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STABILIZATION OF SUB GRADE SOIL USING CALCIUM LIGNOSULFONATE AND GRANITE DUST

Gajulapalli Kiran Kumar Reddy[1], T Suresh Babu [2], and K Pravallika [3]
[1]Research Scholar,[2]Professor, [3]Assistant Professor
Visvodaya Engineering College, Kavali, Nellore, India

Abstract : Soil is the foundation for any civil engineering structures. It is required to bear the loads without failure. In some places, soil may be weak and cannot resist the oncoming loads. In such cases, soil stabilization is needed. Numerous methods are available in the literature for soil stabilization. But sometimes, some of the methods like chemical stabilization; lime stabilization etc. adversely affects the chemical composition of the soil.

In this study, Calcium Lignosulfonate and Granite Dust were mixed with clay soil to investigate the relative strength gain in terms of bearing capacity and compaction. The effect of Calcium Lignosulfonate and Granite Dust on the geotechnical characteristics of clay-Calcium Lignosulfonate and clay-Granite Dust mixtures was investigated by conducting Atterberg limits, standard Proctor compaction tests and CBR tests. The tests were performed as per Indian Standard specifications.

In this test program, without additives and with additives clay was tested to find the optimum moisture content, Maximum dry density, CBR value, plasticity index.

IndexTerms - ***Subgrade, Stabilization, Soil, Calcium lignosulfonate, Granite dust, CBR***

I INTRODUCTION

1.1 General

Transport in the Republic of India is an important part of the nation's economy. Roads are the vital lifelines of the economy making possible trade and commerce. They are the most preferred modes of transportation and considered as one of the cost-effective modes. An efficient and well-established network of roads is desired for promoting trade and commerce in any country and fulfills the needs of a sound transportation system for sustained economic development. To provide mobility and accessibility, all weather roads should connect every nook and corner of the country. To sustain both static and dynamic load, the pavement should be designed and constructed with utmost care. The performance of the pavement depends on the quality of materials used in road construction. Sub grade is the in-situ material upon which the pavement structure is placed. Although there is a tendency to look at pavement performance in terms of pavement structures and mix design alone, the subgrade soils can often be the overriding factor in pavement performance. The construction cost of the pavements will be considerably decreased if locally available low-cost materials are used for construction of lower layer of pavements such as subgrade, subbase etc. If the stability of local soils is not adequate for supporting the loads, suitable methods to enhance the properties of soil need to be adopted. Soil stabilization is one such method. Stabilizing the subgrade with an appropriate chemical stabilizer increases subgrade stiffness and reduces expansion tendencies, it performs as a foundation (able to support and distribute loads under saturated conditions). This report contains a summary of the performance of Calcium Lignosulfonate and Granite Dust used with clay.

1.2 Scope of the project

1. To collect a particular soil sample and determine its basic physical property such as LL, PL, PI and grain size distribution
2. To study the soil under Standard proctor compaction and determine the MDD and OMC for the soil sample.
3. To carry out CBR Test for sample

1.3 Objectives of the project

The major objectives of the project are:

1. To study the suitability of calcium lignosulfonate and Granite dust as a stabilizing material to improve the subgrade performance.
2. To evaluate the strength characteristics of local soil blended with different proportions of Calcium lignosulfonate and Granite dust.

1.4 Research significance

From the past literature the effect of calcium lignosulfonate as stabilizing agent on the clay and expansive soils were studied this shows an increase in the engineering properties of the soil. In the current study the effect of soil blended with lignosulfonate and granite dust is to be studied with varying the dosages of lignosulfonate and granite dust on local soil.

2 LITERATURE REVIEWS

Rama varaprasad et al (2020) studied the Characterization of expansive soils treated Lignosulfonate. During this article, the soil sample were collected from Amaravati and Vijayawada, state of Andhra Pradesh. The was taken totally different percentages 0.5, 1, 2, 4%. The laboratory tests were conducted Waterberg limits, standard compaction, CBR and UCS. The UCS of soil increase when adding Lignosulfonate.

Geethu Vijayan et al (2019) studied the Stabilization of Clayey soil by exploitation Lignosulfonate. during this literature review clayey soil are going to be stable. They are various tests are conducted Standard compaction test, Unconfined compressive strength, C.B.R and Atterberg limits. The optimum wet are going to be decrease with addition of Lignosulfonate, Maximum dry density increase with addition of Lignosulfonate. And shear strength properties increase adidition of Lignosulfonate and Liquid limit decrease and Plastic limit increase with the addition of Lignosulfonate.

Nauman Ijaz et al (January 2020) Studied the Integrating lignosulfonate and hydrated lime for the amelioration of expansive soil. In this article the soil is stabilized with lignosulfonate (LS) with hydrated lime (LM). A detailed experimental study was performed on the proposed composite binary admixture (CBA)The optimum binary admixture (OBA) was determined on the basis of plasticity index. The different Percentages are taken 0.4%, 0.875% and 2% and different percentages of Lime 2.625%, 3.82%, 5%, 6.5% and 9%. The Various geotechnical properties, such as plasticity, swelling, shrinkage, strength and hydraulic conductivity were analyzed based on the test results. addition reduces the lime consumption in expansive soil stabilization and also provides a sustainable solution in curtailing the industrial waste. In this article, numerous geotechnical properties of expansive soil treated with LS, LM were investigated in the laboratory. On the basis of plasticity index and swelling behavior of the treated expansive soil, 2.625% lime in conjunction with 0.875%. R. Tirumala et al (2017) studied the stabilization of black cotton soil victimization granite waste and quarry dust. During this project an attempt has been created to stabilize the soil victimization Granite waste and Quarry dirt. Experimental work has been completed with ten 10%, 15% and 20% of Granite waste and Quarry dirt content. The experimental work is predicated whole on proportion changes of Granite waste and Quarry dirt content in soil on tests for soil Liquid limit, O.M.C., M.D.D, Bulk density, Dry density and Grain size analysis.

İsmail Zorluer et al (2017) studied the Usage of fly ash and Granite mud for Soil Stabilization. During this study, laboratory tests were conducted on granular soil samples adjusted with fly ash and granite mud. The specimens were ready with granular soil and fly ash-granite at completely different ratios. They were standard Proctor compaction energy and, than cured 1, 7, 28 and 56 days. When cure, unconfined compressive tests were conducted to analyze the impact of fly ash-granite mud on granular soil strength. C.B.R test were performed to samples with additive materials. Addition of fly ash-granite mud redoubled soil strength. As a result, fly ash-granite mud are often used with granular soil for increasing strength.

3 MATERIALS AND METHODOLOGY

In this chapter, a brief review of various experiments conducted using clay and the same stabilized with Calcium lignosulfonate and Granite dust are explained.

