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California Bearing Ratio for Cohesive Soils from Quasi Static Cone Tests

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Abstract

The California bearing ratio (CBR) test is the most widely spread method of determining the bearing strength of the pavement material and is fundamental to pavement design practice in most countries. This test is expensive, laborious and time consuming, and to overcome this, Quasi static cone penetrometer machine was fabricated and used to measure the consistency limits (liquid limit-LL, Plastic limit-PL and Plasticity index-PI), which were used to develop an empirical equation to determine CBR. Soil samples were collected and unsoaked CBR, PL, LL and PI were determined according to BS 1377 part 9 and BS 1377-2; 1990. Quasi static penetration forces at 20 mm depth of penetration were also determined at consistency limits. It was found that the force of 1020 gf and 60 gf was achieved at a depth of 20 mm at PI and LL respectively. The correlation and regression analysis between consistency limits, and the experimental CBR obtained showed coefficient of determination, R^2 = 0.907 between CBR and all the parameters using multiple linear regression analysis (MLRA). The regression equation developed was used together with the relationship developed between the Quasi static Penetration force at consistency limits and the tested consistency limits to come up with the General Empirical Equation. Verification of the formula showed that the correlation can be used accurately to determine the un soaked CBR.

Keywords

California Bearing Ratio, Quasi Static Consistency Limits, Plastic Limit, Liquid Limit, Plasticity Index

1. Introduction

Transportation Infrastructure is a key component for any long term development program of any nation. The development of road network is regarded as an index of economic, social and commercial progress of a particular country [1]. No region or country can develop without adequate transportation facilities especially the road network. Consequently 20.8% of the Uganda's National Budget are allocated to the transportation sector [2]. It is important that during early stages of the planning, design and construction of a road network, proper soil characterization is carried out.

Soil bearing capacity plays a very important role for the design of highway structure. It determines the design thickness of the pavement. The bearing capacity of the sub grade is mostly influenced by the type of soil, water content and its density [3].

In Uganda, it is a common practice to determine the subgrade soil bearing capacity for highway pavement design using CBR test measurement. To determine the CBR representative soil samples are compacted at predetermined optimum moisture content and maximum dry density for a given compaction energy of the soil material. Thereafter the CBR value is obtained only after immersion in water for 4 days and a plunger is used to penetrate the soil [4]. Carrying out this exercise on soil samples collected from a limited number of locations cannot be representative of the whole road length due to the variations of engineering properties along the road. Overcoming this entails the collection of a large number of specimens for testing which makes the procedure expensive, time consuming and laborious [1].

Research on correlation between DCP and CBR value has been performed on clay sand and sand soils. The study aimed at relating the result of DCP to CBR value, which takes into account the soil density [5].

However, [4] found out that DCP has limitations in that it needs to be held vertically, a person using it must lift the hummer carefully so as not to lift the whole instrument and releasing the hammer someone must be careful so that it is not out of plumb.

Other methods are available to determine sub grade bearing capacity such as Plate Bearing test, and Hand Cone Penetrometer (HCP) test, also known as Proving Ring Penetrometer [6]. However all these studies need to be carried out in other types of soils such as peat soils. Also the liquid limit (LL) and plastic limit (PL) tests are among the most commonly specified tests in the geotechnical engineering industry and originate from the original research of Atterberg [7], which was subsequently standardized for use in civil engineering applications [5], and adopted for the classification of fine-grained soils. These Atterberg limits have been used for numerous purposes, including the estimation of shear strength, deformation and critical-state soil mechanics parameter values. However, [8] noted that the error in using the Casagrande tool may arise due to the differences in behavior in response to shaking. For plastic limit, the test is also very sensitive to the operator technique. Also, difficulties were reported in using the casagrande apparatus method for soils of low plasticity; for which double edged grooving tools were developed [9]. There are also difficulties in controlling the rate of penetration during fall-cone tests. This complicates their use over the entire plastic range, particularly close to the plastic limit, where slight variations in moisture content may significantly affect the soil strength [10].

The quasi-static cone test procedures are essentially the same as for the fall-cone tests and are also described by [10] and [11]. This instrument was essentially used to determine plastic ranges of test soils as opposed to other approaches that concentrated on testing at moisture contents around the atterburg limits. Both these two tests, the California bearing Ratio and the Atterberg tests, are crucial in determining the soil properties which are helpful in design of civil engineering infrastructures. However, carrying out these tests needs a lot of time and money; this may be one of the reasons why most construction projects in Uganda are delayed to be completed [12].

Therefore to resolve this, it is necessary to determine CBR values using other parameters determined by Quasi static cone tests which can be easily determined by use of statistical analysis.

2. Description of the Study Area/Geographical Setting

The study concentrated on soils in Masaka, Kalungu, Rakai, and Lwengo District in Uganda as shown in **Figure 1**; where by 12 samples were collected in Masaka, 9 samples in Kalungu 10 samples in Rakai and 19 samples were collected in Lwengo district.

3. Previous Research Carried Out

[7] carried out a research on correlation between Dynamic Cone penetrometer (DCP) and CBR value has been performed by Indrawn on clay sand and sand soils and came up with the relationship below;.

$$\mathrm{CBR} = (\mathrm{DCP})\left[\mathrm{Log}(\mathrm{PI}\times\mathrm{C})^{26.51} + 8.89\right]\left[\mathrm{F_i} - \frac{0.0269}{(\mathrm{PI}\times\mathrm{C})^2} + \frac{0.541}{\mathrm{PI}\times\mathrm{C}} - 5.72\right].$$

Equation (1)

where: $\mathrm{F_i}$: is the initial state factor, DCP is the dynamic cone penetration (mm/blow), PI: is the plasticity index, C: is the clay content.

However, [13] found out that to use a DCP it needs to be held vertically, a person using it must lift the hummer carefully so as not to lift the whole instrument and releasing the hammer someone must be careful so that it is not out of plumb which is very difficult to achieve.

[6] Carry out a research to get a relationship between CBR and Hand cone penetrometer and came up with the following relation ship

$$\text{Field CBR}_{\text{prediction}} = C_0 + C_1\gamma + 0.025\text{HCP}.$$

Equation (2)

where C_0 and C_1 are coefficients depending on the type of soil. HCP is the value of Hand Cone Penetrometer test.

For peat soils, the value of C_0, C_1, and C_2 significantly influenced by fiber peat. The value of C_0, C_1, C_2 is −1.250, 0.085, and 0.005 respectively.

Figure 1. Map showing districts in Uganda (samples were collected in Kalugu, Masaka, Lwengo and Rakai).

But all this study needs to be carried out in other types of soils like peat soil.

