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Effect of Saturation on Soil Subgrade Strength

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ABSTRACT- Pavements are made up of many materials. The qualities of the resulting pavement are determined by these materials, their related properties, and their interactions. As a result, knowing these materials, how well they are classified, and how they operate is critical to understanding pavement. The materials used in highway building are of particular interest to highway engineers. This necessitates not only a detailed knowledge of the soil and aggregate parameters that influence pavement stability and durability, as well as the binding elements that may be applied to improve these pavement characteristics. Soil is a mass or deposition of earth material that may be easily mined using heavy machinery in the field or decomposed by mild mechanical methods in the laboratory, generated organically from mineral weathering or decay of plants.

This research deals with the effect of moisture on the strength of soil. Six different soil samples are selected for the experimentation. These samples are classified in to various types. This classification was done based on sieve analysis, liquid limit, plastic limit, plastic index and plasticity chart. The samples were classified as ML, CL, SC and SM. The strength of soil is measured through its CBR value. The CBR value was calculated for unsoaked condition and soaked condition for 1, 2,3 and 4 days and these values were compared. The values of CBR show there is decrease in CBR for all the soil samples.

Key words: CBR, ML,CL,SC,SM

I. INTRODUCTION

The quality of subgrades and sub bases determines the performance of pavements. A stable subgrade and a well-drained sub base aid in the production of a long-lasting pavement. In terms of key engineering, a subgrade and sub base with a high level of spatial uniformity. Shear strength, stiffness, volumetric stability, and permeability are examples of parameters. The pavement system's successful performance is critical. Several environmental issues Temperature and moisture are two elements that influence these geotechnical properties both in the short and long run the subgrade and sub base serve as the basis for the structure. Higher layers of the pavement system and are crucial in mitigating the negative effects of Climate, as well as static and dynamic pressures caused by traffic [1]. The pavement crust, whether flexible or rigid, rests atop a soil foundation on an embankment or cutting, which is sometimes referred to as subgrade. Subgrade is a compacted layer, typically of naturally occurring local soil, considered to be 500/300 mm deep, located directly beneath the pavement crust and serving as a suitable foundation for the pavement [2]. The embankment's subgrade is compacted in two levels, usually to a higher quality than the embankment's bottom part. Because of the traffic loads, the soil in the subgrade is generally stressed to a specific minimum degree of stress, and the subgrade soil should be of good quality and suitably compacted.

1) Subgrade Performance

- The performance of a subgrade is often determined by three basic qualities, which are briefly outlined below:
- The subgrade must be able to support loads transmitted from the pavement structure. The degree of compaction, moisture content, and soil type all have an impact on load bearing capability. A good subgrade is one that can withstand a high level of loading without deforming excessively [3].
- Moisture content affects a variety of subgrade qualities, including load bearing capacity, shrinkage, and swelling. A variety of factors can alter moisture content, including drainage, groundwater table elevation, infiltration, and pavement porosity (which can be assisted by cracks in the pavement). Significantly damp subgrades will generally deform excessively underweight.
- Shrinkage and/or swelling: Depending on their moisture content, some soils shrink or swell. Furthermore, soils with high fines content may be prone to frost heave in colder climates. Any pavement type built over shrinkage, swelling, or frost heave will distort and split [4].

2) Desirable Properties

Subgrade soil has the following desirable features as a roadway material:

- Incompressibility
- Strength permanency
- Withstand capability (Stability)
- Superior drainage
- Low change in volume during adverse conditions of weather and ground water table [5].

The California Bearing Ratio is the most commonly used measure to assess pavement layer strength (CBR). The CBR value is influenced by the water content, dry density, and texture of the soil. In general, the CBR test in the laboratory is performed using test samples generated at the expected dry density and water content in the field Whereas the Field dry density can be predicted.

A. California Bearing Ratio Test

The California Bearing Ratio Test (CBR Test) is a penetration test designed by the California State Highway Department (U.S.A.) to assess the bearing capability of subgrade soil. O.J. Porter of the California Highway Department was the first to establish or invent the CBR test in 1920. It is also known as a load-deformation test, and it is performed in the laboratory or in the field. The results are often used to determine the thickness of pavement layers, base course, and other layers of a particular traffic loading using an empirical design chart. Initially it practiced for the design of surfaced and un-surfaced airfields which is still based upon CBR today [6].

The CBR defines the thickness of the various pavement elements. The CBR test measures the force per unit area required to penetrate soil mass with a 50mm circular plunger at a rate of 1.25mm/min. There are comparisons made between load resistances (penetration) and plunger penetration. The California bearing ratio, CBR, is defined as the proportion of the load resistance (test load) of a given soil sample to the standard load at 2.5mm or 5mm penetration [7].

