# WORKING PLATFORMS AND BEARING CAPACITY ASSESSMENTS OF SAND OVERLYING CLAY USING FINITE ELEMENT LIMIT ANALYSIS


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## ABSTRACT

The bearing capacity of shallow foundations on layered soils is typically based on empirical models assuming a strip footing. Shape factors are then applied to the strip footing solution to account for the specific geometry of the foundation being considered. A common practical application of this methodology is when the ultimate bearing capacity of a granular working platform constructed over a clay subgrade is estimated using the Working Platforms for Tracked Plant BRE-470 guideline. Previous studies using finite element limit analysis have been undertaken to examine a strip footing on a layered soil and how the resulting bearing capacity compares to that derived from BRE-470. This paper presents an extension of previous work by the authors using finite element limit analysis to investigate the three-dimensional influence on the bearing capacity of square and rectangular footings on sand over clay. The finite element limit analysis solutions are used to produce charts to assist designers with estimating the ultimate bearing capacity of granular working platforms overlying clay. The paper also aims to highlight some important considerations when adopting the BRE-470 guideline to design granular working platforms overlying clay.


## 1 INTRODUCTION

The bearing capacity of a footing resting on sand overlying clay is a classic geotechnical engineering problem, commonly encountered in the design of a working platform. Working platforms normally comprise a good quality granular material, such as fine crushed rock, that is placed and compacted in a controlled manner, over a weak clay subgrade. The design of a working platform aims to determine the thickness and strength of the granular layer such that a minimum factor of safety against bearing capacity failure is achieved.

Several studies (Shiau et al 2003, Salimi Eshkevari & Abbo 2015 and Salimi Eshkevari et al 2019a) using Finite Element Limit Analyses (FELA) have investigated the bearing capacity of a footing resting on sand overlying clay. These studies have indicated that the bearing capacity depends on the relative strength of the two layers, which differs from what is assumed in the commonly used design guideline BRE-470 Working Platforms for Tracked Plant (BRE 2004) method (ie the bearing capacity contributions of each layer are decoupled). Whilst these previous studies are insightful to the behaviour of a footing resting on sand overlying clay, they have been limited to plain strain conditions which limits the use of the solutions in engineering practice.

The objective of this paper is to investigate the three-dimensional bearing capacity of square and rectangular footings resting on sand overlying clay using FELA. The results of the modelling are used to develop dimensionless design charts for a range of commonly encountered geotechnical conditions, working platforms and more generally footings resting on sand overlying clay. Comparisons with the BRE (2004) method are also provided for some selected cases. It is emphasised that it is not the intention of this paper to advocate or criticise the BRE-470 guideline, rather it aims to provide an alternative to compare the method with and to provide additional tools for geotechnical engineers to design working platforms, in particular, for situations that may fall outside the scope of the BRE-470 guideline.

## 2 LITERATURE REVIEW AND BACKGROUND

### 2.1 BEARING CAPACITY OF LAYERED SOILS

#### 2.1.1 Empirical methods

In the authors' experience, the most frequently cited method to estimate the bearing capacity of sand overlying clay comes from Meyerhof (1974) and Hanna & Meyerhof (1980). These methods assume that a rigid block of sand is pushed down into the underlying clay and is commonly referred to as a punching shear model. In this instance, the ultimate bearing capacity of a strip footing is assessed using the following Equation 1.

$$q_u = s_u N_c + \frac{\gamma D^2}{B} K_p \tan(\delta) \leq q_s \quad (1)$$

Where $s_u$ is the undrained shear strength of the clay layer, $N_c$ is a bearing capacity factor (typically taken as equal to 5.14 for strip footings with no embedment), $\gamma$ is the unit weight of the sand, $D$ is the thickness of the sand, $B$ is the width of the footing, $K_p$ is the coefficient of passive earth pressure, $\delta$ is the average mobilised frictional angle and $q_s$ is the ultimate bearing capacity of the same footing resting on a layer of uniform sand with the same friction angle and unit weight. Meyerhof (1974) indicated that the average mobilised friction angle $\delta$ was a function of the friction angle of the sand φ', and varied between φ'/2 to 3φ'/4. Meyerhof (1974) proposed an average value of 2φ'/3 for practical applications.