3.1 Materials

3.1.1 Clayey soil

In this study, clayey soil was collected from Kavali located in Andhra Pradesh, India. All natural soil samples were oven dried, crushed and sieved through 425 µ sieve prior to experimentation. The various geotechnical properties of the procured soil are as follows:

Fig 3.1 clayey soil

Table 3.1 Properties of Untreated Soil

Soil Property	Value
Grain size distribution (%)	
Fines	68
Sand	24.8
Gravel	7.2
Consistency limits (%)	
Liquid limit	60
Plastic limit	42
Plasticity index	18
Engineering Properties	
IS Standerd compaction.	
MDD (g/cc)	1.769
OMC (%)	16
CBR (%) @2.5mm	2.1

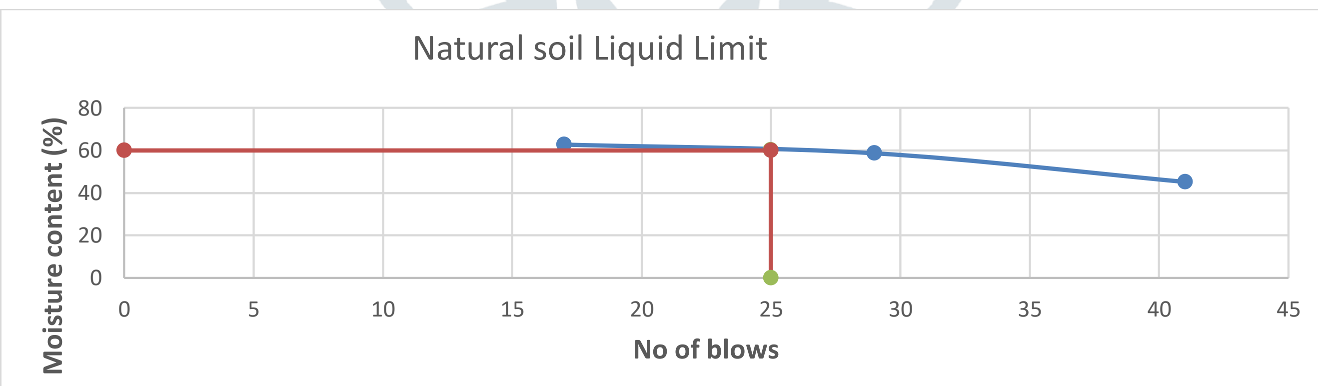


Fig 3.1 Liquid limit for Natural soil

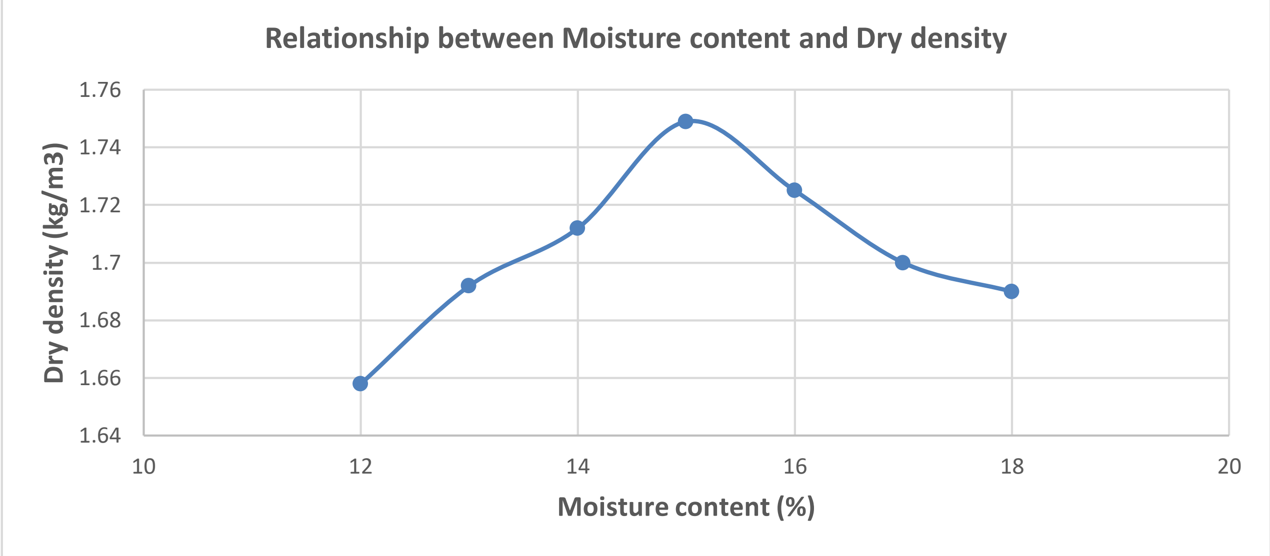


Fig 3.2 MDD & OMC for Natural soil

3.1.2 Calcium lignosulfonate

Lignosulfonate may be a polymer based mostly compound stabilizer derived as a waste by- product from wood/paper trade. Lignosulfonate has shown a promising prospect as a stabilizing agent particularly for soft soils. The main benefits of lignosulfonate over ancient stabilizers are non-toxicity and non-corrosiveness. Stabilization mechanism of ancient stabilizer like lime includes cation exchange, natural action, agglomeration, lime suffusion and pozzolanic reaction resulting in stable soil mass. The lignosulfonate chiefly includes of positive ions, and these reacts with the negative ions present in clay minerals to make stable aggregates by reducing the double layer thickness of clay particles.

Lignosulfonate has shown a promising prospect as a stabilizing agent particularly for soft soils. It belongs to a family of polymer based mostly organic polymers derived as a waste by-product from the wood and paper process trade. it is an environmentally friendly, non-corrosive and non-toxic chemical that does not alter the soil pH upon treatment.

Fig 3.2 Calcium lignosulfonate

3.1.3 Granite dust

The granite waste may be a by-product created in granite factories whereas cutting vast granite rocks to the specified shapes. regarding 3000 MT of granite waste is created per day as a by-product throughout producing of granite tiles and slabs from the raw blocks. Economic means of stabilization as a result of granite that is out there in large quantity from granite industries. The properties of waste rely upon the granite from that it's taken.

Fig 3.4 Granite dust

Mechanism of granite dust:

- When the granite dust is added to a soil, dust particles will fill the missing sizes in the soil and turns the gradation of the curve to a well graded mix.

- With increase in percentage of granite dust in soils the coarser fraction dominates the clay fraction and hence the water absorption capacity of the soil decreases there by OMC decreases.

The Maximum dry density of the stabilized soil increases due to the filling of voids with the particles of granite dust in the virgin soil. This increases the shear strength and CBR of the stabilized soil.

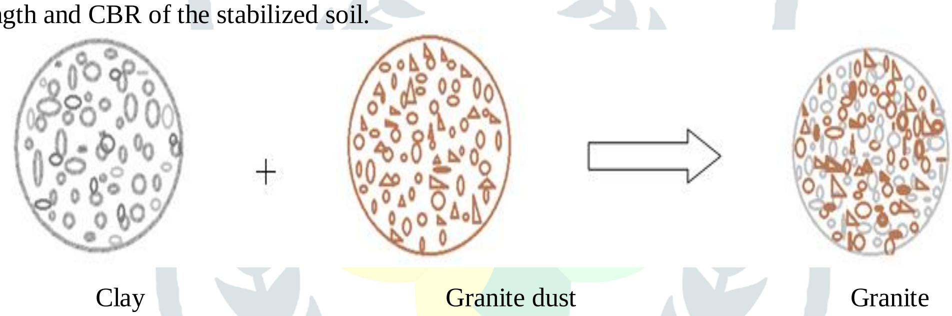


Fig 3.5 Change in the particle gradation of clay soil with addition of granite

- Granite dust can be used as a sustainable stabilizer for poor soils.

