravity-2.61
2	4.75	92	
3	2.36	84.80	
4	1.18	59.90	
5	600µ	35.30	
6	300µ	10.60	
7	150µ	0.62	
8	75µ	0.10	
Total		374.32	

Water: - Potable water which is available in laboratory was used for mixing purposes.

Geosynthetics Fibres (Tensar SS40-Biaxial Type):- Since Geopolymer cement concrete is more brittle than conventional concrete, in these investigations Geosynthetics fibres (GGR Types only) are used to convert GPCC into more ductile or elastic/flexible one. Geogrids fibres with aspect ratio (Lf/ hf= 35.54/1.27 =28) 28 to 50 is used with modulus of elasticity (E) in the range of 12000 -18000 MPa {MN/m^2} say avg. E = 15000 MPa, Specific Gravity ranging from 1.22-1.38, melting point @ 260^0C., unit mass or weight/unit area-345-930 g/m^2, thickness ranges from 10-300 mils or generally 20 mils i.e.0.025 mmx20 = 0.5mm (since 1 mil=0.025mm).

Super-plasticizers:- Sikament 610 ut.250 Kg (Naphthalene based) was used as super plasticizer/ an admixture as per actual requirements of workability.

2.2 Mixture Proportioning:- All in aggregates were prepared by bringing them in Saturated Surface Dry (SSD) conditions. All the GPCC and OPCC samples were made with mix design procedures as follows and mentioned in [36] all respects as far as possible.

1] All ingredients of GPCC/ OPCC are collected separately and mixed in the Pan Mixture for @ 5 minutes.

2] The Alkaline Activated solution(which was prepared one Day before)/liquid was then added to dry mixture with actual quantity of super plasticizers, as necessary for workability and then mixing was done for another 5 minutes so as to obtain a homogeneous and consistent matrix.

3] After this mixing, the flow values /slump values of fresh GPCC/OPCC was determined as per slump tests confirming to the IS: 516-1959[34] same as for OPCC in Concrete Technology.

4] After flow test, the fresh GPCC/OPCC were placed in corresponding moulds as described in the IS: 516-1959[34] and compacted by usual methods that are used in OPCC. The quantity of different constituents of the mixture has been arrived based on a preliminary study conducted and its details are presented elsewhere [31]. Accordingly, the ratio of fine aggregate to total aggregate (0.35), ratio of AAS to fly ash (0.55), molarity of NaOH (13), ratio of Na_2SiO_3 to NaOH (2.5), ratio of H_2O (water) to Geopolymer Solid (= 2.5) were kept constant in the present investigation. The total aggregate contents in the mixture were varied from 60% to 75% of the volume of GPCC in these experimentations. An OPC concrete mixture proportions w.r.t. that of GPCC references has also been arrived based on a trial and error method so as to achieve its compressive strength almost the same as that of the optimized GPCC (concrete having maximum compressive strength).Table 6 presents the total quantity of materials required to produce one m^3 of GPCC with quantity of Geosynthetics fibres, as shown in Table 7, as per previous experimentations [38]and the optimized mix proportions of both GPCC and OPCC taken in this experimentations are presented in Table No.8.

Table 6: Mix proportion of geopolymer concrete per cubic meter (Additional 10%) [38]

Fly ash	NaOH (13M)	Na_2SiO_3	FA	CA	Extra water	Total Quantity of Ingredients (kg)
310	54.25	54.25	744	1387.90	62.00	2612.401
1part	0.175 part	0.175 part	2.40 part	4.477 part	0.20 part	[AAS/FA]=108.5/310 = 0.35

Table 7: Quantity of fibres for various mixes [38]

Geosynthetics fibre (%)	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8
Fibre quantity (kg/m3)	0.0	2.613	5.22	7.84	10.452	13.06	15.678	18.286	20.899
Super plasticizer (kg/m3)	0.0	0.10	0.20	0.225	0.370	0.500	0.723	0.981	1.052

