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Strength Studies on Expansive Clay Treated With Lime and GGBS

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Abstract - *Expansive soils (ES) tend to be problematic due to the alternate swelling and shrinkage they undergo following seasonal moisture changes. This behaviour of soil is attributed due to the presence of mineral Montmorillonite, which has an expanding lattice. Structures built in them are subjected to distress and as a result special foundations have to be adopted in these soils. Sand cushion, CNS cushion and under-reamed piles are some of the foundations that are being used in this country to overcome the problems. However, each one of the above practices suffers from one drawback or the other. In the present study, Expansive soil was treated with combination of Lime and GGBS. The percentage variation of lime was 2,4,6 and 8 and for each percentage of Lime, GGBS was varied as 5,10, 15 and 20 percentages. For each and every percentage of combination of Lime and GGBS Liquid Limit, Plastic Limit, compaction characteristics and CBR test and Tri-axial test(UU) were conducted.*

Keywords: *Expansive clay, Lime, GGBS*

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I. Introduction

Expansive soils are a worldwide problem posing many challenges to civil engineers, construction firms and owners. Black cotton soils of India are well known for their expansive nature. In India, the black cotton soil covers 7 lakh square kilometres approximately 20-25 % land area and are found in the states of Maharashtra, Gujarat, Rajasthan, Madhya Pradesh, Uttar Pradesh, Karnataka, Andhra Pradesh and Tamil Nadu. These are derived from the weathering action of Basalts and traps of Deccan plateau. However, their occurrence on granite gneiss, shale, sandstone, slate and limestone is also recognised (Uppal, 1965; Mohan, 1988; Katti, 1979; Desai, 1985). They are highly fertile for agricultural purposes but pose severe problems to the pavements, embankments and light to medium loaded residential buildings resting on them due to cyclic volumetric changes caused by moisture fluctuation. (Bell, 1996).This volume change behaviour is the reason for cracking to the overlying structures. The reason for this behaviour is due to presence of clay mineral such as montomorillonite that has an expanding lattice structure (Snethen, 1979). During monsoon soil containing this mineral will imbibe water, swell, become soft and their capacity to bear load is reduced, while in drier seasons, these soils shrink and become harder due to evaporation of water (Cokca, et al, 2009). These types of soils are generally found in arid and semi-arid regions. These types of soils are to be stabilized in order to rectify its deficiencies in engineering properties of the soil.

In the present study an attempt is made to study the influence of GGBS on lime treated expansive soil.

II. Materials Used

In the present work the following materials and equipment were used.

2.1. Expansive Soil

Expansive soil (ES) has been obtained from the site Komaragiripatnam near Amalapuram at a depth of 1.5m below the ground level. According to IS classification system, the soil was classified as high plasticity clay (CH) and the results are tabulated in Table-1.

TABLE 1 Properties of Expansive Soil

S.No	Property	Amalapuram Soil
1	**Grain size distribution**	
	Sand (%)	13.33
	Silt (%)	35.4

	Clay (%)	51.27
2	**Atterberg limits** Liquid limit (%) Plastic limit (%) Plasticity index (%)	 88 33 55
3	**Compaction properties** Optimum Moisture Content, O.M.C. (%) Maximum Dry Density, M.D.D (g/cc)	 27.3 1.46
4	Specific Gravity (G)	2.66
5	IS Classification	CH
6	Free Swell Index (%)	140
7	CBR	2.21

2.2 Lime

Lime is obtained in the form of quicklime or hydrated lime. Quicklime is manufactured by calcinations of limestone at high temperatures, which chemically transforms calcium carbonate into calcium oxide. Hydrated lime is created when quicklime chemically reacts with water. Lime can be used to stabilize clay soils and base materials.

When added to clay soils, lime reacts with water in the soil and reduces the soil's water content. The lime also causes ion exchange within the clay, resulting in flocculation of the clay particles. This reaction changes the soil structure and reduces the plasticity of the soil. Lime used for this experimental study is shell Lime that is manufactured from burning of Shale.

2.3 Ground Granulated Blast furnace Slag (GGBS)

Quenching (i.e. sudden cooling with water or air) of hot slag may result into formation of vitrified slag. The granulated blast furnace slag is a result of use of water during quenching process. The GGBS used in this project work is collected from RASHTRIYA ISPAT NIGAM LIMITED Visakhapatnam.

The chemical composition of GGBS

CaO-- 30-38%
SiO_2--30-40%
Al_2O_3 --15-22%
MgO---8-11%
FeO--5% max
MnO--2% max

2.4. Tests conducted in this Investigation

1. Specific gravity test
2. Grain size analysis test
3. Free Swell Index
4. Atterberg limits
5. Compaction
6. California bearing ratio
7. Trai-Axial (UU-test)

2.5 Specific gravity test

Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. The specific gravity of a soil is used in the phase relationship of air, water, and solids in a given volume of the soil. Specific gravity test was carried out by Pycnometer as per IS 2720 Part 3 (1980).

2.6 Grain size analysis test

The distribution of different grain sizes affects the engineering properties of soil. Grain size analysis provides the grain size distribution, and it is required in classifying the soil. Grain size analysis tests are still some of the most valuable guides to the engineering behavior of soils in the context of geotechnical engineering. Coarse particle and fine particle size distributions were determined, in this study, in accordance with test methods IS 2720 Part 4 (1985) respectively. This method is followed for size of particles more than 75micron only. Hydrometer analysis was carried out for size of particles less than 75micron.

2.7 Atterberg limits

Liquid limit and plastic limit are termed as Atterberg limits. Liquid Limit (LL) is defined as the moisture content at which soil begins to behave as a liquid material and begins to flow. Plastic Limit (PL) is defined as the moisture content at which soil begins to behave as a plastic material. Liquid limit and plastic limit tests were carried out as per IS 2720 Part 5 (1985). Liquid limit tests were conducted using two different apparatus namely Casagrande apparatus and cone penetration apparatus.

2.8 Casagrande method

Liquid limit of soil was determined with the help of standard apparatus designed by Casagrande. About 120 g of the soil sample passing 425-micron IS Sieve shall be mixed thoroughly with distilled water in the evaporating dish or on the flat glass plate to form a uniform paste. The paste placed in the brass cup of Casagrande apparatus and the groove is cut with the standard grooving tool. The handle is rotated at a rate about 2 revolutions per second and the numbers of blows are counted until the bottom of the groove along a distance of 10mm due to flow and not by sliding. About 10gm of soil from near the closed groove is taken for water content determination. This process is repeated at different water contents.

Liquid Limit: A flow curve shall be plotted on a semi logarithmic graph representing water content on the arithmetical scale and the number of drops on the logarithmic scale. The flow curve is a straight line drawn as nearly as possible through the four or more plotted points. The moisture content corresponding to 25 drops as read from the curve shall be rounded off to the nearest whole number and reported as the liquid limit of the soil. Casagrande apparatus for determination of liquid limit is shown in Fig-1

Fig 1: Casagrande apparatus for determination of liquid limit.

2.9 Plastic limit test

Plastic limit of clay was determined by taking the soil sample passing through 425μ sieve which is mixed with water until the soil mass becomes plastic enough to be easily remoulded. A ball is formed with about 8 g of paste and rolled between figures up to 3mm diameter and the sample is again remoulded into a ball. This process of rolling and remoulding is repeated until the thread starts just crumbling at a diameter of 3mm.The crumbling threads are kept for water content determination. The test is repeated two or three times and the average water content are taken which is treated as the plastic limit.

2.10 Compaction test

Compaction tests are still some of the most valuable guides to the engineering behaviour of soils in the context of road engineering. Standard Compaction Test was used to determine the OMC (optimum moisture content) and the maximum dry density to which various mixes can be compacted at this moisture content. Although these tests form a part of the procedures of other tests used, they were performed primarily to determine whether or not there was an increase in density upon the addition of various additives to the soil.

Improvement of grading and/or compaction of soil to higher density results in reduction in settlement, reduction in permeability and an increase in shear strength.

Standard proctor compaction tests were conducted on various mixes prepared on basis of dry weight. The mould of standard volume equal to 1000cc is used. The mould is filled up with the material to be compacted by 25 blows in three layers. Standard hammer of 2.45kg weight falling from a height of 30cm is used for compaction. Test is repeated at different water contents. Dry density is calculated at all water contents so as to obtain the compaction curve between moisture content and dry unit weight. The water content corresponding to maximum dry density achieved is taken as the optimum moisture content.

2.11 California bearing ratio test

The CBR test is a penetration test which gives a measure of the load spreading ability of the pavement. This is only justified in the case of flexible pavements. To prepare the samples for CBR test, different mixes chosen were compacted statically in standard moulds at optimum moisture content and maximum dry density. The dimension of the soil sample for CBR test is taken as 150mm diameter and 125mm height. Surcharge weight of 25N was used during the testing. A metal penetration plunger of diameter 50 mm and 100 mm long was used to penetrate the samples at the rate of 1.25 mm/minute using computerized CBR testing machine. Soaked CBR tests were conducted after 96 hours soaking. For soaking samples were placed in a tank maintaining constant water level throughout the period.

