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INFLUENCE OF PLUNGER FRICTION ON CALIFORNIA BEARING RATIO OF LOW PLASTICITY CLAY SOILS

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Abstract: Rehabilitation and retrofitting of existing pavements justify the residual strength assessment to guide implementation of necessary life enhancement exercise and have necessarily become a primary task of the highway construction industry. Laboratory California Bearing Ratio (CBR) tests were performed at various moisture contents such as OMC-1%, OMC and OMC+1% (OMC = Optimum Moisture Content). The values of CBR are calculated with and without plunger side friction. The moisture content increases from OMC -1% to OMC, there is an increase in CBR and further increase of moisture content from OMC to OMC+1%, there is a decrease in CBR. The CBR estimated considering plunger friction showed about 0.32 to 2.72% increment as compared to the laboratory CBR estimated without considering plunger friction. Similarly The moisture content of samples varying from OMC-1% to OMC+1%, there is increase in UCS and cohesive strength up to OMC and from OMC onwards, there is decrease in Unconfined Compressive Strength (UCS) and undrained cohesive strength of the samples. Further, empirical relationships also established between the CBR and UCS.

Keywords: *CBR, plunger friction, UCS, OMC.*

1.0 Introduction

In geotechnical engineering, in-situ or field tests have been widely using to assess the ground reality conditions. Road transport provides greater utility in transport over short and long hauls of lighter weight commodities and of lesser volumes, as also for passenger transport for short and medium hauls. The pavement construction requires assessment of adequacy of subgrade to have satisfactory support beneath them. Good quality road network with sound soil conditions beneath the surface always contributes for better growth. Thus, the development in road network is regarded as an index of socio-economic and commercial progress. No region or country can flourish, if it lacks adequate transport facilities, especially road network. In order to arrive at an effective and reliable pavement design, accurate material characterization by various techniques is essential.

California Bearing Ratio (CBR) was developed by the California State Highway Department. It is a simple penetration test and was developed to evaluate the strength of sub grades for pavements. This test method is used to evaluate the potential strength or distress state of subgrade, sub-base and base course materials, including recycled materials for use in road and airfield pavements. The CBR value obtained in this test forms an integral part of several flexible pavement design methods. Based on these values, the thickness of subgrade, sub-base and base coarse can be assessed. The effect of testing procedures on load carrying capacity of calcareous sediments (marls) utilizing the CBR, Unconfined Compressive Strength (UCS) and Clegg hammer tests using two different marls (Saad and Osman, 2002). It is observed that the maximum particle size has little effect on the CBR and Clegg Impact values of subgrade soils tested. Also concluded that the mold confinement has showed an increase of about 100% in the CBR values.

The effect of wetting-drying cycles on CBR values of silty subgrade soil treated with lime-micro silica additive as a modern additive stabilizer. The CBR values were greatly increased as the soil was stabilized with lime - microsilica additive. In addition, an increase on the CBR values of the stabilized soil by wetting- drying cycles also noticed (Moayed and Lahiji, 2013). The analysis of CBR is commonly presented in CBR-water content relation but, in unsaturated soil, suction is one of the key parameters for understanding the soil behaviour. Suction-monitored CBR test was done by attaching tensiometers on CBR mold and its surcharge. The standard compaction test on various proportions of sand-kaolin clay mixtures starting from 0% (pure sand), 5%, 10%, and 20% of clay were conducted. The tests were performed with different value of water content in both soaked and unsoaked conditions. The results revealed that the CBR versus matric suction followed a bi-linear variation (Purwana *et al.*, 2012). The strength of granular sub base (GSB) underlain by a sub grade layer in terms of CBR and bearing capacity of GSB-soil subgrade composites. The CBR and shear parameters are determined for different combinations of granular sub base layer and subgrade thickness. The results indicated that with ratio of height and thickness is 0.5, an allowable bearing capacity of 520 kN/m^2 can be achieved (Sureka *et al.*, 2013). And this vale satisfying the minimum required compressive stress of 150 kN/m^2 for a subgrade as quoted in literature. The field evaluation of in-situ test method for construction of pavement layers and embankments. The CBR value for a given soil mainly depend upon its density, molding moisture content, and moisture content after soaking. The result of a CBR test also depends on the resistance to the penetration of the piston (plunger) and finally, the CBR indirectly estimates the shear strength of the material being tested (Munir *et al.*, 2003). The regular and extended dynamic cone penetrometer (DCP) is used for pavement sub soil strength evaluation. From the study, few important findings were brought quantitatively verified by examining the change in dynamic efficiency factor for the two penetrating rods. A corrective equation is suggested when the penetration is not performed vertically in order to isolate the skin friction that develops along penetrating rod and that has significant effect on Dynamic Cone Penetrometer (DCP) values (Livneh *et al.*, 2000).

From the aforementioned literature review, it is noticed that many researchers focused on subgrade strength assessment towards pavement construction. Many important observations were brought out about the factors controlling the CBR of subgrade. Grain size, moisture content, mould size and compaction effort can have major influence on the CBR. It is observed that almost no much literature available about influence of plunger friction on CBR, it may be due to the standard conditions as the researchers are following all over the globe. Still, in this investigation, it is planned to conduct the testing in a standard test mould by additionally considering the plunger friction and bring out the influence of estimated plunger friction on the CBR. The further details of soil samples and comparison of the data has been discussed in the following sections.

