SESOC 2021 CONFERENCE
A New Beginning
CLAUDELANDS, HAMILTON | 5 - 6 JULY 2021

# UPDATING VERIFICATION METHOD B1/VM4 – SECTION 3.0 – SHALLOW FOUNDATIONS

**Nick Clendon**[1]

## SUMMARY

The current Verification Method B1/VM4 for Shallow Foundations currently does not provide any guidance on the potential settlement of foundations, although it has always been intended that this would eventually be added. The building code itself, requires settlement to be addressed but does not provide any guidance. Appendix B (B1/VM4) does specifically state acceptable settlement limits for serviceability limit state, but as yet does not provide any guidance on its assessment. In addition, since B1/VM4 was last amended (2008), there has been significant changes within the building industry due to recent earthquake events. New 'best practice' guidelines developed under the New Zealand Geotechnical Society (NZGS) provides additional information on Ultimate Bearing Capacity (UBC) calculations and covers aspects of foundation performance relating to settlement. Lastly, NZS3604:1999 – Timber Framed Buildings, provides a definition of 'good ground' suitable for foundations for residential buildings which will allow it to meet the expectations of the building code. But it deals with settlement by having a sufficiently factored UBC as to ensure excessive deformation does not occur. This is mostly achieved through the use a dynamic cone penetrometer test (Scala).

Bearing capacity failure is extremely rare, however, settlement, ranging from minor aesthetics related damage through to significant movement occurs on a more frequent basis. This paper presents recommendations for B1/VM4 which captures the recent guidelines, updates the nomenclature, and identifies how the definition of 'good ground' from NZS3604, actually fits within a regular UBC calculation.

## INTRODUCTION

This paper is split in to three discreet sections, the first looks at Ultimate Bearing Capacity and the relationship with 'Good Ground' and testing criteria, the second looks at soil moisture deficient and the impact this may have on insitu testing. Thirdly, a simplified method for settlement assessment calculations is presented.

## BEARING CAPACITY (ULTIMATE)

Ultimate Bearing Capacity is the point at which the foundation pressure causes the ground to rupture. Prior to rupture, the ground would have already experienced significant settlement. Foundation settlement would have already occurred prior to the ultimate bearing capacity and can be in the order of 15-30% of foundation width (*The Seismic Assessment of Existing Buildings - Part C4 - Geotechnical Considerations*, 2017). So, for a 0.5m wide foundation, this could be 75-150mm.

[1] 1stAuthor, Coffey Services NZ Ltd Email: nick.clendon@coffey.com

B1/VM4 terms this as Ultimate Bearing Strength, NZS3604, refers to 'Good Ground', with an Ultimate Bearing Capacity of 300kPa, however, the testing required, in most cases does not relate to Ultimate bearing capacity but to a limited bearing capacity to reduce the risk of excessive settlement, without burdening the designer with the need to make a settlement check. The more recent New Zealand Geotechnical Society Modules (MBIE & NZGS, 2016) also refers to Ultimate Bearing Capacity, and provides a more detailed explanation of the risk associated with using higher reduction values such as 0.8, but refrains from discussing over strength checks as detailed in B1/VM4. For the purposes of this paper, the following terminology is utilised and recommended to minimise confusion:

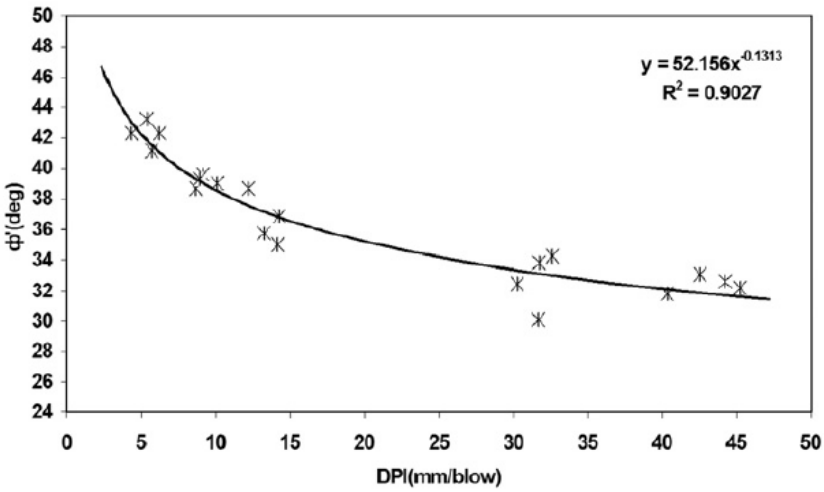
Ultimate Bearing Capacity (UBC) – the point at which excessive movement has occurred and no additional bearing pressure can be carried by the foundation system (similar to NZGS and B1/VM4)