Granite dust is highly explored for its suitability as a stabilizer for sub grades and base courses of pavements and embankments as backfill material in retaining walls

3.2 Methodology

3.3 Experimental procedure

All the tests are conducted as per IS Classifications.

- Grain size Analysis of soil: IS 2720(Part 4)- 1975
- Liquid & Plastic limit test: IS 2720 (Part 5) – 1970
- Compaction test: IS 2720 (Part 7) – 1965
- California bearing ratio test: IS 2720 (Part 16) – 1979

3.3.1 Grain size Analysis of soil

1. Weigh a dry soil sample which should be at least 500grm.
2. Record the weight of the sieves and the pan that will be utilized during the analysis. Each sieve should be thoroughly cleaned up before the test.

3. Assemble the sieves in ascending order, placing those with the larger openings on top. Therefore, the 4.75mm sieve should be on top and the 0.075mm sieve on the bottom of the stack.
4. Place the soil sample into the top sieve and place a cap/lid over it.
5. Place the stack in a mechanical shaker and shake for 10 minutes.
6. Remove the sieve stack from the shaker and measure the weight of each sieve and that of the pan placed at the bottom of the stack.

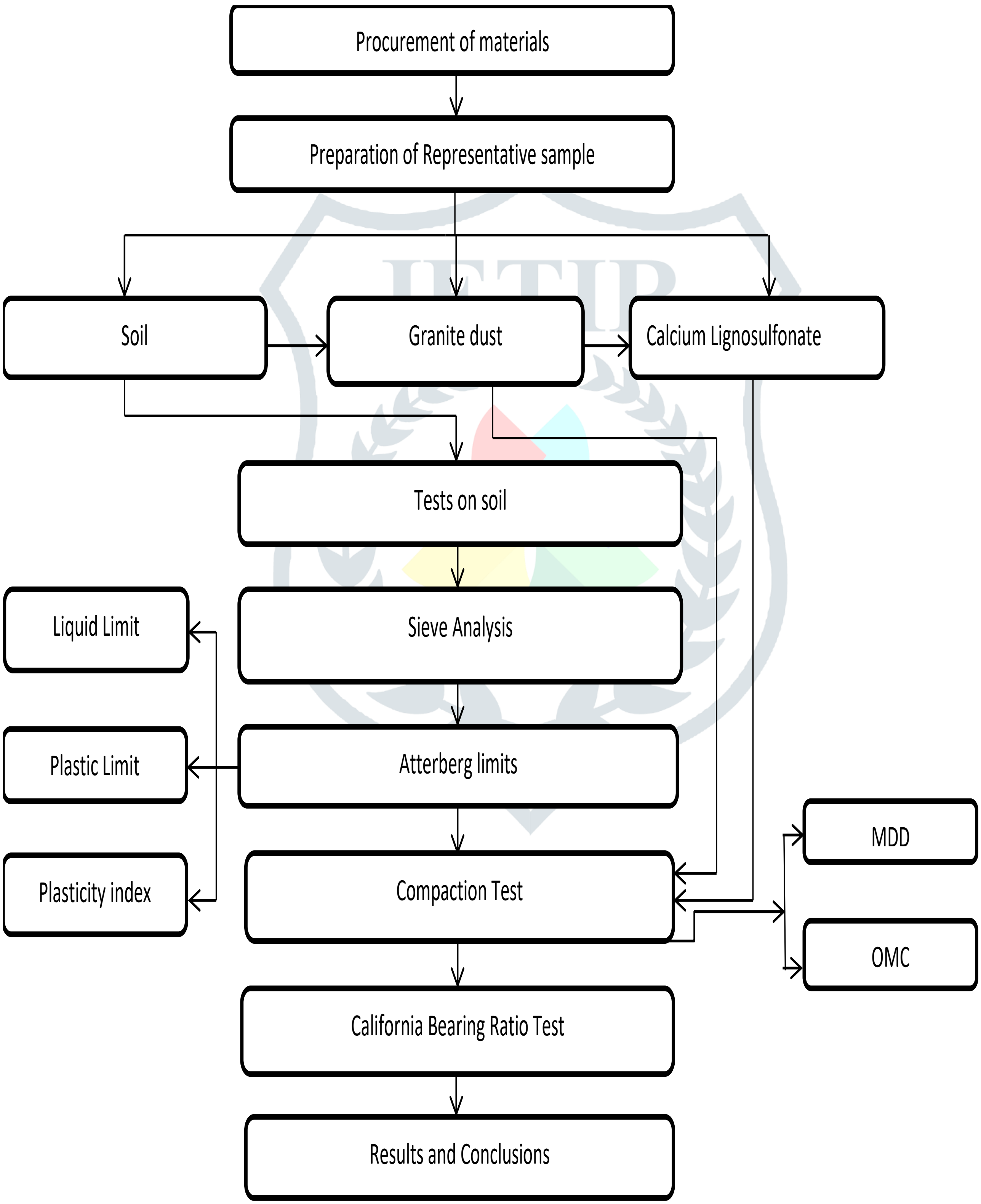


Table 3.2 the sieves typically utilized in the Grain Size Analysis test.

Sieve no	Opening diameter(mm)
4	4.75
8	2.36
16	1.18
30	0.6
40	0.425
50	0.3
100	0.15
200	0.075

3.3.2 Consistency limits

Liquid limit may be defined as that water content at which a standard groove (2mm wide) made in a Part of soil placed in the cup of a standard limit device, closes over about 13mm when the cup drops 25 times from the height of 10mm or hard rubber base.

Determination of Liquid limit:

1. Take about 250g of air-dried soil passing 425microns sieve in a porcelain dish.
2. Add a small amount of water and carefully mix the soil to a uniform color.
3. Adjust the height of fall such that it is equal 10mm.
4. Place a portion of the paste in the cup over the spot where the cup rests on the base, squeeze down and smoothen the surface of the soil pat carefully. By using a grooving tool, cut a clean, straight groove that completely separates the soil

Fig 3.7 Plastic limit

Fig 3.6 Liquid Limit

pat into two parts.

5. Rotate the handle at rate of 120 revolutions per minute and count the number of blows until the two parts of the soil sample come into contact at the bottom of the groove over a distance of 13 mm. Record the number of blows. Continue to add small amount of water until the soil attains that particular consistency at which it will take `about 40 blows to close the standard groove. The is a trial-and-error procedure.
6. Take about 25g of soil from the closed part of the groove in the cup in a pre-weighed moisture can, place the lid on the moisture can and determine the water content.

Determination of Plastic limit:

1. Take about 20g of the air-dried soil passing 425µ m IS sieve. Add water to it to attain a consistency that enables it to be molded into a ball.
2. Take a small portion of the ball and roll it on the glass plate with fingers, using just sufficient but uniform pressure to make it into thread of uniform diamet
3. er over its entire length. When the diameter of the soil thread has reduced to 3 mm knead the specimen together and roll it again. Continue the process until the thread just crumbles at 3 mm diameter.
4. Collect the crumbled soil thread in a pre-weighed moisture can, close the lid and determine its moisture content. The moisture content thus determined is the plastic limit of the soil.
5. Repeat the test two or three time and take the average value.

