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Research Paper

Prediction of Bearing Capacity of Fine Grained Soils from Field Dynamic Cone Penetration Reading: Case of Clayey Soil from Asella, Ethiopia

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Abstract

Bearing capacity is one of the important parameters that needs to be considered for design and construction of foundations of civil engineering structures. The conventional method of estimating this parameter is commonly using Terzaghi's equation, which requires the shear strength parameter of soil. Shear strength test is expensive and time consuming, hence conventional method of estimating bearing capacity is uneconomical especially for small scale projects. Dynamic cone penetrometer (DCP) is one of the simple in situ test methods widely used in site investigation for estimating the strength and density of soils. This work is, therefore, aimed at predicting bearing capacity of shallow foundations from field in situ DCP reading for fine grained soils of Asella town, Ethiopia. The soil index properties and tri-axial tests to determine the shear strength parameters were performed on soil samples extracted from different pits. On the same pits, DCP test have been performed in the field to record D-value (mm/blow). Then, regression analyses have been performed to determine relationship of allowable bearing capacity (q_{all}) with D-value and index properties. Regression analyses results showed that q_{all} have strong relation with D-value having coefficient of determination (R^2=0.89) than index properties. The footing dimension have no effect on the predicted bearing capacity of foundation. The comparison between the developed equation and the actual q_{all} obtained from Terzaghi's bearing capacity equation shows average variations of 11.7%, suggesting the validity of the developed equation.

Keywords: *Dynamic Cone Penetration, Bearing Capacity, Shallow foundation, Shear strength, Correlation*

1. Introduction

The causes of failure of light weight engineering structures built on clayey soils are mostly related to the expansive nature of the soils (Bell, 1996). Improper estimation of bearing capacities of the soil mass can also lead to failure. On the other hand, underestimation of soil load carrying ability leads to increase of cost of project unnecessarily. Bearing capacity is, therefore, one of the important parameters to be accurately investigated for design of foundations of civil engineering structures (Bowles, 1993).

In conventional approach, bearing capacity of foundation soil is mostly calculated from Terzaghi's bearing capacity equation (Murthy, 2007), which requires the shear strength parameters such as cohesion (C) and angle of internal friction (Φ). Tri-axial compression test is a widely used method to measure the shear strength parameters of a soil in the laboratory by controlling confining stress and drainage condition. The aforementioned method, however, requires relatively more time, effort as well as money. Therefore, it may not be economical for small engineering projects such as in residential house development. In light of these challenges, there is a need to find time and cost effective ways that can replace shear strength parameters for determination of bearing capacity of shallow foundation for light structures. In addition, the application of field tests for site investigation are important for professional engineers since present practice is to rely more on field tests. The in situ field test can avoid some of the problems of sample disturbance associated with the extraction of soil samples from ground.

Dynamic cone penetrometer (DCP) instrument is simple and portable. It is one of the in situ penetration test methods widely used in site investigation for estimating the strength and density of soils (Sanglerat, 1972). The principle of DCP test is based on dynamic resistance offered by soil to deformation caused by dynamic penetrometer (Braja, 1985). Its application mostly limited to pavement overlay design. Due to this, extensive attempts have been made by different researchers to find the empirical relationship between DCP reading and California bearing ratio (CBR) (Scala, 1956; Van Vuuren, 1969; Kleyn, 1975; Livney, 1987; Amini, 2003, etc.). Their results suggests that CBR can be predicted satisfactorily from DCP values. However, as the results of previous works showed, the relationship between different engineering parameters with DCP is highly material dependent. Regarding correlation between DCP reading and bearing capacity of a soil, only few works have been carried out on limited types of soil (i.e., lateritic soil) and mainly on re-moulded soils at lab scale (Sanglerat, 1972; Cearns and McKenzie, 1988; Ampadu, 2005; Dzitse-Awuku, 2008). Ampadu & Dzitse-Awuku (2009) also contributed towards the search for a correlation by measuring the bearing capacity of a model ground in the laboratory and correlating it to the DCP test results. In all these mentioned works, the DCP test was conducted on re-moulded and re-compacted materials at optimum moisture contents.

In Ethiopia including the study area, it is common to place the foundation of simple structure on natural ground at depth less than 3 meter. So far no attempt has been made to correlate the in situ DCP result with the bearing capacity of natural ground (uncompacted and at natural moisture content). In addition, previously developed correlation by different researchers to predict bearing capacity of soil have limited application to different study area and are typically only reliable over the range of data from which they were derived. Adopting those developed correlations without improvement can lead to misinterpretation of soil behaviors. This study, therefore, attempts to establish correlation between allowable bearing capacity of shallow foundation and field dynamic cone penetration (DCP) reading for the case of expansive soils of Asella town.