Limiting Bearing Capcity – A reduced bearing capacity based on pre-selected testing values to achieve a conservative design that mitigates the need for design checks of settlement (such as used in NZS3604).

If you look at the definition of 'good ground' and the current acceptance criteria of 5 blows per 100mm with a Dynamic Cone Penetrometer (DCP also known as scala), the following UBC could be calculated in Coarse grained soils.

Based on B1/VM4 for a 0.5m wide strip footing the UBC would equal = 450kPa. The original DCP criteria of 3 blows per 100mm equals = 300kPa. Figure 1 below provides an example of the relationship between DCP count and soil friction angle.

Figure 1. mm/blow versus friction angle for coarse soil (Mohammadi et al., 2008)




The increase from 3 blows to 5 blows per 100mm has reduce the UBC by a third. The increase in the blow count has been associated with reducing settlement, either static or liquefaction induced settlement. However, does this reduce the risk of static settlement related issues and does it adequately provide for fine grained soils?

<u>DCP for Fine Grained Soils (Cohesive Soils or soils with clay like behaviour)</u>

Relationships between DCP blow count (or DPI – mm per blow) are less common, however, there are some papers which cover this topic in details, and Figure 2 below is the example of one such relationship. However, caution needs to be taken as Figure 3 below shows the impact of relatively small variations in soil moisture conditions can have a substantial impact on the site data collected and subsequent interpretations of soil strength.

Figure 2: Correlation between Undrained Shear strength (Cu) and DPI (mm per blow)

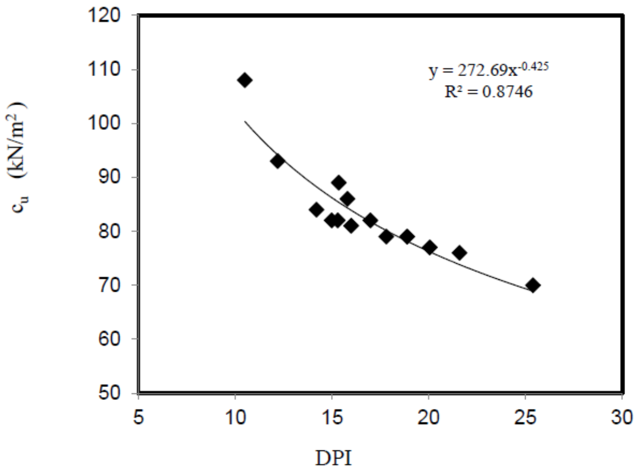
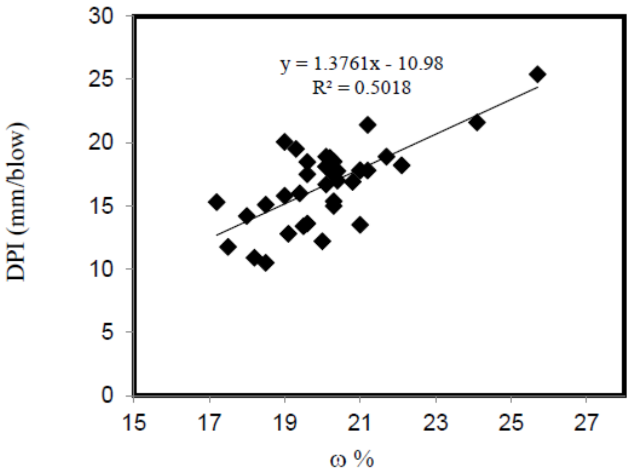



Figure 3: Moisture Content vs DPI




Based on the above two figures, assuming 5 blows/100mm and B1/VM4 for a 0.5m wide strip footing the UBC would equal = 520kPa. The original DCP criteria of 3 blows per 100mm equals = 350-400kPa.