Table8: Optimized Mix proportion of geopolymer concrete per cubic meter (taken for experimentations after trials and errors)

MIX ID	Total Aggre. % by Volume	Ratio (FA /TA)	Coarse Aggregates (Kg)	SAND (F.A.) (Kg)	FLY ASH (Kg)	AAS (Kg)	Super-Plasticizer (Kg)
GPCC60	60	0.35	1063.09	581.11	431.67	282.35	9.31
GPCC65	65	0.35	1149.09	653.79	376.26	260.92	8.2
GPCC70	70	0.35	1235.18	694.09	320.98	221.47	7.12
GPCC75	75	0.35	1321.09	745.40	266.63	190.06	6.06
OPCC	0.67	0.385	1305	525	…….	……	2.03

2.3 Casting of Specimens:- For testing the GPCC/OPCC , Specimens are casted as per IS Codal requirements: Push-off specimen of size: 100 mm x 200 mm x 500 mm were cast in steel moulds, V-groves of 4mm deep were made on either sides of the specimen along the shear plane with the help of standard angles. The push-off specimens were cast with and without dowel bars. Two numbers of 8 mm diameter dowel bars, having yield strength of 440 MPa, were placed across the shear plane (0.985 %), in the form of closed link. Additionally, to prevent the premature failure at the loading points for all specimens, the 10 mm dia. bars and 8 mm dia. stirrups were provided in the specimens. The Schematic sketch of Push -Off specimen showing the dimensions and details of reinforcements is shown in Fig.1. In addition to the push-off specimens,(2) cubes for Compression tests: size (150x150 x150) mm; (3) Cylinders 150 mm diameter x 300 mm height for Split Tensile tests, (4) Beams of sizes (100 × 100) mm c/s and 500mm length with two-point loads applied at the middle third of the span for Flexural strength tests were cast in standard steel moulds to determine the required strength properties; out of which three specimens each were used to determine the corresponding Compressive Strength ,Split Tensile Strength, Flexural strengths; each at 28 days for different grades of GPCC/OPCC with and without incorporating Geosynthetics Fibres-in the form of rectangular pieces of the cube sizes within the cubes /or in the form of circular discs of cylindrical sizes at proper spacing and strips having beam c/s into the beams of beam length etc.{Only Geo-gride type fibres were used} as shown in Table 8 and corresponding reported strengths are shown in Table 9.

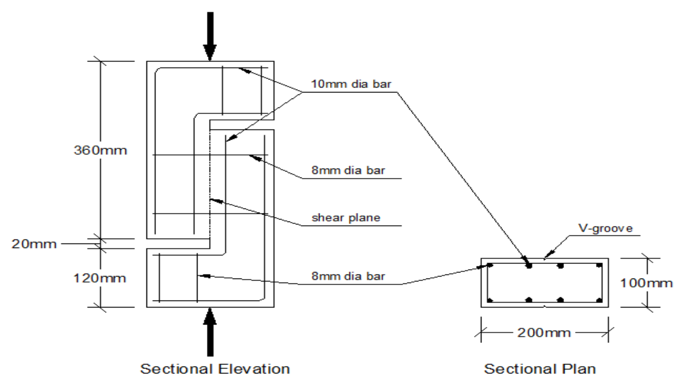


Fig.No.1: Schematic sketch (for casting) of push- off specimen with all dimensions and reinforcements details

The concrete, after placing in moulds, were compacted with the help of a table vibrator. In the case of specimen with GPC concrete, the top side of moulds was covered with a steel plate and edges were sealed properly to avoid the loss of moisture from specimens during heat curing. The GPCC specimens were subjected to heat curing in a thermostatically controlled electric oven at 90◦C for a period of 24 hours, the curing temperature and period were chosen based on preliminary studies [38]. After this the specimens were de-moulded and were kept in room temperature till the expiry of date of curing period i.e. on testing date (on 28 th day).Similarly OPCC specimens were also removed from moulds after 24 hours completion from the time of casting and then kept for water curing till the day of testing as usual in Concrete Technology.