CBR = (Test load/Standard load) ×100

For 2.5mm and 5mm penetrations, the standard loads are 1370 kg and 2055 kg, respectively. The CBR test is performed on a small scale dial reading penetration with probing ring divisions. The proving ring divisions that correspond to the penetrations are used. If the maximum load and penetration occur at a penetration of less than 12.5 mm, the maximum load and penetration are recorded. The curve is mostly convex upwards, however due to surface flaws, the first segment of the curve may be concave upwards. After that, a correction is made by drawing a tangent to the curve at the point of greatest slope. The adjusted origin will be where the tangent intersects the abscissa. CBR values are typically determined for 2.5mm and 5mm penetrations. CBR values at 2.5mm penetration are typically greater than those at 5mm penetration, and in such cases, the former is used as the CBR value for design purposes. If the CBR value for a 5mm penetration surpasses that for 2.5mm, the test is repeated. If the results are identical, the bearing ratio corresponding to 5mm penetration is used for design [8].

Given the foregoing, it has been proposed in this project to investigate the various strength properties of different types of soil made at different moisture and density levels, as well as to draw general conclusions about the effects of moisture conditions on the determination of different strength parameters, in order to achieve the most viable and economical pavement design.

II. OBJECTIVES

- To determine the strength: - The soil's inherent properties, such as density and moisture between its particles, influence its ability to hold up. Under normal situations, burnt construction dirt is the weakest. When there is a significant amount of moisture in the soil, however, the closely packed water molecules give additional support and grip due to the creation of air-water interfaces. You can determine how much weight your field can support by determining that the soil contains enough moisture.
- To determine the compaction: - Soils that have been highly compacted typically have lower moisture content, lower pore volume, and higher density, making them an excellent building material. However, if you are building a road, it is best to keep the soil moist. Before constructing a substructure or foundation, you must have well-compacted soil because rolling dry soil is useless. The reason for this is that it will not acquire the appropriate density and will hence be prone to shrinkage once the heavy weight is added. To avoid these outcomes, you must first determine the moisture content of the ground before and during construction.
- 3. To establish the soil's Optimum Moisture Content capacity: - OMC is generally used to determine the quantity of soil moisture required for complete compaction. OMC is generally used to determine the quantity of soil moisture required for complete compaction. Without establishing this element, you could construct a skyscraper only to discover it leaning on one side later on due to increasing soil compaction. The worst part is that it may not emerge immediately during construction but may manifest later when building is completed, resulting in substantial losses to the owner and endangering the lives of the residents. To avoid this blunder, measure the moisture content before making your initial action.
- To study effect of moisture variation on Direct Shear Test.
- To study effect of moisture variation on CBR.
- To determine index properties of soil.

III. MATERIAL & METHODOLOGY

A. MATERIAL

3) Soil

The entire study was done of the soil collected locally from Ganderbal area of Kashmir.

Initially experiments were conducted to find out different properties of soil such as index properties, grain size distribution and differential free swell index. Later on heavy compaction tests were conducted to find out the optimum moisture content & corresponding maximum dry density. Then CBR tests were made at different moisture contents including OMC and analysis made to investigate the variation of CBR with respect to different days of soaking.

B. EXPERIMENTAL INVESTIGATIONS

- Six samples were collected and named as A, B, C, D, E and F.
- Performing Sieve analysis on soil to classify it as fine and coarse aggregate soil.
- Performing plastic limit test, Liquid limit test.
- Finding the values of plastic index and plotting the plasticity chart.
- Performing the standard proctor test on soil samples to find the OMC and MDD.
- Performing the direct shear test on soil samples to calculate the shear strength.
- Performing the CBR test on soil samples for specimens kept in soaked and unsoaked condition, with the soaking period ranging from 1,2,3 and 4 days.

- Comparing the result and concluding.

IV. RESULTS AND DISCUSSION

A. Sieve analysis

In soil samples A, B and C the results of sieve analysis show that more than 50% of soil particles are of size lesser than 0.075 mm. Thus, these soils are classified as fine aggregate soils which are further classified as silt (M) and clay(C). The further classification of fine aggregate soils is done after performing the liquid limit and plastic limit test on the samples. In samples D, E and F size of particles passing through 0.0075 mm sieve is less than 50% and thus are classified as coarse aggregate soils which are further classified as Gravel (G) and Sand(S).