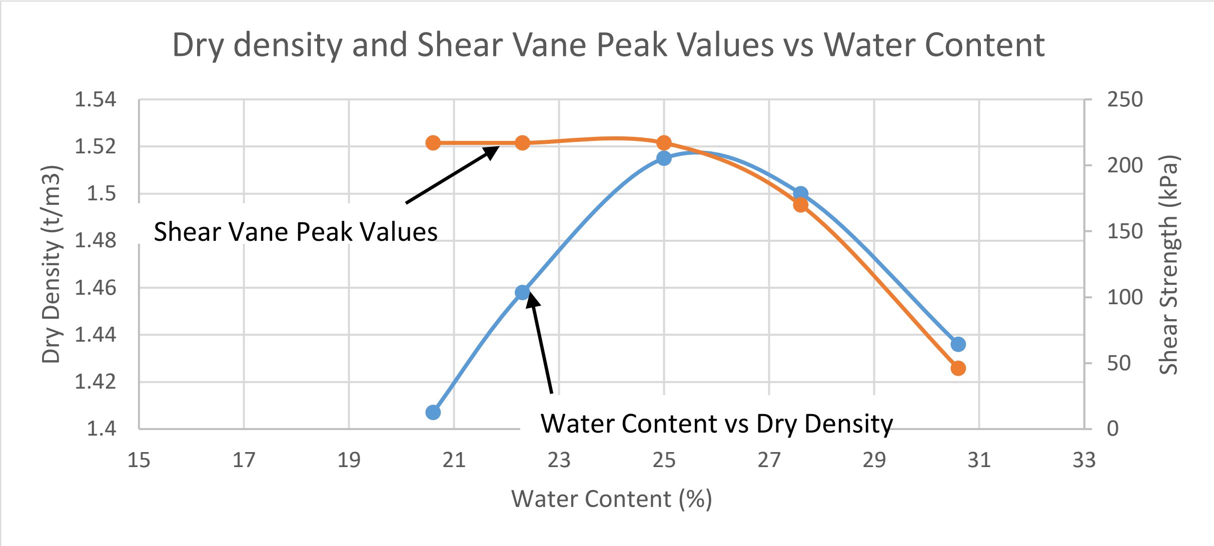
<u>Moisture Content vs Shear Vane</u>

Moisture content has a significant impact on the shear strength of soils for both re-compacted (engineered fill) or natural material.

For natural soils there is a strong relationship between undrained strength and liquidity index (*PN12*, 2009, p. 12). The relationship arises from the coincidence that the undrained strength of soil at its liquid limit is about 1.7kpa, and at its plastic limit it is about 170kPa. This is commonly seen in both fill soils and natural soils. The figure below is taken from a site in Wellington for an alluvial soil, - SILT, some clay moderately plastic.

One of the keys points to note is that the shear strength has a marked drop as moisture content increase but does not necessarily reduce as the soil becomes drier but dry density does. The consequence of this can be that soils are susceptible to wetting at a later date resulting in settlement as the soil softens.

Figure 4: Compaction curve with Shear Vanes for Fine Grained Soil




## SOIL MOISTURE DEFICIENT

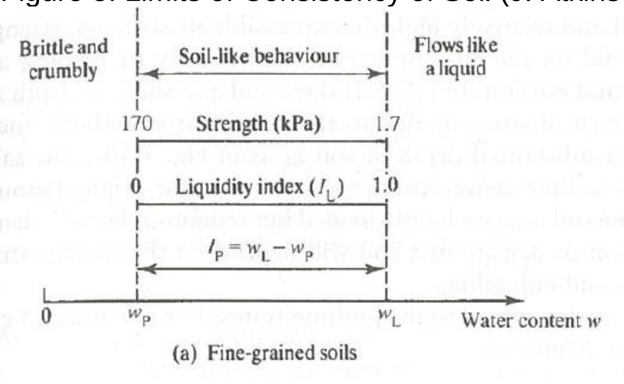
If the soil moisture has such a substantially impact, how can we consider the data collected in design to ensure we have selected appropriate test values and sufficiently allowed for moisture variations.

