Application of Dynamic Cone Penetration (DCP) Testing in Analysis of Shallow Foundations – A Case Study of the Jose Eduardo Dos Santos (JEDS) Campus, Namibia

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ABSTRACT: Methods like Terzaghi's formula for allowable bearing pressure determination are costly and time consuming, especially for shallow foundation designs. Alternative methods, like the DCP, are being explored. This study aimed to establish a reliable correlation between DCP penetration rate and allowable bearing pressure through literature review. Several empirical formulas were derived based on these findings. Tests in five pits yielded results with strong relationship and high coefficient of determination.

1 INTRODUCTION

The foundation is the part of a structure that transmits loads directly to the underlying soil. An important aspect of foundation design involves the development of design parameters, such as the bearing capacity of the soil. The conventional laboratory method for estimating this parameter is known as Terzaghi's equation, which requires the shear strength parameter of the soil Abdela & Chemeda (2019). The shear strength test is expensive and time-consuming. In addition, the modern industry cannot afford the delays and costs of old methods. Considering these challenges, there is a need to find time and cost-effective ways to replace shear strength parameters to determine soil-bearing capacity for shallow foundations Abdela & Chemeda (2019). In-situ standard penetration tests (SPT) and cone penetrometer tests (CPT) are usually used for the computation of soil-bearing capacity (Hassam, 2015). The CPT is an accurate method, but apart from this, it is time-consuming, requires a trained operator with a crew to operate the machine, and it is expensive. Therefore, this test is not economical for small-scale and low-cost projects for shallow foundation design.

The DCP is one of the simple, inexpensive in situ field testing methods widely used in pavement design for estimating the strength and density of soil. It is globally promoted because it does not require the sampling of undisturbed soil, especially when loose or submerged sandy soil is encountered (Hamid, 2015). Since its development, the DCP has been widely used as a simple but effective means of determining the in-situ stiffness of subgrade materials. It is widely used worldwide for its conjunction with various empirical correlations, such as the California Bearing Ratio (CBR) determination. For this purpose, extensive research by Scala (1956), van Vuuren (1969), Kleyn (1975), and Livney (1987) has been carried out to establish a correlation between the DCP penetration rate (DCPI) and the CBR Ampadu (2005). However, the difficulty in successfully applying this equipment lies in the absence of a credible correlation between the DCPI results and the bearing capacity of the soil Ampadu (2005). To enable the utilization of DCP as a standalone device in assessing soil strength for the foundation design, several researchers, as seen in Table 1, have come up with correlations between the DCPI n-value and the allowable bearing capacity (q_{all}).

Table 1. Constants of different authors for the correlation formula of q_{all}.

Authors	Constants		Equation
	a	b	
Sanglerat (1972)	0	48.7	
Dzitse-Awuku (2008)	48	57	aDCPI100+b
Ampadu (2005)	164	504	
Abdela (2019)	167.31	49.8	

*Note: DCP n-Value = DCPI100 (# of blows per 100 mm penetration).

Therefore, this study aims to establish a correlation between DCP field data collected at Jose Eduardo dos Santos (JEDS) campus and the calculated q_{all} derived from Terzaghi's equation. The results will be compared with correlations derived by Sanglerat (1972), David Dzitsse-Awaku (2008), Ampadu (2005), and Abdela & Chemeda (2019). The goal is to determine the effectiveness of DCP as a foundation analysis tool, regardless of testing conditions and soil types employed by other researchers.

2 METHODOLOGY

2.1 *Equipment*

The DCP used in this study Figure 1, consisted of an upper rod containing an 8 kg hammer sliding on a 16 mm diameter rod with a 575 mm drop height, falling on an anvil attached to a 16 mm diameter rod on the top with a 60° cone angle at the bottom.

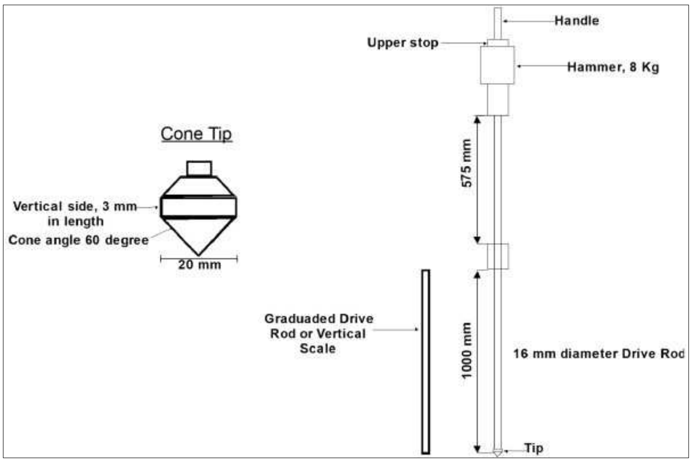

Figure 1. Dynamic Cone Penetrometer (DCP) apparatus.

