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INTRODUCTION

Adequate foundation design is essential for attainment of the intended performance of a building and indeed any engineering project. It requires suitable theories and design parameters obtained on geomaterials representing the field conditions of the ground where the building is to be developed. As such, the foundation design parameters are often preferably a result of extensive field assessments. Drawing on the Ugandan experience, this paper focuses on the evaluation of bearing capacity for shallow foundation design.

The field assessment method(s) adopted in especially developing countries usually depend on cost implications rather than suitability of the methods. This, in Uganda, is evidenced by the fact that for its comparative cost advantage and speed of work, dynamic cone penetrometer (DCP) which has a long history of extensive application in characterisation of highway subgrade strength (Scala 1956, Kley 1975, van Vuuren 1969) is steadily gaining popularity as a method for in situ assessment of soil strength for building foundation design. In Kampala, the capital city and business centre of Uganda, for instance, foundations for a number of buildings have been designed based on bearing capacity determined using DCP device through existing correlations between DCPI and CBR and/or bearing capacity. In this paper, it is shown that most of the correlations cannot give reliable shallow foundations design data without prior calibration.

BACKGROUND AND PRINCIPLE OF DCP

Whereas the actual first version may be difficult to tell, literature suggests that the present day DCP is traceable to 1956 when Scala developed the earliest version called the Scala Penetrometer which had a 9.08 kg hammer, fall height of 508 mm, 16 mm diameter rod and 30° cone (Paige-Green & Du Plessis 2009). Since then, the device has undergone several
modifications and refinements with the most commonly used in Uganda today being the TRL DCP.

In its most basic form, the current TRL DCP consists of a rod fitted with a replaceable 20mm diameter and 60° included angle cone that is driven into the soil using the energy provided by a slide hammer. The 60° cone was introduced (Salgado & Yoon 2003) and is preferred to the earlier 30° cone for durability reasons. The 8 kg hammer is dropped repeatedly through a standard height of 575 mm onto an anvil attached to the rod with the cone at the end while measuring the penetration depth for each hammer drop or set of drops until the desired depth is reached. On impact at the anvil, the kinetic energy associated with the falling mass is transferred to the conic tip through the rod. If the energy (about 144 kNm per unit cone area) is high enough relative to the soil strength, the soil fails in shear and the cone advances. The penetration of the cone into the soil, as a result of the imparted energy, is thus related to the strength of the soil. Guidelines for proper execution of the DCP test are published in various sources and standards. One such a standard is ASTM D6951/D6951M. To provide a wide dynamic range, a dual mass hammer is utilized to better cover the range (Webster et al. 1992). Alternatively, heavier DCP models such as the Borros Penetrometer are also available for relatively strong ground.

3 INTEPRETATION OF DCP TEST RESULTS

In order to use DCP test results to design building foundations, the penetration results have to be converted into bearing capacity. Numerous equations have been developed for this purpose. It was not the intention of the author to enlist all the equations that have been developed in this regard. Therefore, only the commonly cited and used were examined. In so doing, any equations that are not available for free access were not considered. It is believed that the users of the DCP test method are unlikely to look for information not readily and freely available to them. They prefer it for its straightforwardness, inexpensiveness and ease of use.

The equations available for conversion of DCP penetration data into bearing capacity generally fall in two categories, namely, direct and indirect equations. With the former category, DCP penetration indexes (DCPI) are converted directly to bearing capacity while with the latter they are first converted to CBR and then to bearing capacity.

3.1 Direct equations

In an attempt to enable utilisation of DCP as a standalone device in the assessment of soil strength for the design of foundations, several researchers have come up with correlations between the penetration index (DCPI) and bearing capacity. Generally, the equations are of two forms: that of Equation 1 and the second of Equation 2 with \( a \) and \( b \) as curve fitting constants. Table 1 shows the curve fitting constants for some of the equations in this category. In the table, the first three are of Equation 1 form whereas the rest are of Equation 2 form.

\[
q = a + b\text{DCPI}_{100}
\]

\[
q = a\text{DCPI}^b
\]

\( \text{DCPI}_{100} \) is the number of blows per 100 mm penetration (also referred to as \( n \) values) and \( q \) is bearing capacity. In Equation 2, \( \text{DCPI} \) is simply the penetration index.