2.4 Testing of specimen

The laboratory set up for the Push-Off specimen is shown in fig.2. Axial load was applied on push- off specimens at regular intervals until failure occurred. Average shear strength of the concrete was calculated on the basis of the area of shear plane. Two dial gauges were used to measure the relative slip at the shear plane of all the specimens.

Fig.2: Laboratory set up for the Push-off Specimen

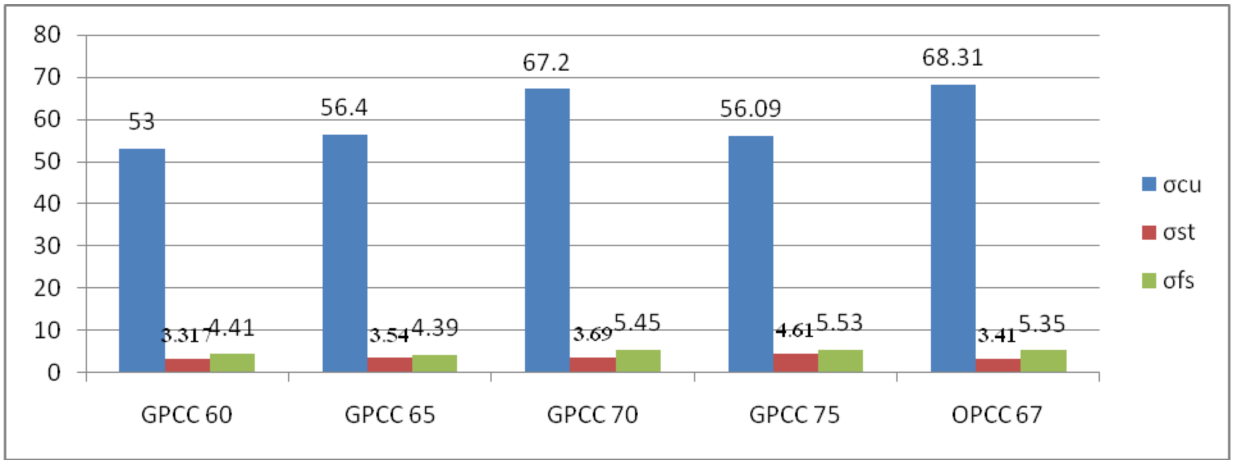
III ANALYSIS OF TEST RESULTS

The values shown in Table 9 and the various strengths calculated therein are as per the previous research [38]corresponds to the average test results of minimum three sample specimens for cube compressive, split tensile, flexural strength at 28 th days of curing completion, wherever applicable.

Table 9: Mechanical properties of GPCC and OPCC (after 28th Day curing)

MIX ID	Cube Compressive Strength(MPa)	Split Tensile Strength(MPa)	Flexural Strength(MPa)
GPCC 60	53	3.317	4.41
GPCC 65	56.4	3.54	4.39
GPCC 70	67.20	3.69	5.45
GPCC 75	56.09	4.61	5.53
OPCC (67%)	68.31	3.41	5.35

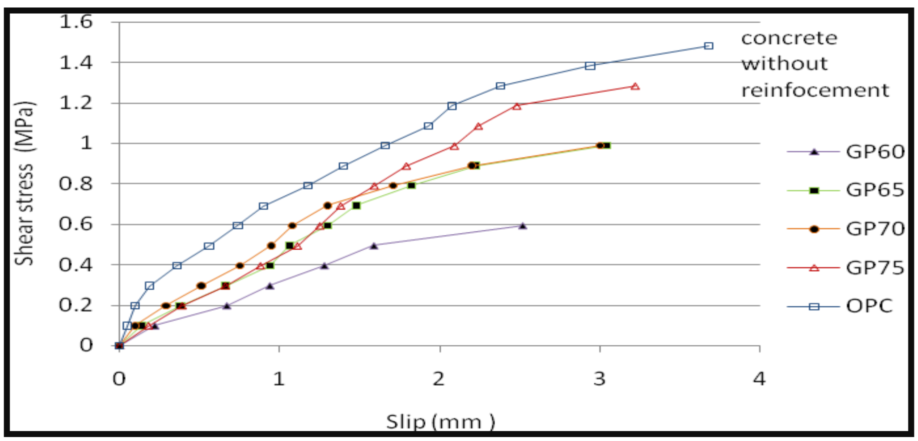
From the Table 9, it is very clear that the OPCC and GPCC 70 concretes are having almost the same cube compressive strengths while their Total Aggregate contents are 67% and 70% as depicted in Bar Chart no.1 below.