2.0 Experimental Investigation

2.1 Soil Samples Used in the Study

In the present study, soils were collected from various locations such-as Patancheru ***-P***, Balanagar ***-B***, Kondapur ***-K***, Vansthalipuram ***-V***, Miyapur – ***M*** and Hi-tech city – ***H,*** around Hyderabad, Andhra Pradesh (A.P), India. Atterberg's limits were quantified and soils were grouped according to IS (Indian Standard) classification types. The soil samples collected from different places are falling under the CL classification as per the IS classification of soils. Intrinsic to belonging class CL, the selected six soil samples are confirmed to have major percentage of fine grained soil. The basic properties with the consistency of all six soils are presented in Table 1.

The CBR tests were conducted with and without considering plunger friction and are studied to understand the variation of CBR values for both the cases. The testing was carried out by maintaining the laboratory controlled conditions.

The samples were processed and stored in airtight containers in the laboratory. The soil samples used in UCS and CBR tests were prepared at the moisture contents such as OMC-1, OMC and OMC+1, where OMC is the optimum moisture content obtained from the standard Proctors compaction test. As the moisture content changes a little in the field with respect to OMC, it is difficult to maintain the exact moisture content (i.e., OMC) in the field as obtained in the laboratory proctors compaction test which is also supported by Rashid *et al.* (2014). So as to account the moisture content variations in the field, the moisture contents such as OMC-1, OMC and OMC+1 are considered appropriately in the current study.

Table 1: Properties of soils used in the study

Soils	*LL*	*PL*	*PI*	*IS Classification*	*Water Content*	*CI*	*Consistency of soil*
					OMC-1%	1.33	Very stiff
Soil P	29	16	13	CL	OMC	1.23	Very stiff
					OMC+1%	1.13	Very stiff
					OMC-1%	1.83	Very stiff
Soil B	36	21.5	14.5	CL	OMC	1.76	Very stiff
					OMC+1%	1.68	Very stiff
					OMC-1%	1.41	Very stiff
Soil K	35	17	18	CL	OMC	1.36	Very stiff
					OMC+1%	1.3	Very stiff
					OMC-1%	1.84	Very stiff
Soil V	38	23	15	CL	OMC	1.76	Very stiff
					OMC+1%	1.69	Very stiff
					OMC-1%	1.6	Very stiff
Soil M	38	21	17	CL	OMC	1.52	Very stiff
					OMC+1%	1.45	Very stiff
					OMC-1%	1.65	Very stiff
Soil H	34	20	14	CL	OMC	1.57	Very stiff
					OMC+1%	1.48	Very stiff

From the Consistency Index (CI) presented at various moisture contents showing very stiff consistency for all soils tested. The consistency index is defined as the ratio of difference in Liquid Limit (LL) and Natural Moisture Content (NMC) to the plasticity index (CI = [(LL-NMC)/PI]). The numerical difference in LL and Plastic Limit (PL) is defined as plasticity index). The grain size distribution curves (Sieve Analysis) for all soils used in the study are presented in Fig. 1. The compaction curves corresponding to standard compaction for all samples tested are presented in Fig. 2. The grain size distribution is presented for all samples in Table 2. From the grain size distributions of soil samples it is observed that the fine to medium sand is varying from 50% to 75% and fines content (passing 0.075 mm sieve) is varying from 10% to 46%. From the standard compaction test results as presented in Table 3, it is noticed that the Optimum Moisture

Content (OMC) of all soils tested is varying from 10.15% to 13% and Maximum Dry Density (MDD) is varying from 18.8 kN/m^3 to 19.8 kN/m^3.

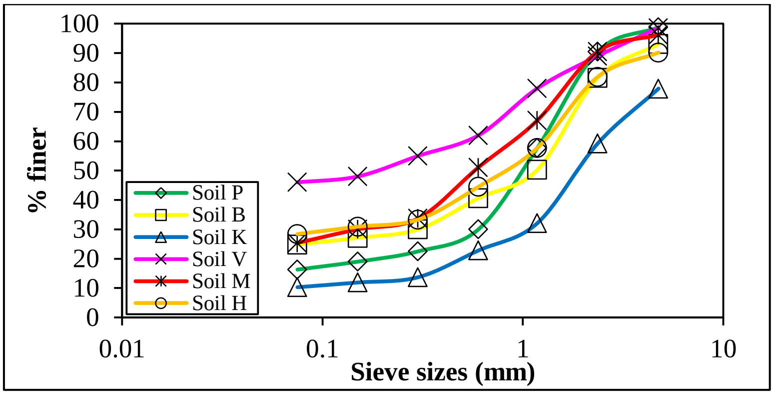

Figure 1: Grain size distribution curves for soils

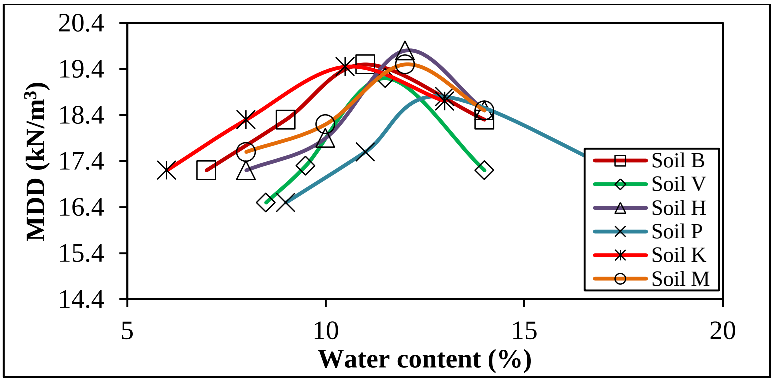

Figure 2: Compaction curves for soils

Table 2: Grain size distribution of soil samples

Soil	*% Gravel*	*% Sand*	*% Fines*
Soil P	3.8	79.9	16.3
Soil B	3.8	70.8	25.4
Soil K	22.1	67.6	10.3
Soil V	3.8	50.2	46
Soil M	3.8	70.8	25.4
Soil H	9.8	61.8	28.4