3.3.3 Compaction Test

Soil at a given site may not be often ideal for the construction of a civil engineering structure or a facility. It may be necessary to improve the engineering properties of such soils. Compaction is one of the methods of making such improvement and involves densification by applying mechanical energy on a soil mixed with suitable water content, which also produces the maximum dry unit weight, is determined from this test. This test has been developed by R.R.proctor in 1933 and is therefore named after him.

Fig 3.8 Compaction test

3.3.4 California bearing ratio.

CBR is the ratio of the force per unit area request to penetrate a soil mass with a standard circular piston of 1875 (mm²) cross section area at the rate of 1.25mm/minute to that required sample of compacted stone was defined as having a CBR of 100%. The standard load (ps) corresponding to 2.5mm penetration of the plunger into the standard sample is reported to be 1370kg and for 5mm penetration it was found to be 2055kg.

Fig 3.9 CBR test

4 RESULTS AND DISCUSSIONS

4.1 Introduction

Several studies have been done on the engineering properties of the soil but the studies on Calcium lignosulfonate and granite dust are limited in literature. In this Chapter, a series of experiments have been done to study the effect of the additives such as Calcium lignosulfonate and Granite dust are added in soil. There are many tests are done like geotechnical properties such as Compaction Characteristics, CBR.

4.2 Atterberg limits

Plastic limit and liquid limit of clayey soil determined as per IS 2720-Part V (1970) and the results are presented in Table 4.1. Atterberg limits done by natural soil and 1%, 2% and 3% of CLS after adding the CLS to decreasing the liquid limit, plastic limit and plasticity index.

Table 4.1 Atterberg limits of natural soil and CLS

Index	Natural soil	Soil + 1%CLS	Soil + 2%CLS	Soil + 3%CLS
Liquid limit (%)	60	56	54	49
Plastic limit (%)	42	39	37	35
Plasticity index (%)	18	17	16	14

Plastic limit and liquid limit of clayey soil determined as per IS 2720-Part V (1970) and the results are presented in Table 4.2. Atterberg limits done by natural soil and 10%, 15% and 20% of Granite dust after adding the Granite dust to similarly decreasing the liquid limit, plastic limit, and plasticity index.

Table 4.2 Atterberg limits of natural soil and Granite dust

Index	Natural soil	Soil + 10%GD	Soil + 15%GD	Soil + 20%GD
Liquid limit (%)	60	43	39	36
Plastic limit (%)	42	34	31	29
Plasticity limit (%)	18	9	7	6

4.3 Particle size distribution:

Grain size distribution of clayey soil is determined by dry sieve analysis as per IS: 2720- Part IV (1985). The particle size distribution curve for the soil samples are presented in figure 4.1

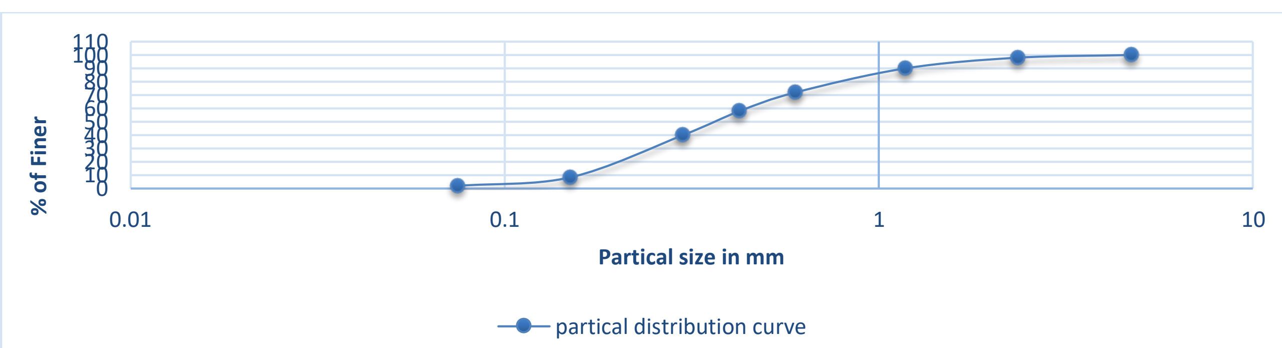


Fig 4.1 Particle distribution curve

Percentage of gravel = 7.2

Percentage of sand =24.8

Percentage of fines =68

4.4 Standard Proctor Compaction Test

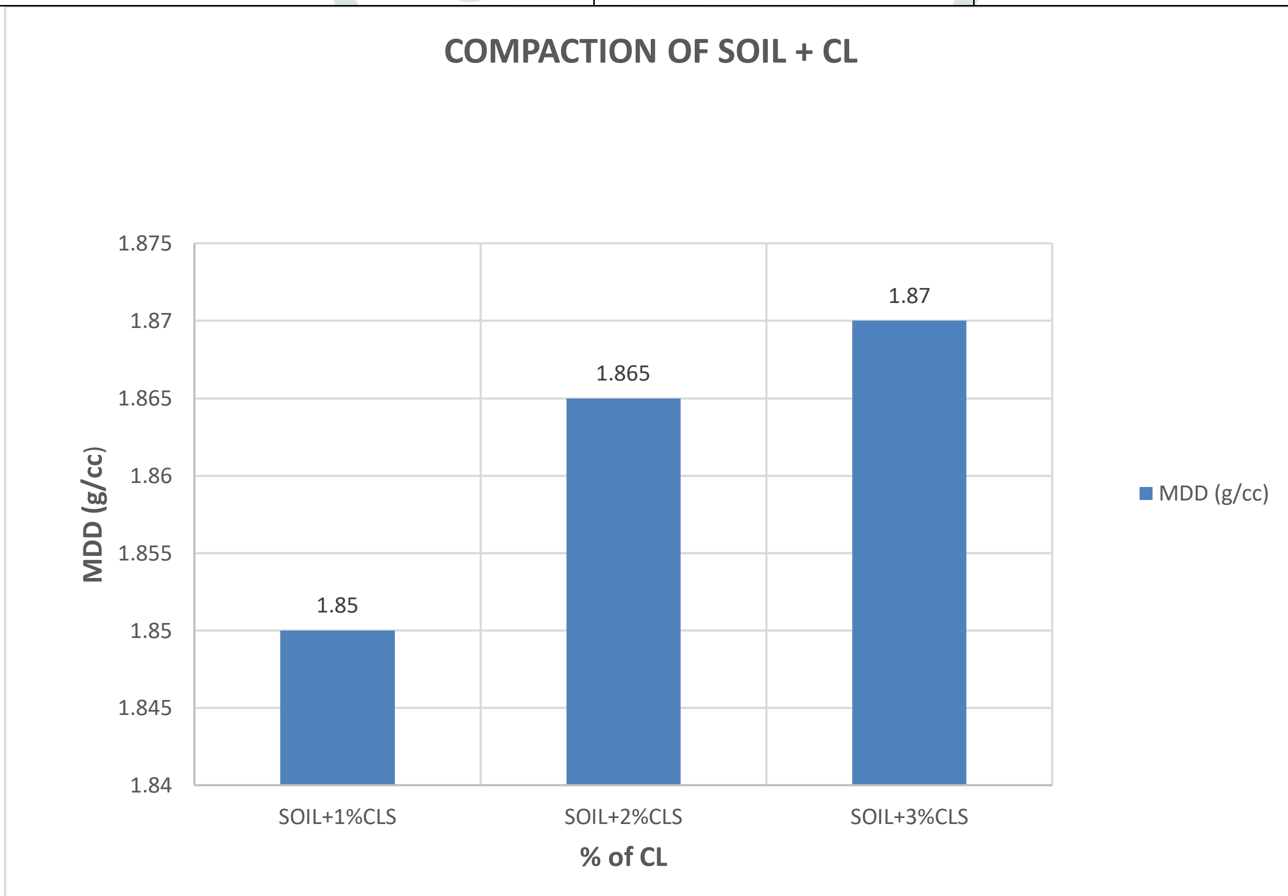
4.4.1 Standard compaction Test of soil stabilization with Soil + 1%, 2% and 3% CLS