Bar Chart No.1 showing Mechanical Properties of GPCC and OPCC.

From the experimental observations of Push-Off specimens, following two graphs may be drawn…

1] The Graph 1:- represents the variation of slip with interface shear stress in push-off specimens with no/zero shear reinforcement across the shear plane -



Graph 1: Variation of slip with interface shear stress in specimen without shear reinforcement

From this graph no.1, it is clear that, for a given value of shear strength of geopolymer specimen, the slip is more with lower aggregate content. Further, as the aggregate content increases, the ultimate shear strength also increases. This is basically may be due to the improvement of the cohesive strength of concrete and better aggregate interlocking at the interface with higher percentage of aggregate content. It has been reported that, for the low steel ratio, the cohesive strength of concrete have remarkable influence on interface shear strength as observed and confirmed by [13].

2] The Graph 2:-depicts the variation of slip w.r.t. the shear stress in push- off specimen with 0.985% shear reinforcement with interface shear stress in specimen along with shear reinforcement.

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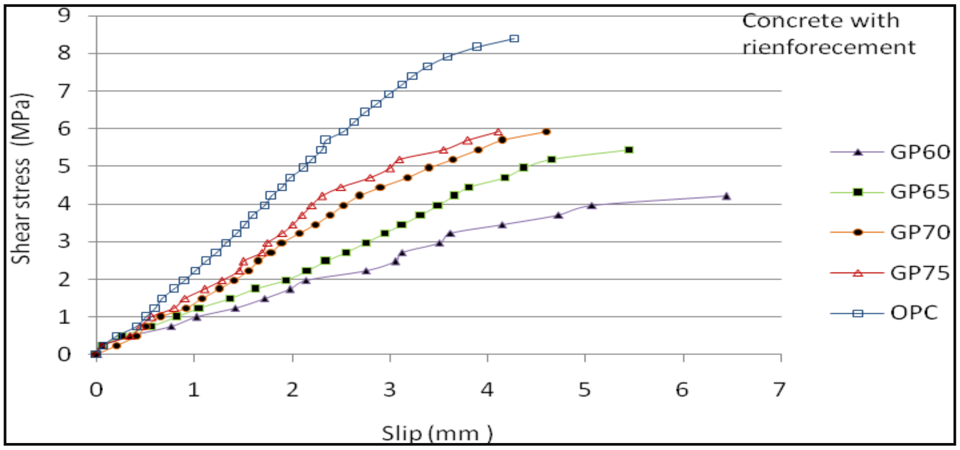


Graph 2: represents the variation of slip with shear stress in push- off specimen with 0.985 % shear reinforcement with interface shear stress in specimen with shear reinforcement.

The Graph 2 shows almost very similar behavior as that of the specimen without shear reinforcement. Hence,

1] It could be predicted that, for a given interface shear stress, a GPCC with Total aggregate content less than 65% , shows large slip values.