2.12 Tri-Axial Test

This test is limited to specimens in the form of right cylinders of nominal diameter 38, 50, 70 and 100 mm and of height approximately equal to twice the nominal diameter. In case of remoulded samples; ratio of diameter of specimen to maximum size of particle in the soil should not be less than 5. The object of the specimen preparation is to produce cylindrical specimens of height twice the specimen diameter with plane ends normal to the axis and with the minimum change of the soil structure and moisture content. The specimen prepared shall be placed centrally on the pedestal of the triaxial cell. The cell shall be assembled with the loading ram initially clear of the top cap of the specimen and the cell containing the specimen shall be placed in the loading machine. The operating fluid shall be admitted to the cell and the pressure raised to the desired value. The test shall be commenced, a sufficient number of simultaneous readings of the load and compression measuring gauges being taken to define the stress strain curve. The test shall be continued until the maximum value of the stress has been passed or until an axial strain of 20 percent has been reached. The specimen shall then be unloaded and the final reading of the load measuring gauge shall be recorded as a check on the initial reading. I have chosen 38 mm diameter and height of 76 mm specimen.

2.13 Methodology

The experimentation program of the present work was conducted in two phases.

Phase 1:

The first phase dealt with finding the properties of the virgin soil.

Properties like Differential Free Swell index, Atterberg limits, specific gravity, and compaction characteristics were found out.

Phase 2:

For various percentages of lime (2, 4, 6, and 8%) with varying percentages of GGBS (5, 10, 15 and 20 %) tests like Liquid Limit, Plastic Limit and Compaction characteristics were found out.

Phase 3:

Tri-Axial (Unconsolidated Undrained) (UU test) was carried out for curing periods of 3, 7, 14and 28 days for different percentages of lime with varying percentages of GGBS. CBR tests are carried out for curing period of 4 days with the above varying percentages.

Phase 4:

SBC for Strip footing was calculated by assuming necessary assumptions.

2.14 Experimental combinations

Flow chart showing different combinations of lime and GGBS with ES is depicted in Fig.2

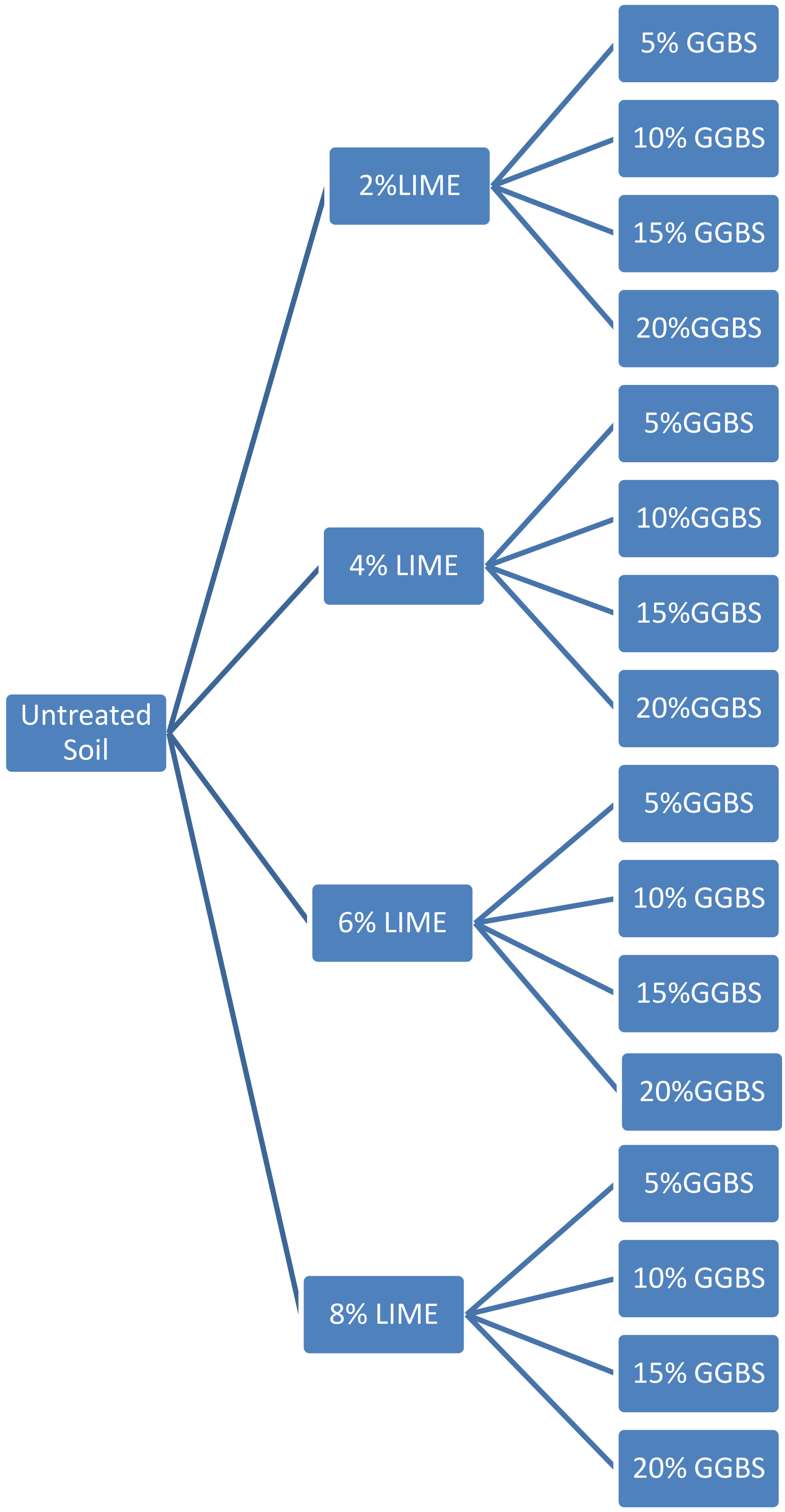

Fig 2 Flow chart showing different combinations of lime and GGBS with ES.

III. Results And Discussion

3.1 General

In this chapter a detailed discussion on the results obtained from various laboratory tests are presented.

3.2 Free Swell Index (FSI)

From the results obtained for different combinations of lime and GGBS with ES a bar chart is drawn Figure 3 shows the variation of FSI which was carried out as per IS: 2720 part XL (1977)

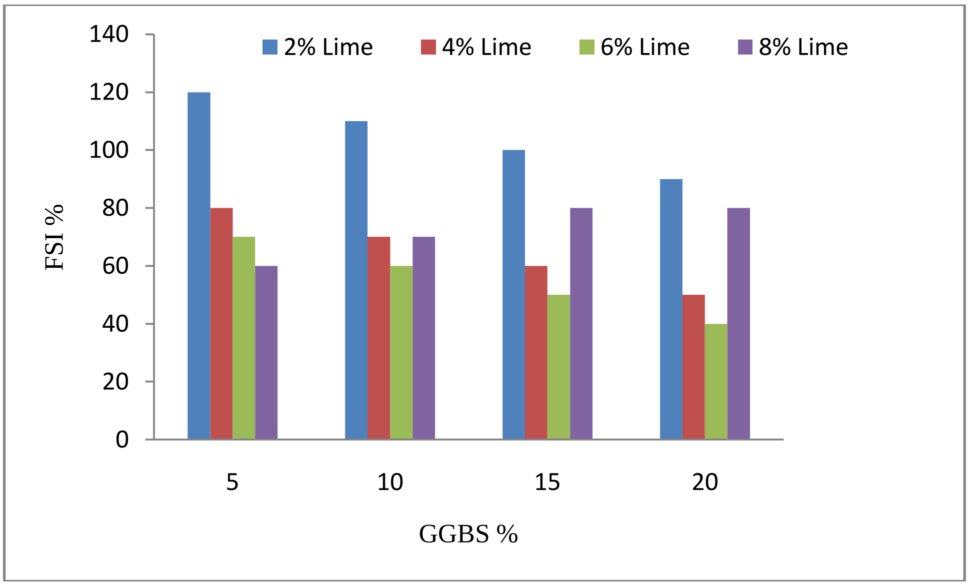

Figure 3 Bar chart showing variation of FSI vs GGBS%

From the above bar chart we can observe that there is a decrease in the FSI value of treated soil from 2% of lime to 6% of lime this is due to the reaction between the lime and GGBS with the ES. But for 8% of lime the FSI for 5% GGBS decreased and for remaining 10% of GGBS, 15% GGBS and 20% GGBS there is a slight increase of FSI value this is due to the remains of free Lime content after reaction.

3.3 Atterberg's Limits

The test results from figures 4,5,6 and 7 shows that LL and PI decreased whereas PL increased for ES mixed with Lime and GGBS. It is known by addition of Lime and GGBS to Expansive Soil can,

- Reduce the thickness of the diffuse double layer,
- Cause flocculation of clay particles,
- Increase the coarser particles content by substitute finer soil particles with coarser particle.
- These reasons all together cause the decrease in LL and PI, and the increase in PL.

Mitchell (1993) indicated that PI is a good indicator of swell potential, the lower PI is, and the lower swell potential will be. Addition of Lime and GGBS to Expansive Soil decreased the plasticity index of expansive soil significantly. This implies there is a significant reduction in swell potential by addition of Lime and GGBS to Expansive Soil.