2.2 *Testing*

As can be seen in Figure 2, five (5) exploratory test pits labelled as L1, L2, L3, L4, and L5 were excavated at a one-meter depth around the German Wing C-block on JEDS campus. These pits extend beneath the 750 mm strip foundation depth of the German Wing C-block unit. One set of the DCP field tests and disturbed soil sampling was carried out in each test pit. The collected samples were analyzed in the laboratory to obtain their engineering properties, and the Atterberg limits were tested from a representative sample following the SANS 3001-GR10:2014 standards.

Figure 2. Location of five test pits (red) at JEDS campus.

3 RESULTS AND DISCUSSION

3.1 *Soil sample characteristics*

A representative soil sample was sieved through various sieves according to SANS 3001-AG1:2014. The retained material was weighed, recorded, and plotted to obtain particle size distribution curves for each sample, as shown in Figure 3. Based on the results shown on the particle size distribution curve, samples L3, L4, and L5 indicated soil with a range of particle sizes of well-graded soil. In contrast, samples L1 and L2 curves were almost horizontal at some ranges, indicating a gap-graded soil material. The soil samples underwent classification based on the Unified Soil Classification System (USCS). The classification revealed a consistent pattern across all five samples, with coarse-grained soils constituting over 50 %, primarily dominated by sand with fines comprising less than 5 %. These findings position all samples within the USCS coarse aggregates chart. Following this chart with the calculated percentages places the soil in the category of well-graded sand with gravel.

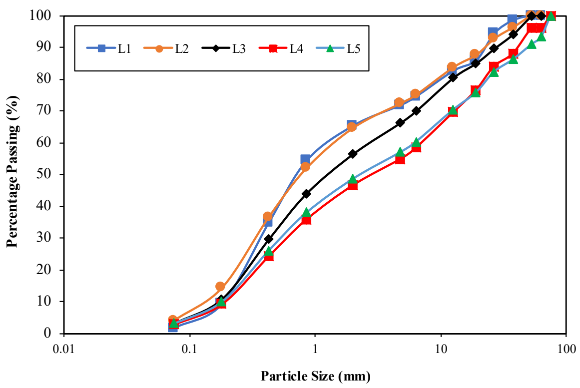

Figure 3:Grading Curves for five samples.

The Atterberg limits could not be obtained from sample L1. Therefore, the sample was concluded to be non-plastic (NP), while samples L2, L3, L4, and L5 gave values of liquid limit (LL) ranging from 25.75 % to 29.46 %, plastic limit (PL) from 21.43 % to 25.64 %, plasticity index (PI) from 2.87 % to 4.32 % and linear shrinkage index (SI) from 0.7 % to 1.94 %. Further analysis and observation of the results show a minimum difference between the LL and PL, giving a low PI (PI<7), indicating a slightly plastic behaviour.

Table 2. Basic soil properties for five samples.

Test pit	Atterberg Limits				Compaction		
	LL (%)	PL (%)	PI (%)	SI (%)	OMC (%)	γdry (kN/m^3)	γ (kN/m^3)
L1	NP	NP	NP	NP	11.6	18.59	19.47
L2	25.75	21.43	4.32	1.41	12.00	18.24	19.29
L3	25.82	22.95	2.87	0.70	12.00	18.08	19.08
L4	27.36	23.00	4.36	1.41	13.00	16.83	18.99
L5	29.46	25.64	3.82	1.94	12.60	17.81	18.97

3.2 *Dynamic Cone Penetrometer (DCP) test*

The DCP test was conducted on natural ground, where the penetration achieved by each blow was recorded and plotted to obtain the dynamic cone penetration index/rate (DCPI) for each test pit. A representative DCPI value of each of the five samples tested was obtained by averaging the DCPI across the entire penetration depth at each test location, given in Table 3. This study obtained the friction angle (ϕ) by using a commonly cited formula derived by Mohammadi et al. (2008) who found the relationship between the DCPI value and the friction angle (ϕ), as shown below:

$$\phi = 52.16DCPI_{0.13} \quad (1)$$

where ϕ = friction angle; and $DCPI$ = Dynamic Cone Penetration Index

Therefore, the average DCPI value for each test pit was converted to a DCP n-value by dividing it by 100 mm. The DCPI n-value indicates the number of blows required to drive the DCP cone 100 mm into the ground.