The compaction test is to obtain the moisture-density relationship of the soil-additive mixtures were conducted according to IS: 2720 (Part VIII)-1965. The Optimum moisture content is 12.5%, 11.5% and 11% and Maximum dry densities of clayey soil are 1.85 g/cc, 1.865g/cc and 1.87g/cc. It is observed that addition of varying proportions 1%, 2% and 3% of calcium lignosulfonate have considerable effect on the maximum dry density (MDD) and optimum moisture content (OMC) of the clayey soil. MDD values increases by the addition of CLS because the water holding capacity of the soil decreases due to its non-c

ohesive nature and there by results in the decrease of OMC.

Table 4.3 MDD and OMC values of CLS stabilized soil

Content (%)	MDD (g/cc)	OMC (%)
SOIL+1%CLS	1.85	12.5
SOIL+2%CLS	1.865	11.5
SOIL+3%CLS	1.87	11



4.2 Compaction of Soil+ 1%CLS, Soil+ 2%CLS and Soil+ 3%CLS

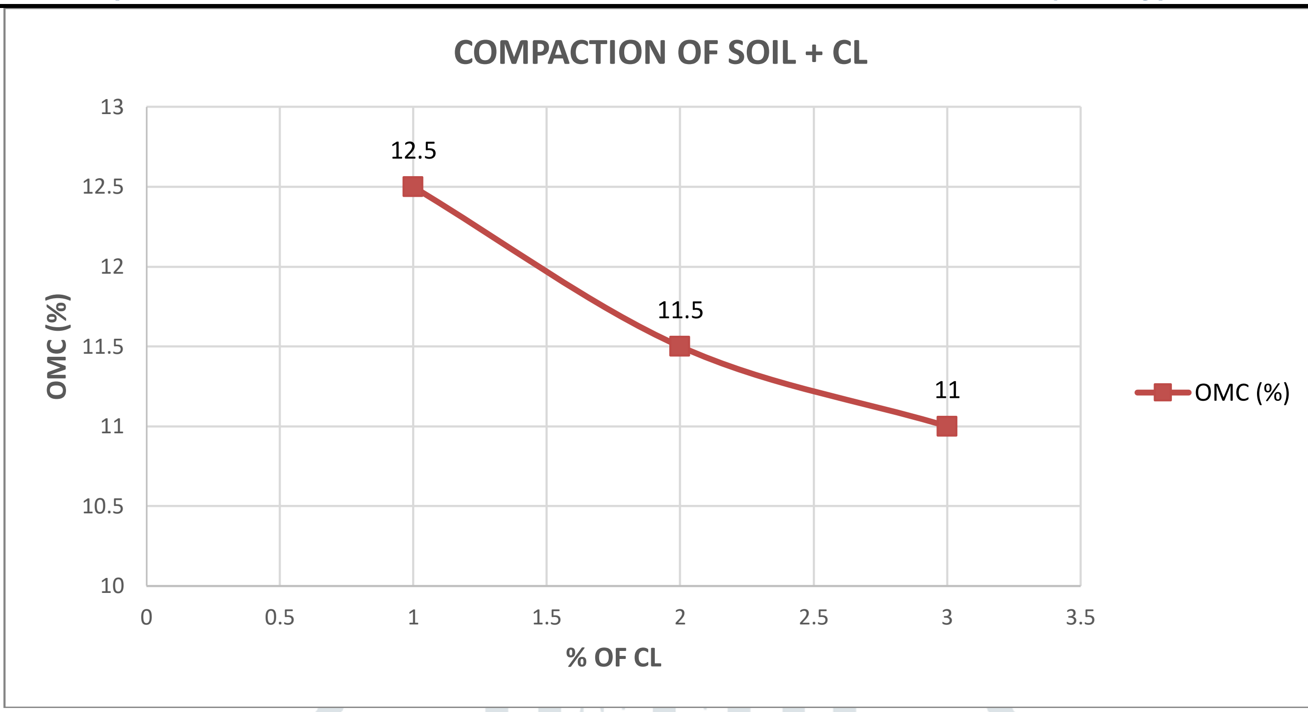
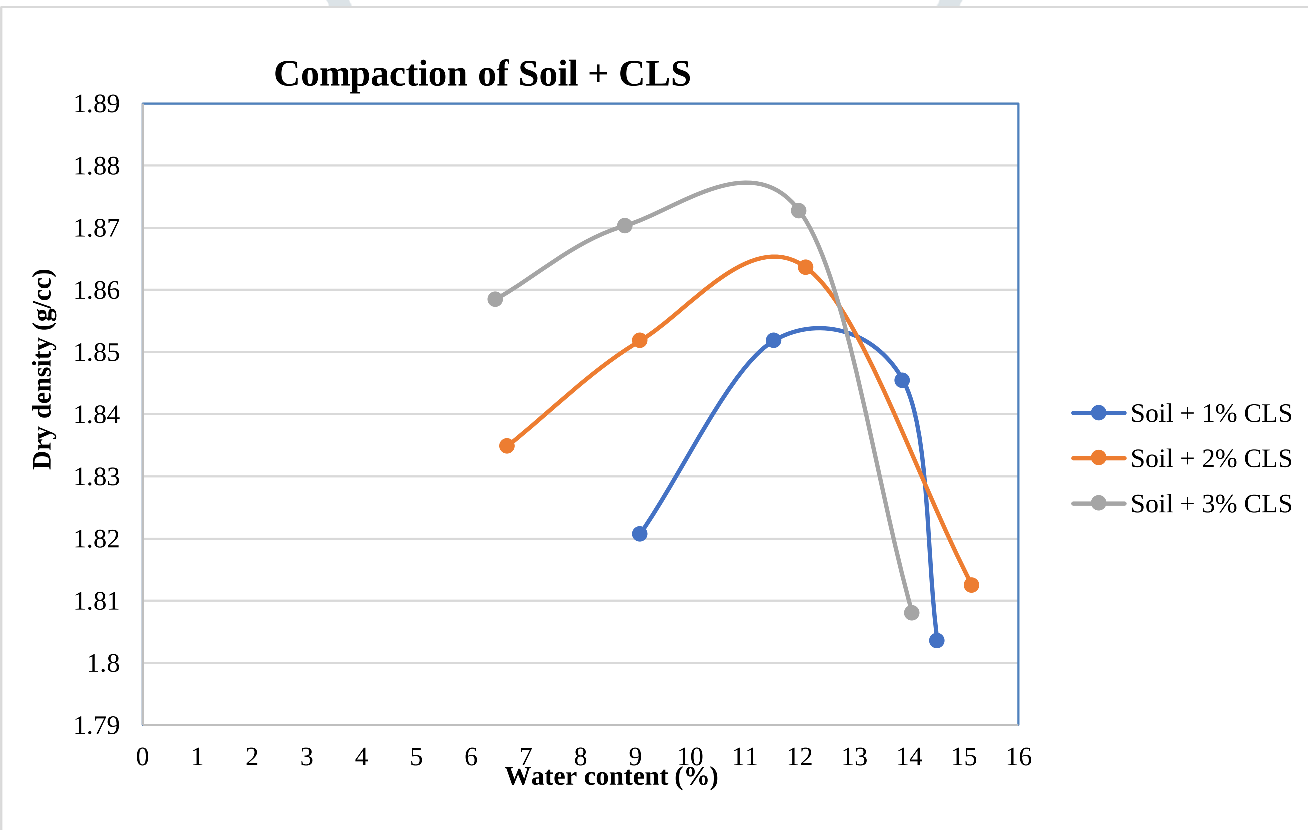