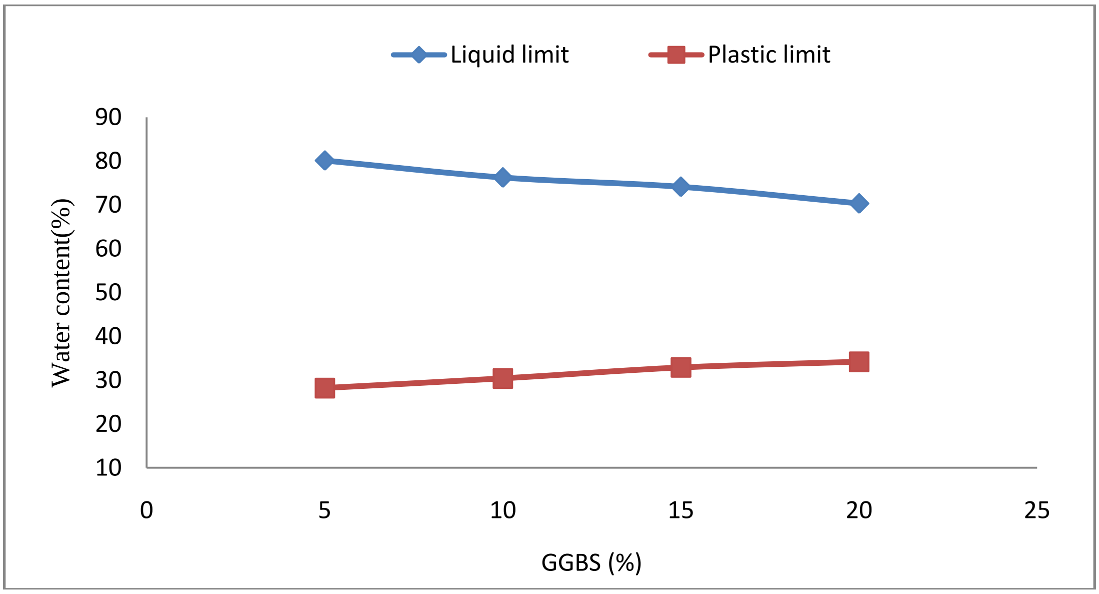

Fig 4 Variation of Liquid limit and Plastic limit for Expansive soil treated ES with 2% Lime and (5%, 10%, 15%, 20%) GGBS.

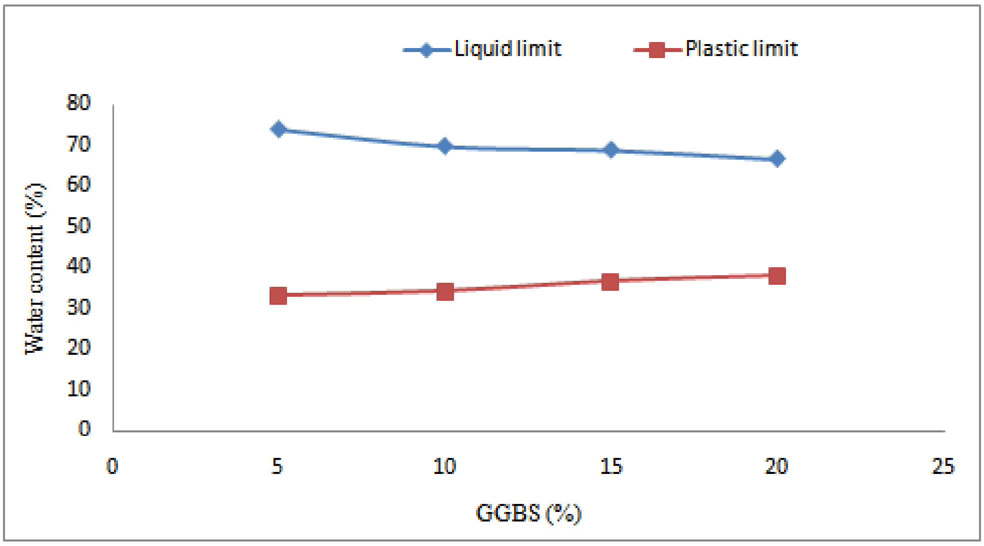

Fig 5 Variation of Liquid limit and Plastic limit for Expansive soil treated ES with 4% Lime and (5%, 10%, 15%, 20%) GGBS

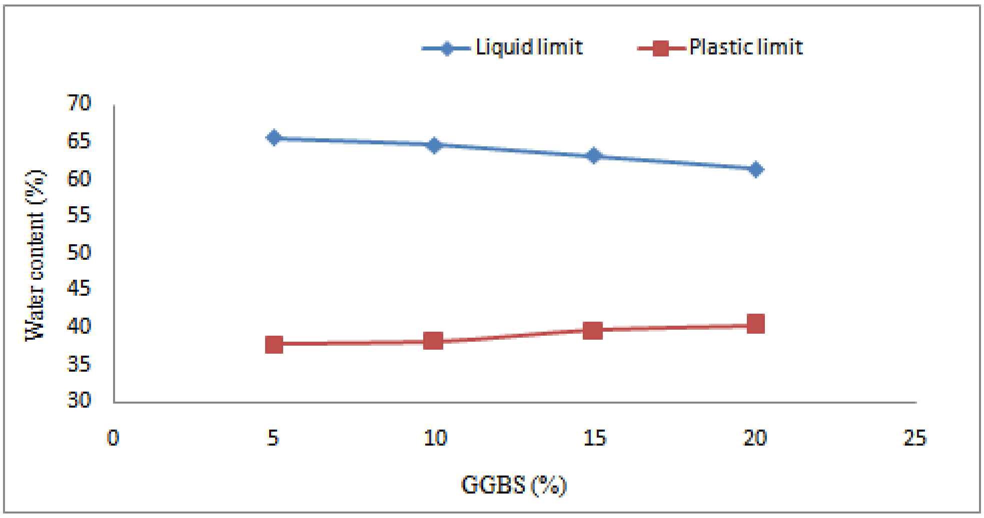

Fig 6 Variation of Liquid limit and Plastic limit for Expansive soil treated ES with 6% Lime and (5%, 10%, 15%, 20%) GGBS

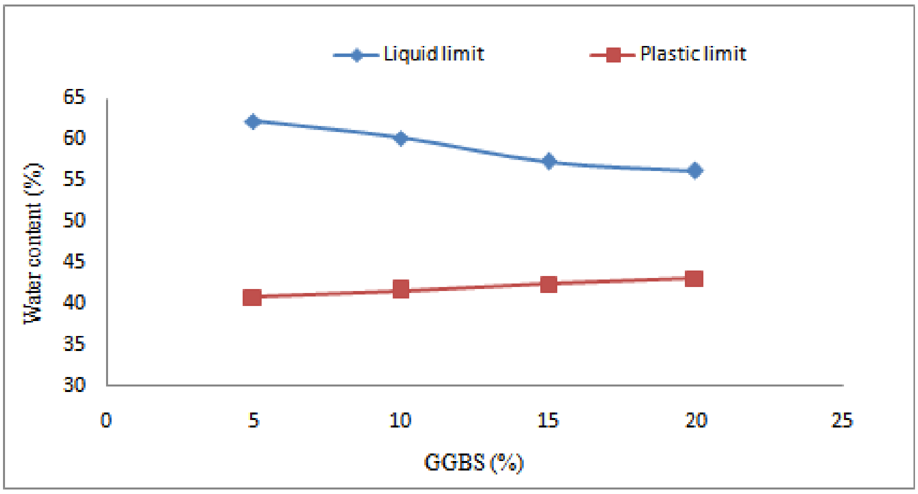

Fig 7 Variation of Liquid limit and Plastic limit for Expansive soil treated ES with 8% Lime and (5%, 10%, 15%, 20%) GGBS.

3.4 Compaction test

Compaction was done for 2%, 4%, 6% and 8% of Lime by varying percentages of GGBS as 5%, 10%, 15% and 20% for each percentage of lime. It was observed for each case of lime percentage with varying percentages of GGBS the maximum dry density was decreasing and the optimum moisture content as well. The compaction curves as outlined in Figures 8,9,10 and 11 show the trend below.

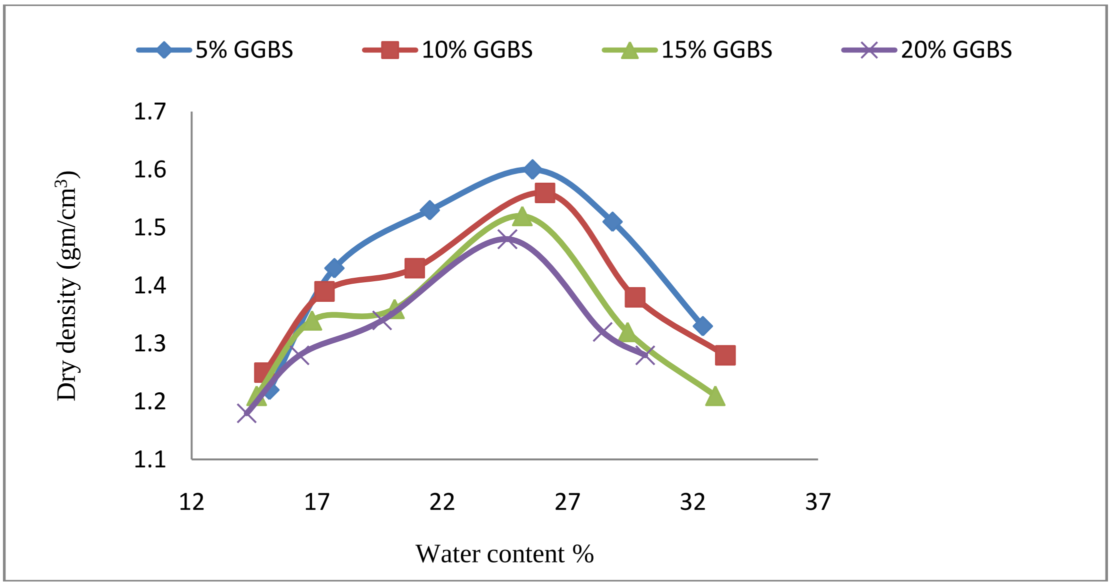

Fig 8 Graph showing compaction cures for 2% Lime with (5%, 10%, 15% and 20%) GGBS

