THE HUMBLE DCP – STAY HUMBLE

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ABSTRACT

The Dynamic Cone Penetrometer (DCP) was developed in Australia in the mid-1950s. At this time, pavement design procedures were created, based on correlations between DCP and California Bearing Ratio (CBR). Fast forward 60 years, and you will see designers and contractors blindly specifying and undertaking testing, then using DCP values to assess CBR, bearing capacity, pile capacity and soil strength.

This paper discusses the limitations of this tool, which will assist designers in selecting when and where DCP testing may be appropriate. A recent case study is presented which demonstrates the limitations and also encourages effective use of the DCP alongside other site investigation techniques. With an appropriate understanding of its place, the humble DCP can remain a time and cost effective option within the suite of tools available to engineers.

1 INTRODUCTION

A quick online search using “DCP” and “correlation” as key words results in multiple hits, leading the unsuspecting engineer to believe that this magical tool can give them all the answers they need. This paper seeks to remind users of the initial purpose and background to this testing apparatus. Extreme caution is recommended when using the multitude of published correlations available. These should not be taken at face value, but require a proper understanding of the apparatus used, testing conditions and data limitations. A number of correlations and equations are presented in this paper, not as reference for use, but rather to highlight the range of information available, of which the engineer must use caution to filter through and select if a method is applicable for their condition.

2 ORIGINS OF THE DCP

2.1 THE NEED FOR IN SITU TESTING

In his 1956 paper, titled “Simple Methods of Flexible Pavement Design”, Scala (1956) recognised a major challenge in road design was the lack of a quick and simple test for determining the subgrade strength. A review of current practice at the time showed that no road authority within Australia used any form of strength test for subgrades, compared to approximately half of the states in the USA. With a focus on pavements, specifically for roads, Scala (1958) reviewed the various methods of pavement design at the time. A summary of these methods is presented in Table 1.

Table 1: Methods of Pavement Design (adapted from Scala, 1956)

Category	Method	Details
Empirical, without the use of soil strength tests	Group Index Method	Uses Atterberg Limits tests to obtain the ‘group index’. Curves relate group index to pavement thickness
	Civil Aeronautic Administration	Similar to Group Index but with more complicated tests (no strength test)
	Country Roads Board, Victoria	Estimation of soaked CBR from simple soil tests
	Department of Main Roads, NSW	Estimation of pavement thickness based on simple soil tests
	State Road Commission of West Virginia	Estimation of the minimum CBR from various soil tests
Empirical, with the use of soil strength tests	California Bearing Ratio (CBR)	Developed by the Californian Division of Highways and adopted by the US Corps of Engineers. Testing can be either in the laboratory or *in situ*.

	North Dakota Cone	Loading a cone with standard weights and noting the penetration in situ. Only a short cone, therefore to test a different layer or depth, the upper material must be removed
	Housel Penetrometer	1¼ inch pipe, driven by 20 lb hammer, falling 34 inches. Count number of blows required to penetrate 6 inches. Early testing showed a fair correlation with the CBR test.
Partly theoretical and partly by experience	Unconfined Compression Test	Testing developed for saturated clays, in the field
	Vane	Measurement of *in situ* shear strength. Quick and easy to use. Difficult to penetrate the vane when CBR > 4%
	Triaxial Test	Fundamental test in soil mechanics but lengthy to prepare and test samples
	Plate Bearing Test	Loading of plates and measuring deflection.

It was obvious that the empirical methods which did not consider soil strength, made no allowance for the condition of the subgrade *in situ*. The design methods which did test the soil *in situ* were considered to be too costly, time consuming and/or labour intensive to be part of routine road pavement design.

2.2 THE DYNAMIC CONE PENETROMETER

The invention of the dynamic cone is attributed to Scala in 1956. In the late 1960's D.J. Van Vuuren continued to develop the DCP in South Africa and further modifications were made to the apparatus in Israel by Livneh and Ishai in 1985 (Luo et al., 1998). The original Dynamic Cone is compared to the current DCP as per AS1289.6.3.2 (1997) in Table 2.