Table 3: OMC and MDD of soil samples

Soil	*OMC (%)*	*MDD (kN/m^3)*
Soil P	13	18.80
Soil B	11	19.50
Soil K	10.5	19.45
Soil V	11.5	19.20
Soil M	12	19.60
Soil H	12	19.80

2.2 Soil Testing

Soil tests were conducted as per the Indian Standard (IS) code of practice of testing of soils as mentioned below. The liquid limit and plastic limit tests were conducted as per IS: 2720 (Part 5) - 1985. Grain size distribution is as per IS: 2720 (Part 4) – 1985. Standard proctor compaction tests were carried out according to IS: 2720 (Part 8) -1983. The California Bearing Ratio (CBR) tests were carried out as per the IS: 2720 (Part 16) – 1987. The unconfined compressive strength tests were carried out as per the IS: 2720 (Part 10) – 1973.

2.3 Plunger Friction

To understand the effect of friction of plunger on CBR, an analysis is made by using the pile shaft friction concept for estimating the plunger friction. The soils used in the present study are confirming the low compressible clay (CL) and every soil tested in the study are having % fine fraction (< 0.075 mm) from 10 to 46%. Out of six samples, four samples are having % fine fraction more than 30%. Based on the soil classification, all the soil samples are considered under clayey category and accordingly the samples were subjected to unconfined compression strength tests at OMC for obtaining the cohesive strength of soils. As it is known that the pile shaft friction can be estimated by using the static formulae, the same procedure is used in the current study to estimate the friction of plunger which is penetrating into the compacted soil in the CBR mould. The reduction factor or adhesion factors were taken as per the consistency index values for different soils. The adhesion factor, α values used in the study is presented in Table 4. As all the soils tested in the study are falling under the category of very stiff to hard, the adhesion factor 0.3 was chosen from the table for plunger friction calculation.

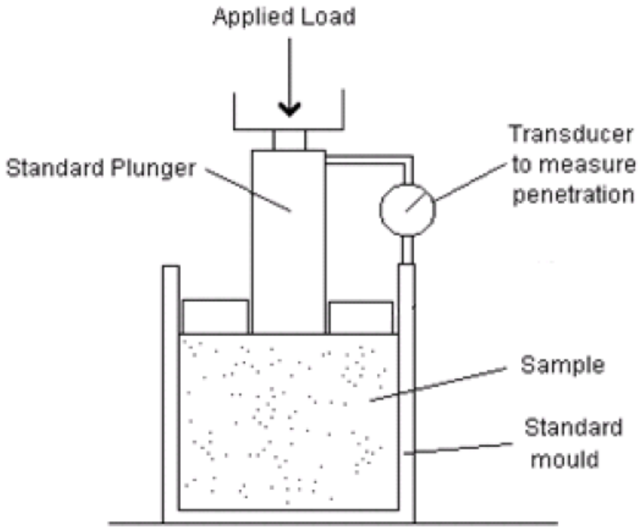

Figure 3: Schematic view of plunger details

Table 4: Values of adhesion factor, α

Consistency	*α - value*
Soft to very soft	1.0
Medium	0.7
Stiff	0.4
Stiff to hard	0.3

The formulae and adhesion factors used for estimating the plunger friction is as follows (IS: 2911 (Part 1/ Sec 1) - 1979):

Plunger friction, $f_P = \alpha\, c$ (1)
Plunger friction load, $F_P = f_{P.}\, A_s$ (2)

Where,
c = cohesive strength (kN/m^2)
α = adhesion factor
A_s = plunger surface area in soil = $\pi\, D\, L_s$
L_s = length or penetration of plunger in soil
D = diameter of plunger.

3.0 Results and Discussion

Stabilization of rural roads on weak subgrade material is greatly assisted by the knowledge of their basic engineering properties. On similar lines, relating properties of existing subgrade to its extent of distress state or for understanding their probable remaining life span of service to decide upon necessary actions to infuse life span in them seems viable. From the tests conducted on soil samples, the discussion of CBR and UCS results are presented in below sections.

3.1 CBR With and Without Plunger Friction

The CBR tests are performed at different moisture contents such OMC, OMC+1% and OMC-1% to study the effect of CBR with water content for all samples collected. The respective load - penetration curves are plotted and presented in Figures 4 to 6. From these figures, it is noticed that for the plunger penetration of up to 2 mm, the resistance offered by the soil against the plunger area is minimal and thereafter for increased plunger penetration there is drastic resistance from the soil. Soil K is showing more resistance amongst all other soils against the plunger penetration for the samples prepared at OMC and OMC-1% and this can be attributed that presence of more gravel fraction as compared to other soils. The CBR of all soil samples tested are deduced from the curves after applying the appropriate correction to the initial concave prevailed in the curves. Similarly the load – penetration curves are plotted by considering the plunger friction for all soil samples tested at water contents OMC-1%, OMC and OMC+1% and are presented in Figures 7 to 9.