2. Materials and Methods

Field DCP test and soil sampling were carried out in Asella town on 15 test pit locations (Figure 1) where expansive soils are identified visually. Depth of sampling varies from 1.5 – 2.5 m as it is common on the study area to place the foundation at this depth. Both disturbed soil (for index properties test) and undisturbed soil (for tri-axial compression test) samples were extracted from test pits.

For index properties test, the bulk soil samples were first air dried for about 3 days to almost constant moisture content. All the tests (i.e. grain size distribution, liquid limit (by Casagrande method), plastic limit, specific gravity, free swelling index, etc.) were conducted on representative sample following the ASTM standards. Two methods were used to find the particle-size distribution of the soil samples: sieve analysis, for particle sizes larger than 0.075 mm (No. 200) in diameter; and hydrometer analysis for particle-sizes smaller than 0.075 mm in diameter. Sodium hexametaphosphate ($NaPO_3$) was used as a dispersion agent.

The DCP test was performed in the test pit after excavation to the desired depth. The test device used in this study consists of two 16 mm diameter rods. The lower rod contains anvil and replaceable 60° apex cone. The upper rod contains 8 kg slide hammer with 575 mm drop height. The cone diameter (20 mm) is made wider than rod diameter (16 mm) so that penetration resistance is provided by cone alone and not side friction of the rods. The penetration achieved by each blow was recorded and plotted to obtain the dynamic cone penetration index (D-value). D-value is the penetration produced by one drop of the sliding hammer and it is obtained as the gradient of the line of best fit of the graph of cumulative blow against penetration in mm (Figure 2).

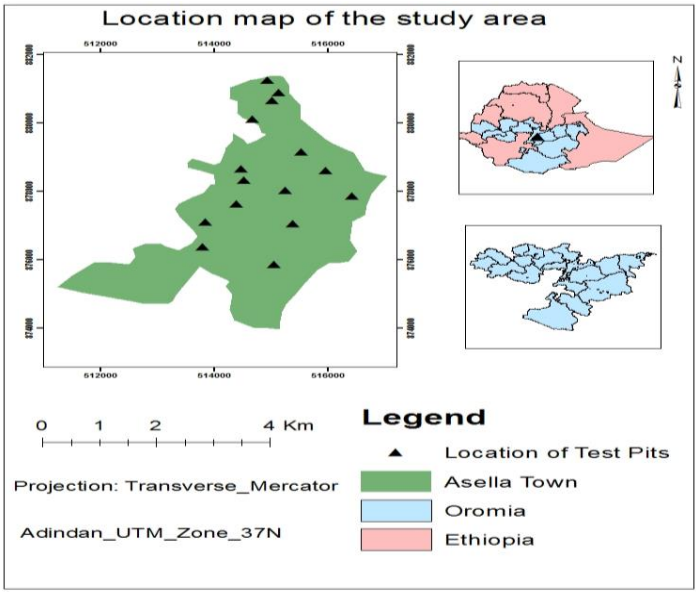

Figure 1: Location map of the study area and test pits

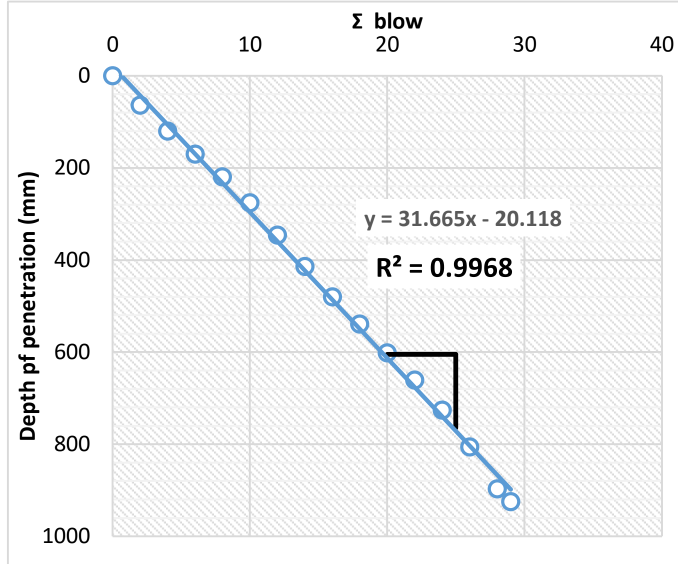

Figure 2: Typical cumulative blows-penetration curve from DCP test (Test pit 4)