Table 2: Comparison of DCP apparatus

	Scala (1956)		D.J. Van Vuuren (1960s)	AS1289.6.3.2 (1997)
	Imperial Units	Metric Units	Metric Units	Metric Units
Hammer weight	20 lb	9.08 kg	10 kg	9 kg
Fall distance	20 inches	508 mm	460 mm	510mm
Angle of cone	30°	30°	30°	30°

3 USE OF THE DCP

Due to the 'simple, quick and cheap' testing available through the DCP, its application as a site investigation tool has become widespread.

In addition to the initial correlations with CBR, other correlations exist in the literature to convert DCP results to bearing capacity, SPT N-value, unconfined compressive strength (UCS), shear strength in granular materials as well as the resilient modulus of subgrade soils. Two such correlations are presented in the following sections, though the engineer must use caution to filter through and select if a method is applicable for their condition.

3.1 CORRELATIONS TO CBR

In Scala's original paper, the theoretical dynamic resistance to penetration (Rd) was calculated by Equation 1 using energy and work done principles. This dynamic resistance to penetration was then empirically correlated to CBR using data from over 70 tests, on a wide range of materials, arriving at the relationship presented in equation 2. The resulting CBR value could then be used to determine pavement thickness, by the established methods.

$$Rd = \left(\frac{1}{A}\right)\left[\left(\frac{W^2 h}{(W+w)e}\right) + (W+w)\right] \tag{1}$$

Where:

Rd = dynamic resistance to penetration

W = weight of hammer (in pounds)

h = height of fall of hammer (in inches)

w = weight of rods and cone (in pounds)

e = penetration per blow (inches)

A = area of section of cone (inches)

$$CBR = Rd/64 \tag{2}$$

(Scala, 1956)

Many more studies have been carried out to correlate DCP to CBR in various locations and ground conditions. Some of the more accepted correlations are shown in Equations 3 to 5.

$$\log CBR = 4.66 - 1.32(\log PI) \tag{3}$$

$$CBR = 405.3(PI)^{-1.259} \tag{4}$$

$$\log CBR = 2.0 - 1.3\log(PI - 0.62) \tag{5}$$

Where:

PI = penetration index (mm/blow)

(Livneh, 1989)

Variability of the testing apparatus and ground conditions must be understood, before using a specific formula, developed from a specific dataset.

Scala (1956) states "It is not the intention that these (DCP tests) should displace the California bearing ratio test, but in view of the difficulties associated with the latter test… it is essential that some simple, quick and cheap method must arise". The vision going forward was that "dynamic cones will be performed each 100ft along the road. (Then) the static cone, which is recognised as being the more accurate test, shall then be put down at certain specific sites, selected from the dynamic cone penetration results." This implies that the device is more suitable as a profiling tool, rather than being an accurate way to measure strength.

3.2 BEARING CAPACITY

Stockwell (1977) proposed a method for determining the allowable design bearing pressure under a *small* structure, by way of a graph relating allowable bearing pressure (kPa) to penetration with a Scala Penetrometer (mm/blow), presented in Figure 1. A small structure was defined as a one or two storey building. There are many modification factors which must then be applied, depending on the depth and width of strip footings or for saturated soil conditions in sand. It is important to note that the factor of safety for bearing failure ranges between 1.6 and 5.8 when using this graph (Stockwell, 1977).

An investigation by Ampadu (2006) for a local soil in Ghana concluded that the correlation predicts higher allowable bearing pressure when compared with the results measured in the field and derived from the literature.

Mogotsi and Merwe (2017) recently addressed the relationship between bearing capacity and DCP. They recognised that the DCP (as developed in South Africa in 1975) was originally intended to be used for pavement applications as an indicator test. A review of South African literature uncovered an estimate for bearing capacity as shown in Equation 3.

$$Bearing\ Capacity = 3426\ DN^{-1.014} \tag{6}$$

Where DN = Penetration (mm/blow)

It is unclear whether this is ultimate or allowable bearing capacity.