[14] Carried out a research on Prediction of CBR using DCP for local subgrade materials and conclude the relationship as follows;

$$\log 10\text{SCBR} = 0.397 + 0.917 \log 10\text{UCBR} \; R = 0.847 \text{ Strong relationship} \quad \text{Equation (3)}$$

where;

SCBR = California bearing ratio for soaked soil samples;

UCBR = California bearing ratio for unsoaked soil samples.

The above relationship were carried out on fine grained soils and can used to determine the soaked CBR incase Unsoaked CBR or DCPI are tested. And [3], carried out a research on Co-relationship between California bearing ratio and index properties of Jamshoro soils and came up with the following results

$$CBR_s = 0.2807(CBR_u) + 5.0352;\ R = 0.718 \quad \text{Equation (4)}$$

$$CBR_u = 293.4964 + 25.4466(LL) - 59.5422(PI) \quad \text{Equation (5)}$$

where CBR_s = California bearing ratio for soaked soil samples;

CBR_u = California bearing ratio for unsoaked soil samples;

LL = Liquid limit;

PI = Plastic limit.

It was observed that CBR values decrease with increase in plasticity index and increase with increase in liquid limit. Also unsoaked CBR was largely dependent on Liquid limit and plastic limit. However, their research was carried out on soils with unsoaked CBR ranging 65 - 85. There is a need to consider also the soils with unsoaked CBR below 65. Although Equation (i) can be used to determine soaked CBR ranging from 65 - 85.

The liquid limit (LL) and plastic limit (PL) tests are among the most commonly specified tests in the geotechnical engineering industry and originate from the original research of Atterberg [7] which was subsequently standardized for use in civil engineering applications [5], and adopted for the classification of fine-grained soils. These Atterberg limits have been used for numerous purposes, including the estimation of shear strength, deformation and critical-state soil mechanics parameter values. However [8] noted that the error in using the Casagrande tool may arise due to the differences in behavior in response to shaking. For plastic limit the test is also very sensitive to the operator technique. Also, difficulties were reported in using the casagrande apparatus method for soils of low plasticity; for which double edged grooving tools were developed [9]. There are also difficulties in controlling the rate of penetration during fall-cone tests. This complicates their use over the entire plastic range, particularly close to the plastic limit, where slight variations in moisture content may significantly affect the soil strength [11].

4. Materials and Methods

The study concentrated on determining the atterberg limits, quasi static cone penetrations forces at atterberg limits and California bearing ratio of soil samples and finding their empirical relationship. 50 samples of soils were tests and compared to find the relationship between these tests.

Materials tested were soils of different plasticity where the particles in the soil can bond to one another. The samples collected were kept in the polythene bags to avoid loss of the moisture content in the soil. The samples were taken to laboratory to be tested. Particles larger than 0.425 mm diameter were sieved out then the soils were mixed with distilled water and molded using palette knives. Particular care was taken to break down aggregated particles by hand powdering.

4.1. Sample Collection

In order to have sufficient and reliable data, laboratory tests were conducted on soil samples obtained from Masaka, Kalungu, Lwengo ,and Rakai District in Uganda. A total of 50 distributed samples were collected. The representative samples selected on the basis of visual identification of the soils with different properties. Soil samples were 12 samples from Masaka, 9 samples from Kalungu 10 samples from Rakai and 19 samples from Lwengo district.

4.2. Geotechnical Tests

Tests carried out were thread rolling plastic limit as per clause 5 of BS 1377-2: 1990, liquid limit test using Fall-cone Tests as per section 2.3.2 to BS 1377-2: 1990, determination of unsoaked CBR at optimum moisture content and determination of quasi static force at 20 mm depth of penetration for soils at consistency limits. These tests were carried out to all 50 samples of soil collected as shown in Table A1 in **Appendix A**.

5. Results and Discussion

The results of the experimental testing program reported in from Table A1 in **Appendix A** were used.

For the development of alternative plasticity index parameters based on quasi-static cone penetration tests which provide upper and lower strength indices similar to the conventional BS 1377 liquid limit (*LL*) and plastic limit (*PL*), to develop the empirical relationship between conventionally derived consistency limits and CBR and to develop the empirical relationship between Quasi static cone Penetrations and California bearing ratio

The results in Table A1 shows that soil materials are all cohesive soils with Plasticity index ranges from 12.5 - 23.7 with CBR ranges from 9.3% - 83.4%. Therefore most of these soils are suitable to be used as subgrade in road construction.

5.1. California Bearing Ratio Test (CBR)

The laboratory CBR tests were conducted for 50 samples comprised of soil collected in different areas in Uganda. The tests were conducted for unsoaked CBR. at optimum moisture content.

5.2. Consistency Limit Tests: Fall-Cone and Thread Rolling Plastic Limit Tests

Fall-cone tests were conducted for 50 test soils of different plasticity obtained from different areas of Uganda. All fall-cone tests were conducted in accordance with the BS 1377-2: 1990 procedure. The derived LL values varied between 28 and 50.3 are presented in Table A1.

Also Thread rolling plastic limit tests were conducted for 50 test soils comprising of soils from different areas of Uganda following the procedure set out in part 2 of BS 1377. For all the tested soils the PL ranged between 11 and 36, and is presented in Table A1.

5.3. Quasi-Static Cone Tests

This section presents the results of quasi-static cone tests from which quasi-static Liquid and plastic limits were derived. Also presented are results of preliminary investigations on quasi-static cone penetration load versus depth relationships.

Quasi-Static Cone Tests for Fine Soils

This was done using the fabricated mechanically driven cone devices. The quasi static cone penetrometer was consist of the 20 mm diameter bar cone shaped at the penetration tip similar to that fall cone penetrometer attached to the frame similar to tri axial frame as shown in Figure 2. The frame was made in such a way that it moves the soil sample in cup to be penetrated by a cone at constant penetration of 1.33 mm/s. The force applied was determined using a load cell placed at the bottom of the cylindrical cup containing the specimen. The penetration was determined using Ultra sound sensor attached at the bottom of the frame. The load cell and ultra sound sensor were both connected to Urduino mother board which acts as data logger. The results were read directly from the computer using urduino mother board so as to improve in the accuracy of reading the penetration and force at the same time.

Quasi-static cone tests were conducted for various soils of different plastic ranges. The results are presented in Table A1.

5.4. Preliminary Tests

Before coming up with results in Table A1, Preliminary tests were carried out by recording the force at each depth of penetration and penetration force versus depth curves were plotted for all the soil samples. A typical plot is shown in Figure 3 and Figure 4.

From Figure 3 and Figure 4 the points seem to trace a parabola, and the best fit line leaves a number of them.

Therefore a new plot of penetration force versus depth square was made. This is shown in Figure 5 and Figure 6.