Fig 4.3 Compaction of Soil+GD10%, Soil+GD15% and Soil+GD20%



4.4.2 Standard compaction Test of soil stabilization with Soil + 10%, 15% and 20% GD

The compaction test is to obtain the moisture-density relationship of the soil-additive mixtures were conducted according to IS: 2720 (Part VIII)-1965. The Optimum moisture content is 13.5%, 12% and 11.5% and Maximum dry densities of clayey soil are 1.95 g/cc, 1.96g/cc and 1.97g/cc. It is observed that addition of varying proportions 10%, 15% and 20% of granite dust have considerable effect on the maximum dry density (MDD) and optimum moisture content (OMC) of the clayey soil. MDD values increases by the addition of GD because the water holding capacity of the soil decreases due to its non-cohesive nature and there by results in the decrease of OMC.

Table 4.4 MDD and OMC values of Soil + GD stabilized soil

Content (%)	MDD (g/cc)	OMC (%)
Soil+10% GD	1.95	13.5
Soil+15% GD	1.96	12
Soil+20% GD	1.97	11.5

The compaction test is to obtain the moisture-density relationship of the soil-additive mixtures were conducted according to IS: 2720 (Part VIII)-1965. It is observed that addition of varying proportions of CLS and GD properties of CLS and Granite Dust

4.5 Test Results of CBR Test for soil stabilization

4.5.1 CBR Test of soil stabilization with 1% CLS

The CBR test is done as per IS: 2720 Part XVI- 1979 on the soil. In this experiment we have taken soil + 1% CLS. The standard load (PS) corresponding to 2.5mm penetration of the plunger into the standard sample is reported to be 1370 kg and 5mm penetration it was found to be 2055kg. In this proportion 2.5mm penetration value is 2.4% and 5mm penetration is 2.2% hence it is accepted.

Table 4.6 Load vs. Penetration soil sample with 1%CLS

Penetration in mm	Load (Kg/cm2)
0.5	5.65
1	16.95
1.5	28.25
2	33.9
2.5	34.1
3	39.55
3.5	44.2
4	45.85
4.5	47.5
5	51.23
5.5	57.21
6	62.15
6.5	67.8
7	69.32
7.5	73.45
8	75.65
8.5	79.55
9	84.75
9.5	89.72
10	92.33

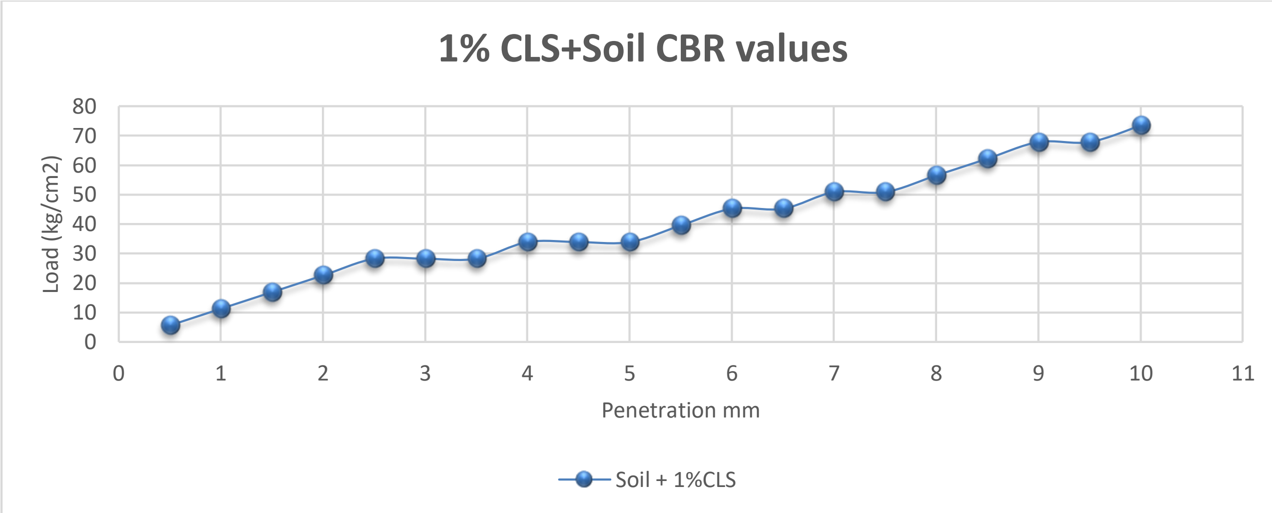


Fig 4.5 Load vs. Penetration of soil sample with 1% CLS + Soil

4.4.2 CBR Test of soil stabilization with 2% CLS

The CBR test is done as per IS: 2720 Part XVI- 1979 on the soil. In this experiment we have taken soil + 2% CLS. The standard load (PS) corresponding to 2.5mm penetration of the plunger into the standard sample is reported to be 1370 kg and 5mm penetration it was found to be 2055kg. In this proportion 2.5mm penetration value is 2.5% and 5mm penetration is 2.1% hence it is accepted.

Table 4.7 Load vs. Penetration soil sample with 2% CLS

penetration in mm	Load (Kg/cm2)
0.5	5.65
1	11.3
1.5	16.95
2	28.25
2.5	35.1
3	39.9
3.5	39.55
4	42.5
4.5	45.2
5	47.85
5.5	50.85
6	52.36
6.5	56.5
7	62.15
7.5	64.36
8	67.8
8.5	69.74
9	73.45
9.5	79.1
10	80.33