The limitations associated with DCP testing to predict bearing capacity were noted to include:

- Depth of testing relative to the size of the footing (considering a 2B influence zone)
- Effect of water table within sand material
- Influence of sloping ground or proximity to a slope

(Mogotsi and van der Merwe, 2017).

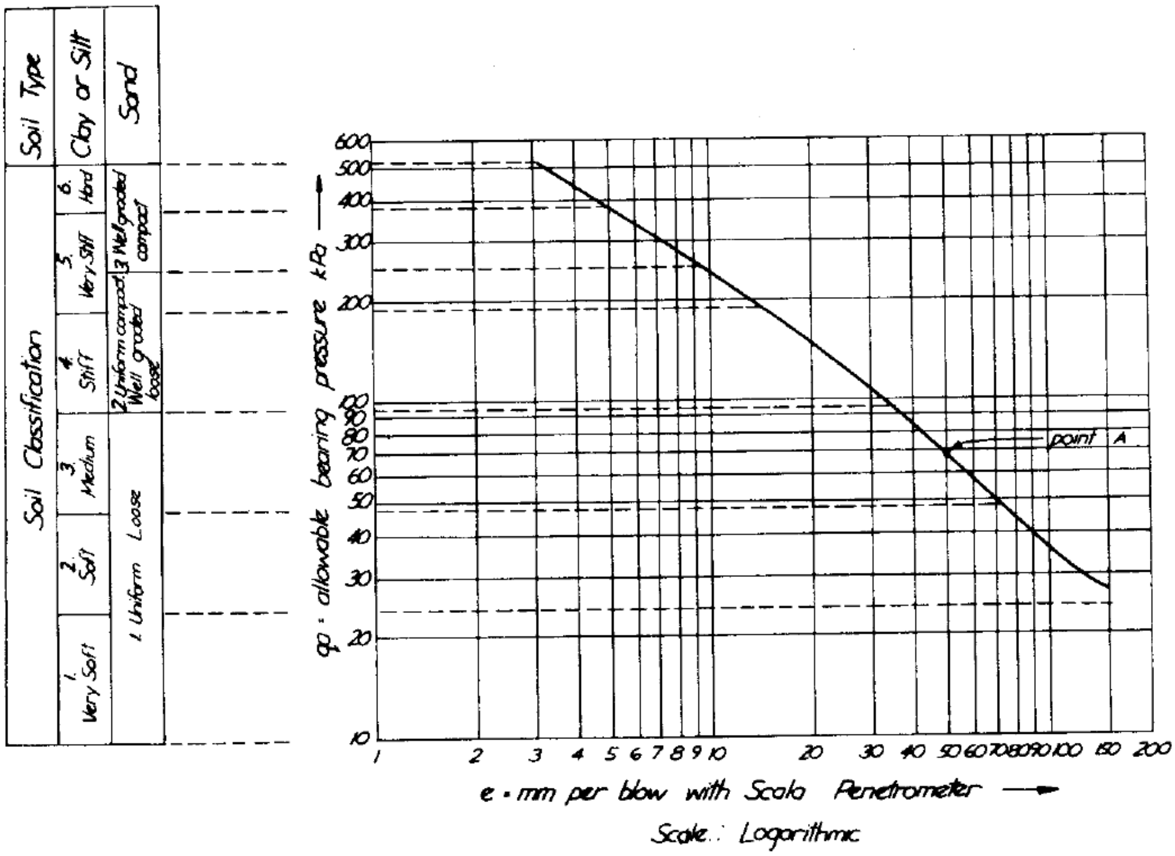

Figure 1: Relationship between DCP and Allowable Bearing Capacity (Stockwell, 1977)

Another website presents the relationship in the form of a table comparing DCP (blows per 100mm) to indicative allowable bearing pressure (www.samw.org/e/DCP-Stockwell), 2018). The published table values are plotted in Figure 2. A key warning is also given with the data, stating that *“DCP test results are to be used as a guide only to relative density and consistency of solid. Changes in moisture contents or the presence of coarse grained material can greatly influence the outcome of this test.”* The author’s concern is that these warnings and disclaimers are often overlooked in the search for a simple answer.