Table 1: Sieve analysis of Sample C

Sieve size	Weight retained (gm)	% Weight retained	% Weight passing
4.75 mm	0	0	100
2 mm	26.36	2.19	97.81
1 mm	32.68	2.72	97.28
0.6 mm	37.64	3.13	96.87
0.425 mm	39.48	3.29	96.71
0.3 mm	43.64	3.63	96.37
0.212 mm	50.78	4.23	95.77
0.15 mm	65.68	5.47	94.53
0.075mm	903.84	75.32	

Table 2: Sieve analysis of Sample D

Sieve size	Weight retained (gm)	% Weight retained	% Weight passing
4.75 mm	0	0	100
2 mm	61.86	5.15	94.85
1 mm	68.34	5.69	94.31
0.6 mm	79.31	6.60	93.40
0.425 mm	88.42	7.36	92.64
0.3 mm	97.48	8.12	91.88
0.212 mm	113.67	9.47	90.53
0.15 mm	124.47	10.37	89.63
0.075mm	141.65	12.05	87.95
PAN	425.40	35.45	

Table 3: Sieve analysis of Sample E

Sieve size	Weight retained (gm)	% Weight retained	% Weight passing
4.75 mm	0	0	100
2 mm	86.97	7.24	92.76
1 mm	95.45	7.95	92.05
0.6 mm	109.42	9.11	90.89
0.425 mm	118.53	9.87	90.13
0.3 mm	138.01	11.50	88.5
0.212 mm	143.78	11.98	88.02
0.15 mm	154.58	12.88	87.12
0.075mm	355.68	29.64	

The tables 1,2 and 3 shows the values of sieve analysis for the soil samples C, D and E. Based on these values these samples are classified as fine and coarse aggregate.

B. Liquid Limit

Table 4: Liquid Limit values

Sample	Liquid limit			Average Liquid limit
	Specimen 1	Specimen 2	Specimen 3	
A	21.32	24.41	25.17	23.63
B	23.45	25.36	22.58	23.79
C	28.67	31.87	33.38	31.30
D	36.46	37.84	42.25	38.85
E	37.58	40.38	43.67	40.54
F	27.35	25.46	27.58	26.79

The values in Table 4 show the values of Liquid Limit for the three specimens of all the six samples and the variation in the Liquid Limit for all the samples can be seen.

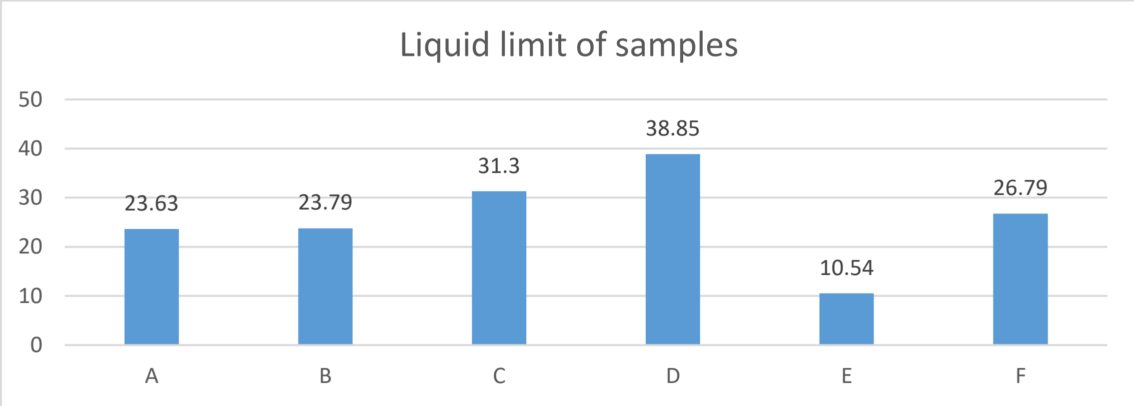

Figure 1: Liquid limit of soil samples

The Figure 1 graph shows the liquid limit for all the samples. The sample with the highest Liquid Limit is the sample D and the one with the lowest Liquid Limit is E.

C. *Plastic Limit*

Table 5: Plastic Limit values

Sample	Plastic Limit			Average Plastic limit
	Specimen 1	Specimen 2	Specimen 3	
A	17.11	20.83	21.15	19.69
B	19.58	21.11	18.62	19.77
C	20.03	23.23	22.14	21.80
D	20.81	21.72	26.02	22.85
E	21.46	25.04	28.81	25.10
F	21.48	23.58	22.50	22.52

The values in Table 5 show the values of Plastic Limit for the three specimens of all the six samples and the variation in the Plastic Limit for all the samples.