In tri-axial compression tests, a cylindrical soil sample of standard size (diameter = 38 mm and height = 76 mm) was trimmed from undisturbed core sample and encased by a thin rubber membrane and placed inside a chamber. For conducting the test, the chamber is filled with water and the sample is subjected to a confining pressure of 150 and 300 kPa by application of pressure to the water in the chamber. Axial (or deviator) stress is applied through a vertical loading ram. No drainage is permitted during the test and specimen is sheared in compression without drainage at constant rate of axial deformation (strain controlled) of 1 mm/min. From stress-strain curve of tri-axial test, the deviator stress at failure and confining pressure were used to plot Mohr circle. From Mohr circle un-drained cohesion, C_u, (intercept) and internal angle of friction, Ø, (slope) were deduced and used to calculate the bearing capacity. The typical stress-strain curve and the Mohr circle for the determination of the shear strength parameters are shown in Figure 3 and 4, respectively.

3. Results and Discussion

3.1. The characteristics of the soil samples

The index properties of the fifteen samples tested in the laboratory are summarized in Table 1, while Figure 5 shows the grading curves of the soils. All of the studied soil samples are described as fine grained soil with materials finer than 0.075 mm and 0.002 mm are 73% - 91% and 35-53%, respectively.

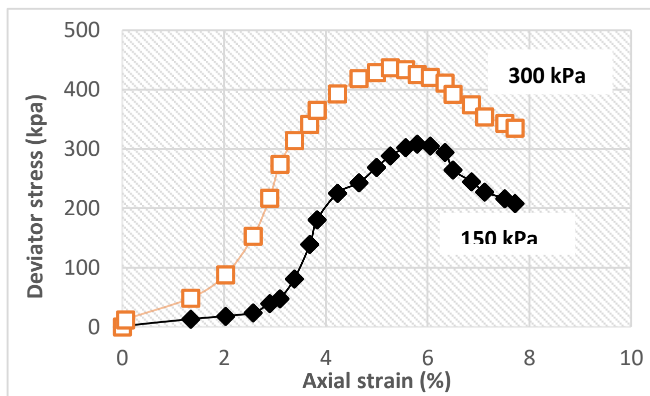

Figure 3: Typical stress-strain curve at different confining stress (Test pit 12)

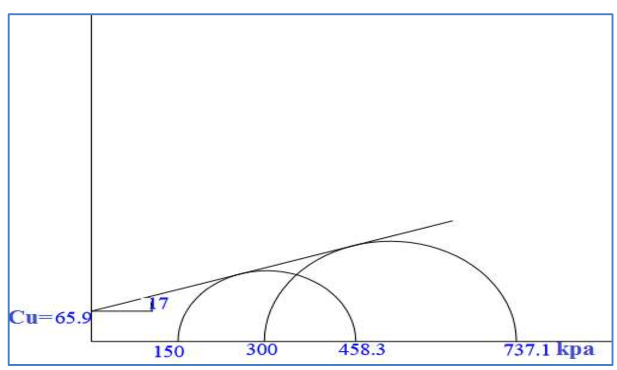

Figure 4: Mohr circle drawn from stress-strain curve (Figure 3) for determining cohesion (intercept) and internal friction angle (slope of tangent line)

Table 1: Basic properties of the soils used in this study

Test pit no.	Depth (m)	Clay fraction (%)	NMC (%)	LL (%)	PL (%)	PI (%)	SI (%)	CI	γ_{dry} (kN/m^3)	γ (kN/m^3)
1	1.5	37.1	48.8	64	33.3	30.7	130	0.5	12.9	17.95
2	2.5	52.5	47	64	26.8	37.2	105	0.46	12.2	17.65
3	2	28.2	42.5	61	39.4	21.6	62.5	0.74	13.1	17.9
4	1.5	50	44.6	70	29.3	40.7	107.5	0.62	12.5	17.6
5	2.5	48.5	43.3	68	31.7	36.3	82.5	0.68	12.6	17.9
6	2	47.8	41.6	64	28.3	35.7	102.5	0.63	12.5	17.6
7	1.5	58.5	35.3	80	33.3	46.7	132.5	0.98	12.4	17.4
8	1.5	47.4	37.9	56	23.4	32.6	112.5	0.71	12.8	17.65
9	2	49.7	37.3	58	26.8	31.2	125	0.66	12.5	17.65
10	2.5	48.3	35.4	66	28.6	37.9	135	0.82	12.7	17.9
11	2	43.4	40.5	60	31.4	28.6	117.5	1.03	12.8	17.85
12	1.5	49.5	40.1	70	24.4	45.6	115	0.66	12.8	17.85
13	1.5	46.7	37.8	63.5	31	32.5	122.5	0.91	13	17.9
14	1.5	47.4	39.3	67.5	31.7	35.8	97.5	0.79	12.7	17.6
15	2	34.3	37.5	58.5	33.3	25.2	85	0.84	12.3	17.7