### Table 1. Constants for some of the direct equations

<table>
<thead>
<tr>
<th>Author(s)</th>
<th>Constants</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ampadu (2005)</td>
<td>( a = 164; b = -504 )</td>
</tr>
<tr>
<td>Dzitse-Awuku (2008)</td>
<td>( a = 48; b = 57 )</td>
</tr>
<tr>
<td>Sanglerat (1972)*</td>
<td>( a = 0; b = 48.7 )</td>
</tr>
<tr>
<td>Wilches et al. (2018)</td>
<td>( a = 112.03; b = -0.803 )</td>
</tr>
<tr>
<td>Paige-Green &amp; Du Plessis (2009)</td>
<td>( a = 3426; b = -1.014 )</td>
</tr>
<tr>
<td><strong>USACE</strong> **</td>
<td>( a = 292; b = -1.12 )</td>
</tr>
<tr>
<td>* Takes into account the features of the TRL DCP device. ** US Army Corps of Engineers.</td>
<td></td>
</tr>
</tbody>
</table>

3.2 Indirect equations

Table 2 shows the examined correlations in the category of indirect equations. From the table, it can be seen that the general form of these equations is a power one as depicted in Equation 3 (\( a, b \) and \( c \) are curve fitting parameters) and that given CBR values bearing capacity is estimate-able. Luckily, it is possible to compute CBR from DCP test data and numerous correlations have been developed for this purpose. Among the commonly cited and readily available equations are those presented in Table 3 from which it can be deduced that the equations relating DCPI to CBR take the general form of Equation 4 in which \( a \) and \( b \) are curve fitting parameters as was the case for Equations 1, 2 and 3.

\[
q = a(\text{CBR} - b)^c
\]

\[
\log_{10} \text{CBR} = a + b\log\text{DCPI}
\]

### Table 2. Constants for some of the indirect equations

<table>
<thead>
<tr>
<th>Author(s)</th>
<th>Constants</th>
</tr>
</thead>
<tbody>
<tr>
<td>Black (1961)</td>
<td>( a = 70; b = 0; c = 1 )</td>
</tr>
<tr>
<td>Portland Cement Association</td>
<td>( a = 180; b = 0; c = 0.664 )</td>
</tr>
<tr>
<td>Zumrawi and Elnour (2016)</td>
<td>( a = 65; b = 1.5; c = 1 )</td>
</tr>
<tr>
<td>Zumrawi and Elnour (2016)</td>
<td>( a = 113; b = 12.5; c = 1 )</td>
</tr>
</tbody>
</table>

### Table 3. Constants for conversion of DCPI to CBR

<table>
<thead>
<tr>
<th>Author(s)</th>
<th>Constants</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kleyn (1975)</td>
<td>( a = 2.62; b = -1.27 )</td>
</tr>
<tr>
<td>Sampson (1984)</td>
<td>( a = 5.8; b = -0.95 )</td>
</tr>
<tr>
<td>Livneh (1989)</td>
<td>( a = 2.56; b = -1.16 )</td>
</tr>
<tr>
<td>Webster et al. (1992)</td>
<td>( a = 2.46; b = -1.12 )</td>
</tr>
<tr>
<td>TRL (1993)</td>
<td>( a = 2.48; b = -1.057 )</td>
</tr>
</tbody>
</table>
3.3 Remarks on the reviews equations

A close look at the equations enlisted above reveals that within a given category they are generally of the same form. This is interesting because then it means the framework of describing the relationship between the different sets of parameters is well established. What remains is probably an understanding of the factors that control the fitting constants. Until such is done, the practice of undertaking an extensive testing programme to establish the most suitable equations to use or to calibrate any preferred equations will continue.

In as much as it is not shown or demonstrated herein this paper, the other aspect that is associated with the equations presented in the tables is that they are generally further divisible into two categories, namely, those for coarse soils and the others for fine cohesive soils and that regardless of the category of the equation, the different equations have a wide range of the fitting parameters: \( a \), \( b \) and \( c \) as is applicable. The range is wider across category while it is narrower for a given category. Nonetheless, given that Equations 3 and 4 are of power and logarithmic form respectively any small changes in the fitting parameters cannot be taken for granted. Sensitivity analysis presented in Figures 1-3 clearly demonstrates the fact that changes within the range of fitting parameters can lead to substantial differences in the computed results.

It therefore means that the data analyst must be familiar with these aspects if any meaningful results are to be realised from the conversions. Where un-skilled inexperienced operator is deployed to con duct or oversee investigations, the level of understanding expected of them may not guarantee meaningful results. Thus, care should be taken in selecting a person to oversee the task of DCP testing and/or data analysis.