1. Soil logging – clear description of layers and soil behaviour. Moisture condition. Does moisture need to be added to roll a good thread. If the soil is dry of the plastic limit it can be brittle and crumbly, and may absorb moisture in the future. Soils with too much moisture are more easily identified by their low shear strength.
2. Lab testing – Atterberg testing, moisture content, fines content
3. Soil moisture deficient maps – understanding of recent conditions, timing of investigation and sample collection.

<u>Soil Logging</u>

For soil logging, reference should be made to New Zealand Geotechnical Society guidelines (NZGS, 2005). Also an understanding of the impact moisture content can have is represent in Figure 55 below.

Figure 5: Limits of Consistency of Soil (J. Atkinson, 2007)




<u>Lab testing</u>

Lab testing is beyond the scope of this paper, however, simple classification testing should be done whenever possible to allow for cross checking of the soil logging. There should be consistency of description between soil logs and the classification test results. In addition, there are numerous correlations between classification tests and soil properties, such as friction angle or soil strength, which can provide a second check on the strength properties selected.

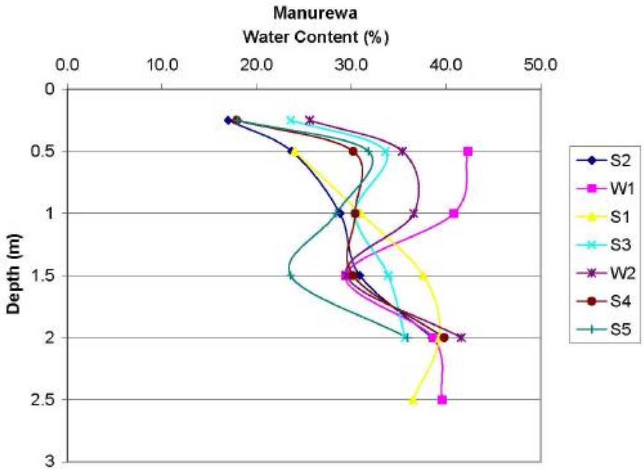
Soil Moisture Deficient

The major factor affecting varying insitu soil moisture is climate, which will affect the level of soil suction and the base of the zone of season moisture variation which can in turn lead to ground movement.

Soil Moisture Deficient maps are produced by NIWA and provide a good overview of the current and relative condition of the soil. Typically for your local jobs, the local engineer will have a good awareness of recent ground conditions, and will unlikely need this additional information, but when working outside your normal region, these can provide a useful guide. During very dry conditions, it can be very difficult to penetrate the upper soil lending to miss-leading data collection.

Typically, in New Zealand, due to a fairly temperate climate the base of the soil moisture zone would be within 1-2m. however, many intrusive investigations using DCP and shear vanes often don't penetrate far past 2m, and with the upper 2m often being critical to the design of shallow foundations. Care should be taken to assess the soil moisture condition at the time of the investigation. As can be seen in Figure 6 below, soil moisture can vary 5-20% seasonally.

Figure 6: Seasonal moisture change (Brown et al., 2008)




Summary of DCP, Shear Vane and Moisture Content

Shallow site test data forms the backbone of shallow foundation assessments for bearing capacity. If the quality and variability of the site data is unknow then the suitability of the data for calculating bearing capacity is unknown. With reference to Figure 6 and Figure 3 it can be seen that 10% change in moisture content can produces a reduction in DPI from 10mm/blow to 25mm/blow (from 10 blows per 100mm to 4 blows to 100mm). Which can change the site from being good ground to not such good ground.