An alternative to the punching shear model is the load spread model (Terzaghi and Peck 1948). This bearing capacity model assumes that the load is distributed through the upper sand resulting in a hypothetical footing of an equivalent width resting on the clay layer beneath. The method ignores any contribution from the strength of the overlying sand. The bearing capacity of a strip footing is then assessed using classical bearing capacity theory as shown in the following equation:

$$q_u = s_u N_c \frac{B'}{B} \quad (2)$$

Where $B'$ is the effective width of the hypothetical footing as defined in the following equation:

$$B' = B + 2D tan(\theta) \quad (3)$$

Terzaghi & Peck (1978) proposed $\theta$ be taken as 26.6°, which is commensurate with a 2V:1H stress distribution. Later studies have indicated 2V:1H assumption may not be appropriate where a layered soil profile is present as the angle is a function of the relative strength of the upper and lower layers (Burd & Frydman 1997). Further discussion on the load spread angle is provided in Burd & Frydman (1997) and Salimi Eshkevari et al (2019a).

### 2.1.2 Numerical methods

Although conventional displacement finite element analysis can be used to predict the ultimate bearing capacity of a footing on layered soil, the estimate obtained is neither a lower nor upper bound (Shiau et al 2003). Furthermore, a phenomenon referred to as ‘locking’ can occur and is characterised by a constantly rising load deformation curve and can lead to inaccurate estimates of the ultimate bearing capacity (Sloan & Randolph 1982 and Shiau et al 2003). One method to overcome these limitations is to apply the limit analysis theorems to estimate the rigorous plasticity solutions using linear finite elements (commonly referred to as finite element limit analysis or FELA). Further details on the background and theory of FELA applied to bearing capacity problems is provided in Merifield et al (1999), Shiau et al (2004), Salimi Eshkevari et al (2019a) and Salimi Eshkevari et al (2019b).

FELA has been successfully applied to estimate the bearing capacity of strip footings on layered clays (Merifield et al 1999), sand over clay (Shiau et al 2003 and Salimi Eshkevari et al 2019a) and layered sands (Salimi Eshkevari et al 2019b). However, to the authors’ knowledge, limited studies using FELA have considered three-dimensional footing effects on the resulting ultimate bearing capacity of a layered soil. For example, solutions appear to be limited to Merifield & Nguyen (2006) who provide the ultimate bearing capacity for square and circular footings resting on a layered clay.

## 2.2 WORKING PLATFORM THICKNESS DESIGN

In Australian practice, it is common to refer to the design guideline ‘Working Platforms for Tracked Plant’, published by the Building Research Establishment (BRE 2004) for guidance when designing a working platform. Even though BRE (2004) is aimed at tracked plant, it has found its way into routine working platform assessments for both tracked and outrigger plant without much consideration of the limitations or applicability of the design guide.

Even though the BRE (2004) approach has proven to be useful in assessing and preparing the design of working platforms, some authors have indicated that BRE (2004) can result in both thick working platforms and over predictions of the bearing capacity (Corke & Gannon 2010 and Eshkevari & Abbo 2015). It is important to highlight that, even though the method has been shown to yield conservative working platforms, there are many other factors that may contribute to the perceived conservatism of the BRE (2004) method. For example, pessimistic / lower bound subgrade strengths adopted in design and that the friction angle of the granular layer is usually much higher than what is adopted in design. From a designer's perspective, this should seem sensible, since most of the time the subgrade strength is assessed using simple methods (eg a dynamic cone penetrometer or pocket penetrometer to assess subgrade strength is common) and the friction angle of the granular material is not normally determined from laboratory testing. This would usually be acceptable on simple sites for routine working platform assessments, however, where large / high risk lifts are proposed or there are difficult / poor ground conditions, a more rigorous assessment of the geotechnical parameters should be considered.

In the authors experience, there also appears to be some misunderstandings or misconceptions in industry regarding the track bearing pressure and loading geometry for working platform thickness design. All too often, we (consultants) receive requests like 'just design the working platform for 300 kPa', without any information on how or what situation the track pressures are to be considered. And when queried, or clarifications requested, responses like 'this has not been an issue in the past, why can't you just design it for 300 kPa' or 'here is the technical drawing for the rig, just use the track dimensions' or 'what do you mean travelling / standing and drilling / extracting load cases'. From a geotechnical perspective, it is apparent that the resultant thickness of any working platform is directly proportional to the magnitude and geometry of the loading. It is also very apparent from BRE (2004) how important what situation the loading occurs in. Unfortunately, this message is either not always conveyed or understood by contractors and designers alike and has the potential to lead to both unsafe and overly conservative situations.