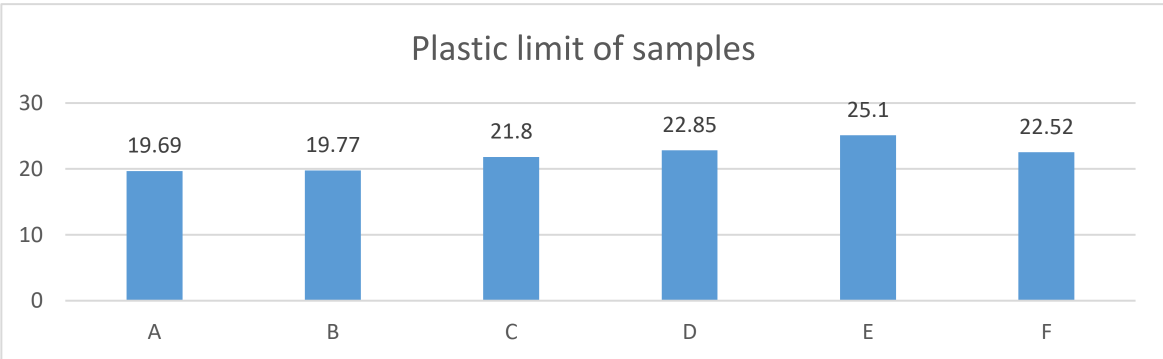

Figure 2: Plastic limit of soil samples

The Figure 2 graph shows the Plastic limit for all the samples. The sample with the highest Plastic Limit is the sample E and the one with the lowest Plastic Limit is A.

Table 6: Plasticity index values

Sample	Plasticity index
A	3.94
B	4.02
C	9.5
D	16.00
E	15.44
F	4.27

The values in Table 6 show the values of Plasticity index for the three specimens of and the variation in the Plastic Limit for all the samples.

D. *Classification Of Soil*

A-line PI = 0.73(LL-20)
U-line = 0.9(LL-8)

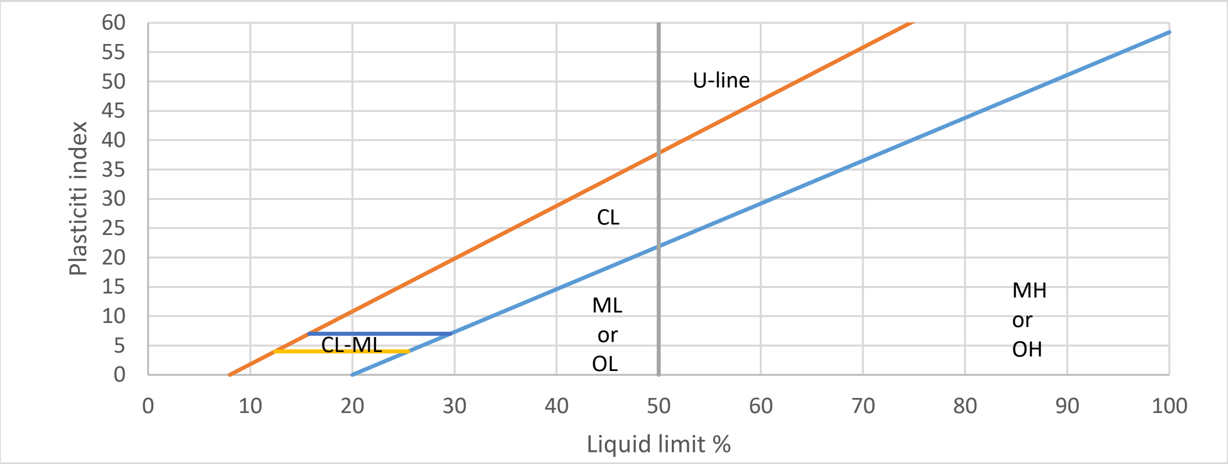

Figure 3: Plasticity Chart

The plasticity chart is shown in figure 3. The values of A and U line are calculated and the graph is plotted and depending on the values of Plasticity index and liquid limit the soil is classified as Clay, Silt, Sand and Organic Soil. These can be further classified as being of high, low or medium plasticity.

Table 7: Soil classification

Samples	Classification
A	ML
B	ML
C	CL
D	SC
E	SC
F	SM

The Table 7 shows the samples being classified in to various types of fine soils. The soil samples are classified based on the results obtained from sieve analysis, liquid limit, plastic limit, plasticity index and plasticity chart.

E. Standard Proctor Test

Table 8: Water content and dry density values

Sample	Water content				Dry density			
	1	2	3	OMC	1	2	3	MDD
A	12.15	13.85	14.12	13.85	1.75	1.92	1.64	1.92
B	11.34	12.17	13.25	12.17	1.64	1.85	1.58	1.85
C	13.95	14.24	14.88	14.24	1.73	1.96	1.86	1.96
D	14.74	15.65	16.17	15.65	1.44	1.67	1.51	1.67
E	10.14	11.38	11.92	11.38	1.58	1.78	1.43	1.78
F	8.87	9.86	10.14	9.86	1.73	1.96	1.79	1.96

The Table 8 shows values of OMC and MDD for the various samples of soil. Three values of water content and their dry density values were taken and the value at the optimum was chosen as the OMC and MDD.