Figures 4 to 6 present the load – penetration curves of soil samples tested at OMC-1%, OMC and OMC+1% without considering plunger friction. Irrespective of moisture contents, the load – penetration curves are showing similar trend for samples tested at OMC-1% and OMC. The load penetration curve corresponding to Soil K is overriding as compared to other soil samples. Up to 1 mm penetration, the load – penetration curves are merging can be seen spreading in curves. Soil V, Soil M and Soil H the load penetration curves are comparatively riding close to the horizontal axis as compared to the curves of Soil B, Soil P and Soil K. From Figure 6, it can be seen that the load – penetration curves up to 5mm penetration are merged closely and there after further increase of penetration the load levels are increasing for all soil samples tested at OMC +1%. This can be attributed that, 1% increase in moisture content is causing further saturation in samples and accordingly though there is an increase in penetration up to 5 mm, the load levels are seen low. Further the load – penetration curves are plotted by considering the plunger friction and are presented figures 7 to 9. The load – penetration curves plotted by considering plunger friction are almost following the similar trend as observed in case of the curves drawn without considering plunger friction. The load levels of the curves drawn with plunger friction are little higher for the same penetration as compared to the penetration curves drawn without plunger friction.

From load-penetration curves presented in figures 4 to 9, the CBR values are estimated by applying suitable correction for the curves. The CBR values obtained from the curves after correction corresponding to moisture contents of OMC-1%, OMC and OMC +1% and with the consideration of plunger friction are presented in Table 5. From this table it can be clearly seen that as the moisture content increases from OMC-1% to OMC +1%, the CBR is decreasing for all soil samples tested in this study. Similar decrease in CBR is noticed with the increased moisture content, in case of CBR obtained by considering plunger friction for all the soil samples.

From table – 5, based on the comparisons of CBR with and without plunger friction consideration, the percentage increase in CBR is noticed as varying from 0.32% to 2.72%. Depending upon the soil constituents present in the samples tested in study, especially in soft clay soils, the adhesion factor can be taken equal to 0.8 to 1. In this study, consideration of plunger friction may contribute higher CBR. From table 5, it can be further seen that as the moisture content increases from OMC -1% to OMC, there is an increase in CBR and further increase of moisture content from OMC to OMC+1%, there is a decrease in CBR. This may be due to further saturation in samples. The CBR with and without plunger friction has been plotted in the form of histograms as shown in figures 10 to 12 for samples tested at OMC-1%, OMC and OMC+1% respectively.