From Figure 5 and Figure 6 it is seen that penetration Force versus depth squared gives the line of best fit. This was also supported [15] who stated that the shear strength of the soil is directly proportion to weight of the cone divide by the depth of penetration squared.

$$\tau_f = KQ/h^2 .$$

where τ_f is the undrained shear strength is the cone factor, Q is the weight of the cone and h is the depth of penetration.

Therefore the 50 tests for penetration were all made plotted by Penetration Force versus depth squared.

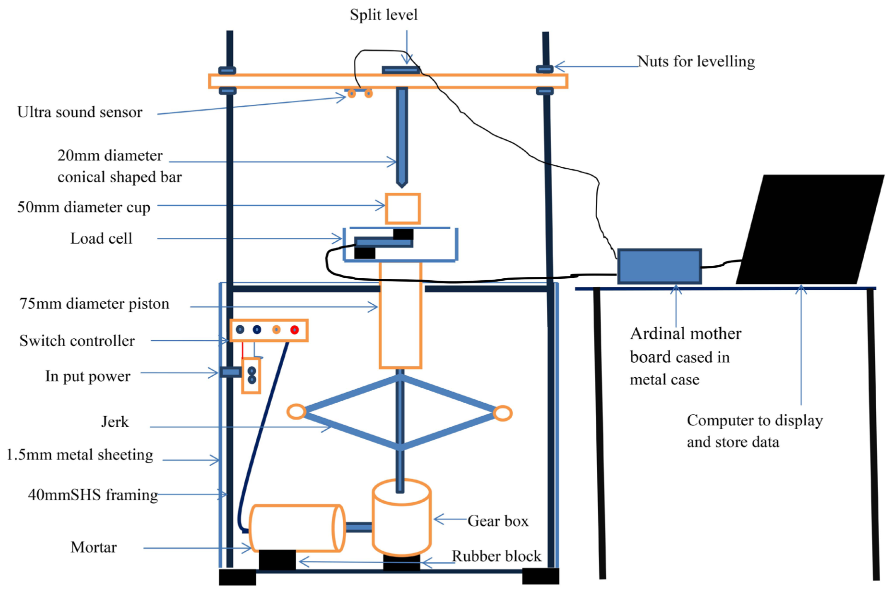

Figure 2. Shows systematic drawing for quasi static machine.

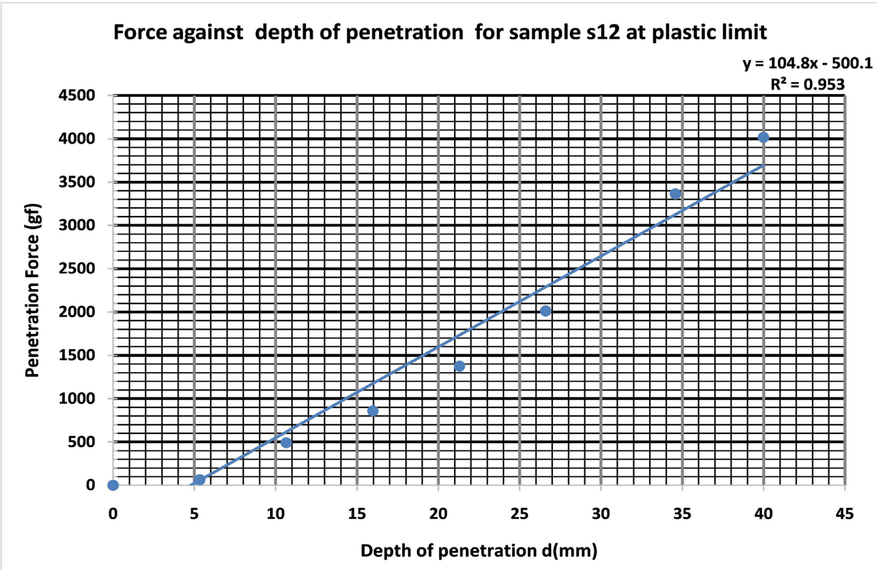

Figure 3. Shows a graph of Force against depth of penetration at plastic limit.

However, for some of the soils it is observed that some of the loads versus depth squared relationships are not entirely linear. This may be, among others factors, attributed to the load recording mechanism (load-cell) of the quasi-static cone devices. However, even with the curvature in the load versus depth squared relationships, linear regression curves indicated correlation factors (R) generally above 0.92.

From the graph of Force Versus penetration squared, the Force corresponding to penetration of 10 mm, 20 mm and 30 mm were determined for all the 50 samples. The aim was to determine which penetration depth matches the drop cone at plastic Limit and Liquid Limit.

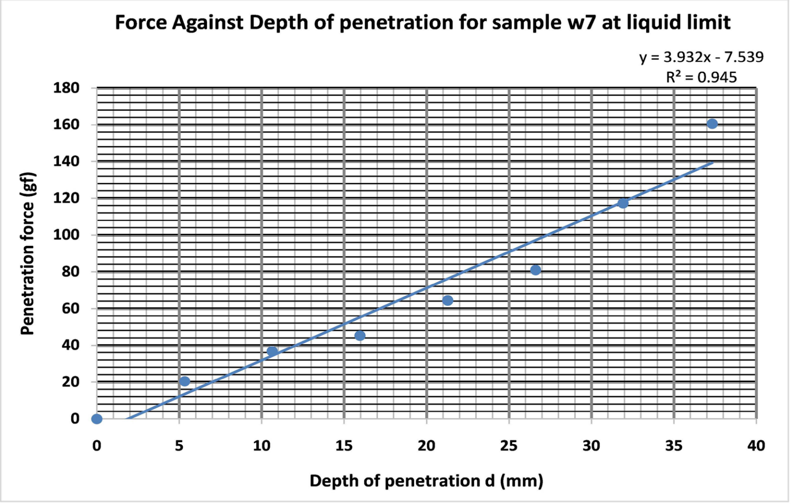

Figure 4. Shows a graph of force against depth of penetration at Liquid limit.

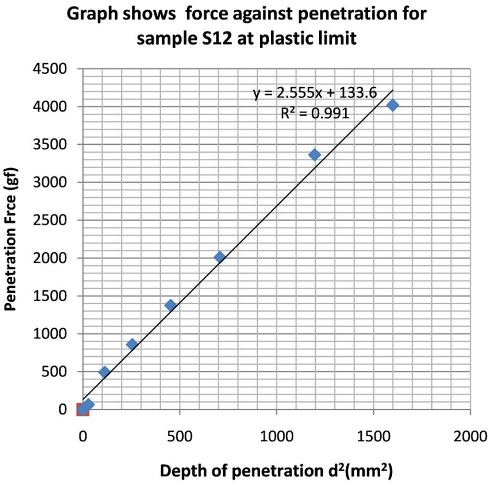

Figure 5. Force against depth of penetration square at plastic limit.