Table 3. Basic soil properties for five samples.

Test pit No	Depth (m)	DCPI (mm/blow)	ϕ (°)	DCP n-value (blow/100 mm)	q_{all} (kPa)
L1	1	51.84	31.22	1.93	197.17
L2	1	31.58	33.30	3.17	257.84
L3	1	40.11	32.28	2.49	224.89
L4	1	45.00	31.80	2.22	209.67
L5	1	46.84	31.63	2.13	204.46

3.3 *Correlation between bearing capacity and engineering properties*

The q_{all} of the soil results, summarized in Table 3, was computed using Terzaghi's bearing capacity equation for strip foundation with a safety factor of 3, as shown below:

$$q_{all} = \frac{\gamma Nq+\gamma 2BN\gamma+cNc}{3} \quad (2)$$

where γ = Bulk unit weight of soil; N_q, N_γ, and N_c = Bearing capacity factors; B = Foundation breadth; and c = cohesion.

The results obtained were modelled using the statistical linear regression method as shown in Figures 4a-b, which shows q_{all} having a moderate negative correlation with LL and PL, with a coefficient of determination (R^2) of 0.59 and 0.69. Meanwhile, in Figures 4 c-d, the q_{all} had a weak negative and positive correlation with PI and SI, with a coefficient of determination of 0.03 and 0.09. Finally, the results of Figure 5a-c showed a weak negative and positive correlation of q_{all} with maximum dry unit weight (γdry), OMC, and bulk unit weight (γ), with a coefficient of determination of 0.03, 0.03, and 0.01. Meanwhile, in Figure 5d, the DCPI values showed a strong correlation in with the q_{all}, with a high coefficient of determination of 97 %.

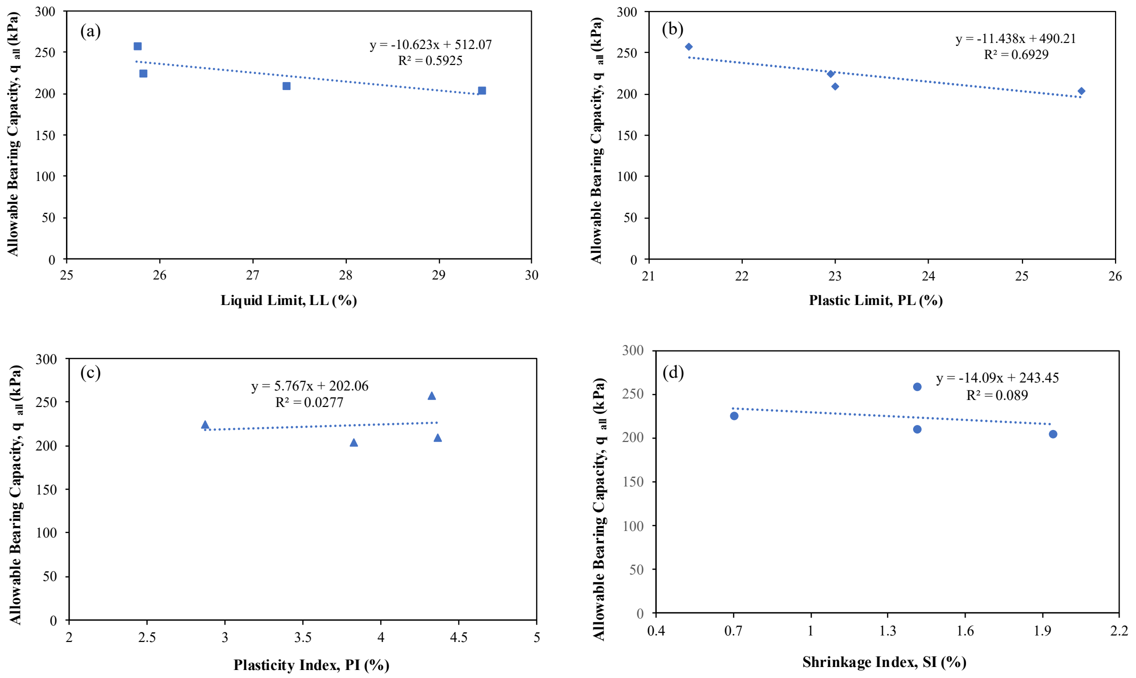

Figure 4: Plots showing variation of allowable bearing capacity (q_{all}) against: (a) liquid limit (LL), (b) plastic limit (PL), (c) plasticity index (PI) and (d) shrinkage index (SI).