4 LIMITATIONS OF DCP TEST

Given its simplicity in carrying out, the DCP test appears to be an obvious test to many. Its execution is usually entrusted with unskilled inexperienced operators. This would not have been a problem if the test was free of any limitations. Unfortunately, research has shown that the reliability of DCP data is compromised by a number of factors. It becomes serious when even data analysis is left to such operators.

Whereas literature is rich with the equations from which one can choose to convert the DCP data into bearing capacity, directly or indirectly, there have been instances where unsuitable equations have been deployed. Having reviewed geotechnical investigations reports for various clients the author has observed that use of unmodified Terzaghi & Peck (1967) SPT data analysis equation to compute bearing capacity from DCP data is a common occurrence in Uganda. Although relationships between DCP data and the SPT N-value exist (Sowers & Hedges 1966, Lacroix & Horn 1973, Cearns & McKenzie 1988), direct use of those developed purely for SPT or indeed any other test without any calibrations is risky.

Furthermore, it is not uncommon for one to find reports in Uganda where DCP data has been used without taking into consideration the effect of mois-
ture. While this may be advantageous for wet condition in terms of increased safety due to the associated conservatism, it is disastrous when the tested ground constitute of especially very dry cohesive soils. Nevertheless, even where moisture condition has been accounted for it has been estimated by the operator with no supportive scientific basis. Whether or not unskilled inexperienced operator can properly judge the prevailing soil moisture condition relative to the optimum moisture content of the material without performing the relevant laboratory tests is debatable. In consideration of the fact that DCP penetration rate is very sensitive to moisture changes (Hassan 1996, Kleyn et al. 1982) and that the effect of the latter on the former depends on whether the moisture regime is dry or wet of optimum moisture content, one should be extremely careful to reasonably estimate the moisture condition of the material at the time of testing. This can only be guaranteed by accompanying the DCP data with moisture content dry density relationship tests. In lieu of building footings where compaction is not performed, saturation moisture content appears to be a suitable reference.

In addition to the material property factor of moisture condition, particle size and distribution has also been observed to limit the applicability and reliability of DCP test (Hassan 1996, Kleyn et al. 1982, Amini 2003). It is understood that the test works best in isotropic fine soils. As particle size increases its efficiency reduces; the test becomes inapplicable when significant proportion of soil constitute of particle size in excess of 50mm. The dilemma with this is that the subsurface material being penetrated is not visible to the operator. Fortunately, in materials larger than the established limit of 50mm, the hammer will generally bounce on the anvil and so this can easily be used as the basis for judgment provided particle size distribution is more or less uniform and that the particles do not crush with ease. Otherwise it becomes necessary to compliment the DCP test data with gradation results together with qualification of what significant proportion of soil particles with size greater than 50mm means.

Friction is the other factor. It reflects verticality of the device rod (George & Uddin 2000, Amini 2003) and is probably the least important as anyone may easily gauge the extent to which the rod is bent during the test especially when it is tending to the established limit of 75mm deviation from the vertical of top part of the rod. According to Livneh (2000) a correction factor for side friction is necessary if DCP test results are to be meaningful. Later, Paige-Green & Du Plessis (2009) observed that friction effect considerations are only needed when a DCP test is continuously extended passed 1.0m. Apparently, unlike with road works related investigations the scope for building footings tends to exceed the 1.0m depth threshold implying that for all assessments made in respect of building footings design, friction is not negligible.

Unfortunately, at the moment there seems to be no clear and reliable way of accounting for the friction effect. Retraction and re-drive of the DCP rod through a distance of about 300 mm has been proposed to measure friction (Paige-Green & Du Plessis 2009). The validity of this approach requires that the cone remains at the farthest point reached; a condition possibly achievable with only disposable cones.

Lastly but not the least, the other source of error has been failure to incorporate the effect of geometric characteristics of the ground where the footing is to be constructed and those of the footing itself plus the nature of the load. It is now common knowledge that the capacity of the ground to support footings is also influenced by non-material factors such as ground slope, footing base slope, footing depth, footing shape and load inclination among others. However, since DCP is more or less a vertical point test in comparison to the real footing size, it should be free of the effect of the aforementioned factors. The challenge then would be to have the factors individually accounted for in the developed correlations.

5 ADEQUANCY OF DCP TEST DATA IN SHALLOW FOUNDATION DESIGN

The background, principle, benefits and limitations of the DCP test have been highlighted in the preceding parts of this paper. The question now is to do with whether or not the test data can be relied on to design shallow footings for buildings. This section addresses this question by way of comparing cases where the test has been used together with other more reliable and established methods. Two cases are compared. The results are presented in Figure 4 whereof the DCP based bearing capacity is plotted against that obtained using other established methods. The cases are denoted case one (C1) located in Kisasi and case two (C2) located in Lugala. These places are both within Kampala and for each of them trial pits were dug and both disturbed and undisturbed samples taken and tested.