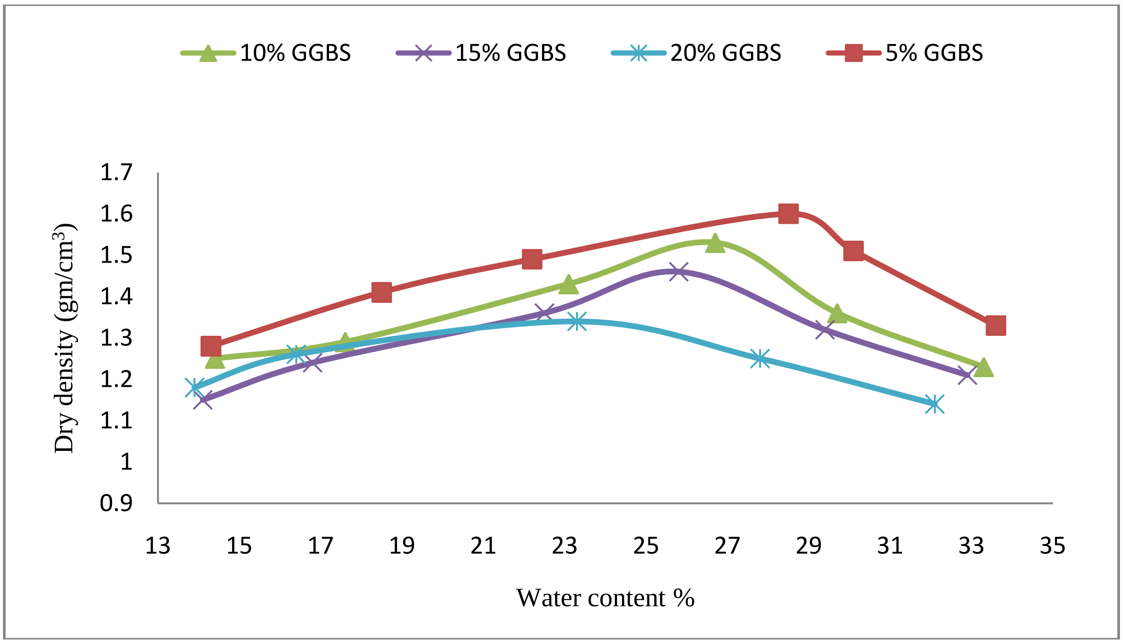

Fig 9 Graph showing compaction cures for 4% Lime with (5%, 10%, 15% and 20%) GGBS

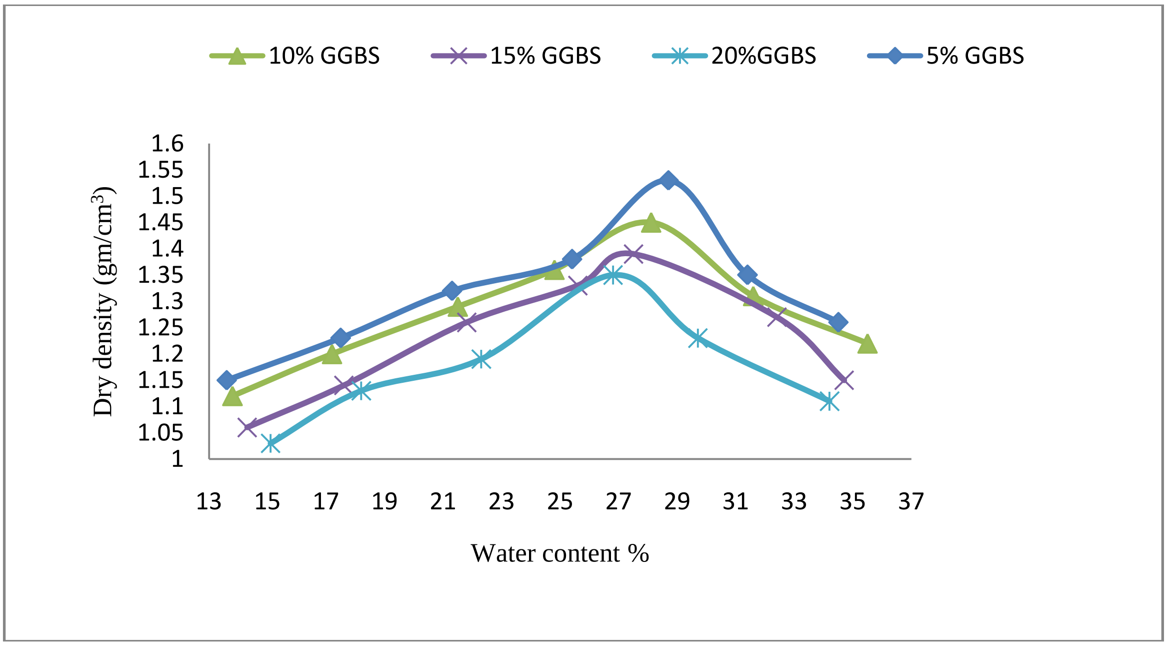

Fig 10 Graph showing compaction cures for 6% Lime with (5%, 10%, 15% and 20%) GGBS

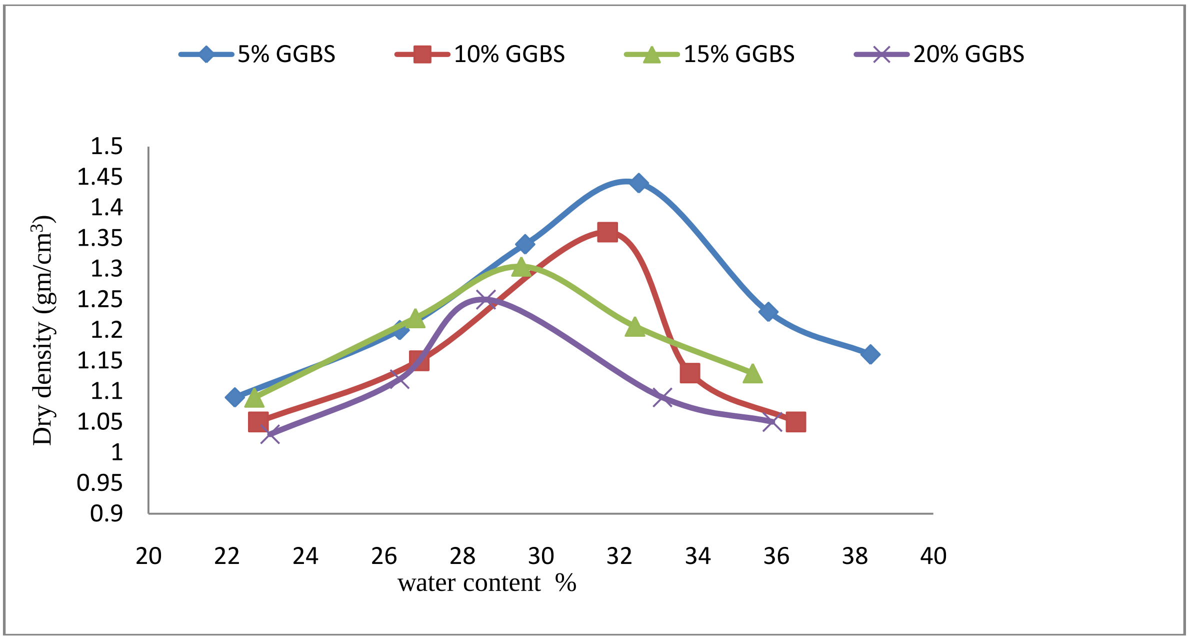

Fig 11 Graph showing compaction cures for 8% Lime with (5%, 10%, 15% and 20%) GGBS

The decrease in the OMC is due to presence of GGBS coarser particle than that of soil which absorbs less water content than that of finer ones. The decrease of MDD accounts due to the frictional forces that oppose the compaction effort, this frictional forces are due to irregular shape of granular particles of GGBS.

3.5 California Bearing Ratio (CBR)

CBR tests were conducted for Expansive soil stabilized with different percentages of Lime (2%, 4%, 6%, and 8%) and for each percentage of Lime varying GGBS percentages as 5, 10, 15, and 20. All the specimens are compacted at optimum moisture content. The soaked CBR test was carried out after soaking the specimens for 4 days. From the figures 12 and 13 we can observe the variation of CBR value for different combinations of Lime along with GGBS added to ES. There is a significant increase of CBR value from 2.21 of untreated ES. For 8% of lime with varying percentages of GGBS the CBR value seems to be constant.