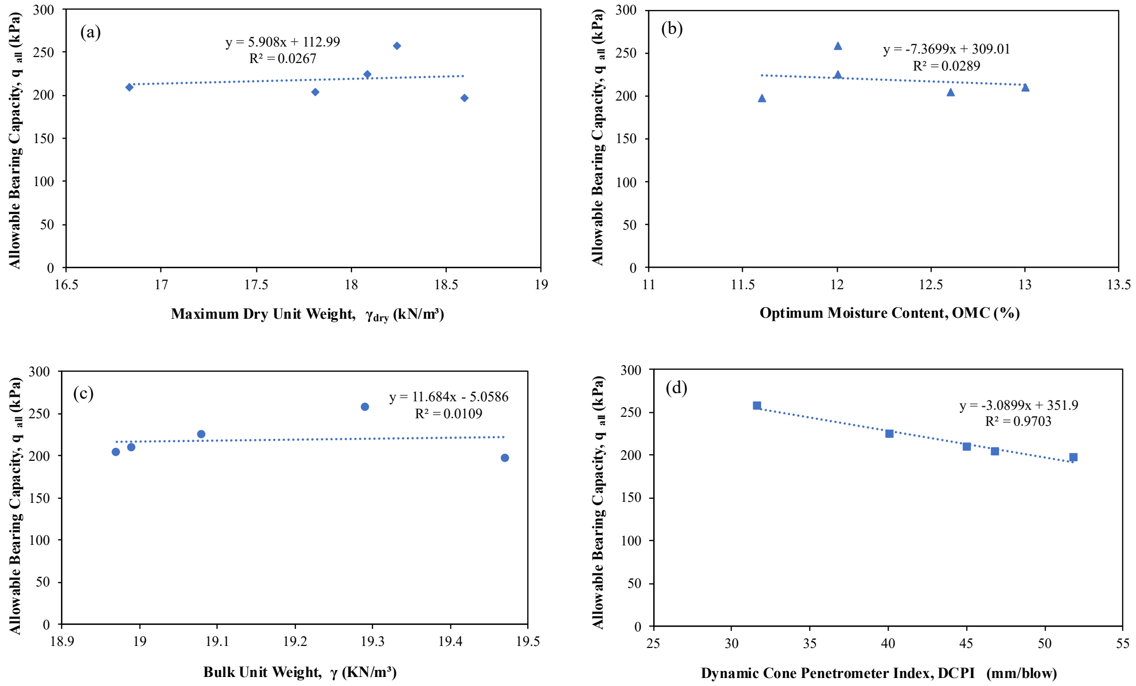

Figure 5: Plots showing variation of allowable bearing capacity (q_{all}) against: (a) maximum dry unit weight (γdry), (b) optimum moisture content (OMC), (c) bulk unit weight (γ) and (d) dynamic cone penetrometer index (DCPI).

The relatively strong correlation between the q_{all} and DCPI indicates the potential of the DCP test conducted on natural ground, complementing Terzaghi's conventional geotechnical testing methods to determine the q_{all} of shallow foundations.

3.4 *Comparison to literature empirical formulas*

In the comparison of the results of this study with work done by other researchers, the DCPI value was converted into DCPI n-value, and its correlation was observed with a high coefficient of determination of 99 % (Fig. 6). Empirical formula for q_{all} from the DCPI n-value was obtained below:

$$q_{all} = 50.153n + 98.98$$
$$q_{all} \approx 50.2n + 99 \quad (3)$$

where n = DCPI n-value.

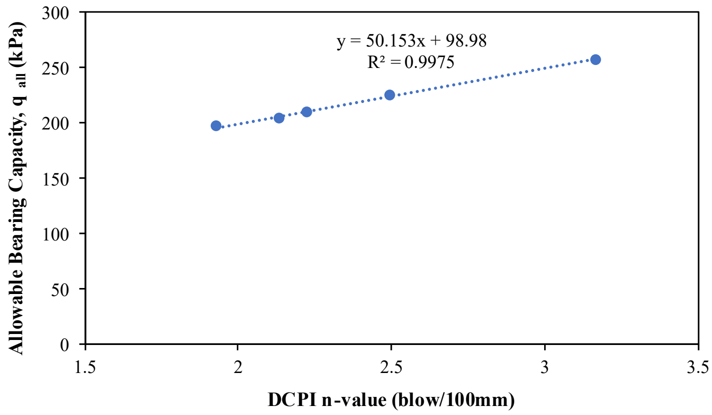

Figure 6: Plot showing variation of allowable bearing capacity (q_{all}) against DCPI n-value.