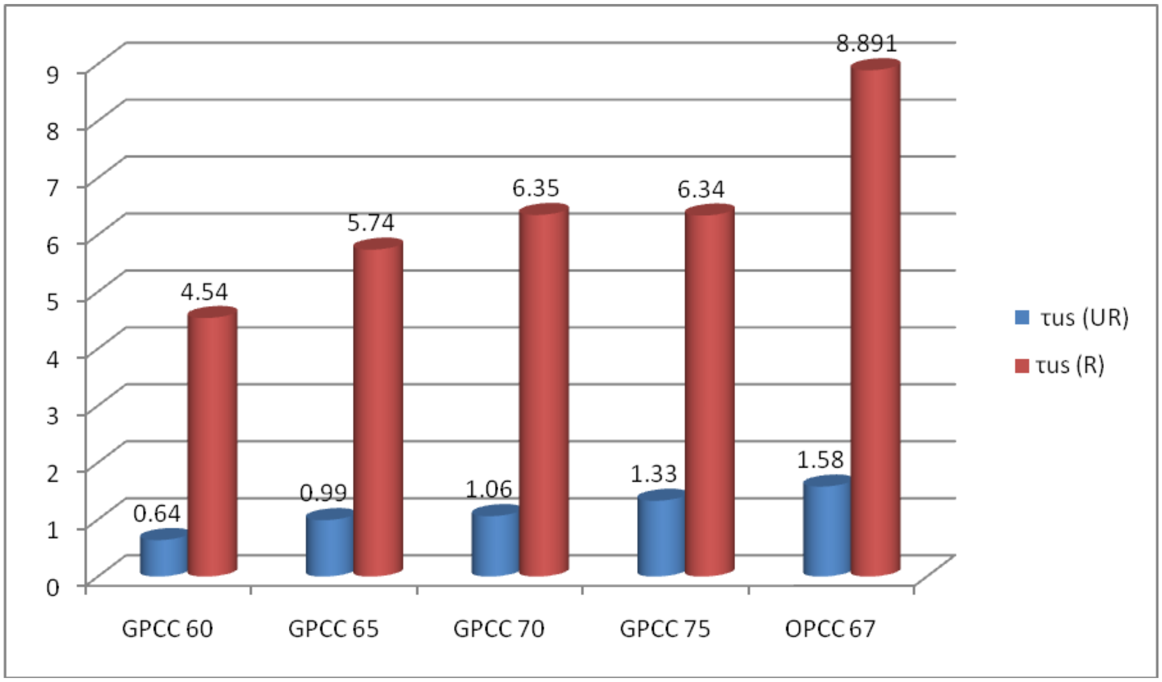
2] It may be further noted from Graphs 1 and 2 respectively that, for a given shear strength, the slip of GPCC specimen is more than that of the OPCC specimen; which has almost the same compressive strength of GPCC specimen (GPCC 70) for both specimens with and without shear reinforcement. This clearly shows that the cohesive strength of GPCC concrete is inferior to OPCC concrete w.r.t the interface shear resistance.

The ultimate interfacial shear strength of specimens tested is presented in Table 10.