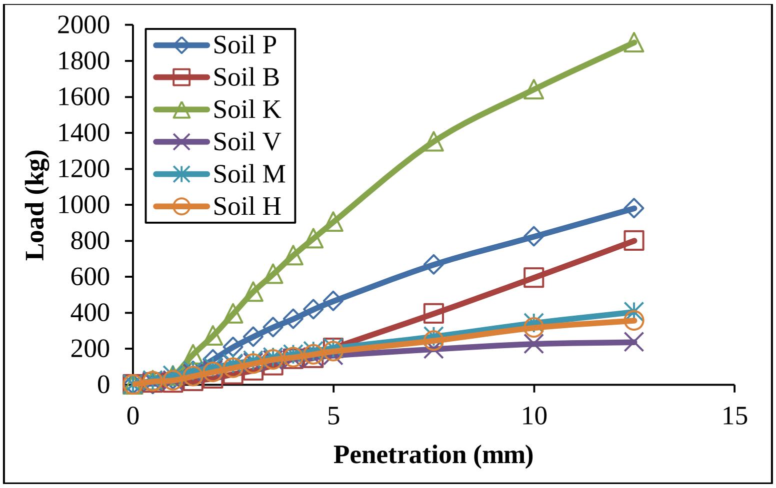

Figure 4: Load – penetration curves of soils without plunger friction at OMC-1%

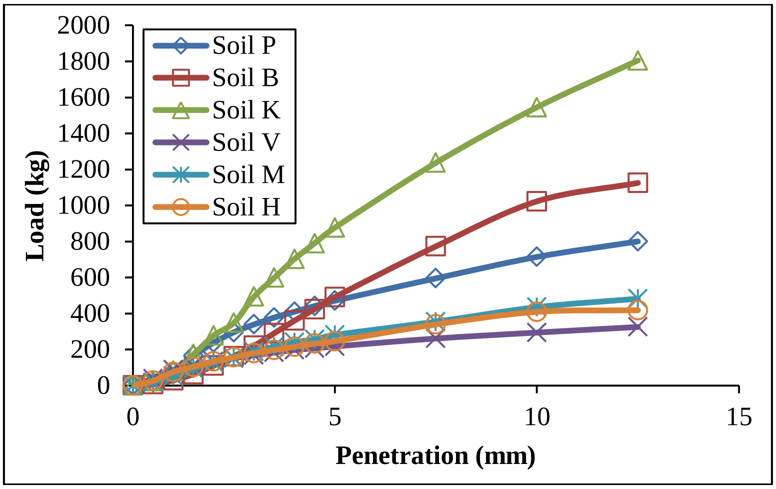

Figure 5: Load – penetration curves of soils without plunger friction at OMC

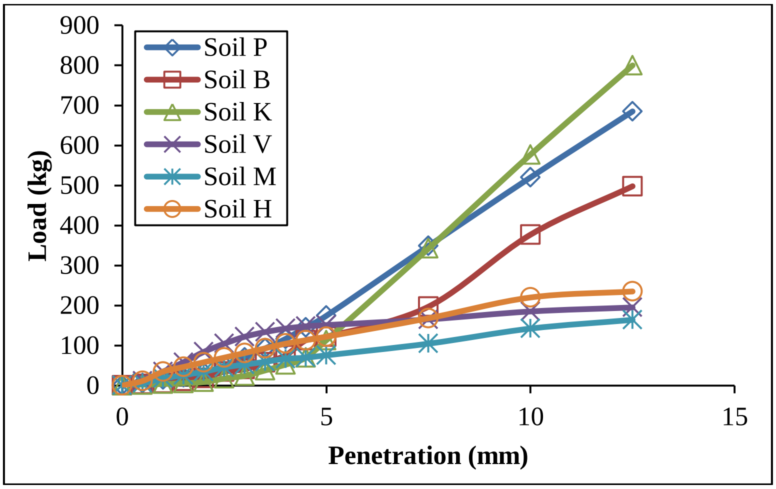

Figure 6: Load – penetration curves of soils without plunger friction at OMC +1%

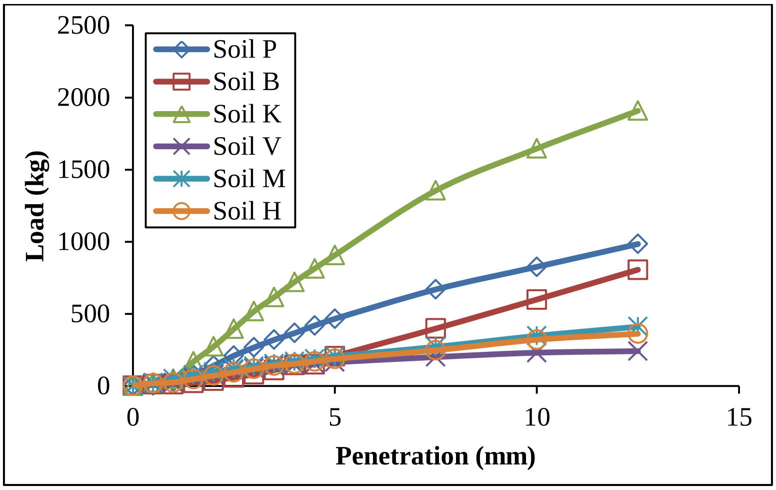

Figure 7: Load – penetration curves of soils with plunger friction at OMC-1%

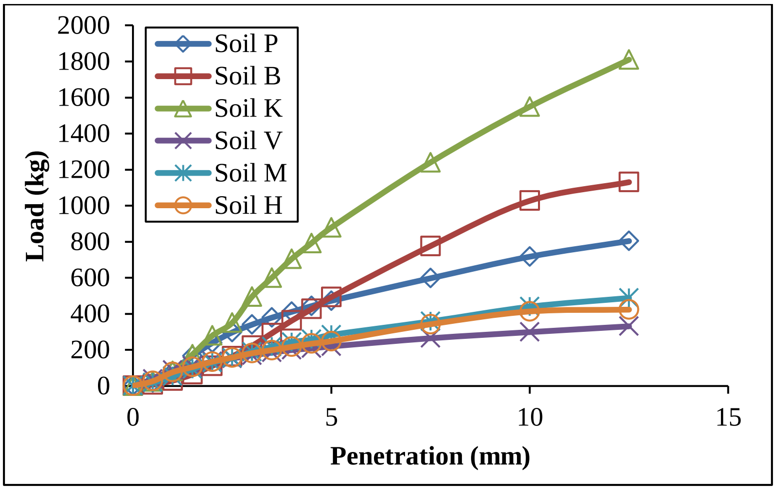

Figure 8: Load – penetration curves of soils with plunger friction at OMC

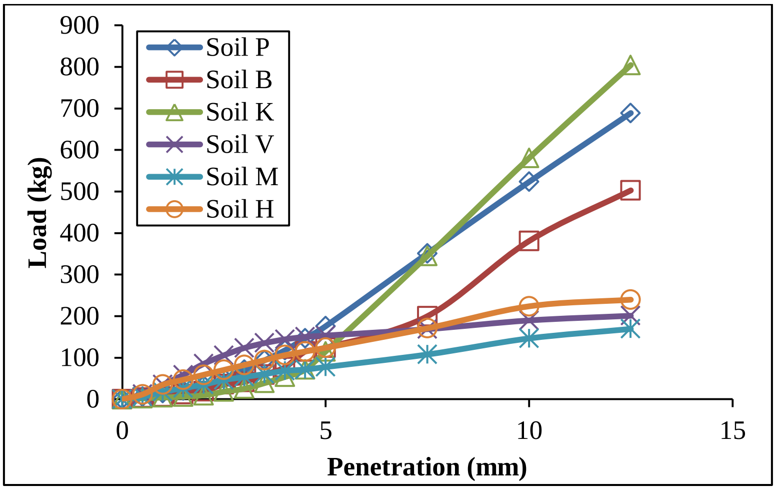

Figure 9: Load – penetration curves of soils with plunger friction at OMC+1%

The respective CBR of soil samples tested at OMC-1%, OMC and OMC+1% are further presented in Table 5.