The current definition of 'Good Ground' and the testing requirements, relies on its ability to sufficiently 'Limit' bearing capacity such as to ensure settlement is acceptable. Reducing bearing capacity, can increase footing dimensions and lead to a greater depth of influence.

**SIMPLIFIED SETTLEMENT CALCULATIONS**

Acceptable Settlement

There is limited guidance on acceptable settlement limits within the building code with the exception of Appendix B in B1/VM4, where foundation design should limit the probable maximum differential settlement to 25mm over a distance of 6m for the serviceability limit state. This relates to a slope of 1:240.

In traditional timber framed structures, many buildings could accommodate this movement, however, much of the modern building practice is moving to concrete floor slabs and more brittle claddings. In addition, significantly more effort is going into the insulation of homes, to reduce heat loss and improve heating. This means minimising gaps, leaks and pathways for air to enter or leave structures. Having poorly fitting door or windows is generally considered less acceptable.

It is recommended that the 1:240 tolerance for serviceability limit state is reviewed, with modern building practices in mind. A criterion of 1:500 may be considered more acceptable and is often achieved in the majority of developments already.

However, for the purposes of this paper, we have completed calculations in line with the 25mm for consistency with existing documentation. It should be noted, that for any settlement assessment, a range of 0.5-2.0 is always recommended on the values calculated, unless a more detailed assessment of the upper and lower bounds of the soil properties is to be completed. As such this relates to an acceptable calculated settlement of approx. 16mm at any location 16mm x 2 (upper bound) – 16mm x 0.5 (lower bound) = 24mm differential settlement

Simplified Settlement Assessment

There are numerous ways to calculate settlement, with most being specific to a certain soil type, sand, clay, silt or to the investigation method, SPT, CPT etc. The purpose of this guidance is not to technically advanced settlement assessment techniques, but a simplified approach, to allow checking of the potential for excessive settlement, in cases which may not be well captured by a bearing capacity assessment alone.

The current bearing capcity approach as outlined above, deals with settlement by having sufficient reduction factors on the bearing capacity as to create a sufficient separation between large displacements associated with bearing failure and acceptable serviceability settlement.

This approach deals with the simplified problem by utilising the following soil properties and considering the following design case:

Long term drained Young's Modulus – $E_s'$

The process following that outline by Atkinson (J. H. Atkinson, 2000) and utilises the equation (1) below.

$$\frac{\Delta p}{B} = \Delta\sigma \frac{(1-\nu^2)}{E_s} I_g \qquad \text{Equation 1}$$

Δp is the change is settlement due to the change in bearing pressure (Δσ), $E_s$ is the secant Young's Modulus (corresponding to the increment of loading, $I_g$ is an influence factor dependent of the foundation geometry. B is the foundation width and v is poisons ratio.

The reason for choosing elastic solutions for the approach is in their simplicity. There are limited inputs, and those required are relatively simple to establish. The four key inputs are

Young's Modulus, Poisson's Ratio, Foundation Size, Bearing Pressure. With the key soil input being Young's Modulus ($E_s$) which is not overly sensitive to small changes in soil properties.

Recommended values for $E_s'$ and v are tabulated below.

Table 1: Recommended Drained Young's Modulus Values

| Consistency | γ (kN/m$^3$) | Friction Angle (°) | Cu (kPa | $E_s'$ (MPa) | v' |
|---|---|---|---|---|---|
| Very Dense | 22 | 40 | | 100 | 0.2 |
| Dense to Very Dense | 21 | 38 | | 80 | 0.2 |
| Dense | 20 | 36 | | 60 | 0.2 |
| Medium Dense to Dense | 19 | 34 | | 45 | 0.3 |
| Medium dense | 18 | 32 | | 35 | 0.3 |
| Looe to medium dense | 17 | 30 | | 20 | 0.3 |
| Loose | 16 | 28 | | 10 | 0.3 |
| Very loose to loose | 15 | 27 | | 7.5 | 0.3 |
| Very loose | 14 | 25 | | 5 | 0.3 |
| Hard | 20 | | 250 | 55 | 0.3 |
| Very stiff to Hard | 19 | | 200 | 40 | 0.3 |
| Very stiff | 19 | | 150 | 25 | 0.3 |
| Stiff to very stiff | 18 | | 100 | 17.5 | 0.3 |
| Stiff | 18 | | 75 | 10 | 0.3 |
| Firm to stiff | 17 | | 50 | 8.5 | 0.3 |
| Firm | 17 | | 35 | 7 | 0.3 |
| Soft to firm | 16 | | 25 | 5 | 0.3 |
| Soft | 16 | | 20 | 3.5 | 0.3 |
| Very soft | 16 | | 5 | 2 | 0.3 |