The specific gravity of most of the soil was found to be in the range of 2.67 to 2.83 which is typical range for black cotton expansive soils (Murthy, 2007). The liquid limit (LL) and plastic limit (PL) range from 56% - 70% and 23% - 40%, respectively. The plasticity index (PI) range from 12 to 47%. Most of the soils of the study area fall in high plasticity type, except the soil from test pit 3 which is low plasticity type. Similarly, the free swelling index (SI) values and natural moisture content (NMC) are relatively higher indicating the soils under investigation possess significant water adsorption and retention capacity and higher swelling-shrinkage potential.

Classification of the soils according to unified soil classification system (USCS) is shown in Figure 6. The plot shows that except the soil from test pit 3 and 15, which are high plastic inorganic silt (MH), the soil samples are classified as high plastic inorganic clay (CH).

The results of the tri-axial tests showed that for all of the tested soils, a clear maximum deviator stress occurred at axial strain less than 7% at which the failure defined. The shear strength parameters (i.e. underained cohesion (Cu) and internal friction angles (Ø)) of the soils that used for computing bearing capacity are summarized in Table 2. As indicated in the table, the Cu ranges between 45 and 197.8 kPa and Ø value ranges between 3° and 21°. The cohesion value is relatively higher and friction angle is lower as expected for fine grained cohesive soils.