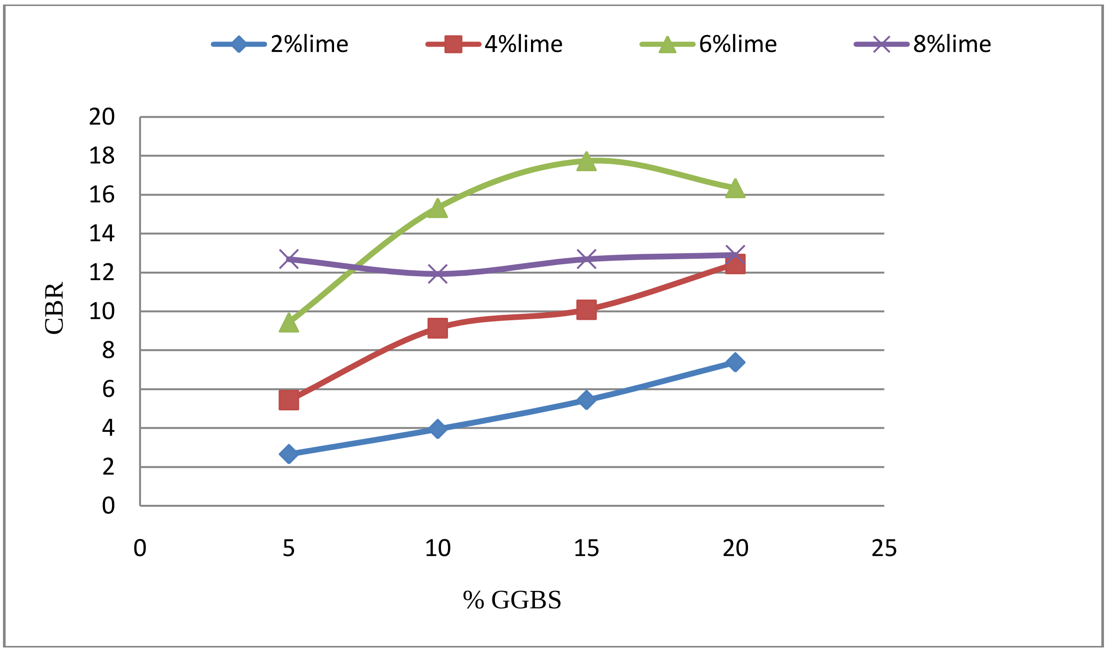

Fig 12 Graph showing CBR variation for different combinations of Lime with (5%, 10%, 15% and 20%) GGBS

From below bar chart we can observe that CBR value has increased for 2% Lime and 4% Lime for different percentages of GGBS. For 6% of Lime there is increase in the CBR value up to 15% of GGBS but for 20% of GGBS the value has decreased. For 8% of Lime for different percentages of GGBS the CBR value seems to be constant varying between the CBR values of 11-12.

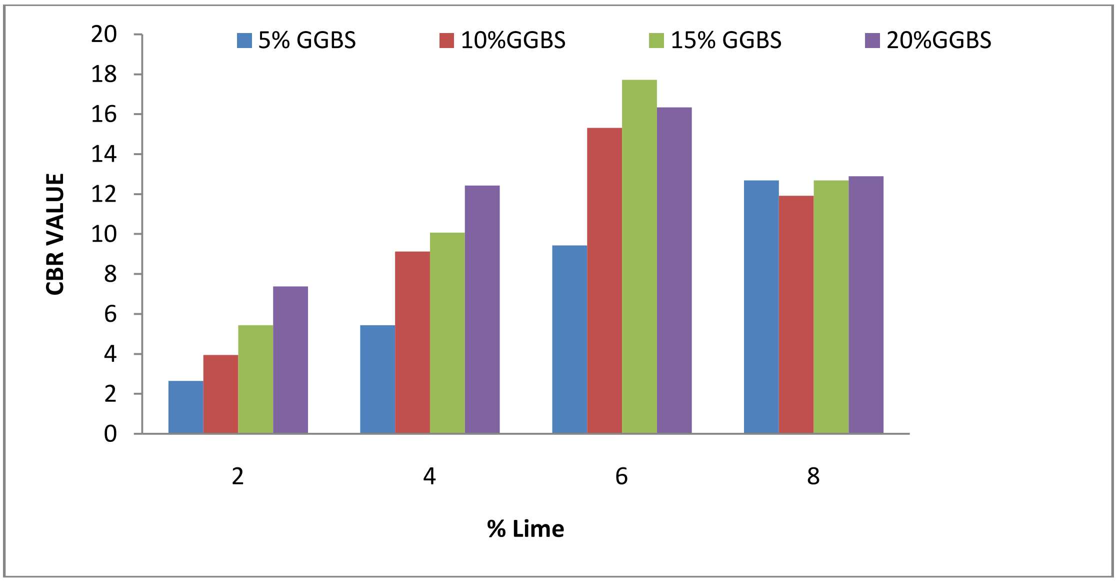

Fig 13 Bar chart showing CBR variation for different combinations of Lime with (5%, 10%, 15% and 20%) GGBS

3.6 Unconsolidated Undrained Tri-Axial test (UU Tri Axial Test)

The treated soil for different percentages of Lime with varying percentages of GGBS was compacted to its maximum dry density and optimum moisture content that was observed from the compaction test.

Trai –Axial specimens of standard size of 38 mm diameter and 76 mm height was prepared and kept in the desecator for curing periods of 3days, 7days, 14days and 28days to avoid the loss of moisture content from the specimens. After completing the curing period Unconsolidated Udrained test was conducted for each combination with varying confining pressures as 0.5, 1.0 and 1.5 kg/cm^2 the values of deformation with their respective load was recorded, for each case of confining pressure maximum and minimum stress were found and Mohr's circle's were developed with the help of MS-Excel and failure envelope was identified along with the

values of cohesion (c) and angle of internal friction (Ø) was determined. The figures 14 and 15 shown below represent the variation of c, Ø with variation of %GGBS and number of days of curing.
Results of Trai-Axial test on ES treated with 2% Lime and (5%, 10%, 15% and 20%) GGBS for (3, 7, 14 and 28) days of curing are shown in Table-2

Table 2 Showing values of c and Ø for different combinations and different curing periods for 2% Lime

curing days	3	3	7	7	14	14	28	28
%GGBS	c in kg/cm^2	Ø °	c in kg/cm^2	Ø °	c in kg/cm^2	Ø °	c in kg/cm^2	Ø °
5%	0.39	3	0.47	3	0.63	4	0.71	5
10%	0.41	4	0.49	5	0.65	5	0.74	7
15%	0.43	5	0.51	6	0.72	6	0.81	9
20%	0.46	6	0.54	7	0.76	8	0.92	10

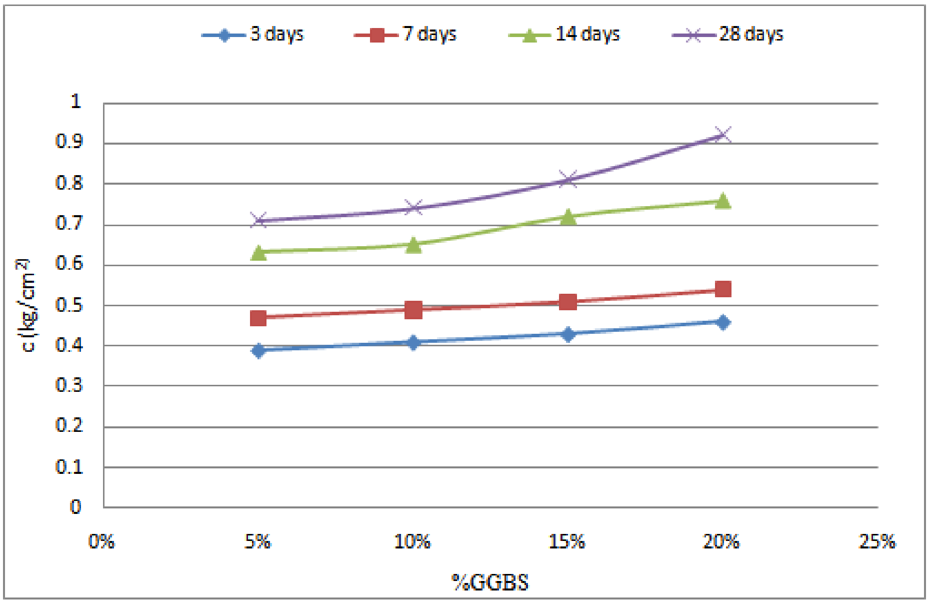

Fig 14 showing variation of ‘c’ with variation of GGBS for different curing periods for 2% Lime

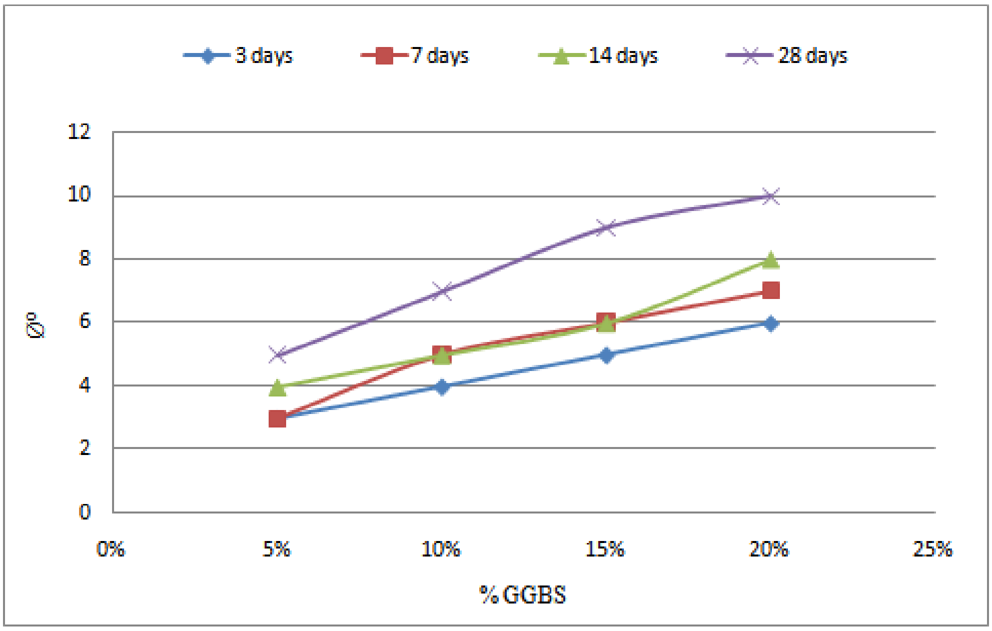

Fig 15 showing variation of ‘Ø’ with variation of GGBS for different curing periods for 2% Lime

Results of Tri-Axial test on ES treated with 4% Lime and (5%, 10%, 15% and 20%) GGBS for (3, 7, 14 and 28) days of curing (Table-3 and Figures 16 and 17).