The results of the predicted empirical formula of this study were compared with those derived by Ampadu (2005), David Dzitsse-Awaku (2008), Abdela & Chemeda (2019) and Sanglerat (1972). Ampadu (2005) and David Dzitsse-Awaku (2008) carried out DCP tests in the laboratory mould with soil having been compacted at OMC. Meanwhile, Abdela & Chemeda (2019) performed the DCP on natural ground and Sanglerat (1972), utilized the "Dutch formula" which was based on the Newton's principle of energy. The empirical formulas derived by these researchers were substituted with the DCPI n-value obtained in this study to calculate q_{all}, and the results are tabulated in Table 4.

Table 4. Comparison of the derivation of q_{all} from literature formulas.

Authors	Empirical equations (q_{all})	q_{all} (kPa)				
		L1	L2	L3	L4	L5
Sanglerat (1972)	48.7n	93.99	154.38	121.26	108.11	103.73
Dzitse-Awuku (2008)	48n+57	149.64	209.16	176.52	163.56	159.24
Amapadu (2005)	164n-504	-187.48	15.88	-95.64	-139.92	-154.68
Abdela (2019)	167.31n-49.8	273.11	480.57	366.80	321.63	306.57
This study (2020)	50.2n+99	195.89	258.13	224.00	210.44	205.93

The results were plotted in Figure 7 and showed a comparable gradient between this study, Sanglerat (1972) and David Dzitsse-Awaku (2008) irrespective of the testing conditions and soil types, while Abdela & Chemeda (2019) had a more significant gradient. This difference could be explained by the different soil types tested by each researcher from the different study areas since the DCP tool used by all the researchers mentioned above had the same characteristics. The different soil types tested included the well-graded sand with gravel in this study, while researcher David Dzitsse-Awaku (2008) tested on sandy clay material, Abdela & Chemeda (2019) tested on high plastic fine-grained soils, and Sanglerat (1972) used the "Dutch formula". The work done by Ampadu (2005), as stated by the literature, was limited to a calibrated DCPI n-value greater than 6; thus, negative values were obtained and were not plotted for comparison, as the DCPI n-values obtained from this study were below 6.

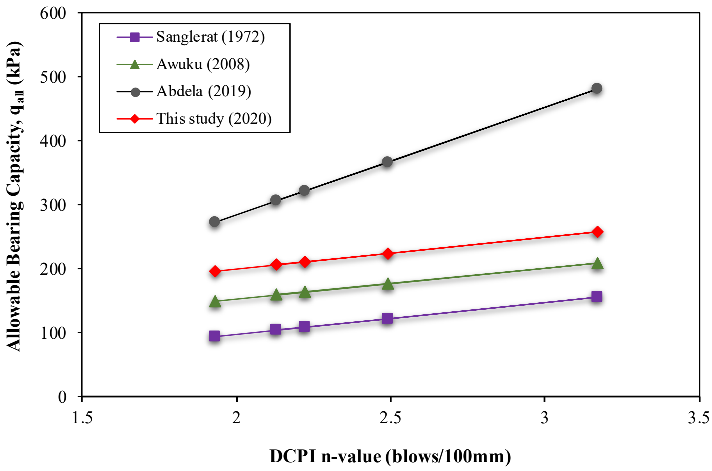

Figure 7: Comparison of q_{all} obtained from different empirical formulas from literature.

3.5 *Evaluation and validation of the Empirical equation*

The validity of the predicted empirical correlation formula obtained in this study was examined by comparing the q_{all} determined using Terzaghi's equation. The average variation of the predicted q_{all} and that from Terzaghi's equation gave a low ±0.06 % marginal error, as shown in Table 5. These results indicate the potential of the q_{all} of the study area to be reliably predicted from the field DCP readings.

$$Variation\ (\%) = \frac{(Terzaghi - Predicted)}{Terzaghi} \times 100 \quad (4)$$

Table 5. Comparison of calculated and predicted q_{all}

Test pit	q_{all} (kPa)		Variation
	Terzaghi	Predicted	(%)
L1	197.16	195.84	0.67
L2	257.84	257.96	-0.05
L3	224.89	224.16	0.33
L4	209.67	210.56	-0.42
L5	204.46	206.17	-0.84
Average			±0.06

4 CONCLUSION

This paper evaluated the reliability of using the DCP as a standalone tool for assessing soil strength for shallow foundation design. The results of this paper have concluded that the allowable bearing capacity (q_{all}) of safety factor 3 of shallow foundations can be predicted reliably from the field DCP test reading. Therefore, this tool can be considered as a cost-effective alternative for the design of foundations for simple structures. However, the developed equations cannot be used for all soil types, since adequate calibration of empirical correlation formulas is required, as well as some further research.

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