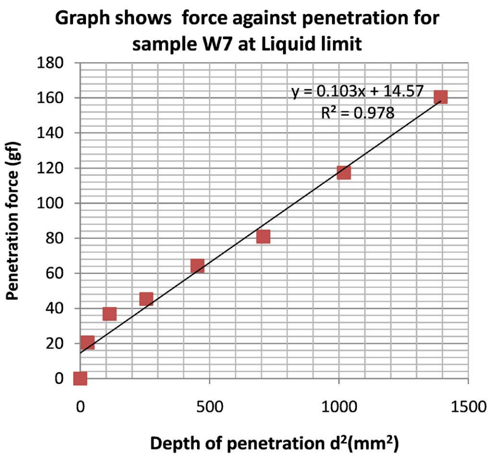

Figure 6. Force against depth of penetration squared at Liquid limit.

Figure 7 and Figure 8 show a plot of force at penetration depth Verus corresponding plastic limit and liquid limit.

From Figure 7 it is seen that the accuracy is most at penetration depth of 20 mm. This is because the penetration force over a wide range of plastic limit values are too close with a narrow range (902.5 - 1176.2 gf) whereas for 10 mm penetration it varies from (41.1 - 451.5 gf) and for 30 mm, it varies widely from (1912.9 - 2790.1 gf).

Similarly Figure 8 it is also seen that the accuracy is most at penetration of 20 mm. This is because the penetration force over a wide range of liquid limit values are too close with a narrow range (40.5 - 64.5 gf), f or 10 mm penetration it varies from (−8.8 - 36 gf) and for 30 mm, it varies widely from (81.6 - 176.3 gf).

Therefore the 20 mm values are to be used for determination of quasi static force for both plastic and liquid limit.

5.5. 20 mm Depth Quasi-Static Cone Penetration Load at LL (LL_{qc})

Denoted LL_{qc}, the quasi-static cone load associated with LL is obtained from the load versus penetration depth squared at liquid limit as shown in Figure 6 which

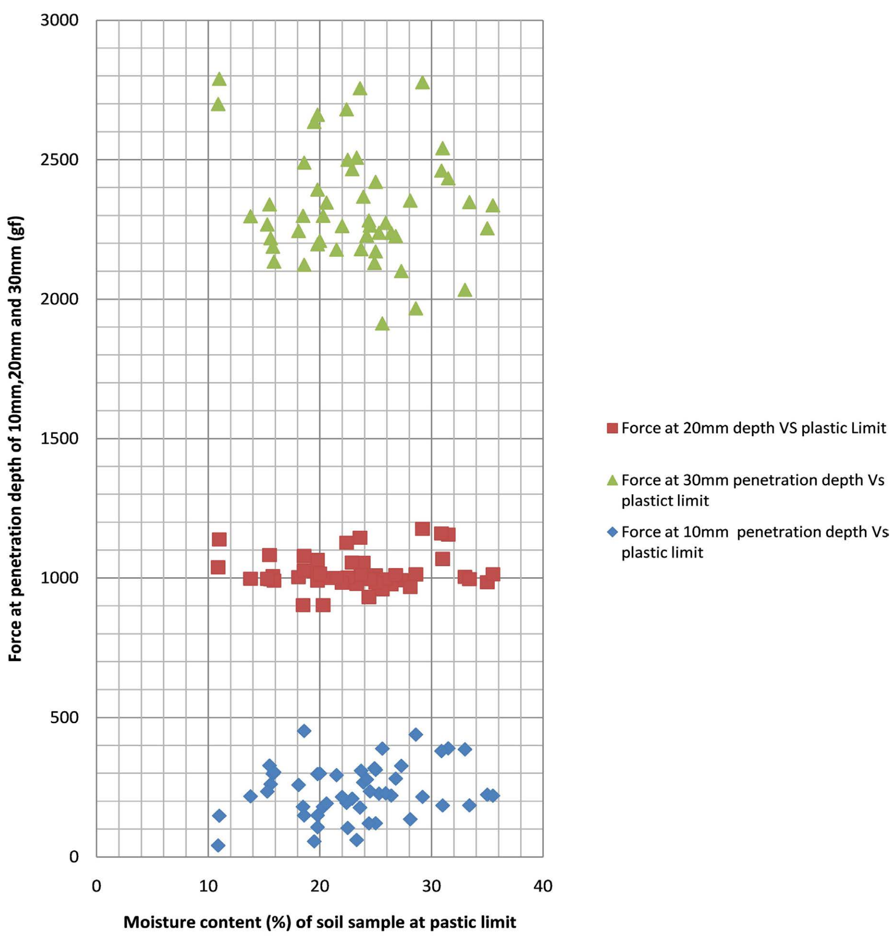

Figure 7. Showing a graph of force at penetration depth of 30 mm, 20 mm and 10 mm against moisture content of soil samples at plastic limit.

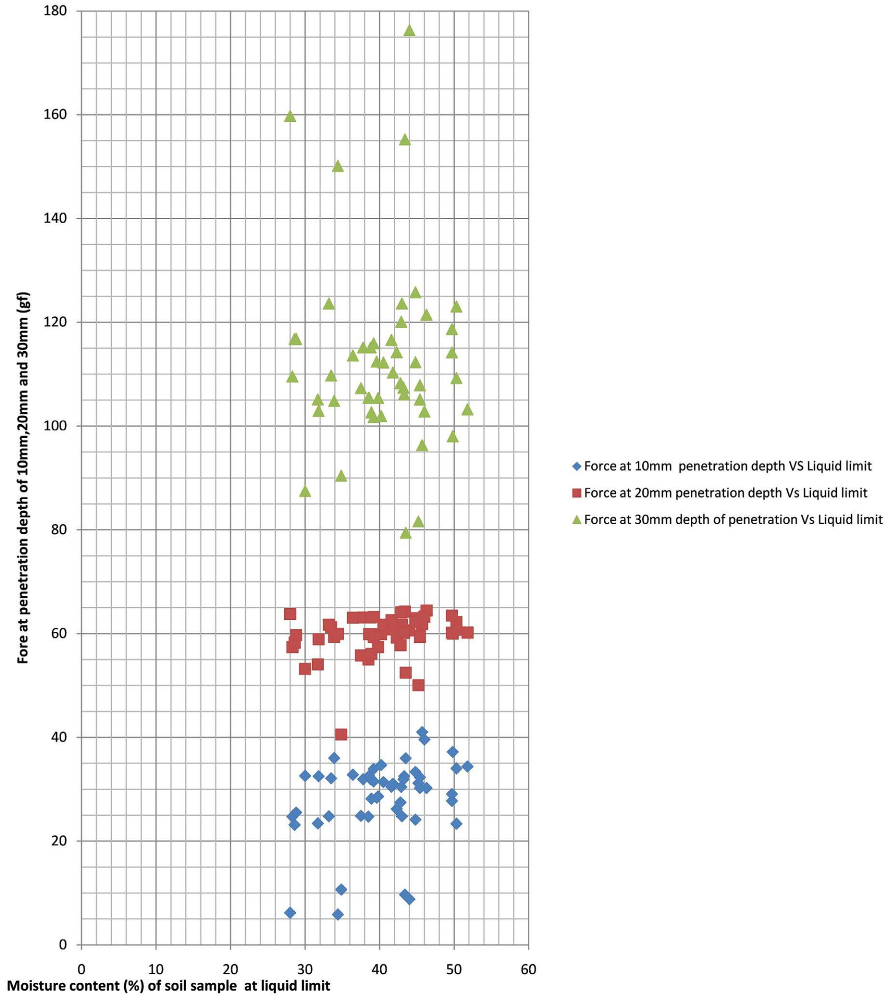

Figure 8. Showing a graph of force at penetration depth of 30 mm, 20 mm and 10 mm against moisture content of soil samples at liquid limit.