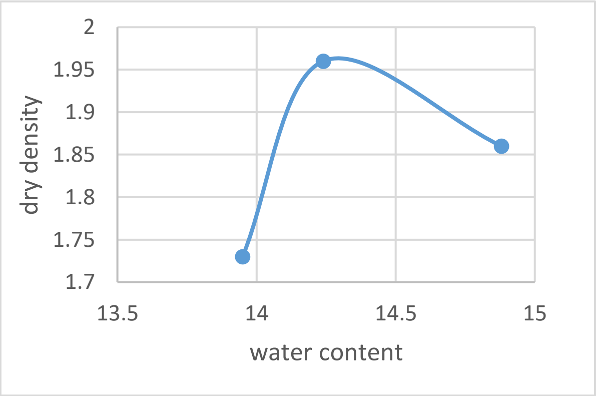

Figure 4: Dry density vs water content for sample C

Above the figure 4 shows the values of MDD and OMC for sample C which are takes as the optimum values for dry density and the water content of the said dry density.

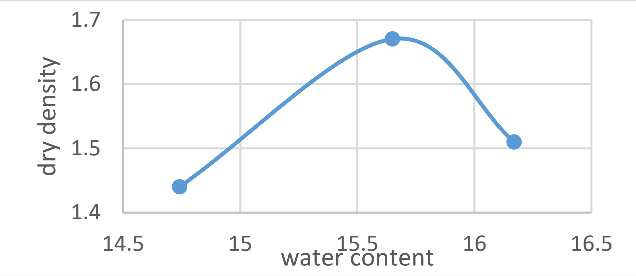

Figure 5: Dry density vs water content for sample D

Above the figure 5 above shows the values of MDD and OMC for sample D which are takes as the optimum values for dry density and the water content of the said dry density.

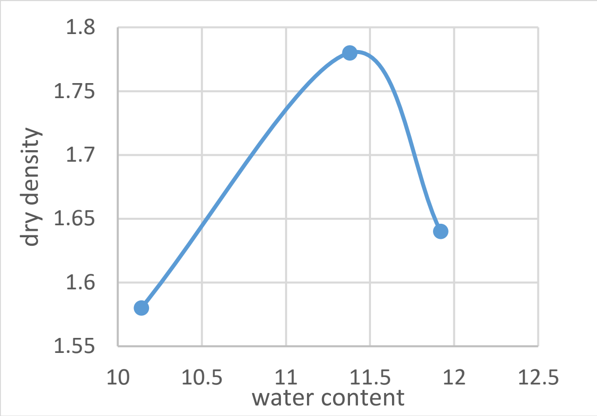

Figure 6: Dry density vs water content for sample E

The figure 6 above shows the values of MDD and OMC for sample E which are takes as the optimum values for dry density and the water content of the said dry density.

F. DIRECT SHEAR TEST

The shear force values are used to calculate the shear stress values and the shear stress and normal stress values are plotted with the maximum of shear stress for every normal stress value. The values of cohesion and angle of internal friction are also calculated from the graph.

Table 9: Shear stress values for sample A

50 kPa		100kPa		200 kPa	
Shear force (N)	**Shear stress (KN/m^2)**	**Shear force (N)**	**Shear stress (KN/m^2)**	**Shear force (N)**	**Shear stress (KN/m^2)**
0	0	0	0	0	0
22	7.33	9	3	40	13.33
45	15.00	10	3.33	107	35.67
60	20.00	54	18	151	50.33
70	23.33	78	26	190	63.33
78	26.00	97	32.33	221	73.67
84	28.00	116	38.67	241	80.33
88	29.33	134	44.67	264	88
89	29.67	144	48	280	93.33
89	29.67	153	51	293	97.67
88	29.33	159	53	301	100.33
86	28.67	164	54.67	304	101.33
84	28.00	167	55.67	306	102
		168	56	309	103
		170	56.67	315	105
		169	56.33	312	104
		169	56.33	310	103.33
		166	55.33	310	103.33