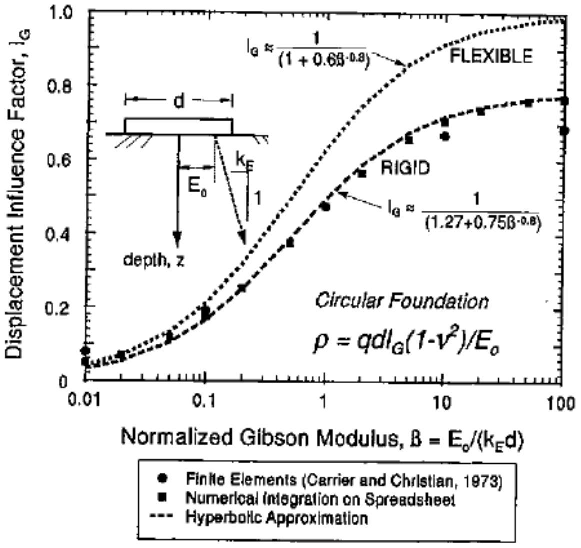
For assessing $I_g$ the following two approaches can be applied for a square or strip foundation (H.G. Poulos & E.H. Davies, 1974), refer to Figures 7 & 9 below.

For a square foundation, it is reasonable to use a circle of equal area and then calculate the equivalent diameter as follows in Equation 2:

$d_e = 2.(A/\Pi)^{0.5}$, A = foundation area Equation 2

For length to width rations greater than about 5, it is recommended to treat them as strip foundations.

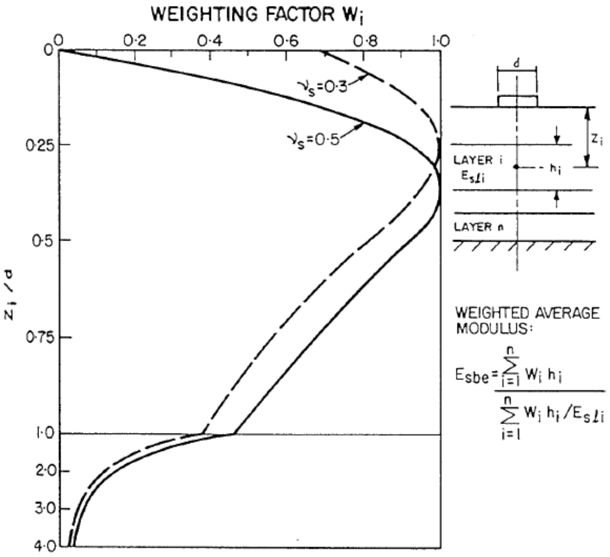
Figure 7: Influence Factors for Circular Foundations resting on Gibson Soil of Infinite Thickness (Mayne & Poulos, 1999)



## Layered Profiles

Although complex settlement assessment is beyond the scope of this paper, most sites come with a layered profile. A simple approach is to develop a weighted average following the process outlined in Figure 8 below.

Figure 8: Weighting Factos for a Layered Profile (Poulos, 1994)




## Strip Foundations

For strip foundation another simple elastic solution is proposed. The solutions depend on the relative stiffness of the strip as shown below in Equation (3):

$$K = \frac{E_p I_p (1-\nu_s^2)}{\pi E_s a^4} \qquad \text{Equation 3}$$

$E_p$ = modulus of strip, $I_p$ = moment of inertia of strip section = $b.t^3/6$, $E_s$, $\nu_s$ = Moduli of foundation, $a$ = ½ length of strip, $b$ = ½ width of strip.