shows the sample of a graph drawn for each soil sample tested at Liquid limit. For 50 test soils, $\mathbf{LL_{qc}}$ ranged from 40.5 gf to 64.21 gf: overall averaging about 59.58 gf. Values of $\mathbf{LL_{qc}}$ are summarized in Table A1.

The above quasi static force of 59.58 gf is approximately 60 gf similar to Swedish fall cone of 60 gf. Therefore the $\mathbf{LL_{qc}}$ at 60 gf and can be denoted by QL_{60}.

Therefore, QL_{60} (Quasi static liquid limit) can be defined as the moisture content in the soil sample at which the quasi static force of 60gf can penetrate a soil sample up to a depth of 20 mm.

5.6. 20 mm depth Quasi-Static Cone Penetration Load at PL (PL_{qc})

Denoted PL_{qc}, the quasi-static cone load associated with PL is obtained from the load versus penetration depth squared at Plastic limit. Figure 5 shows the sample of a graph drawn for each soil sample tested at plastic limit. For fifty (50) test soils, PL_{qc} ranged from 902.51 gf to 1176.2 gf: overall averaging about 1019.85 gf. Values of PL_{qc} are summarized in Table A1.

Therefore the PL_{qc} is 1020 gf and can be denoted by QP_{1020}.

Therefore, QP_{1020} (Quasi static Plastic limit) can be defined as the moisture content in the soil sample at which the quasi static force of 1020 gf can penetrate a soil sample up to a depth of 20 mm.

Thereafter the results were analyzed using single and multiple regression analysis. Accordingly, the 50 laboratory test of the independent and dependent variables were used in regression analysis and the Pearson correlation coefficient between CBR and soil index properties *i.e.* PL, LL, and PI were as follows.

5.7. Single Regression Analysis

5.7.1. Correlation between CBR and Liquid Limit (LL)

The regression analysis after correlating CBR with LL is expressed by Equation (6):

$$CBR = 1.244LL - 4.003 \quad \text{with, } R^2 = 0.187. \qquad \text{Equation (6)}$$

Therefore, LL can be used to explain 18.7% of the variation in CBR. The statistical output's specifics show that the association between liquid limit and CBR that has been developed is not statistically significant ($\alpha > 0.05$). This suggests that for all soil samples, there is a modest correlation between LL and CBR. According to the calculated Pearson's correlation coefficient (R), liquid limit is a very poor predictor of unsoaked CBR. The above relationship indicates that as the liquid limit increases there is a slight increase in CBR.

Figure 9 below shows the above relationship.

5.7.2. Model 2: Correlation between CBR and Plastic Limit (PL)

The regression analysis after correlating CBR with PL is expressed by Equation 7

$$CBR = 2.285PL - 6.797 \quad \text{with } R^2 = 0.669. \qquad \text{Equation (7)}$$

Therefore, PL can explain 66.9% of the variation in CBR. The statistical output data show that there is a statistically significant association between the plastic limit and CBR ($\alpha > 0.05$). This implies there is a relationship between plastic limit and CBR. According to the calculated Pearson's correlation coefficient (R), the plastic limit is a predictor for unsoaked CBR. The above relationship shows that as soils with high plastic limit had high unsoaked CBR. Figure 10 below shows the above relationship

5.7.3. Model 3: Correlation between CBR and Plasticity Index (PI)

The regression analysis after correlating CBR with plasticity index is expressed by Equation (8);

$$CBR = -3.636PL - 108.30 \quad \text{with } R^2 = 0.491 \qquad \text{Equation (8)}$$

Consequently, the dependent variable is predicted by the PI. The statistical output details show that there is a considerable correlation between the plasticity index and CBR ($\alpha < 0.05$). This suggests a connection between PI and CBR. The plasticity index is a predictor for unsoaked CBR, according to the Pearson's correlation coefficient (R) that was obtained.

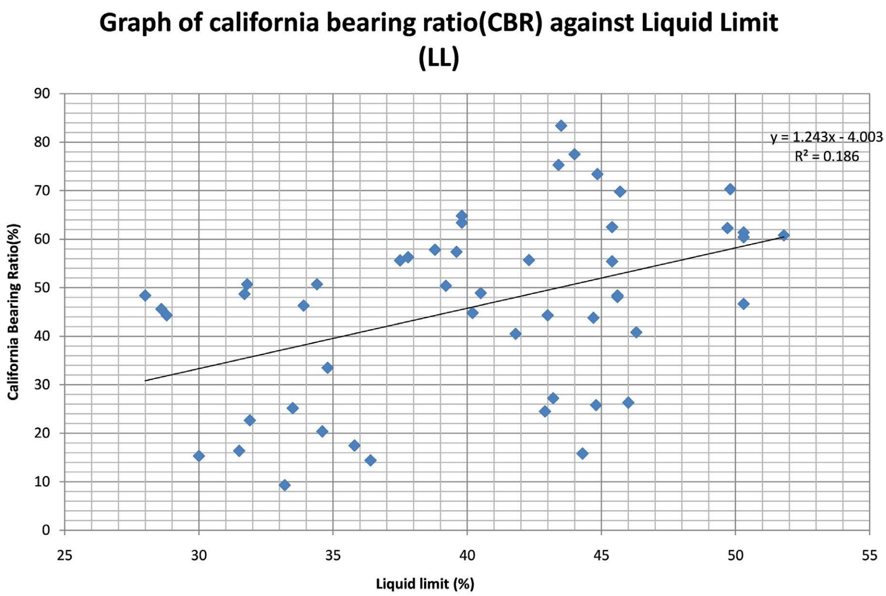

Figure 9. Shows relationship between CBR and LL.