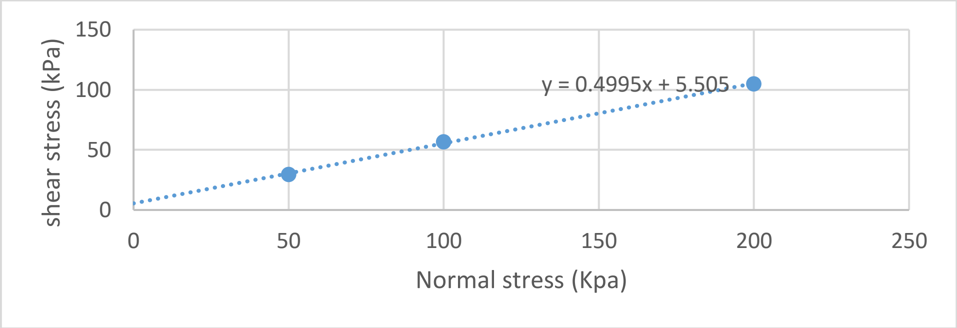

Figure 7: Shear stress vs normal stress for sample A

The normal stress for the test is 50,100 and 200 kPa. These values are represented in the table above and in figure 10. The figure 10 is plotted between the shear stress and normal stress.

Table 10: Cohesion and angle of friction for sample A

Cohesion	Slope	Angle of internal friction
5.5	0.4995	0.461 radians or 26.54

The value of cohesion is found from the intercept that the line makes with the y axis. The angle of friction is calculated using the slope value.

Angle of internal friction = Tan-1 slope = Tan-1 0.4995= 0.461 radians = 24.560.

Table 11: Shear stress values for sample B

50 kPa		100kPa		200 kPa	
Shear force (N)	Shear stress (KN/m^2)	Shear force (N)	Shear stress (KN/m^2)	Shear force (N)	Shear stress (KN/m^2)
0	0	0	0	0	0
20	7.14	8	2	39	13.13
46	14.68	9	3.13	106	35.27
59	18.95	53	17	150	49.93
76	23.14	77	25	189	63.13
79	25.56	96	32.13	220	73.47
82	27.85	115	38.47	240	80.13
86	28.63	133	44.47	263	87
87	29.47	143	47	279	93.13
87	29.47	152	50	292	97.47
86	29.03	157	52	300	99.73
84	28.27	163	54.17	303	100.33
82	27.86	166	55.17	305	101
		167	55	307	102
		168	56.17	313	104
		167	56.13	310	103
		167	56.03	308	102.33
		165	55.13	308	102.33

The normal stress for the test is 50,100 and 200 kPa. These values are represented in the table above and in figure 10. The figure 10 is plotted between the shear stress and normal stress.

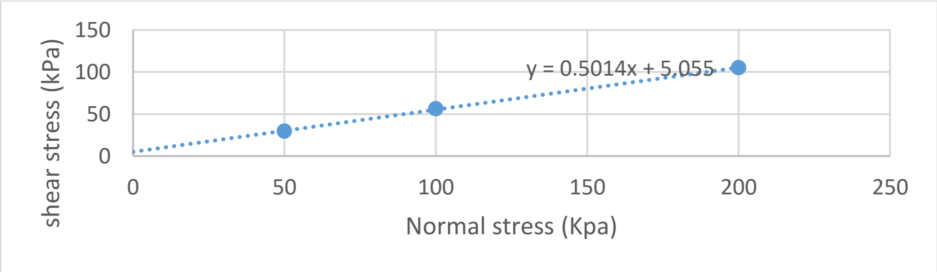

Figure 8: Shear stress vs normal stress for sample B

Table 12: Cohesion and angle of friction for sample B

Cohesion	Slope	Angle of internal friction
5.055	0.5014	0.463 radians or 26.55

The value of cohesion is found from the intercept that the line makes with the y axis. The angle of friction is calculated using the slope value.

Angle of internal friction = Tan-1 slope = Tan-1 0.5014= 0.463 radians = 26.550.

Table 13: Shear stress values for sample C

50 kPa		100kPa		200 kPa	
Shear force (N)	Shear stress (KN/m^2)	Shear force (N)	Shear stress (KN/m^2)	Shear force (N)	Shear stress (KN/m^2)
0	0	0	0	0	0
18	6.54	7	2	38	12.13

44	14.18	8	3.03	105	34.27
58	18.15	50	15	149	48.93
75	22.14	75	24	187	62.13
77	24.56	94	32.03	218	72.47
80	26.85	114	37.47	238	79.13
84	27.63	130	43.47	260	86
85	28.47	140	46	275	92.13
85	28.47	150	48	290	96.47
84	28.03	154	50	298	98.73
82	27.27	160	52.17	301	99.33
80	26.86	164	54.17	300	100
		163	54	305	100
		164	53.17	310	101
		163	53.13	308	100
		163	53.03	306	101.33
		160	53.13	306	101.33