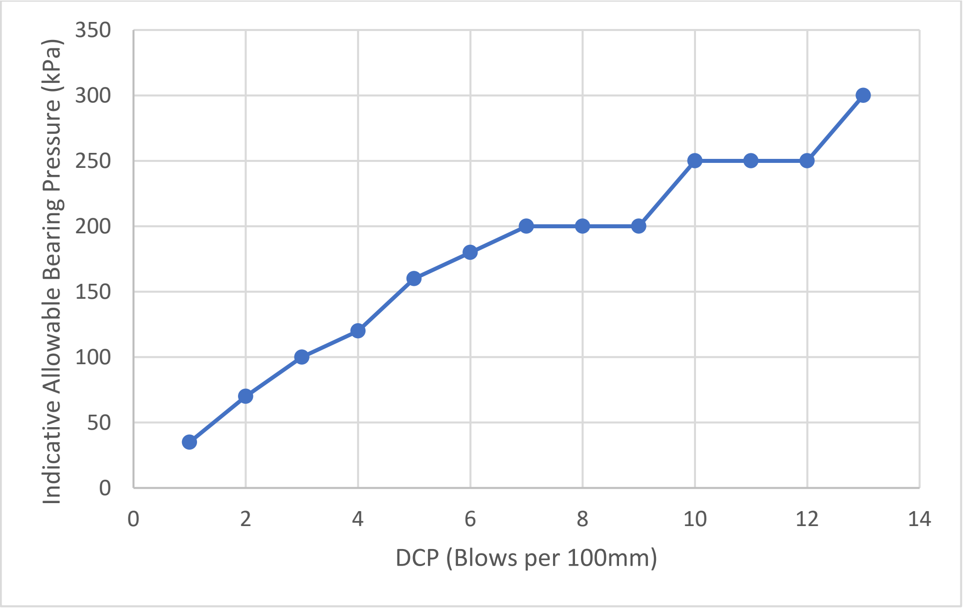

Figure 2: Relationship between DCP and Indicative Allowable Bearing Pressure (www.samw.org, 2018)

According to Sanglerat (1972) the DCP vs bearing capacity correlation is reliable in granular materials. However, for cohesive soils, particularly testing below the water table, the rod friction and effects of pore water pressure are both major components of the resistance, and therefore the correlation is questionable.

4 CASE STUDY

The testing and observations presented in Section 3 serve to show that a DCP value should not be blindly correlated to a design value, based on a published chart, table or equation. The potential issues associated with this approach are demonstrated in the following case study.

The designer's requirements at the underside of a 2100mm wide reinforced concrete box culvert called for an allowable bearing capacity of 150 kPa at the founding level. This was to be confirmed by DCP values ≥ 4 blows/100mm. If this was not achieved, the founding level was to be inspected by the geotechnical engineer, to advise suitable treatment.

The excavation in clay was opened to a depth of 0.7m below founding level and DCP tests were undertaken by the contractor and provided to the Engineer. The results are shown in Table 3.

Table 3: DCP Results (blows per 100mm)

	Depth (m)*	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**
DCP (blows/100mm)	Northern End	1	1	1	1	2	1	2	6	6	7
	Centre	1	1	1	1	2	2	3	4	5	6
	Southern End	1	1	1	1	2	2	3	5	6	7

*Depth is measured below excavation level (note that excavation level was 0.7m below the founding level).

According to one source of published data (refer to Figure 2), to achieve an allowable bearing capacity of 150 kPa, a DCP value ≥ 5 blows/100mm would be required. Relying solely on this correlation would require a further 0.8 to 0.9m of material to be removed, resulting in a total depth of 1.5 to 1.6m. This has significant cost implications with regards to additional excavation and imported material. The stability of the excavation at this depth would also need to be considered and additional works, such as batters or benching, would likely be required to allow workers into the excavation. Near surface material was observed to have been softened by ponding water.

The geotechnical engineer was able to consider the actual site conditions and recognise that at a depth of 1.05m (0.5 x footing width) below the founding level, the requirement for bearing capacity would be half that initially specified due to load spread. Therefore, a revised target for allowable bearing capacity ≥ 75 kPa was set for this depth. Vane shear testing was undertaken at two points across the base, each at two depths. The results are presented in Table 4.