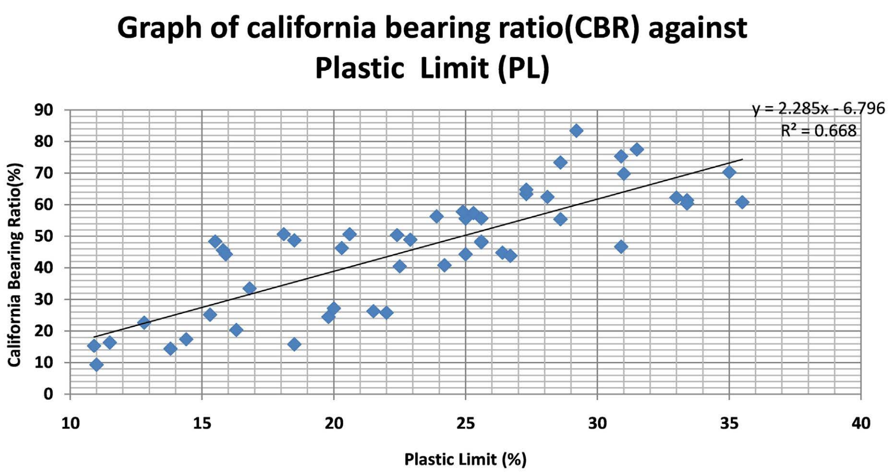

Figure 10. Shows relationship between CBR and PL.

The above relationship indicates that as the PI increase the CBR reduces which is in agreement [7] and [16]. Figure 11 below shows the above relationship

5.8. Multiple Linear Regression Analysis

A multiple linear regression analysis was carried out on fifty samples (n = 50) and after trying a set of alternative combination of predictors the following result were obtained.

1) Model 4: Correlation between CBR and Plastic index and Plastic Limit was carried out. The regression model obtained is a single linear expression with its corresponding coefficients as given by Equation (9) below;

$$\text{CBR} = -2.601\text{PI} + 1.868\text{PL} + 47.340 \quad R^2 = 0.897. \qquad \text{Equation (9)}$$

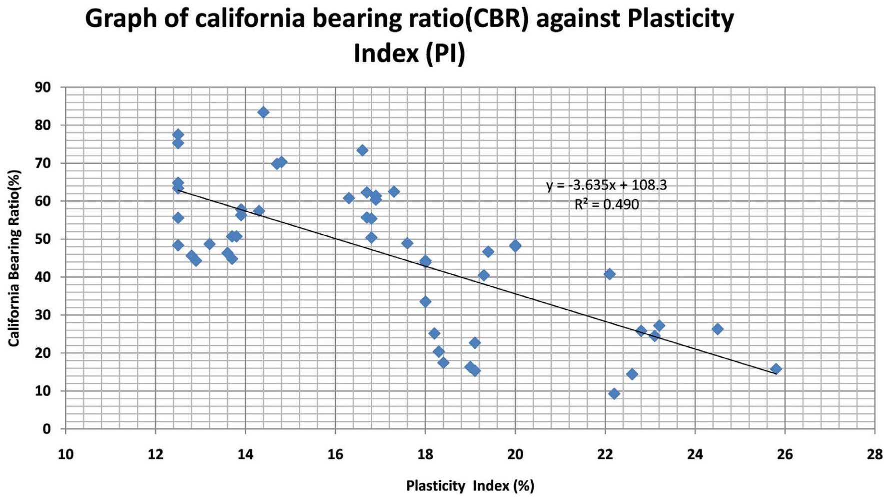

Figure 11. Shows relationship between CBR and PI.

As a result, the independent variables can explain 89.7% of the variance in CBR. According to the specifics of the statistical output, there is a statistically significant association between the plasticity index, plastic limit, and CBR ($\alpha < 0.05$). The above relationship indicates that as the plastic limit increases the CBR increase this is because as the plastic limit increases the plasticity index reduces hence the CBR increase.

2) Model 5: Correlation between CBR and Plastic index and Liquid limit was carried out. The regression model obtained is a single linear expression with its corresponding coefficients as given by Equation (10) below;

$$\mathrm{CBR} = 1.960\mathrm{PI} - 4.55\mathrm{PL} + 44.216\,,\ \mathrm{R}^2 = 0.812. \qquad \text{Equation (10)}$$

As a result, the independent variable can explain 81.2% of the variance in CBR. The statistical results show that the relationship between the plasticity index, the liquid limit, and the CBR is statistically significant ($\alpha < 0.05$). The above relationship indicate that as the Liquid limit increases the CBR reduces this is because as the liquid limit increases the plasticity index increase hence the CBR reduces as seen in 6.3.3. The relationship is also similar to that of [16].

3) Model 6: Correlation between CBR and Plastic Limit and Liquid limit was carried out. The regression model obtained is a multiple linear expression with its corresponding coefficients as given by Equation (11) below;

$$\mathrm{CBR} = 4.566\mathrm{PI} - 2.650\mathrm{PL} + 46.705\,,\ \mathrm{R}^2 = 0.841 \qquad \text{Equation (11)}$$

As a result, the independent variables can explain 84.1% of the variance in CBR. The statistical results show that there is a statistically significant association between the plastic limit, liquid limit, and CBR ($\alpha < 0.05$). The above relationship indicate that as the Liquid limit decreases the CBR increase this is because as the liquid limit decreases the plasticity index decrease hence the CBR increases as seen in 4.6.3. **Table A1** explains more of the above relationship.

4) Model 8: Correlation between CBR and Plasticity Index, Plastic Limit and Liquid limit was carried out. The regression model obtained is a single linear

expression with its corresponding coefficients as given by Equation below.

$$\text{CBR} = 1.645\text{PI} + 6.040\text{PL} - 4.250\text{LL} + 49.534,\ R^2 = 0.907. \qquad \text{Equation (12)}$$

As a result, the independent variables can explain 90.7% of the variance in CBR. According to the specifics of the statistical output, there is a significant correlation between the plasticity index, plastic limit, liquid limit, and CBR ($\alpha >$ 0.05).

The above relationship indicate that the plastic limit and liquid limit are the major determinant of CBR which actually true because plasticity index which is another determinant depends on liquid and plastic limits**.**

5.9. Empirical Relationship between CBR and Quasi Static Consistency Limits

Basing on the correlation between CBR and consistency limits, an empirical relation relationships was developed basing on replacing quasi static consistency limits with conventionally derived consistency limits. Since Equations (6)-(8) indicates a weak relationship, only Equations (10), (11) and (12) were considered in developing empirical relationships as fallows;

5.9.1. Empirical Relationship between CBR, Quasi Static Plastic Limit and Quasi Static Plastic Index

From Equation (9); $\text{CBR} = -2.601\text{PI} + 1.868\text{PL} + 47.340$ the empirical relationship between the CBR, QP_{1020} and QPI was developed as shown in Equation (13) below

$$\text{CBR} = -2.601\text{QPI} + 1.868\text{QP}_{1020} + 47.340 \qquad \text{Equation (13)}$$

5.9.2. Empirical Relationship between CBR, Quasi Static Liquid Limit and Quasi Static Plastic Index

From Equation (10); $\text{CBR} = 1.960\text{LL} - 4.55\text{PI} + 44.216$, the empirical relationship between the CBR, QP_{1020} and QPI was developed as shown in Equation (15) below

$$\text{CBR} = 1.960\text{QL}_{60} - 4.557\text{QPI} + 44.216 \qquad \text{Equation (14)}$$

where: CBR = California Bearing ration;

QL_{60} = Quasi static Liquid limit (Moisture content in the soil sample a which the quasi static force of 60 gf penetrate the soil sample up to 20 mm);

QPI = Quasi static plasticity index (QPI = $QL_{60} - QP_{1020}$).