Further discussions on BRE (2004) can be found in Look & Honeyfield (2016) and the accompanied discussion paper by Buttling (2016). Although not discussed further here, these two references include an insightful case study, discussions around the limitations of BRE (2004) and issues of conservatism and how these might be managed.

The general approach in BRE (2004) for assessing the bearing capacity of a granular layer overlying clay has been based on the work of Meyerhof (1974) and Hanna & Meyerhof (1980) (eg punching failure bearing capacity model) and adopts the average mobilised friction value proposed by Meyerhof (1978). BRE (2004) qualifies this by stating that a simple approach to assess the ultimate bearing capacity of a working platform of a relatively shallow thickness placed on a weak subgrade is to adopt a punching shear model. However, BRE (2004) provides limited guidance or background to determine if the granular layer is considered 'relatively shallow', and only states the punching shear model is not appropriate when the ratio of *D/B* is larger than 1.5. Where a cohesive subgrade is being assessed, BRE (2004) states that it is limited to clay subgrades with undrained shears strengths between 20 kPa to 80 kPa.

BRE (2004) adopts the following equation to calculate the ultimate bearing capacity:

$$q_u = s_u N_c s_c + \frac{D^2\gamma}{B} K_p \tan(\delta) s_p \tag{4}$$

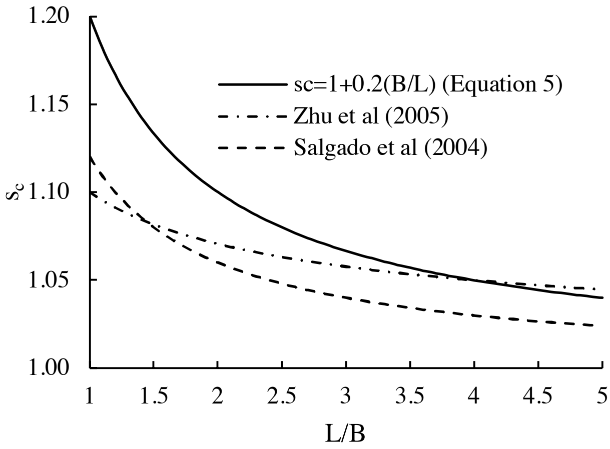
Where $s_c$ and $s_p$ are shape factors as defined in the following equations:

$$s_c = 1 + 0.2\left(\frac{B}{L}\right) \tag{5}$$

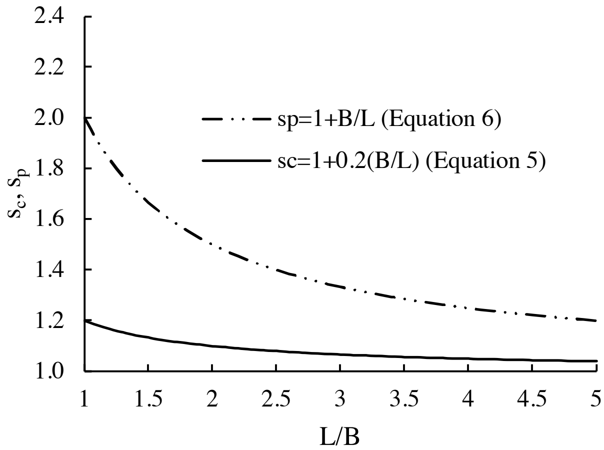
$$s_p = 1 + \left(\frac{B}{L}\right) \tag{6}$$

Where $L$ is the length of the footing.

The value of the shape factor $s_c$ has been derived from experimental data from Meyerhof (1951), Meyerhof (1963) and Skempton (1951). The shape factor intends to account for the additional slip surfaces in front and behind the footing (Salgado et al 2004). More recently, the shape factor $s_c$ has been derived numerically (Salgado et al 2004 and Zhu et al 2005). The shape factor $s_c$ derived from the various authors and equations presented above are illustrated in Figure 1 (a).