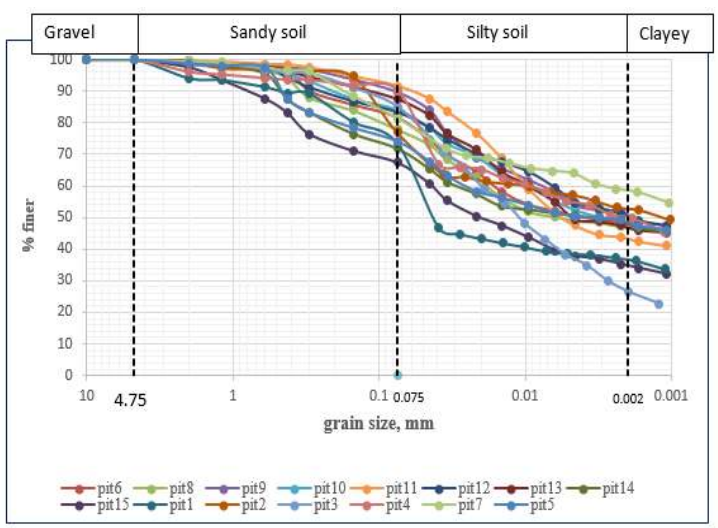

Figure 5. Grain size distribution of the soils

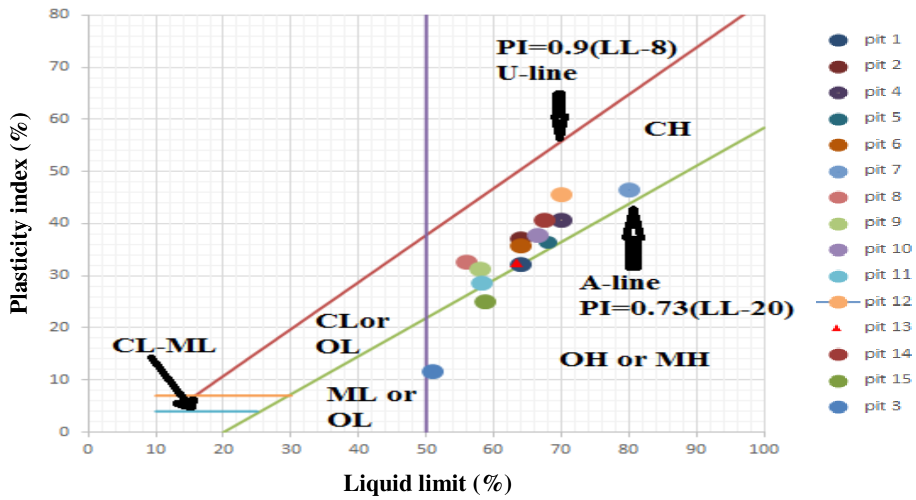

Figure 6: Classification of the soils according to USCS

Table 2: The shear strength parameter and computed ultimate bearing of the soils

Test pit no.	Depth (m)	C_u (kPa)	Φ_u (^{0})	q_{all} (kN/m^2)	DCP D-value (mm/blow)	DCP n-value (blow/100 mm)
1	1.5	98.75	3	267.5	43.6	2.29
2	2.5	94.15	3	266.95	52	1.54
3	2	181.93	10	696.94	20.67	4.84
4	1.5	197.77	4	533.16	31.66	3.16
5	2.5	58.53	21	643.97	29.52	3.39
6	2	152.45	4	421.56	35.87	2.79
7	1.5	57.42	11	269.22	41.97	2.38
8	1.5	86.33	10	345.0	37.88	2.64
9	2	72.43	13	387.91	36.62	2.73
10	2.5	120.41	3	333.59	38.2	2.62
11	2	101.6	15	568.34	31.72	3.15
12	1.5	65.9	17	468.15	32.25	3.1
13	1.5	56.5	19	449.33	32.98	3.03
14	1.5	110	10	429.15	33.7	2.97
15	2	45	19	403.78	38.54	2.59

3.2. Correlation between bearing capacity and engineering parameters

From consistency index (Table 1), physical state of the soils of study areas is dominantly stiff. For stiff cohesive soils and soils that show peak value in stress-strain curve at strain about 5%, types of bearing capacity failures are mostly general shear failure. Hence, the allowable bearing capacity (q_{all}) of the soil is computed using Terzaghi's bearing capacity equation (given below) with a factor of safety of 3.

$$q_{all} = [Cu\ Nc\ (1+0.3B/L) + \boldsymbol{\gamma} D_f\ Nq\ (1+B/L) + 1/2\boldsymbol{\gamma} BN\gamma\ (1-0.2B/L)] / 3$$

Where N_C, Nq and $N\gamma$ are Terzaghi's bearing capacity factors, D_f is depth of footing, B and L are the width and length of footing, respectively and $\boldsymbol{\gamma}$ is the bulk unit weight.

Rectangular footing of 1.5 m wide and 4 m long, which is a common footing dimension in the study area, and depth of foundation at proposed pit depth were used for ultimate bearing capacity determination. The results are summarized in Table 2.

To visually evaluate the data for potential relationship, a simple linear regression analysis between allowable bearing capacity (dependent variable) and DCP n-value is carried out. For comparison, the correlations with other properties of soils are also included. The scatter plots are shown in Figure 7 to 12. From the figures, it can be observed that q_{all} has weak negative correlation with LL and PI and weak positive correlation with CI, bulk and dry unit weight. On the other hand, q_{all} has relatively strong negative correlation with DCP D-value as indicated by the coefficient of determination (R^2) of 0.89. The relatively strong correlation between q_{all} and DCP D-value indicates the potential of DCP test in complementing the conventional geotechnical testing methods to determine the bearing capacity of shallow foundation. The model is able to explain over eighty nine percent of the variation in bearing capacity of soil.

To examine the effect of width of footing on prediction of bearing capacity from DCP test results, bearing capacity for different width of footing have been calculated from Terzaghi's bearing capacity equation and correlated with DCP D-value (Figure 12). The coefficient of correlation remained almost the same for different width of footing, suggesting the width of footing has less effect on the prediction of bearing capacity from DCP reading.

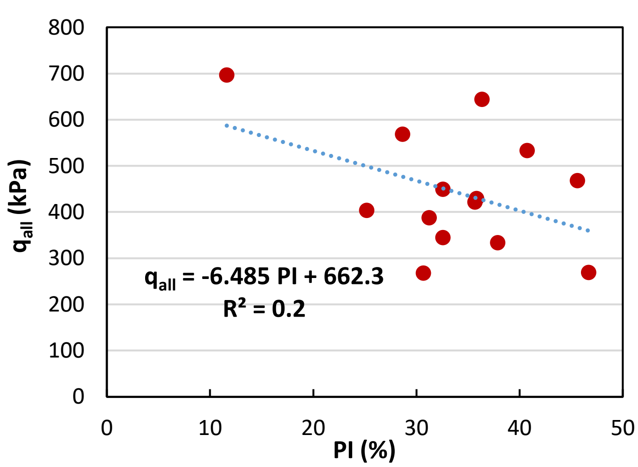

Figure 7: Plot of allowable bearing capacity (q_{all}) and plasticity index (PI)