Table 3 Showing values of c and Ø for different combinations and different curing periods for 4% Lime

Curing days	3	3	7	7	14	14	28	28
%GGBS	c in kg/cm2	Ø o	c in kg/cm2	Ø o	c in kg/cm2	Ø o	c in kg/cm2	Ø o
5%	0.21	4	0.45	5	0.68	6	0.85	8
10%	0.32	5	0.6	6	0.72	8	0.93	11
15%	0.57	6	0.63	7	0.81	10	1.2	12
20%	0.65	6	0.75	9	0.94	11	1.6	14

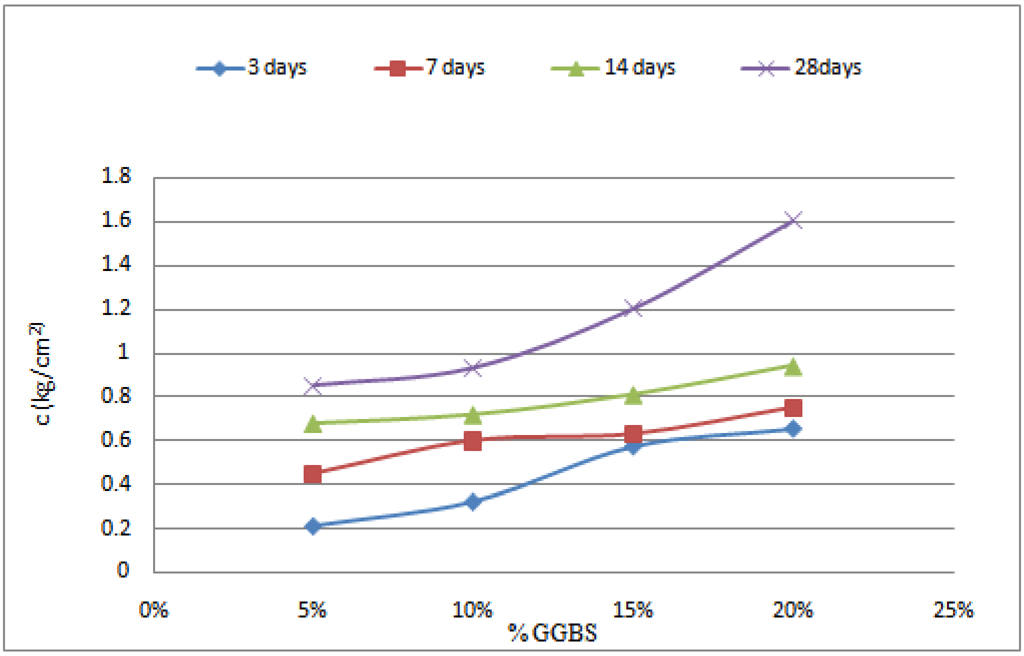

Fig 16 showing variation of 'c' with variation of GGBS for different curing periods for 4% Lime

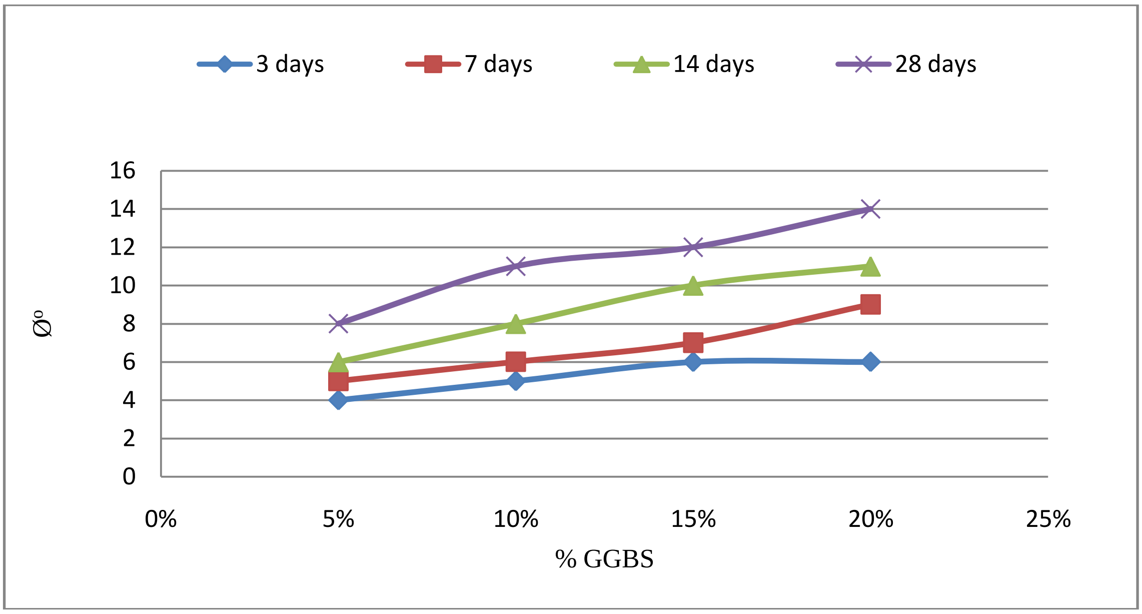

Fig 17 showing variation of 'Ø' with variation of GGBS for different curing periods for 4% Lime

Results of Trai-Axial test on ES treated with 6% Lime and (5%, 10%, 15% and 20%) GGBS for (3, 7, 14 and 28) days of curing (Table-4 and Figures 18 and 19).

Table 4 Showing values of c and Ø for different combinations and different curing periods for 6% Lime

Curing days	3	3	7	7	14	14	28	28
%GGBS	c in kg/cm^2	Ø °	c in kg/cm^2	Ø °	c in kg/cm^2	Ø °	c in kg/cm^2	Ø °
5%	0.58	5	0.7	5	0.8	6	1.15	10
10%	0.61	6	0.72	6	0.82	8	1.3	12
15%	0.64	7	0.73	8	0.95	9	1.4	14
20%	0.7	8	1.15	9	1.58	10	1.9	15

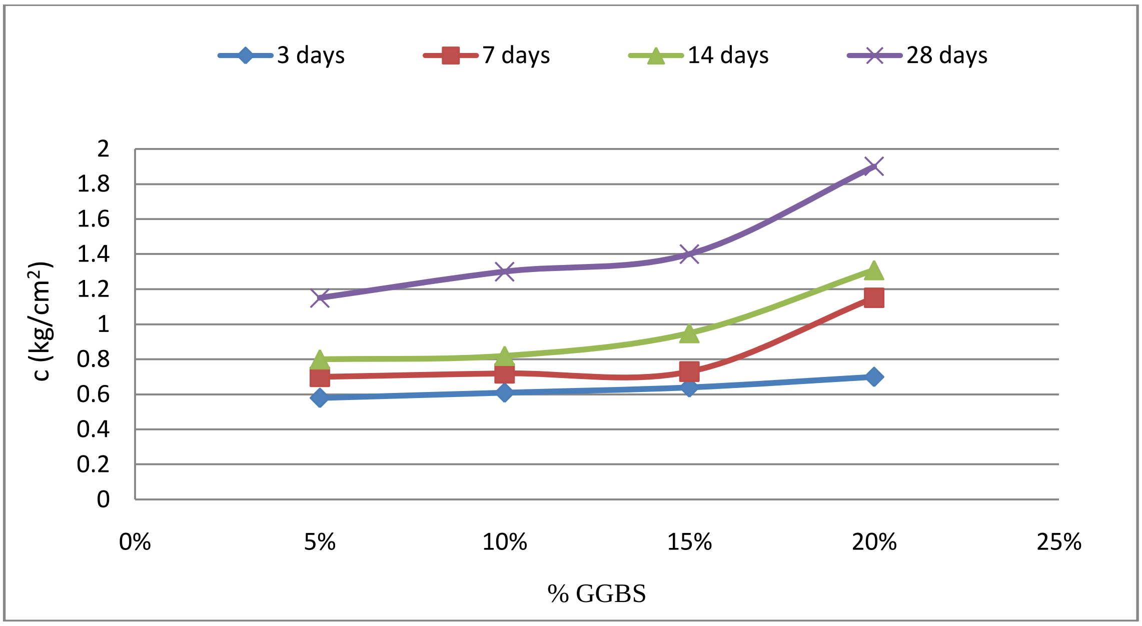

Fig 18 showing variation of 'c' with variation of GGBS for different curing periods for 6% Lime