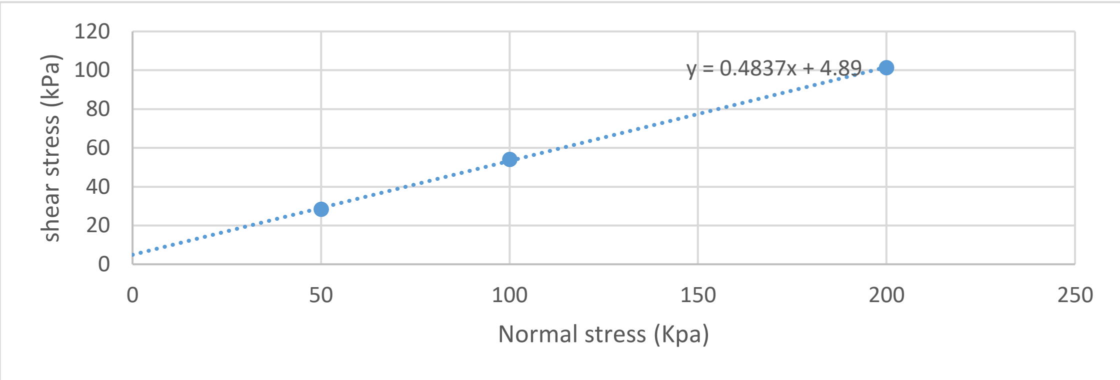

Figure 9: Shear stress vs normal stress for sample C

Table 14: Cohesion and angle of friction for sample C

Cohesion	**Slope**	**Angle of internal friction**
4.89	0.4837	0.449 radians or 25.75

The value of cohesion is found from the intercept that the line makes with the y axis. The angle of friction is calculated using the slope value.

Angle of internal friction = Tan-1 slope = Tan-1 0.4837= 0.449 radians = 25.750.

Table 15: Shear stress values for sample D

50 kPa		**100kPa**		**200 kPa**	
Shear force (N)	**Shear stress (KN/m^2)**	**Shear force (N)**	**Shear stress (KN/m^2)**	**Shear force (N)**	**Shear stress (KN/m^2)**
0	0	0	0	0	0
24	9.33	9	4	40	15.33
46	17.00	10	5.33	107	39.67
62	22.00	54	20	151	52.33
72	26.33	78	28	190	66.33
80	28.00	97	33.33	221	78.67
86	30.00	116	40.67	241	83.33
90	31.33	134	46.67	264	90
91	31.67	144	50	280	94.33
91	32.67	153	53	293	99.67
90	30.33	159	56	301	106.33
88	29.67	164	55.67	304	109.33
86	29.00	167	56.67	306	110
		168	58	309	106
		170	59.67	315	110.50
		169	58.33	312	104
		169	58.63	310	104.33
		166	57.43	310	106.33

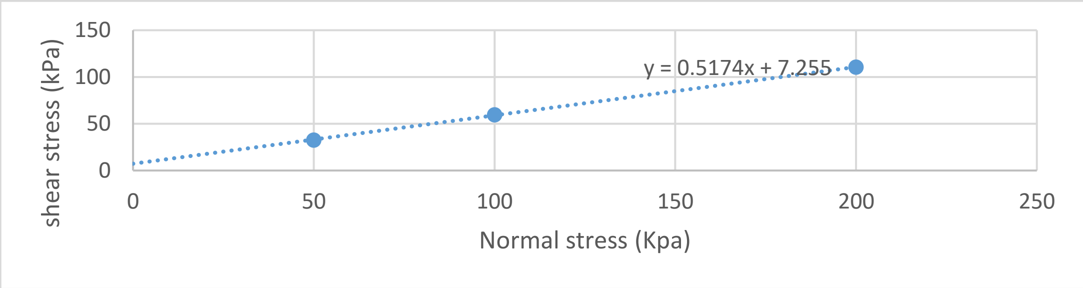

Figure 10: shear stress vs normal stress for sample D

Table 16: Cohesion and angle of friction for sample D

Cohesion	Slope	Angle of internal friction
7.255	0.5174	0.475 radians or 27.26

The value of cohesion is found from the intercept that the line makes with the y axis. The angle of friction is calculated using the slope value.

Angle of internal friction = Tan-1 slope = Tan-1 0.5174= 0.475 radians = 27.260.