Each of the cases compared in Figure 4 had bearing capacity computed using DCP test data analysed through the indicated equation or set of equations in the category of those earlier presented (see Tables 1, 2 and 3) as well as direct shear box test data in conjunction with Terzaghi’s equation for shallow footings. In addition, and for demonstration purposes, Terzaghi & Peck (1967) SPT test analysis equation was also deployed. Results are plotted on the same Figure 4 for ease of comparison.
It should be noted that whereas it was easy to observe the necessary procedures to minimise the impact of the various DCP test limitations earlier presented it was not possible to match the foundation dimensions with those assumed in the various equations developed in terms of allowable bearing capacity. For this reason, only the ultimate bearing capacity equations were considered appropriate. Therefore, the computed bearing capacities in Figure 4 are ultimate. From the figure, a very big scatter is noted. The scatter notwithstanding, Black (1961) equation was found to perform better than all the others. The soils involved were predominantly fine grained lateritic soil. Their grading curves are presented in Figure 5.

In order to also check how the values computed using equations that give safe bearing capacity compares with the results of Terzaghi’s shallow footing equation and for completeness purposes, it was deemed necessary to assume a factor of safety of 3.0 on the latter in as much as the factor applied on the former is unknown to the author. Results are presented in Figure 6. Comparing Figure 5 to Figure 6, it is observable that equations that give safe bearing capacity perform better than those that give ultimate bearing capacity and that such a relatively good performance disappears at higher values of bearing capacity computed from Terzaghi’s equation.

6 CONCLUSIONS

The objective of this paper was to assess the reliability of using DCP test as standalone method for the assessment of soil strength for building foundation design. Bearing capacity obtained from DCP data was compared with that determined using Terzaghi’s shallow footing equation on direct shear box test results. The major conclusions drawn from the study are presented below:

As of present, whereas there are numerous equations relating DCP penetration index to bearing capacity most of them cannot give reliable design data for shallow foundations without prior calibration. Any deviations from the footing width used in the development of the equations will result in significant departure from the expected bearing capacity.

Equations giving safe bearing capacity were better than the ones for ultimate bearing capacity within the range of about 500kPa. Thereafter, the reverse was true. Therefore, unless one is aware of the assumptions made in the development of a given equation, they should adopt equations depending on the expected range of bearing capacity values. Such assumptions might be the factor of safety and the width of footing used.

7 RECOMMENDATIONS

This paper is about a proposal on how to go about obtaining reliable building foundation design using DCP test data. Literature and experience have revealed that several theories are being adopted in strength characterisation of in situ soils for building foundation design. The analysis herein shows that not all the theories have the same level of reliability. Based on the findings of this study therefore, stakeholders are advised to be mindful of the theories they
adopt as dangerously underestimation is likely. Among the indirect equations, the one by Black (1961) gave better estimation of ultimate bearing capacity while the one by Dzitse-Awuku (2008) yielded better estimations of safe bearing capacity.

The relative importance of the structure under consideration notwithstanding, use of correlations not developed and validated for the version of the DCP device adopted and material being tested should be avoided.

The several limitations of the DCP test require that additional relevant routine tests are carried out on samples taken from the penetrated ground to supplement the DCP test and/or to confirm applicability of the test. Such tests can be undertaken on samples recovered from trial pits. For as long as the footing width does not exceed 2.5 m trial pitting should be adequate as the incremental stress due to the foundation load are limited to only about 1.5 times the footing width below the footing depth. Therefore, where excavations can be safely advanced to 5.0 m (e.g. in lateritic soils) there should not be any concerns.

In terms of further studies, research to further improve the reliability of DCP based bearing capacity is necessary. It appears that for as long as the fitting parameters for the correlations are not related to the influential material properties, the reliability and adequacy of the DCP data for the design of shallow building foundations will remain lacking.

8 REFERENCES


Dzitse-Awuku, D. 2008. Correlation Between Dynamic Cone Penetrometer (n-value) and Allowable Bearing Pressure of Shallow Foundation Using Model Footing. MPhil Thesis: Kwame Nkrumah University of Science and Technology, Kumasi.


Webster, S.L. Grau, R.H. & Williams, T.P. 1992. Description and Application of Dual Mass Dynamic Cone Penetrometer. Army Engineer Waterways Experiment StationVicksburg MS Geotechnical Laboratory.