**(a)**




**(b)**

**Figure 1: Shape factors from various authors:**

**(a) after Meyerhof (1951), Salgado et al (2004), Zhu et al (2005). (b) , BRE (2004)**

Figure 1 (a) shows that the relationship after Meyerhof (1951) (Equation 5), as adopted in BRE (2004)) is only comparable to the numerically derived shape factors for large *L/B* ratios beyond about 3.5, which is typical of tracked plant. However, when considering outrigger plant, which typically have outrigger pads with *L/B* ratios of between 1 and 2, Meyerhof (1951) leads to unconservative estimates of the ultimate bearing capacity in the order up to about 10%.

The shape factor $s_p$ proposed in BRE (2004) is said to be based on the volume of soil undergoing punching shear. To the authors' best knowledge, the shape factor $s_p$ has not been proven either experimentally or numerically, and in comparison to the shape factor $s_c$, is some 15% to 70% larger for *L/B* ratios ranging between 1 to 5 (see Figure 1 (b)). Historically, shape factors have only been developed to be used with the general bearing capacity formula of a strip footing resting on a uniform homogenous soil. In the authors opinion, the applicability of using shape factors in a layered soil problem is dubious and should be undertaken with caution. It is important to point out BRE (2004) provides no discussion or evidence for the use of shape factors in calculating the ultimate bearing capacity of a footing resting on sand overlying clay.

Whilst not the aim of this paper, another relevant consideration is the factor of safety (FoS) adopted in the design of working platforms in order to derive the allowable (or safe) bearing capacity. Some guidance may be obtained from BRE (2004) on the choice of a FoS according to the design case being considered, although readers should be mindful that the definition presented in BRE (2004) is in the context of a load factor, which varies according to the load case being considered and also whether a working platform is required (ie the load factor varies between 1.2 to 2.0). In the authors experience, the FoS adopted in design of a working platform thickness can vary between 1.3 to 3.0. Ultimately, the FoS adopted for working platform design needs to be assessed and determined by the designer since this depends on many considerations including but not limited to the consequence of failure, the amount and quality of geotechnical data, design method / philosophy etc.

## 3 FINITE ELEMENT LIMIT ANALYSIS

### 3.1 PROBLEM DEFINITION

A series of FELA simulations were performed using the software OptumG2 and OptumG3 (Optum CE). Over 250 analyses were conducted from which values of limit loads were estimated for a range of geometric and material parameters (Table 1) and assumptions (rough and smooth footings). OptumG2 was used to assess the lower and upper bound limit loads for strip footings. The analyses in OptumG3 were performed using a mixed element type to assess the limit loads for square and rectangular footings. The mixed element type is based on the requirements of the upper and lower bound theorems and is found to be very accurate (Olsen & Krabbenhoft 2021). It is also the recommended element in Optum G3 (Optum 2020). Further discussion on the validation and application of the mixed element is given by Optum (2020) and Olsen & Krabbenhoft (2021). Both analyses have considered adaptive meshing. A simple schematic of the problem is given in Figure 2 along with examples of a typical upper bound mesh for a strip footing and mixed element mesh for a square footing.

**Table 1: Problem Variable Considered**

| Parameter | Values |
|---|---|
| Friction angle of sand, φ' (°) | 35, 40 and 45 |
| Undrained shear strength of clay, $s_u$ (kPa) | 10, 20, 30, 40, 50, 60, 80, 100 |
| Unit weight of sand, γ (kN/m$^3$) | 18 |
| Sand thickness, D (m) | 0.25, 0.5, 1.0 and 2.0 |
| Foundation width, B (m) | 1.0 |
| Foundation length, L (m) | 1.0, 2.0 and 5.0 |