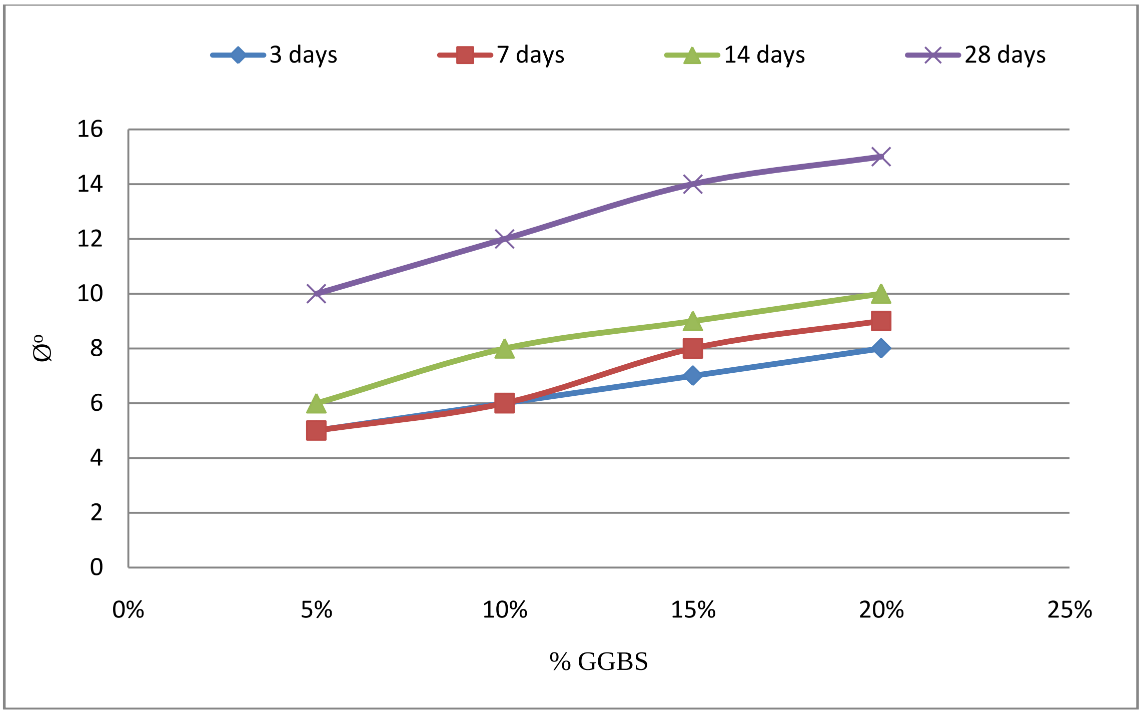

Fig 19 showing variation of 'Ø' with variation of GGBS for different curing periods for 6% Lime

Results of Trai-Axial test on ES treated with 8% Lime and (5%, 10%, 15% and 20%) GGBS for (3, 7, 14 and 28) days of curing (Table-5 and Figures 20 and 21).

Table 5 Showing values of c and Ø for different combinations and different curing periods for 8% Lime

curing days	3	3	7	7	14	14	28	28
%GGBS	c in kg/cm^2	Ø º	c in kg/cm^2	Ø º	c in kg/cm^2	Ø º	c in kg/cm^2	Ø º
5%	0.71	6	1.15	8	2.2	9	2.3	11
10%	0.78	8	1.43	9	2.5	10	2.6	13
15%	0.82	9	1.54	10	2.7	11	3.2	14
20%	1.23	11	2.13	12	2.9	13	3.3	16

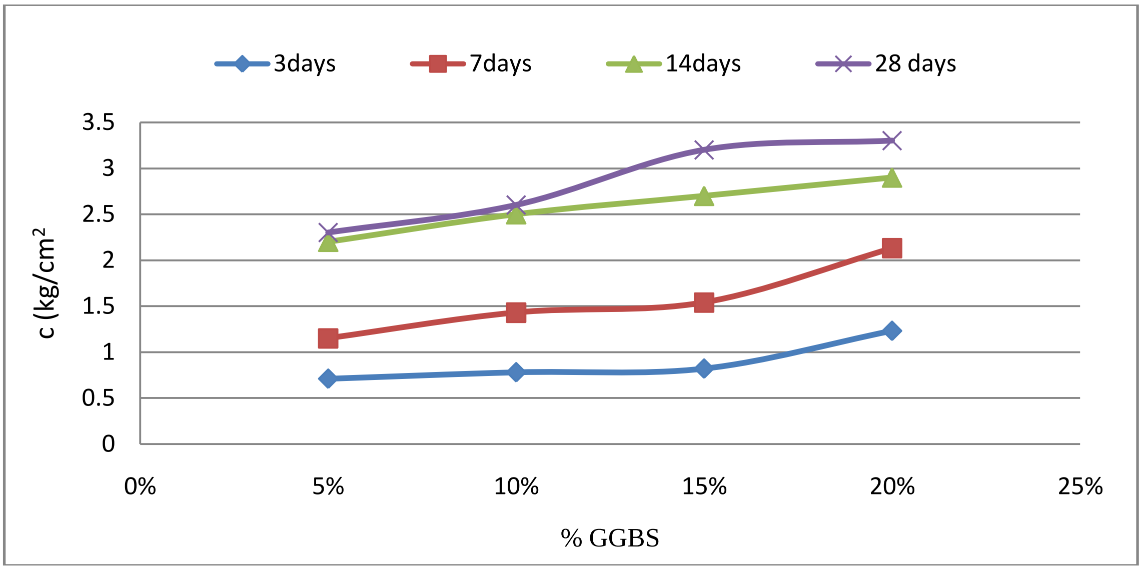

Fig 20 showing variation of 'c' with variation of GGBS for different curing periods for 8% Lime

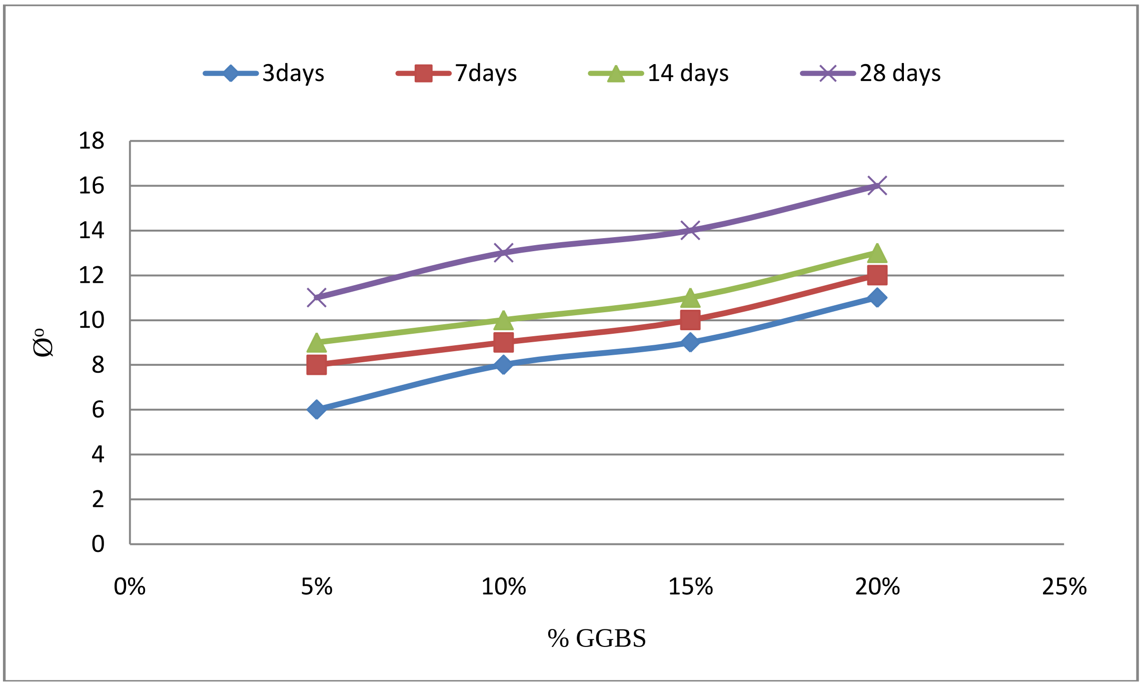

Fig 21 showing variation of 'Ø' with variation of GGBS for different curing periods for 8% Lime

From the above graphs there is increase in the value of cohesion ‘c’ and angle of internal friction ‘Ø’. This is due to the reason that forming of CSH gel due to the reaction of Lime with available moisture and silica from ES and GGBS respectively. The increase of angle of internal friction is due to the presence of granular material GGBS.

3.7 Safe Bearing Capacity (SBC)

For the obtained values from the Tri-Axial test the SBC values for a Strip footing of width one meter.
The following assumptions were made while calculating SBC

- The footing was a strip footing.
- The width of footing is 1meter.
- The footing is located at a depth of 1.5 meters below the ground level.
- The water table was assumed to be much deeper.
- The constants used were Meyerhof’s.
- The factor of safety for calculating SBC was 2.5.
- No shape and inclination was considered.

Figures 22, 23, 24 and 25 illustrate the graphs showing SBC vs % GGBS for 2%, 4%, 6%, 8% lime.