Table 4: Allowable Bearing Capacity results from vane shear testing

Location	**Depth below excavation level* (m)**	**Allowable bearing capacity**
1	0.15	> 75 kPa
	0.40	> 75 kPa
2	0.15	> 75 kPa
	0.40	> 150 kPa

*Note that excavation level was 0.7m below the founding level

Based on this testing, the instruction was to remove 0.4m of material, to be replaced with rockfill, wrapped in a separator geotextile (i.e. 1.1m below founding level). This depth of replacement was supported by the *relative* values of the DCP test (1 blow/100mm down to 0.4m), but is a significant improvement on the treatment that would have been required if DCP tests were solely relied upon for strength values.

For this specific site, the control of water was a key factor in achieving the bearing capacity requirements. An adjacent section of the foundation was opened up to the replacement depth (1.1m) the following day and a further inspection was undertaken. This portion of the foundation was found to be covered in water, with the top 200mm softened due to the ponding water. Therefore, the removal of this softened material was required and appropriate groundwater control measures were needed to ensure that the bearing capacity requirements were met. This highlights the importance of testing and assessing the ground conditions at the time of construction, and the significant influence that ponded water can have on near-surface strength.

5 CONCLUSION

The intention of this paper was NOT to present a myriad of ‘useful’ correlations between DCP and other material parameters, but rather to highlight the pitfalls of adopting published correlations without a proper understanding of the correlation background. A couple of key quotes from the literature illustrate this point.

“Although good correlations have been obtained, all studies have found that the results are material and moisture dependent, and that equations should be used with care and only with a full understanding of the material properties of the soils on which the equation was developed and the soil being tested.” (Jones and Harvey, 2006)

“The greatest strength of the DCP device lies in its ability to provide quickly and simply a continuous profile of *relative* soil strength with depth.” (Luo et al., 1998).

“Only *relative* information regarding classification and stratification should be inferred from the penetration rate of the DCP.” (Sanglerat, 1972).

It is the authors opinion that the DCP is a useful tool for characterisation of relative strength or density with depth, which can be useful in highlighting the variability of the profile and provide a guide to locations for future testing by other methods. It should not be viewed as the solution to all *in situ* strength testing problems.

6 REFERENCES

Ampadu, S.I.K. (2006). A correlation between the dynamic cone penetrometer and bearing capacity of a local soil formation, *Proceeding of the 16th International Conference on Soil Mechanics and Geotechnical Engineering,* PLACE, 655-658.

DCP vs Allowable Bearing Capacity. (2018). [online] www.samw.org/e/DCP-Stockwell, accessed 2018.

Jones, D. and Harvey, J. (2006). Relationship between DCP, stiffness, shear strength and R-value. University of California, Berkeley

Livneh, M. (1989). Validation of correlations between a number of penetration tests and in situ California bearing ratio tests, Transportation Research Record 1219, 56-67.

Luo, X., Salgado, R., Altschaeffl, A. (1998). Dynamic Cone Penetration test to assess the mechanical properties of the subgrade soil. Indiana Department of Transport, West Lafayette.

Mogotsi, K.A. and van der Merwe, F.H. (2017) Can one use the dynamic cone penetrometer to predict the allowable bearing pressure? *9th South African Young Geotechnical Engineers Conference,* Durban. 271-280.

Sanglerat, G. (1972). The Penetrometer and Soil Exploration. Elsevier.

Scala, A.J. (1956). Simple methods of flexible pavement design. *2nd Australian and New Zealand Conference on Soil Mechanics and Foundations,* Christchurch, 34-44.

Standards Australia (1997). AS1289.6.3.2: Methods of testing soils for engineering purposes: Soil strength and consolidation tests – Determination of the penetration resistance of a soil – 9kg dynamic cone penetrometer test. SAI Global, Sydney.

Stockwell, M.J. (1977). Determination of allowable bearing pressure under small structures. *New Zealand Engineering.* Vol 32, 6. 132-135.