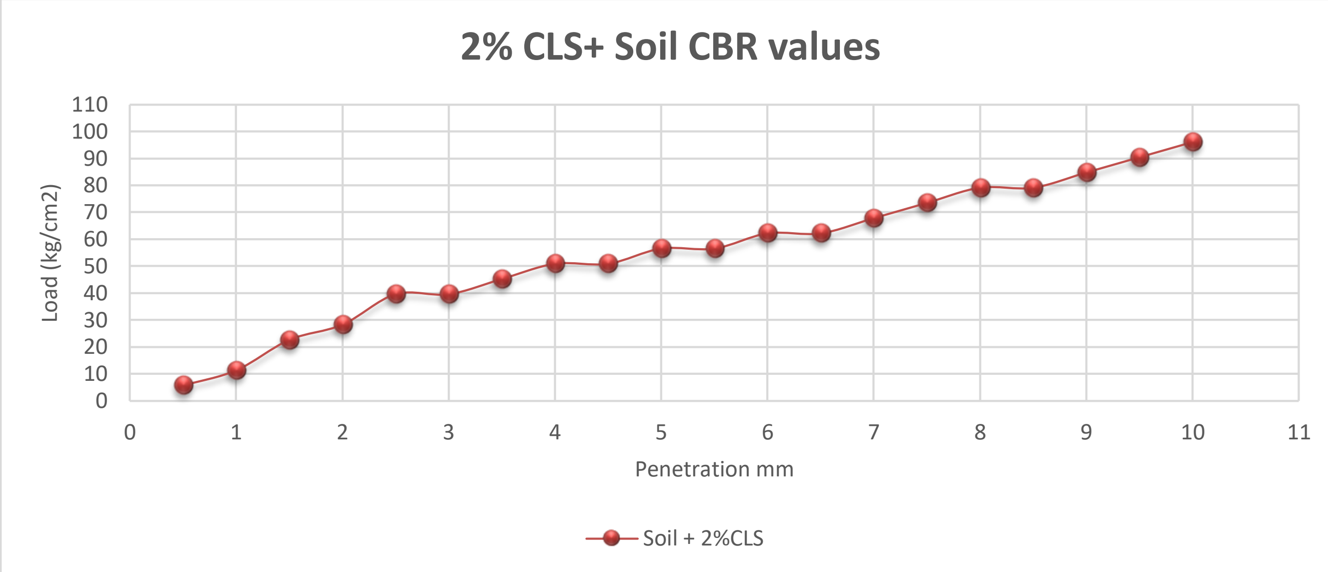


Fig 4.5 (a) Load vs. Penetration of soil sample with 2% CLS + Soil

The standard load (ps) corresponding to 2.5mm penetration of the plunger into the standard sample is reported to be 1370kg and for 5mm penetration it was found to be 2055kg. California Bearing ratio values are useful in estimating the thickness of flexible pavements. Based on the CBR values of sub grade and on the traffic volume expected on the road one can design the thickness of flexible pavement. The soil sample is treated with optimum percentage of additives the maximum CBR values are obtained. The unsoaked condition CBR values for 1%, 2% and 3% CLS + treated soil are 2.4%, 2.5% and 2.6%. The variation in CBR values for treated soil with various percentages of unsoaked conditions are shown in below figures.

Fig 4.5 (b) Load vs. Penetration of soil sample with 3% CLS + Soil

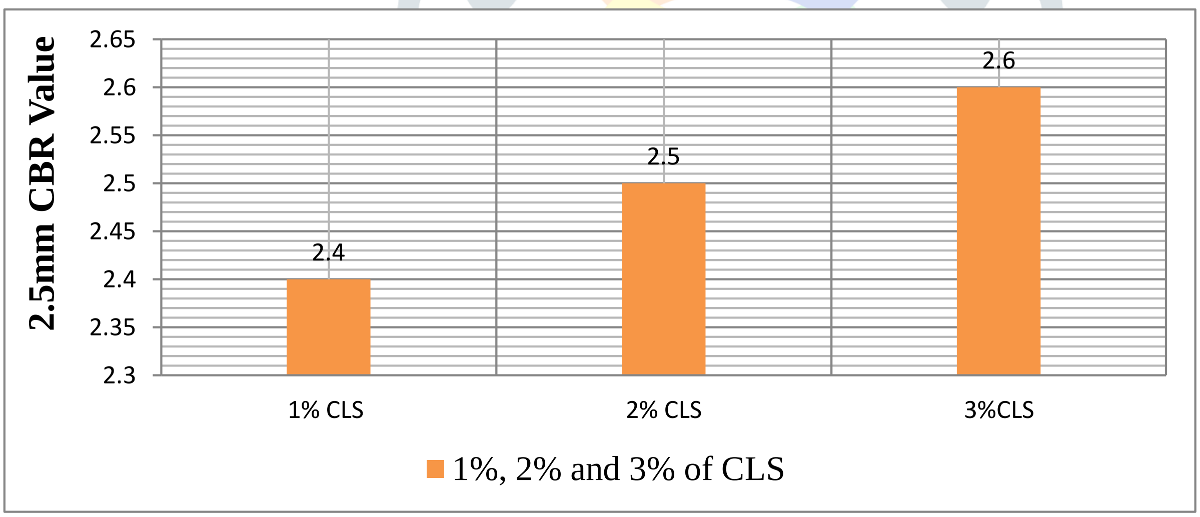


Fig 4.6 2.5mm @ 1%, 2% and 3% of CLS CBR values

4.4.4 CBR Test of soil stabilization with 10% GD

The CBR test is done as per IS: 2720 Part XVI- 1979 on the soil. In this experiment we have taken soil + 10% GD. The standard load (PS) corresponding to 2.5mm penetration of the plunger into the standard sample is reported to be 1370 kg and 5mm penetration it was found to be 2055kg. In this proportion 2.5mm penetration value is 2.5% and 5mm penetration is 1.6% hence it is accepted.

4.4.5 CBR Test of soil stabilization with 15% GD

The CBR test is done as per IS: 2720 Part XVI- 1979 on the soil. In this experiment we have taken soil + 15% GD. The standard load (PS) corresponding to 2.5mm penetration of the plunger into the standard sample is reported to be 1370 kg and 5mm penetration it was found to be 2055kg. In this proportion 2.5mm penetration value is 2.7% and 5mm penetration is 2.1% hence it is accepted.

4.4.6 CBR Test of soil stabilization with 20% GD

The CBR test is done as per IS: 2720 Part XVI- 1979 on the soil. In this experiment we have taken soil + 20% GD. The standard load (PS) corresponding to 2.5mm penetration of the plunger into the standard sample is reported to be 1370 kg and 5mm penetration it was found to be 2055kg. In this proportion 2.5mm penetration value is 2.8% and 5mm penetration is 2.4% hence it is accepted.

5 Design of Flexible pavement by CBR method

Sample 1:

The CBR value of Natural subgrade soil @2.5mm penetration is 2.1 and the thickness of pavement can be found out by using the design formula given below.