G. CALIFORNIA BEARING RATIO TEST

Table 17: CBR values for all samples

Sample	Unsoaked CBR	1 day soaked CBR	2 days soaked CBR	3 days soaked CBR	4 days soaked CBR
A	11.85	10.24	9.57	8.26	7.02
B	9.87	9.36	8.84	8.04	7.23
C	7.64	7.14	6.76	5.20	4.34
D	7.46	6.97	6.13	5.25	3.93
E	8.83	8.25	7.64	6.56	5.24
F	6.56	5.85	4.82	3.74	2.85

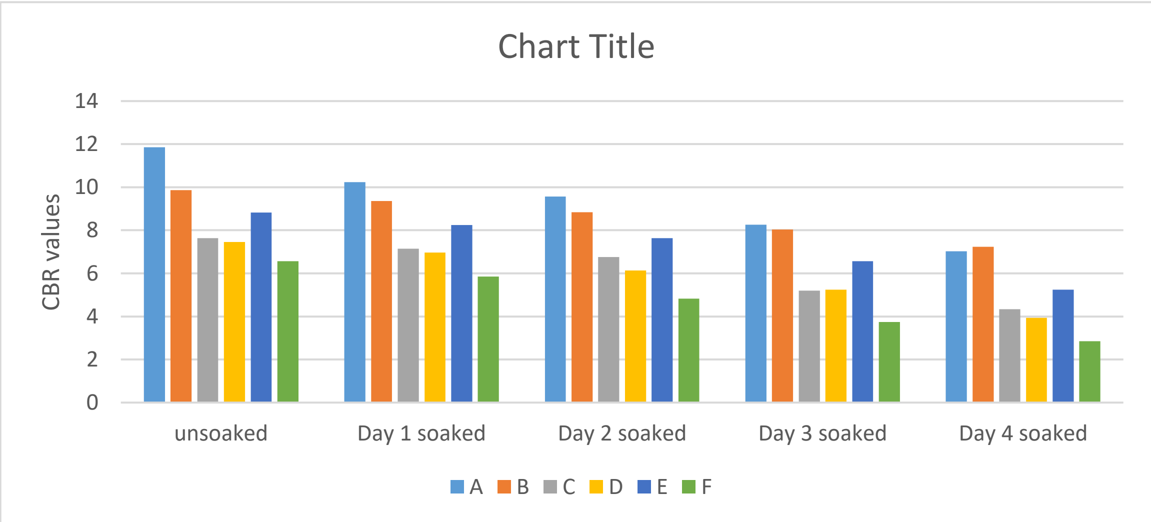

Figure 11: CBR values for all samples

The CBR test was performed for unsoaked specimen and soaked specimen ranging from 1,2,3 and 4 days. The CBR test were performed for all the samples. The results from the CBR test show that there is a decrease in CBR values as the soaked period of the specimens is increased. The decrease in CBR was 13.58% for sample A for a period of 1 day of soaking. The decrease in CBR was 19.24% for sample A for a period of 2 days of soaking. The decrease in CBR was 30.29% for sample A for a period of 3 days of soaking. The decrease in CBR was 40.75% for sample A for a period of 4 days of soaking. The decrease in CBR was 5.16% for sample B for a period of 1 day of soaking. The decrease in CBR was 10.43 % for sample B for a period of 2 days of soaking. The decrease in CBR was 18.54% for sample B for a period of 3 days of soaking. The decrease in CBR was 26.74% for sample B for a period of 4 days of soaking. The decrease in CBR was 6.54 % for sample A for a period of 1 day of soaking. The decrease in CBR was 11.51.% for sample C for a period of 2 days of soaking. The decrease in CBR was 31.93% for sample C for a period of 3 days of soaking. The decrease in CBR was 43.19 % for sample C for a period of 4 days of soaking. The decrease in CBR was 6.56% for sample D for a period of 1 day of soaking. The decrease in CBR was 17.82% for sample D for a period of 2 days of soaking. The decrease in CBR was 29.62 % for sample D for a period of 3 days of soaking. The decrease in CBR was 47.31 % for sample D for a period of 4 days of soaking. The decrease in CBR was 6.79 % for sample E for a period of 1 day of soaking. The decrease in CBR was 13.47 % for sample E for a period of 2 days of soaking. The decrease in CBR was 25.70 % for sample E for a period of 3 days of soaking. The decrease in CBR was 40.65% for sample A for a period of 4 days of soaking. The decrease in CBR was 10.82 % for sample F for a period of 1 day of soaking. The decrease in CBR was 26.52% for sample E for a period of 2 days of soaking. The decrease in CBR was 42.98 % for sample E for a period of 3 days of soaking. The decrease in CBR was 56.55 % for sample E for a period of 4 days of soaking.

V. CONCLUSIONS

In this research, six different soil samples A, B, C, D, E and F were collected and were subjected to various tests to study the effect of moisture on the soil samples.

The soils were classified as A, B as ML C as CL D, E as SC and F as SM. This classification was done based on liquid limit, plastic limit, plasticity index and plasticity chart. The CBR test show that there is decrease in the CBR value for soaked specimen when compared with unsoaked specimens.

This reduction in CBR percentage increases as the soaking period for the specimen is increased. The maximum reduction in CBR value is seen for F sample for soaking period of 4 days and the reduction is56.55%.

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