Table 5: CBR without and with plunger friction of soils at OMC-1%, OMC and OMC+1%

	California Bearing Ratio (CBR), %					
	Without plunger friction	*With plunger friction*	*Without plunger friction*	*With plunger friction*	*Without plunger friction*	*With plunger friction*
	OMC -1%		OMC		OMC +1%	
Soil P	15.24	15.29	21.6	21.97	8.48	8.55
Soil B	9.87	9.98	23.84	23.99	5.93	6.03
Soil K	25.47	25.54	28.75	28.88	5.51	5.6
Soil V	7.88	7.98	11.02	11.11	7.63	7.72
Soil M	9.87	9.99	11.24	11.32	3.67	3.77
Soil H	9	9.11	11.38	11.46	5.93	6.02

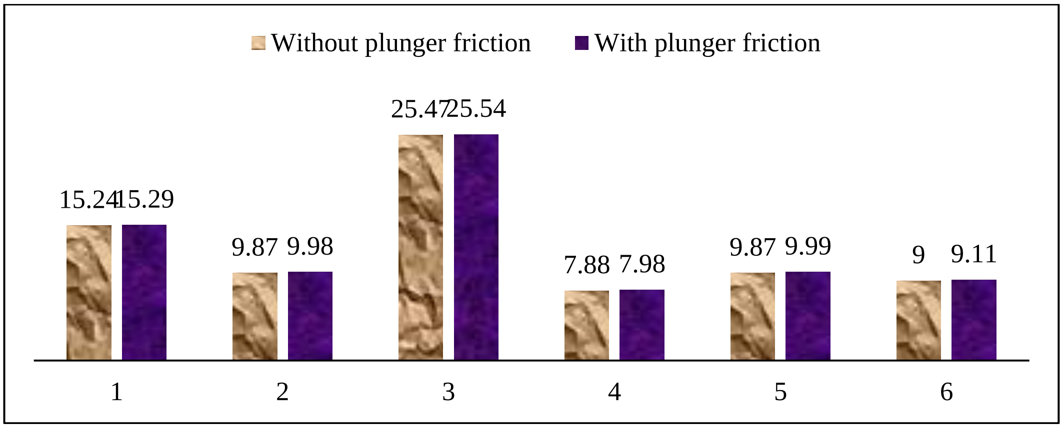

Figure 10: Variation of CBR with and without plunger friction at OMC-1%

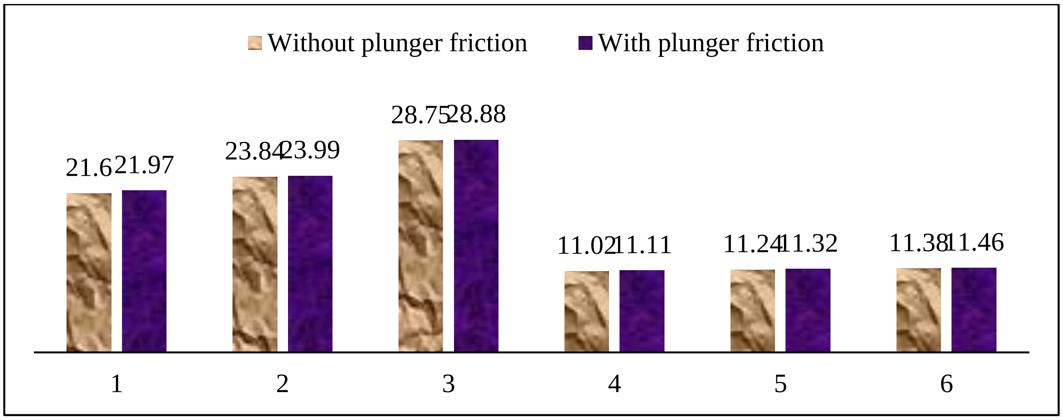

Figure 11: Variation of CBR with and without plunger friction at OMC

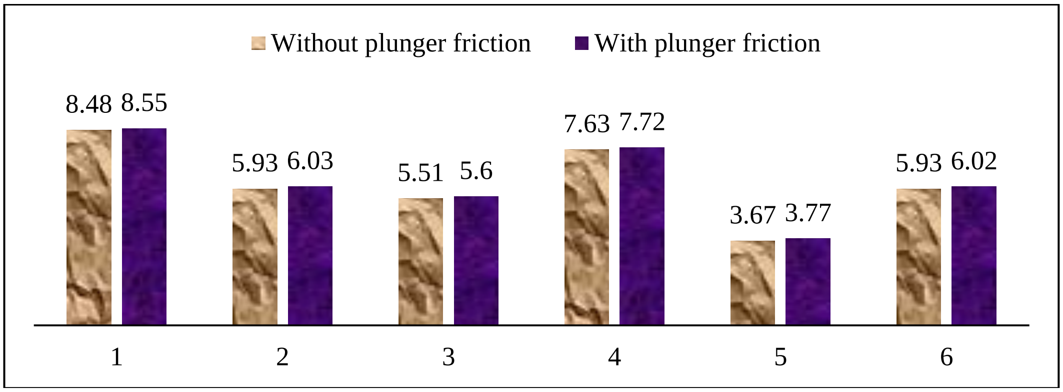

Figure 12: Variation of CBR with and without plunger friction at OMC+1%

3.2 Unconfined Compressive Strength (UCS)

Unconfined compressive strength test can be used to find out the shear strength and unconfined compressive strength of clayey samples as well sandy clayey soils, whose sample can stand without any confinement or support. In the present study, the tests were conducted on soil samples prepared at different moisture contents such as OMC-1%, OMC and OMC+1%. Figures 13 to 15 present the axial compressive stress - strain curves for all samples for moisture contents OMC-1%, OMC and OMC+1% respectively.