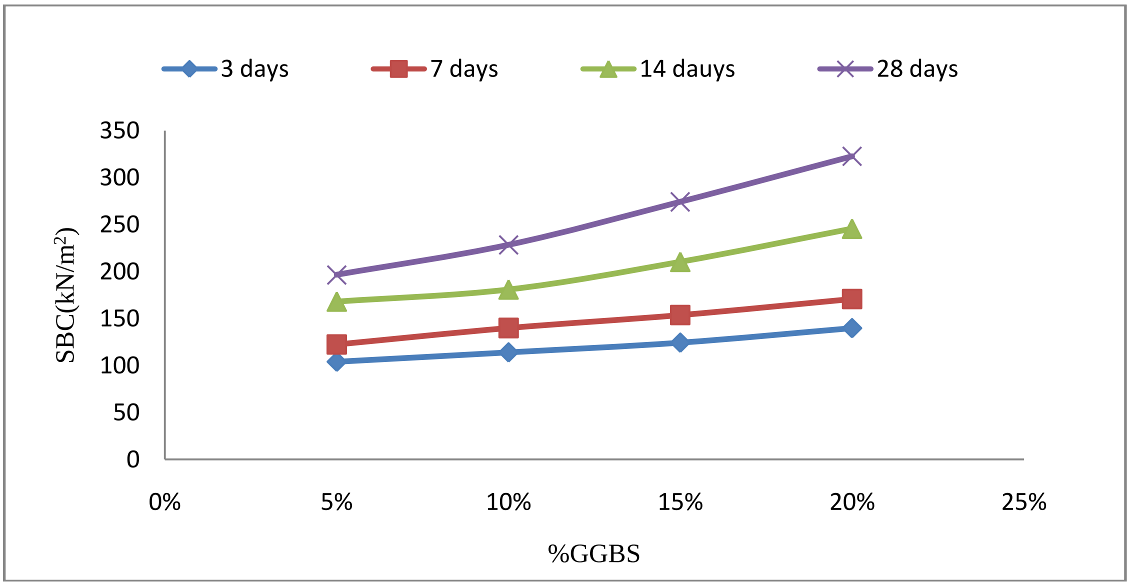

Fig 22 Graph showing SBC vs %GGBS for 2% Lime

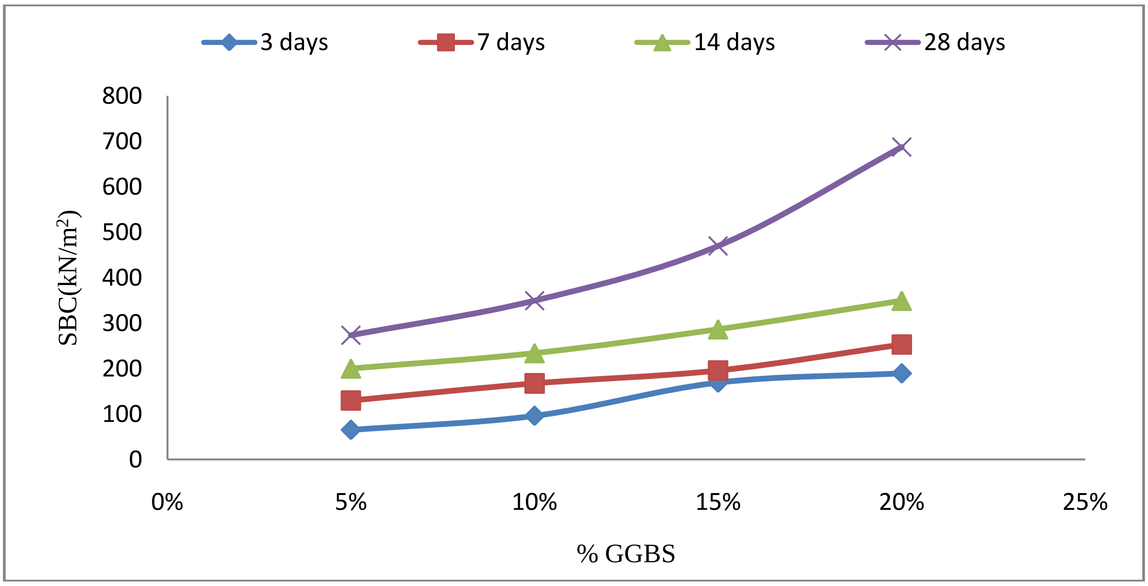

Fig 23 Graph showing SBC vs %GGBS for 4% Lime

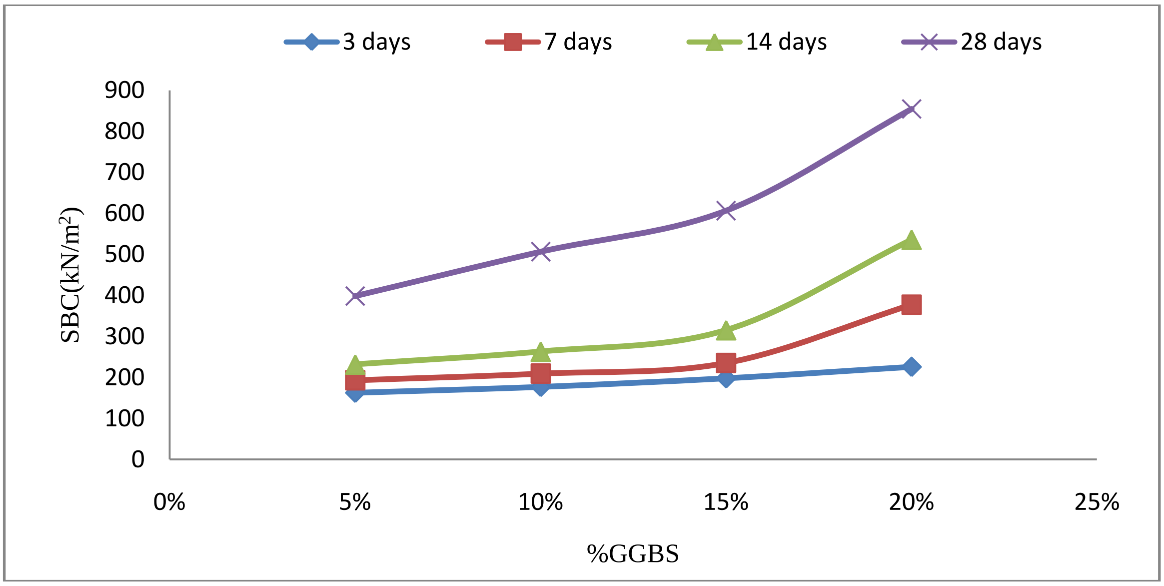

Fig 24 Graph showing SBC vs %GGBS for 6% Lime

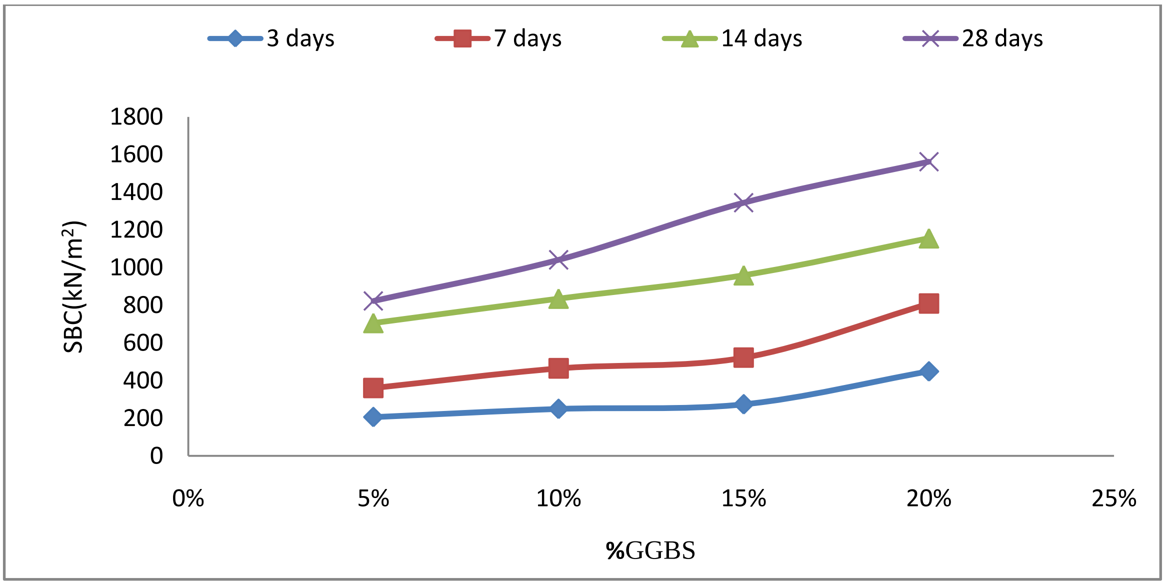

Fig 25 Graph showing SBC vs %GGBS for 8% Lime

The SBC value has been increased due the reason that the values of cohesion 'c' and angle of internal friction have been increased up on increasing the Lime and GGBS percentages with increase in the curing period.

IV. Conclusions

The following conclusions are drawn based on the laboratory studies carried out in this work.

1. On addition of Lime + GGBS the Free Swell Index has been decreased. The maximum decrease was observed by addition of 6% Lime+20% GGBS. The decrease in the FSI is71.4%.
2. The Liquid limit has been decreased up on addition of Lime and for varying percentages of GGBS for a particular Lime content it has been reported as decrement.
3. The plastic limit was increasing by increasing the Lime content and for constant Lime percentage by varying GGBS content it has been reported increment.
4. The plasticity index which is an index of swelling potential has been decreasing.
5. The MDD value has been decreasing by increasing the Lime content and for a constant Lime percentage by varying the percentage of GGBS the MDD value has been reported as decrement.
6. The OMC value has been increasing by increasing the lime content and for a constant lime percentage by varying the percentage of GGBS the OMC value has been reported as decrement.
7. The CBR value for 2% - 6% Lime content has been increasing. But, for 8% of Lime for varying percentages of GGBS the CBR value seems to be constant between 11-12. The maximum value obtained is

17.73 for 6% Lme+15% GGBS. The CBR value has been increased almost seven time that of untreated soil.

8. The values of cohesion 'c' and angle of internal friction 'Ø' has been increasing up on varying Lime and GGBS contents. The values have been reported to be maximum only after 28 days of curing.
9. The SBC values calculated from the obtained values of 'c' and 'Ø' have increased by reaching the peak at 28 days of curing.

From this experimental study the strength parameters have been increasing by increasing the Lime content and GGBS. It is very difficult to arrive at optimum content by considering all the strength parameters. So, based on the CBR value it was reported that 6% Lime + 15% GGBS holds good for all purposes.

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