5.9.3. Empirical Relationship between CBR, Quasi Static Liquid Limit and Quasi static Plastic Limit

From Equation (11): $\text{CBR} = 4.566\text{PL} - 2.650\text{QPI} + 46.705$, the regression model obtained is a single linear expression with its corresponding coefficients as given by Equation (16) below.

$$\text{CBR} = 4.566\text{QP}_{1020} - 2.650\text{QL}_{60} + 46.705. \qquad \text{Equation (15).}$$

5.9.4. Empirical Relationship between CBR and Quasi Static Consistency Limits

From the Equation (12); $\text{CBR} = 1.645\text{PI} + 6.040\text{PL} - 4.250\text{LL} + 49.534$ the empirical relationship between the CBR, QP_{1020} and QL_{60} was developed as shown in Equation (17) below

$$\text{CBR} = 1.645\text{QPI} + 6.040\text{QP}_{1020} - 4.250\text{QL}_{60} + 49.534 \qquad \text{Equation (16)}$$

Therefore there is strong relationship between CBR and the three consistencies limits *i.e.* Plastic limit, Liquid limit and plasticity index.

where: CBR = California Bearing ration

QP_{1020} = Quasi static plastic limit (Moisture content in soil sample at which the quasi static force of 1020 gf penetrate the soil sample up to 20 mm)

QL_{60} = Quasi static Liquid limit (Moisture content in the soil sample a which the quasi static force of 60 gf penetrate the soil sample up to 20 mm)

QPI = Quasi static plasticity index (QPI = $\text{QL}_{60} - \text{QP}_{1020}$)

Empirical Equation (16) happens to be the most accurate being that it was developed from regression Equation (12) and this equation makes use of the consistency limits with the coefficient of determination R^2 = 0.907. Therefore, 90.7% of the variance in CBR can be predicted using the independent variables and finally due to the ease with which these tests are carried out. Empirical Equations (13)-(15) can be possible alternatives when it comes to cost effectiveness but they show fair coefficient of determination hence less predictability.

5.10. Validation of the Empirical Equation 16

Using the actual CBR from tests and the predicted CBR from Equation (16), a control graph was plotted between actual experimental CBR and predicted CBR and is shown in **Figure 12**. The straight line represents the point at which experimental CBR equals predicted CBR. Nearly all points are found closer to the straight line. Only about Seven points tend to deviate away from the line. This indicates that the predicted CBR values may be applied for preliminary characterization of the strength of the soil. Furthermore, a comparison graph is plotted to verify the suitability of the developed correlation as shown in **Figure 13**. There is a mismatch between the two curves observed at soil sample number 17, 22, 40, 41, 48 and 49. This is may be attributed to errors when carrying out the laboratory tests. The graph shows a variation between the two CBR values. Generally, both graphs follow the same pattern. The percentage variation for each of the sample's CBR value is obtained from Equation (18).

$$\text{Percentage Variation} = \left(\frac{\text{Predicted CBR} - \text{Actual CBR}}{\text{Actual}}\right) \times 100 \quad \text{Equation (17)}$$

The average percentage variation obtained from the model is 10.9%, which is a good value proving that the predicted values of CBR are not far from the experimental values.

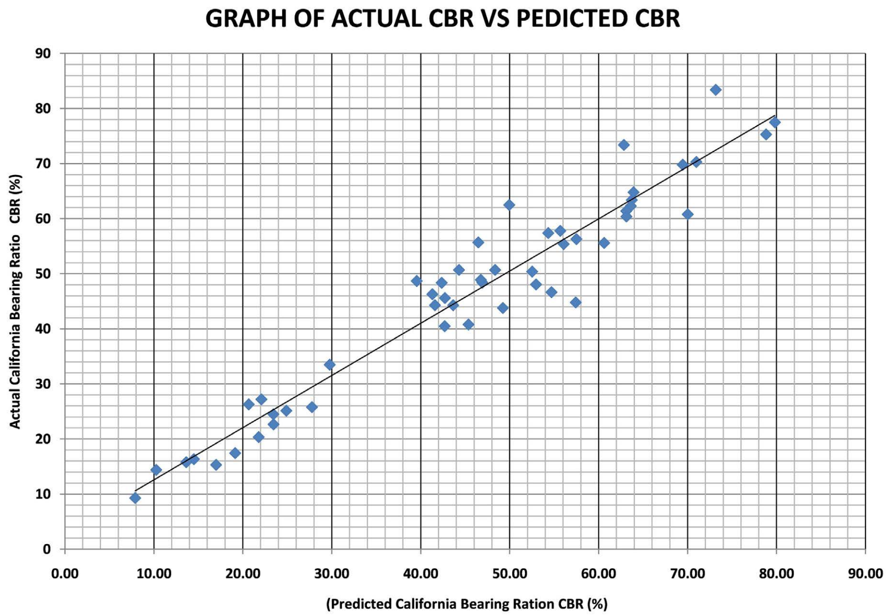

Figure 12. Shows relationship between Actual CBR and Predicted CBR.

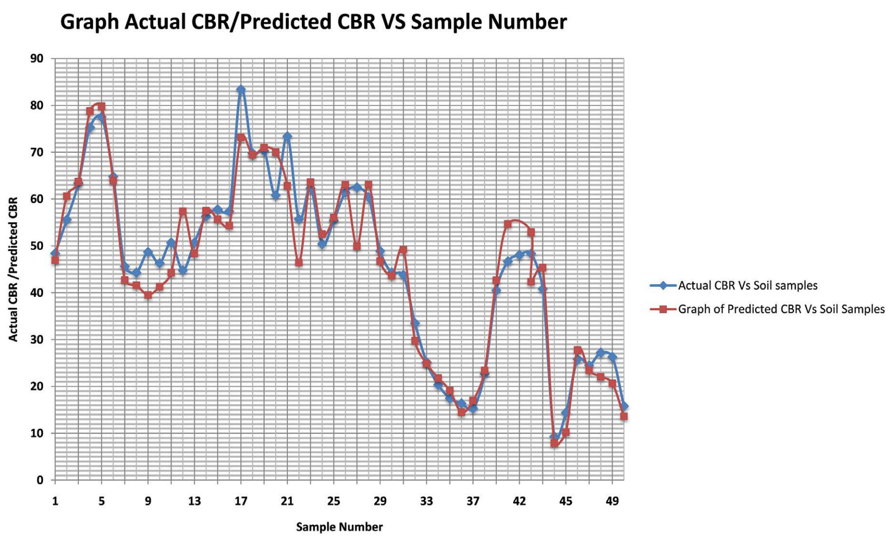

Figure 13. Showing comparison in variation of Actual CBR and Predicted CBR.