Solutions are provided for calculating bending moments using the above method but are not reproduced here.

Figure 9: Deflection Factors for Strip Foundation (H.G. Poulos & E.H. Davies, 1974)

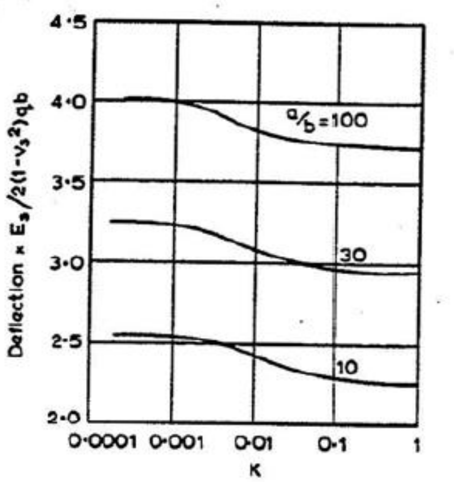
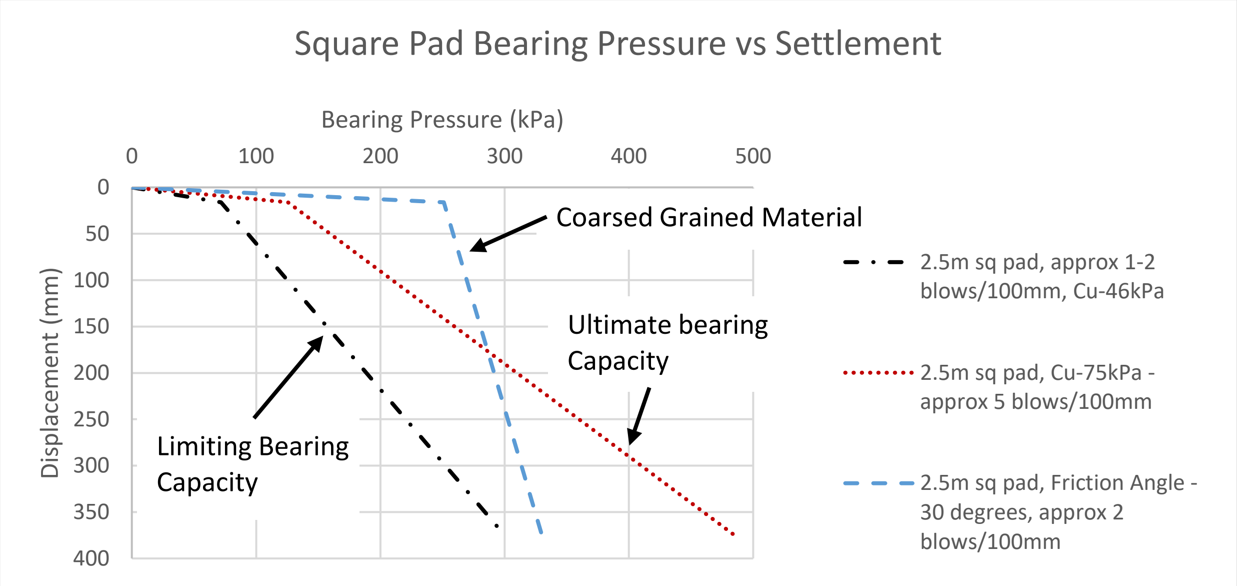



**FIG.12.21 Central deflection of uniformly loaded strip (Brown, 1969c).**

Results

Some simplified calculations have been completed to show the relationship between the definition of 'Good Ground', Ultimate Bearing Capacity, Settlement and site testing data.

Figure 10: Square Pad Bearing Pressure vs Settlement




Discussion

As can be seen above, there is a factor of about two, between the 'Good Ground' – Limiting Bearing Capcity and Ultimate Bearing Capacity – or bearing pressure at acceptable settlements.

However, for all of the undrained shear strength values utilised above, they are potentially within a range of 2-3% change in moisture content for Fill material, and 10-20% for natural material.