Table10: Ultimate interfacial shear stress in Push-off specimen

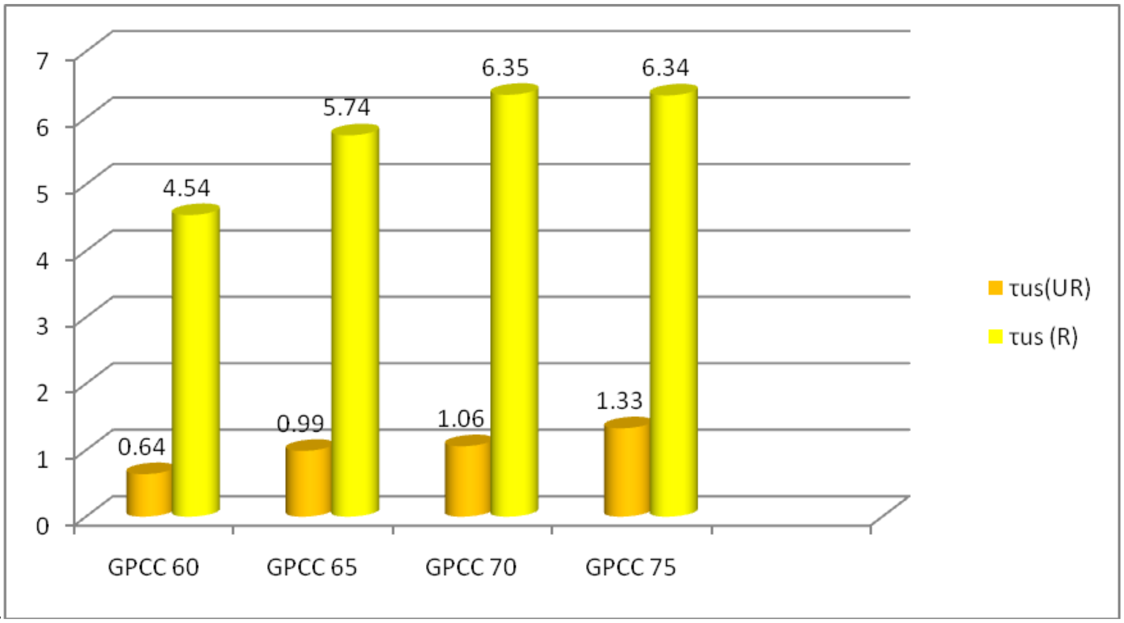
Specimen ID	Un-Reinforced Specimen		Reinforced Specimen		Comparison of % increase in Ult. Stress Bet^{n} Reinforced and Un-Reinforced Specimens due to incorporation of fibres and shear rfts.(0.985%)
	Ultimate Load (kN)	Ultimate Shear Strength (MPa)	Ultimate Load(kN)	Ultimate Shear Strength (MPa)	
GPCC60	13.01	0.64	92.40	4.54	4.54x100/0.64 = 709.375.i.e.7.1 times
GPCC65	20.12	0.99	116.70	5.74	5.74x100/0.99=579.79i.e.5.8 times
GPCC70	21.51	1.06	125.07	6.35	6.35x100/1.06 = 599.05 i.e.@ 5.99 times
GPCC75	27.12	1.33	125.02	6.34	6.34x100/1.33 = 476.70i.e.@ 4.76 times
OPCC	32.12	1.58	180.84	8.891	8.891x100/1.58 = 562.66@ 5.62 times

From Table10, it could be observed that, the shear strength of both GPCC specimens (unreinforced and reinforced) reduces rapidly when the Total aggregate content is lower than 65 % as clearly depicted below in Bar Chart No.2



Bar Chart No.2:- Depicting Ult. Shear Strength of GPCC (Both Reinforced & Un-Reinforced) w.r.t. Total Aggregate Contents from Table no.10.i.e.inferity of GPCC w.r.t.O.P.C for Total Aggregates Percentages.

Further, while unreinforced GPCC specimen shows an increase in shear strength with increase in aggregate content, the GPCC specimen with shear reinforcement shows no significant variation (about 9% only) in shear strength for an aggregate content more than 65%. This proves that, the contribution of cohesive strength in the development of ultimate interface shear reinforcement resistance of GPCC is negligible if its aggregates content is more than 65%. From Table10, it could be observed that with 0.985 %, shear reinforcement, the ultimate shear strength of OPCC specimen is increased by about 5.62 times and that of GPCC70 is increased by about 5.99 times. The other GPCC concrete specimens had increased in ultimate shear strength, as shown in Table 10 and Bar Chart No.3 when shear reinforcement was provided(i.e. One of the assumptions in the development of interface shear friction theory—see bar chart no.3)



Bar Chart No.3:-Depicting increase in Ultimate Shear strength due to Shear rfts.& Geosynthetics fiber incorporations.

CRACK PATTERNS AT FAILURE LOAD:-

The crack patterns at failure load for both unreinforced and reinforced GPCC concrete specimens were the same and a typical crack pattern is shown in Fig.3.