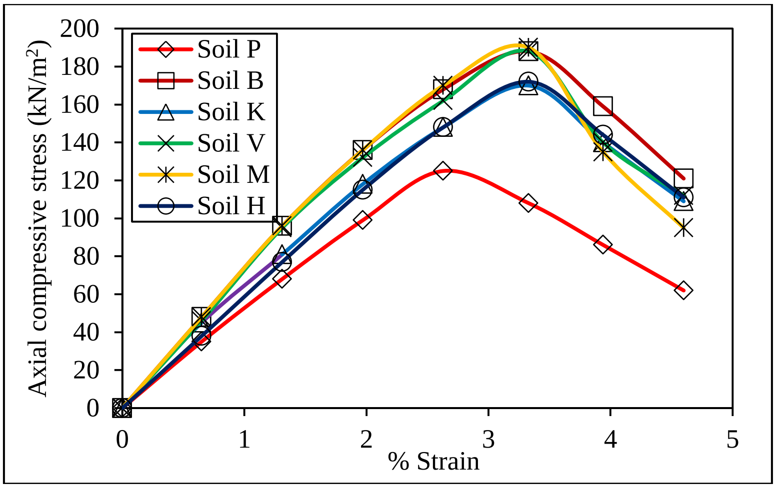

Figure 13: UCS curves for soils at OMC-1%

Figure 13 presents the unconfined compressive strength curves for soils tested at OMC-1%. From this figure, it can be seen that the peak value of compression strength for all samples is ranging from 125 to 190 kN/m^2. Up to about 2.5% strain the stress strain behaviour is linear and there after the behaviour is shifting towards curve linear. This behaviour can be in general seen in the stiff sandy clayey soil samples subjected to unconfined compression strength. Similarly the undrained cohesive strengths of soils from the above figure are ranging between 62.5 to 95 kN/m^2.

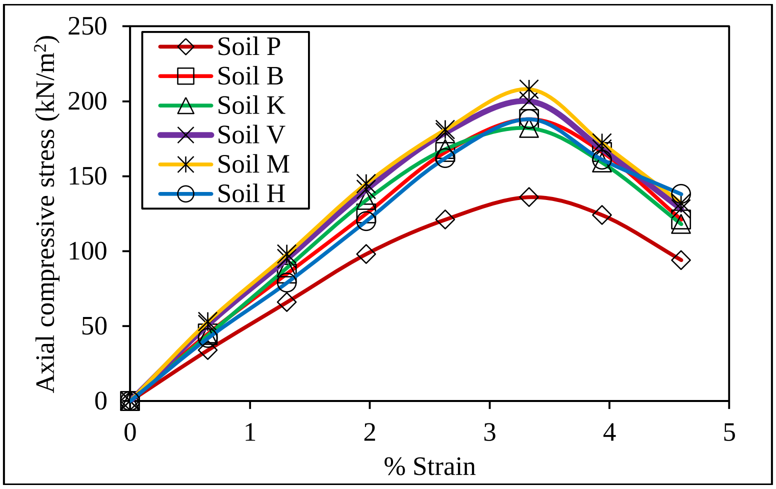

Figure 14: UCS curves for soils at OMC

Figure 14 presents the unconfined compressive strength curves for soils tested at OMC. From this figure, it can be observed that the unconfined compression strength for samples is ranging from 136 to 208 kN/m^2. Up to about 2 to 2.5% strain the stress strain behaviour is linear and there after the behaviour is shifting towards curve linear. Similarly the undrained cohesive strength of soils prepared and tested at OMC is ranging between 68 to 104 kN/m^2. For the samples prepared and tested at OMC+1%, the stress- strain curves obtained from UCS test are presented in Figure 15. From this figure, the UCS of all samples tested is varying from 116 to 176 kN/m^2. The undrained cohesive strengths are ranging between 58 to 88 kN/m^2 for different soil samples tested. Further, the UCS and undrained cohesive strength (c) of soil samples tested at moisture contents of OMC-1%, OMC and OMC+1% are presented in Table 6. From this table, it is clearly noticed that as the moisture content of samples varying from OMC-1% to OMC+1%, there is increase in UCS and cohesive strength up to OMC and from OMC onwards (i.e., at OMC+1%), there can be seen decrease in UCS and undrained cohesive strength of samples.