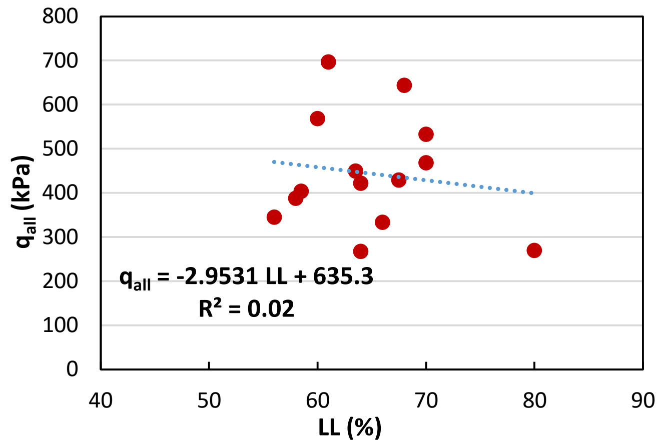

Figure 8: Plot of allowable bearing capacity (qall) and liquid limit (LL)

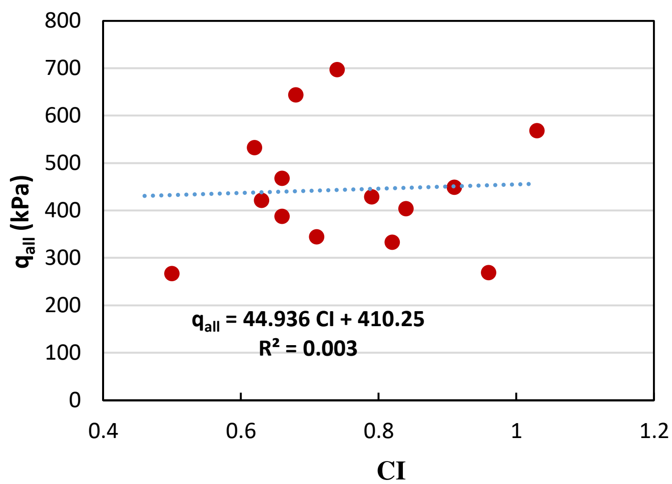

Figure 9: Plot of allowable bearing capacity (qall) and consistency index (CI)

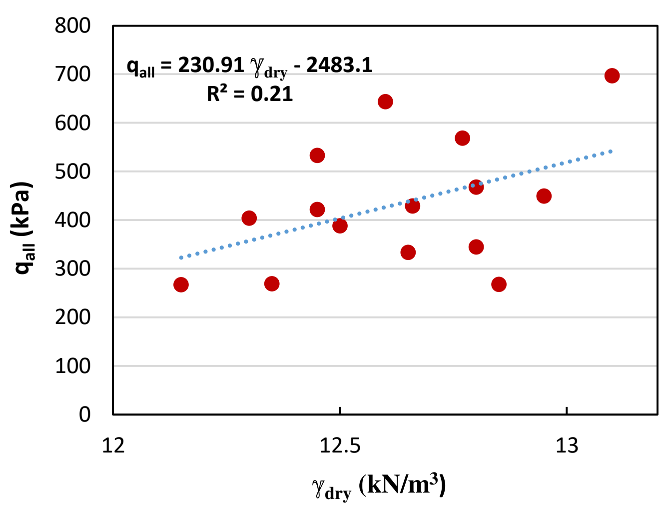

Figure 10. Plot of allowable bearing capacity (q_{all}) and dry unit weight (γ_{dry})

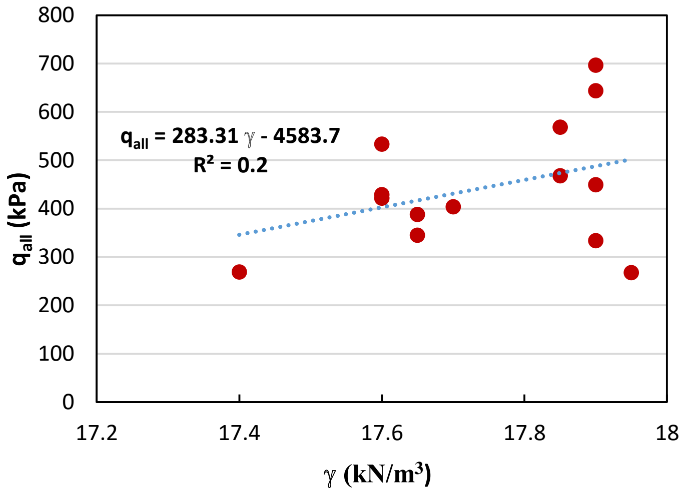

Figure 11: Plot of allowable bearing capacity (q_{all}) and bulk unit weight (γ)