6. Conclusion

This research set out to address limitations of the current approaches used to evaluate the California bearing ratio and consistency limits through testing. These limitations are summarized below:

a) penetration rate effects in fall-cone tests,

b) Reliability associated with the thread rolling plastic limit test,

c) Health and safety issues in handling contaminated material,

d) Limitation in interpreting index tests for California bearing ratio, particularly at the conventional thread rolling, plastic limit and liquid limit.

In order to address the above issues, this research focused on developing alternative methods of determining California Bearing ratio and consistency limit testing using quasi-static cone penetration tests.

Therefore, the following conclusions were based on the findings of the research and limited to the sample sizes and soils tested within this research as follows:

1) An alternative quasi-static cone plastic limit (PL_{qc}) and quasi-static cone liquid limit (LL_{qc}) were proposed and were defined in Section 5.5 and 5.6 respectively.

2) Semi-empirical expressions were proposed for derivation of California bearing ratio and consistency limits in Equations (8)-(12) were based on correlations of California bearing ratio with liquid limits, Plastic limit and Plasticity index respectively.

3) Semi-empirical expressions were proposed for derivation of California bearing ratio and Quasi static consistency limits in Equations (13)-(17) were based on relationship between quasi static: liquid limits, Plastic limit, Plasticity index and California bearing ratio.

4) Using the derived relationship between Unsoaked CBR and Soaked CBR for other researchers for example [14] and [16] one can determine soaked CBR ounce the unsoaked CBR is known.

7. Recommendations

Based on findings of this research, the following proposals are made for further development of the quasi-static cone approaches and consistency limits testing, for cohesive fine and mixed soils.

1) Undertaking extensive experimental programmes for soils of varied geology and plasticity, which may provide improvements of the alternative quasi-static cone approaches developed in this research.

2) Laboratory Unsoaked CBR were employed, and although the effect of soaked CBR testing as a reference may be otherwise evaluated, it is essential to evaluate consistency limits based on soaked CBR testing for consistency evaluations.

3) Further development should be undertaken on this fabricated Quasi static cone penetrometer to find out whether it can test the CBR directly without using consistency limit approach.

Conflicts of Interest

The authors declare no conflicts of interest regarding the publication of this paper.

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Appendix A

Table A1. Laboratory Test results for collected soil samples.

sample Number	SAMPLE	force 20 mm (FPL) gf	force 20 mm (FLL) gf	Plasticity Index (PI) %	Plastic limit (PL) %	Liquid limit (LL) %	California Bearing Ration (CBR) %
1	s2	1081.89	63.75	12.5	15.5	28	48.4
2	W7	1009.26	55.77	12.5	25	37.5	55.6
3	W4	991.7	57.4	12.5	27.3	39.8	63.4
4	w1	1159.8	64.21	12.5	30.9	43.4	75.3
5	s12	1155.6	60.6	12.5	31.5	44	77.5
6	W10	994.9	57.4	12.5	27.3	39.8	64.8
7	W13	1006.42	58.22	12.8	15.8	28.6	45.6
8	W15	990.52	59.72	12.9	15.9	28.8	44.3
9	s6	902.51	54.05	13.2	18.5	31.7	48.7
10	W17	902.51	59.36	13.6	20.3	33.9	46.3
11	W19	1002.934	58.9	13.7	18.1	31.8	50.7
12	W21	976.59	59.87	13.7	26.4	40.2	44.8
13	m5	999.67	59.94	13.8	20.6	34.4	50.7
14	W22	1054.643	63.11	13.9	23.9	37.8	56.3
15	W2	997.35	63.11	13.9	24.9	38.8	57.8
16	W5	981.07	59.87	14.3	25.3	39.6	57.4
17	s1	1176.2	52.45	14.4	29.2	43.5	83.4
18	W8	1068.3	61.8	14.7	31	45.7	69.8
19	W11	984.65	59.98	14.8	35	49.8	70.3
20	W14	1013.42	60.17	16.3	35.5	51.8	60.8
21	m7	1078.5	40.546	16.6	28.6	44.85	73.4
22	W18	959.9	59.18	16.7	25.6	42.3	55.7
23	w16	1003.5	60.14	16.7	33	49.7	62.3
24	M1	1126.5	63.17	16.8	22.4	39.2	50.4
25	W20	1012.9	59.34	16.8	28.6	45.4	55.4
26	W3	995.96	62.2	16.9	33.4	50.3	61.4
27	W6	967.1	59.58	17.3	28.1	45.4	62.5
28	W9	995	60.11	16.9	33.4	50.3	60.4
29	W12	1055.19	61.68	17.6	22.9	40.5	48.9
30	s11	983.3	61.69	18	25	43	44.3
31	S10	1010.33	59.31	18	26.7	44.7	43.8
32	Y4	997	62.94	18	16.8	34.8	33.5
33	M2	978.3	61.21	18.2	15.3	33.5	25.15

Continued

35	s5	1023.2	57.74	18.4	14.4	35.8	17.45
36	Y7	990.6	54.99	19	11.5	31.5	16.36
37	Y2	1002.2	53.16	19.1	10.9	30	15.32
38	m10	1038.1	56.09	19.1	12.8	31.9	22.67
39	S7	1064.46	60.77	19.3	22.5	41.8	40.5
40	Y5	1026.5	59.338	19.4	30.9	50.3	46.67
42	s9	1144.4	59.88	20	25.6	45.6	48.07
42	Y11	1009.03	50.068	20	25.6	45.6	48.37
43	m6	1138.5	64.42	22.1	24.2	46.3	40.8
44	Y9	997.12	61.69	22.2	11	33.2	9.3
45	S4	982.79	63.07	22.6	13.8	36.4	14.4
46	Y3	1009.73	62.25	22.8	22	44.8	25.8
47	S3	1015.26	64.06	23.1	19.8	42.9	24.51
48	Y8	999.77	60.192	23.2	20	43.2	27.21
49	S8	995.77	63.27	24.5	21.5	46	26.3
50	Y1	1010.36	60.71	25.8	18.5	44.3	15.8