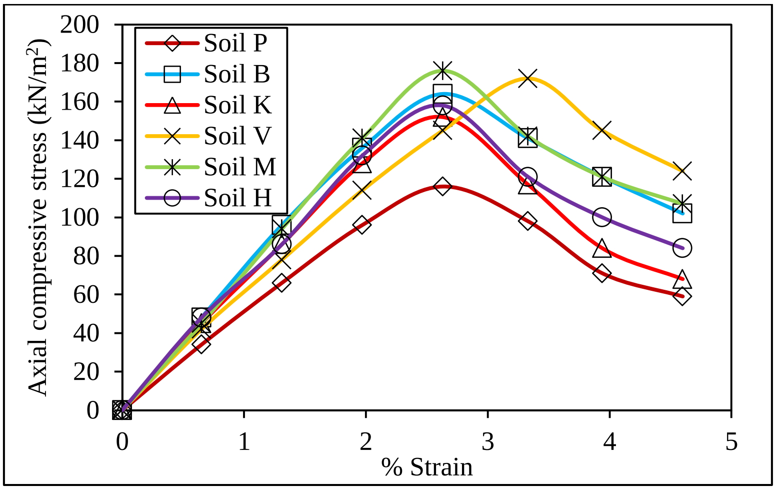

Figure 15: UCS curves for soils at OMC+1%

Table 6: Values of UCS and c used in the Study

	UCS (kN/m^2)			*c (kN/m^2)*		
Soil	*OMC-1%*	*OMC*	*OMC +1%*	*OMC-1%*	*OMC*	*OMC +1%*
Soil P	125	136	116	62.5	68	58
Soil B	180	188	164	90	94	82
Soil K	170	182	152	85	91	76
Soil V	188	200	172	94	100	86
Soil M	190	208	176	95	104	88
Soil H	172	188	158	86	94	79

3.3 Relationship between UCS and CBR

Further to understand the variation of CBR with the UCS or to establish a relation between CBR and UCS, the results are plotted and presented in Figures 16 and 17. Figures 16 & 17 present the relation between UCS and CBR with and without plunger friction and the UCS is varying linearly with CBR

$$UCS = 7.007\ CBR + 117.9 \quad (3)$$
$$UCS = 7.007\ CBR + 118.5 \quad (4)$$

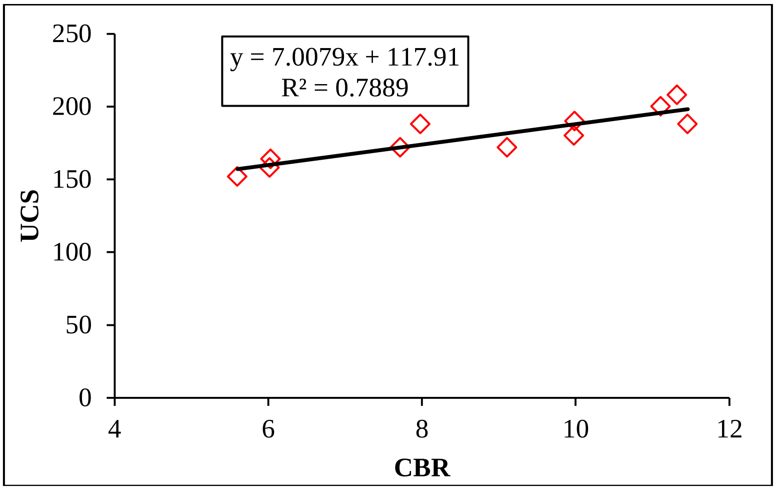

Figure 16: UCS values with CBR (with plunger friction)

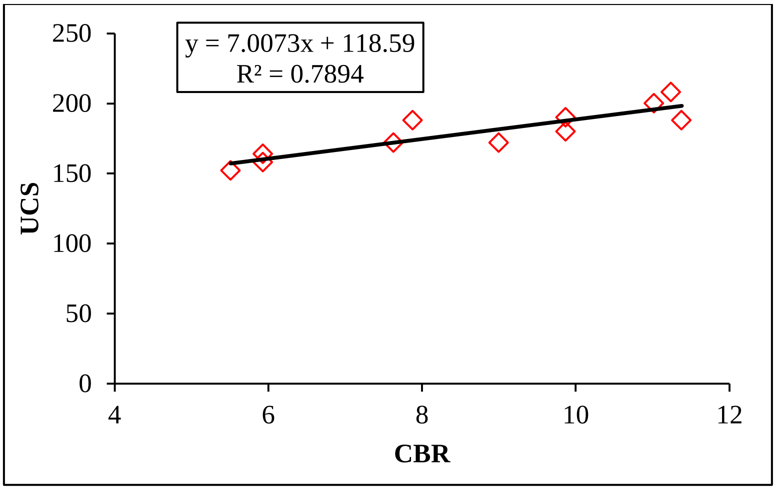

Figure 17: UCS values with CBR (without plunger friction)

The relation between UCS and CBR are without and with plunger friction are presented in the Equations 3 and 4 respectively and their respective regression coefficients are 0.788 and 0.789.

4.0 Conclusions

From the results and discussions the following conclusions are made.

1. The moisture content increases from OMC -1% to OMC, there is an increase in CBR and further increase of moisture content from OMC to OMC+1%, there is a decrease in CBR was observed. Further the moisture content increases from OMC-1% to OMC +1%, the CBR is decreasing for all soil samples tested in this study. Similar decrease in CBR also noticed with the increased moisture content, in the case of CBR obtained by considering plunger friction for all the soil samples.
2. The moisture content of samples varying from OMC-1% to OMC+1%, there is an increase in UCS and undrained cohesive strength (c) up to OMC and from OMC+1% onwards, there is decrement in UCS and undrained cohesive strength (c) of samples. Moreover the laboratory CBR values are affected by considering plunger friction by 0.32 to 2.72%.

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