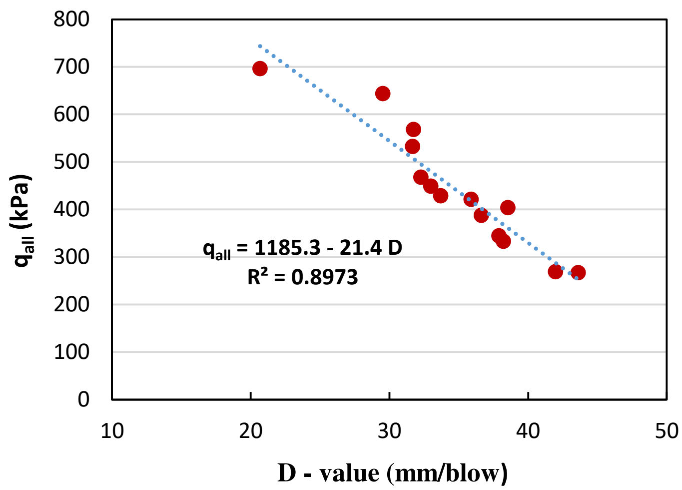

Figure 12. Plot of allowable bearing capacity (q_{all}) and DCP reading (D-value)

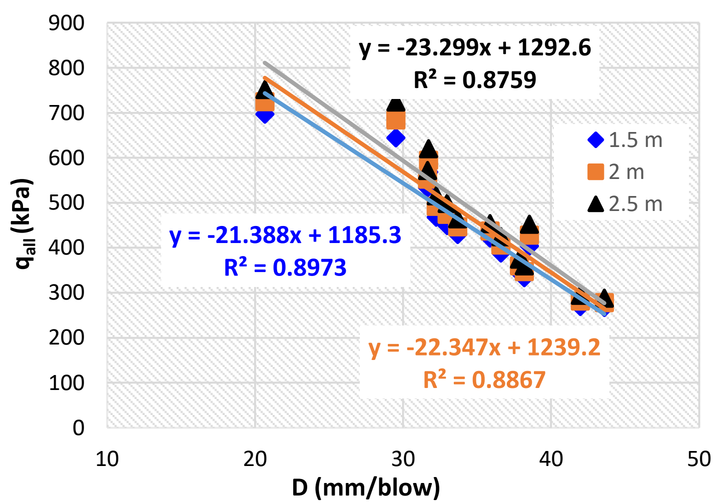

Figure 13. Plot of allowable bearing capacity (q_{all}) and DCP reading (D-value) for different footing width

3.3. Evaluation and validation of the developed equation

The validity of the developed empirical correlation has been examined by calculating ultimate bearing capacity using developed equation and comparing it with the actual value (i.e. computed using Terzaghi's equation) and also with previous works.

As indicated in Table 3, the average variation of the predicted bearing capacity from actual bearing capacity is about ±11.7%. It indicates that bearing capacity of expansive soil of study area can be reliably predicted from the field DCP readings.

The result of this study is compared with the work of Ampadu (2005) for the relationship between q_{all} and DCP n-value (Figure 14). The DCP device and the type of soil (i.e. high plastic fine grained soil) used by Ampadu (2005) are similar with the present work. It must however be noted that, the test conditions are different. In the present case the DCP test was conducted in field at natural moisture content and density, whereas Ampadu (2005) conducted the test in-mould on re-moulded and re-compacted soil at optimum moisture content. Hence, the range of n-values and computed bearing capacity are relatively lower in the present study. It can however be noted that, irrespective of testing conditions, the correlation equation for both works have comparable gradients i.e., as the n-value increases, the bearing capacity of the soil increases nearly at similar rate.