CBR corresponding to 2.5mm penetration = 2.1%

Assume, Average Daily Traffic (ADT) = 250

Annual rate of growth of traffic (r) = 5%

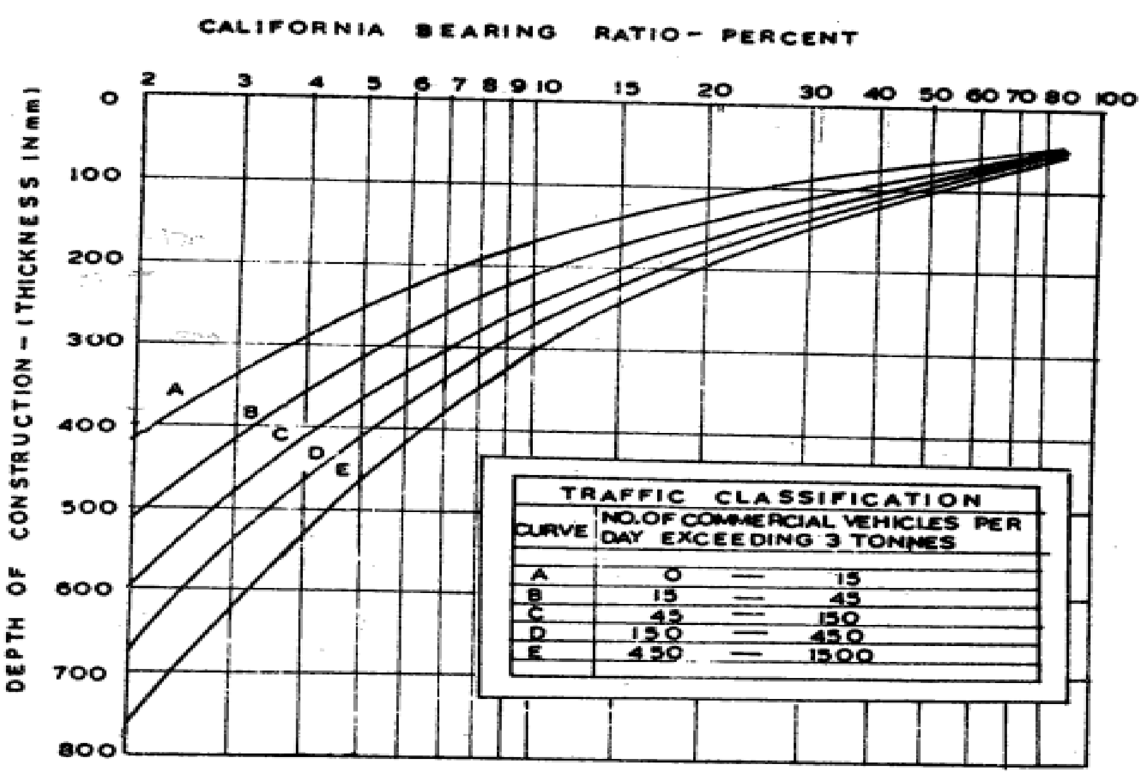
Time taken for pavement construction (n) = 1 year

No. of vehicles for design (A) = P (1 + r)(n + 10)

= 250(1 +5/100)(1 + 10)

= 427.58 vehicles/day

= 430 vehicles/day



CBR Curves for Flexible Pavement Design

Thus 580 mm of pavement materials is required to cover the natural soil subgrade having 2.1% CBR value.

Sample:2

The CBR value of Natural subgrade soil+3% Calcium lignosulfonate+20% Granite dust @2.5mm penetration is 3.98 and the thickness of pavement can be found out by using the design formula given below.

CBR corresponding to 2.5mm penetration = 3.98%

Assume, Average Daily Traffic (ADT) = 250

Annual rate of growth of traffic (r) = 5%

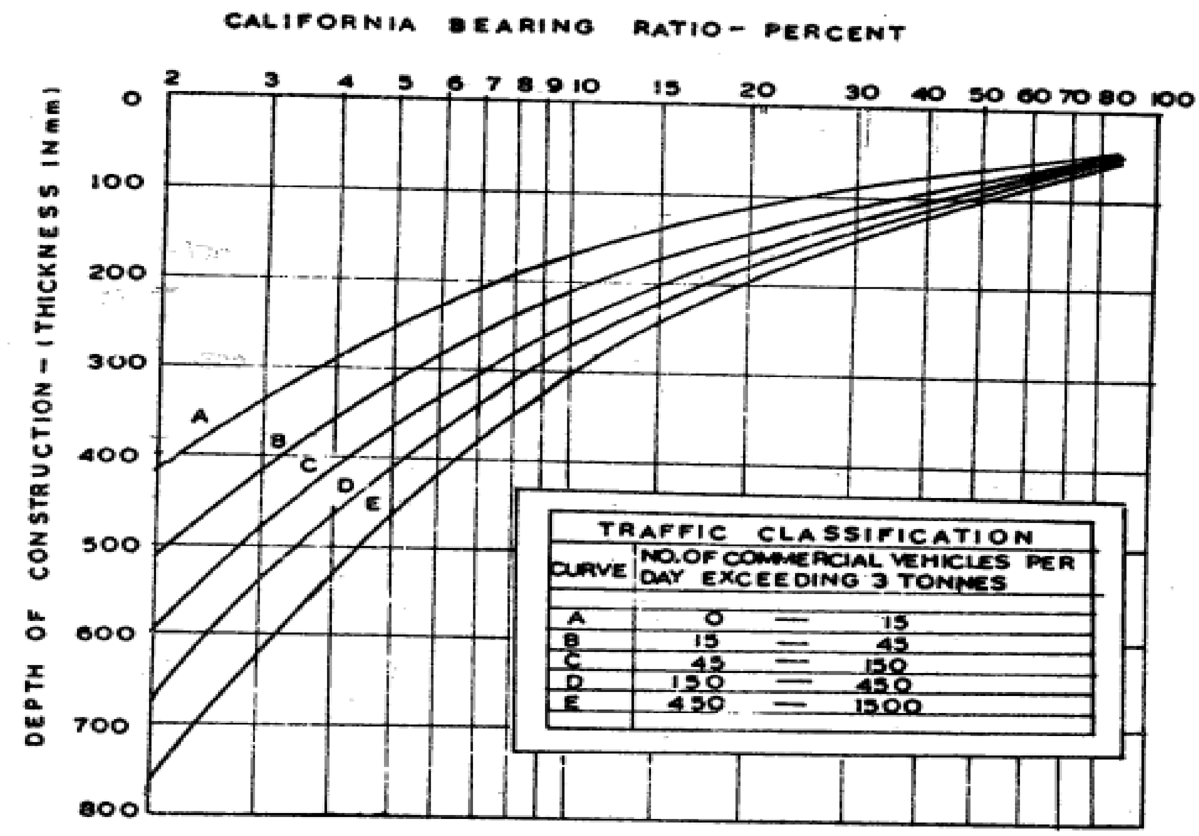
Time taken for pavement construction (n) = 1 year

No. of vehicles for design (A) = P (1 + r)(n + 10)

= 250(1 +5/100)(1 + 10)

= 427.58 vehicles/day

= 430 vehicles



CBR Curves for Flexible Pavement Design

Thus 460 mm of pavement materials is required to cover the natural soil+3% Calcium lignosulfonate+20% Granite dust subgrade having 3.98% CBR

CONCLUSIONS

Based on the results obtained and comparisons made in the present study, the following conclusions can be drawn:

- The chosen soil is clayey soil with fines 68% which is not suitable for construction activities, so it is required to be stabilized.
- The plasticity index of soil at natural state is 18%, by the addition of CLS it decreases due to it is non cohesive nature and the same is observed in Granite Dust also.
- MDD values increases by the addition of CLS and GD because the water holding capacity of the soil decreases due to its non-cohesive nature and there by results in the decrease of OMC
- Addition of both CLS and Granite Dust to the soil in different proportions gives better results compared to individual combinations due to material properties of CLS and Granite Dust which reduces the high swelling nature of the original soil
- Based on the CBR values for the combination of CLS and granite dust mixes, it may be suitable as a subgrade material for low volume roads & medium volume roads.
- Based on the CBR values of Natural soil taken at site for finding the total thickness of flexible pavement came 580MM.Whereas adding the additives of CLS and granite dust mixes the total thickness of flexible pavement may found as 460MM. Hence it may be suitable as a subgrade material for medium volume roads due to increase of safe bearing capacity.

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