Settlement on coarse grained material (sand) is significantly less, even for material with relatively low UBC. Coarse grained material is also not as sensitive to moisture content change, although this has not been directly assessed.

The above figures along with Figure 4, would explain why we more commonly see issues of settlement associated with Fill material (from personnel experience), as it can be very sensitive to the moisture content at both time of placement and potentially at a later date

Also, it identifies why simply raising the DCP blow count may not significantly reduce the amount of settlement identified within the industry but may result in significant overdesign in some soils.

**CONCLUSIONS**

In general, for sites with coarse grained soils (with the exception of liquefaction, or ground movement due to instability), the requirement for 5 blows per 100mm is more than sufficient, provided that these number of blows extend to a level sufficiently below the footing. The former 4 blows per 100mm would general be considered sufficient. Depending on the location it is likely that other processes would govern over settlement, such as on sand dunes where scouring or erosion may occur.

For fine grained soils, the use of a generic 5 blows per 100mm, may just be misleading. Variations in soil moisture can have a substantial impact of soil strengths and the level of understanding of the variation of moisture content within the upper 2.0m of soil is not well established. Although there is some margin for error, it could result in both overly conservative or un-conservative designs, without the designer being aware. For example, 4 blows per 100mm in the middle of winter may provide higher UBC than 5 or 6 blows in the middle of summer.

The impact of moisture variation in compacted fill is probably better understood due the volume of lab and site testing completed on engineered fill. As such, it is very clear a small change in moisture content can substantially reduce the undrained shear strength.

The following would be recommendations for moving forward the assessment of ground and settlement performance for structures on shallow foundations.

Assess the current settlement criteria of 1:240, and whether this is still valid for modern dwellings. Many new homes have higher specification around airtightness.

Investigate the impact of natural moisture variation on insitu deposits and the variation in strength / testing results

Re-assess the use of a limiting bearing capacity (such as used in NZS3604), where the assessment is reliant on DCP testing, noticeable in Clay soils. i.e. in addition to shear vanes, should lab testing be a requirement for shallow soil sites to ensure seasonal variations in moisture content are captured. Basic testing such as moisture content and Atterberg's could provide sufficient guidance.

It would be beneficial to standardise terminology across the industry, and avoid the use of the same terminology where they refer to different calculations (Good Ground Ultimate Bearing Capcity vs Ultimate Bearing Capcity).

**REFERENCES**


Atkinson, J. (2007). *The Mechanics of Soils and Foundations*.

Atkinson, J. H. (2000). Non-linear soil stiffness in routine design. *Géotechnique*, *50*(5), 487–508.

Brown, B. J., Shorten, J. P. M., Dravitzki, D. N. R., & Goldsmith, P. R. (2008). *Soil Expansivity in the Auckland Region* (No. 120A). BRANZ.

H.G. Poulos & E.H. Davies. (1974). *Elastic Solutions for Soil and Rock Mechanics*. The University of Sydney.

Mayne, P. W., & Poulos, H. G. (1999). Approximate Displacement Influence Factors for Elastic Shallow Foundations. *Journal of Geotechnical and Geoenvironmental Engineering*, *125*(6), 453–460. https://doi.org/10.1061/(ASCE)1090-0241(1999)125:6(453)

MBIE, & NZGS. (2016). *Module 4: Earthquake Resistant Foundation Design*. NZGS.

Mohammadi, S. D., Nikoudel, M. R., Rahimi, H., & Khamehchiyan, M. (2008). Application of the Dynamic Cone Penetrometer (DCP) for determination of the engineering parameters of sandy soils. *Engineering Geology*, *101*(3–4), 195–203. https://doi.org/10.1016/j.enggeo.2008.05.006

NZGS. (2005). *Field Description of Soil and Rock*.

Poulos, H. G. (1994). *Settlement prediction for driven piles and pile groups*.

*Practice Note 12—Estimation of Design Parameters from Description and Classification Tests*. (2009). Coffey International Ltd.

*The Seismic Assessment of Existing Buildings—Part C4—Geotechnical Considerations*. (2017). MBIE.