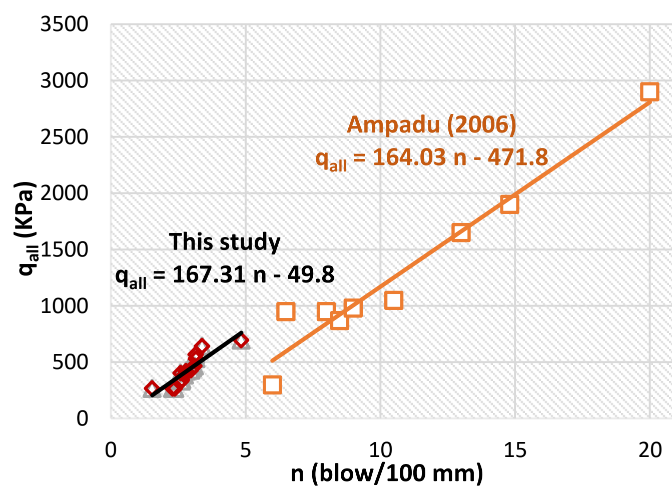

Figure 14. Comparison of the developed empirical correlation between DCP reading and bearing capacity with the work of Ampadu (2005).

Table 3. Comparison of calculated and predicted allowable bearing capacity

Sample no.	Actual q_{all} (kN/m^2) from Terzaghi equation A	Predicted q_{all} (kN/m^2) a	Variation % (A-a)*100/A
1	267.5	252.3	5.7
2	267.0	72.5	72.8
3	696.9	743.0	-6.6
4	533.2	507.8	4.8
5	644.0	553.6	14.0
6	421.6	417.7	0.9
7	269.2	287.1	-6.7
8	345.0	374.7	-8.6
9	387.9	401.6	-3.5
10	333.6	367.8	-10.3
11	568.3	506.5	10.9
12	468.2	495.2	-5.8
13	449.3	479.5	-6.7
14	429.2	464.1	-8.1
15	403.8	360.5	10.7
	Average		**$\pm$ 11.7**

4. Conclusion

There is better correlation between bearing capacity and in situ DCP reading than between bearing capacity and index properties. The width of footing has no effect on the empirical correlation equation between DCP reading and bearing capacity. Based on the results of this research study, it can be conclude that bearing capacity of shallow foundation can be predicted reliably from field DCP reading. Therefore, it can be considered as cost effective alternative for preliminary site investigation of simple structures. However, for soils outside of the study area and/or on different soil types, users are advised not to fully rely on the developed equation or it should be used with extreme caution.

Additional works need to be done by increasing sample number and for different footing shape so as to validate the established correlation over the wide range.

Acknowledgment

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Reference

Amini, F. (2003). *Potential application of dynamic and static cone penetrometers in Mississippi department of transportation pavement design and construction*. Final report, Mississippi department of transportation, USA.

Ampadu, S.I.K. (2005). A correlation between the Dynamic Cone Penetrometer and bearing capacity of a local soil formation. *Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering*, 655-659.

Ampadu, S.I.K. and Dzitse-Awuku, D. (2009). Model tests for bearing capacity in a lateritic soil and implications for the use of the dynamic cone penetrometer. *Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering*.

Bell, F.G. (1996). Lime stabilisation of clay minerals and soils. *Eng. Geol. 42*, 223–237.

Bowles, J. E. (1993). *Foundation Analysis and Design*, 4th Edition, McGraw-Hill, Inc., New York

Braja, M.D. (1985). *Principles of Foundation Engineering*, PWS Publisher Boston

Cearns, P.J. and McKenzie, A. (1988). *Application of Dynamic Cone Penetrometer in East Anglia, Proc. of the Symposium on Penetration Testing in the UK*, Thomas Telford, London.

Dzitse-Awuku, D. (2008). Correlation between dynamic cone penetrometer and allowable bearing pressure of shallow foundation using model footing: Thesis submitted to department of civil engineering, Kwame Nkrumah University of science and technology.

Kleyn, E.G. (1975). *The Use of the Dynamic Cone Penetrometer (DCP)*, Report No. 2/74 Transvaal Road Dept, South Africa.

Livney, M. (1987): *Validation of Correlations between a Number of Penetration Tests and In Situ California Bearing Ratio Tests*, Transportation Research Record 1219, Transportation Research Board, Washington D.C.

Murthy, V. (2007). *Geotechnical Engineering, Principles and Practice of Soil Mechanics and Foundation Engineering*. New York: Marcel Dekker, INC.

Sanglerat, G., 1972, *The Penetrometer and Soil Exploration-Interpretation of Penetrometer Diagrams-Theory and Practice*, Elsevier Scientific Publishing Company

Scala, A.J. (1956). Simple Methods of flexible pavement design using cone penetrometers, *New Zealand Engineer, 11* (2), 33-44.

Van Vuuren, D.J. (1969). Rapid Determination of CBR with the Portable Dynamic Cone Penetrometer, *The Rhodesian Engineer, Paper No. 105*.