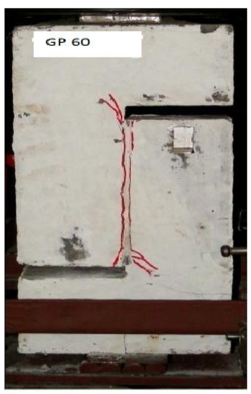


Figure 3: Crack pattern in reinforced push-off Specimen GPCC60

Comparing OPCC concrete with that of GPCC concrete which has almost same compressive strengths of (GPCC70) {see table no.9}, it could be observed from Table 10 that, the ultimate shear strength of GPCC concrete is inferior to OPCC concrete. For the present study, compared to OPCC concrete, from Bar Chart no.2 a reduction factor in the strength £ by 36% calculated as..

£ ={0.64+0.99+1.06+1.33=4.02/4 =1.005, Hence (1.58-1.005 =0.575) gives0.575x100/1.58=36.39 say 36%

and a reduction factor in the strength £ by 35 % ,£ ={4.54+5.74+6.35+6.34=22.97/4=5.74, hence (8.891-5.74 =3.151) gives 3.151x100/8.891=35.44 say35% ;was observed for unreinforced and reinforced GPCC specimens respectively.

Hence, the equations available to calculate shear capacity of OPCC concrete may over-estimate the interface shear capacity of GPC concrete. By using the literature survey and by using the various empirical formulae, we may verify the above experimental results as follows. Table11: compares the experimental interface shear capacity of reinforced specimen with the empirical formula available in the literature for the same [14, 16, and 19].

Table11: Comparison of Shear Capacity of reinforced concrete with the calculated value using Empirical Formulae byACI,Mast and Mattock

Specimen ID	Ultimate load (Experimental Value) P_{EXP}	Ultimate load (Theoretical Value) P_{THEO}			P_{EXP}/P_{THEO}		
		ACI	MAST	MATTOCK	ACI	MAST	MATTOCK
GPCC60	92.40	127	133	139	0.727	0.695	0.67
GPCC65	116.70	127	133	139	0.92	0.877	0.84
GPCC70	125.07	127	133	139	0.985	0.940	0.899
GPCC75	125.02	127	133	139	0.984	0.94	0.899
OPCC	180.84	127	133	139	1.43	1.36	1.301

From this table 11, it may be observed that the empirical formula proposed for OPCC concrete, when used in GPCC concrete overestimates the interface shear capacity of GPCC concrete if its aggregate content is equal to and less than 65%. In the present study, while Mast [13] overestimates the shear strength by about 44 %{ 133X100/92.40 =143.94 for 60% aggregate content (GPCC60), While the value is only about 6% i.e.{133x100/125.02 = 106.38} for GPCC concrete with 75% aggregate content (GPCC75). On the other hand, the shear strength of OPCC concrete specimen is underestimated by about 27% to 35% when different formulae [14, 16, and 19] are used to predict the shear strength. Since no equation is available for the prediction of shear strength of GPC concrete; it is recommended that, only 60% of the predicted shear strength based on the available equations can be considered as the shear strength of GPCC concrete which has Total Aggregate content > OR = 65%.

III CONCLUSIONS

Following conclusions may be derived based on the present study….

1] For a given interface shear stress, Geopolymer concrete specimen (GPCC) shows more slip compared to OPCC concrete specimen.

2] The interface shear strength of Geopolymer concrete (GPCC) is inferior to OPC concrete. In the present study, compared to OPC concrete, a reduction in the strength by 33% and 29% was observed for unreinforced and reinforced Geopolymer specimens respectively.

3] The interface shear strength of both unreinforced and reinforced geopolymer specimens reduces rapidly when the total aggregate content is lower than 65%.

4] The enhancement in shear strength of reinforced (with 0.985% steel) Geopolymer concrete specimen is not significant for an increased aggregate content above 65%.

5] The equations available to calculate shear capacity of OPCC concrete are giving very much over estimated values of the shear capacity of Geopolymer concrete if its aggregate content is less than 65%.

6]By using a strength reduction factor ranging from 30 to 40% for the value obtained by using the prediction equations available (Mattock, Mast and ACI) for the shear capacity of OPCC concrete can be considered as the predicted shear capacity equations of Geopolymer concrete(GPCC) with an aggregate content 65% and above.

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