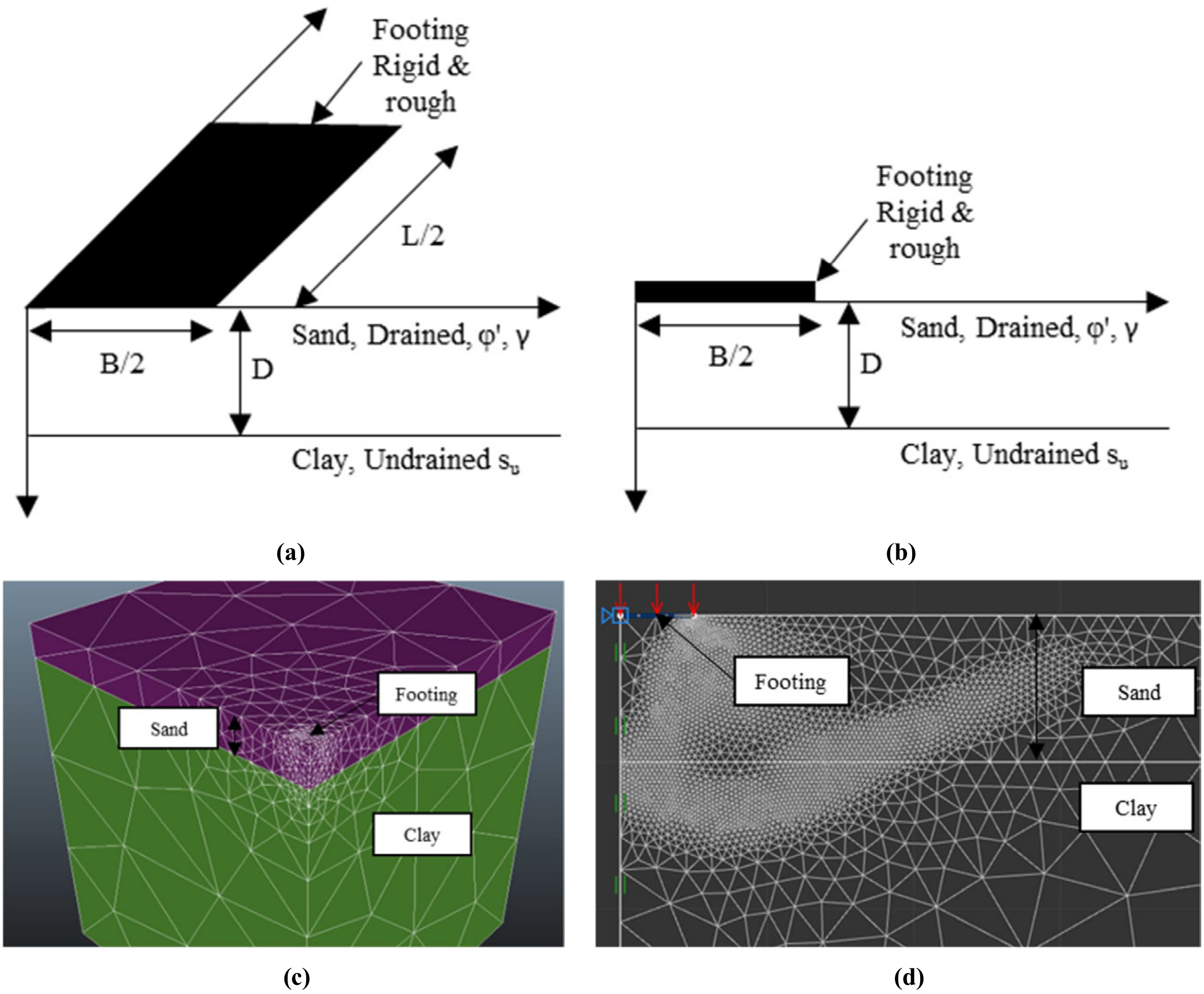



**Figure 2: Problem definition and typical meshes:**

**(a) Three-dimensional problem definition. (b) Plain strain problem definition.**

**(c) Mixed element mesh in three-dimensional analysis. (d) Upper bound mesh in plain strain analysis**

The sand and clay layers were modelled using Mohr-Coulomb and Tresca models respectively. The analysis is based on an associative flow rule. This assumption is considered reasonable since the problem is not strongly constrained due to the freely deforming ground surface and semi-infinite domain (Sloan 2013). The footing was modelled using a weightless shell / plate element and was assumed to be perfectly rigid and rough for the majority of the analyses. Due to symmetry a quarter of the foundation has been modelled in the three-dimensional analyses where possible. The 2D analyses of strip footings considered half of the footing width where possible. The analyses have been limited to footings resting on the surface.

The ultimate bearing capacity from the FELA for the strip footing ($q_{u,\ strip}$) is taken as the average of the lower and upper bound limit loads (Salgado et al 2004). The relative error in the solution was assessed using the following equation and was found to be generally less than about 5 %, with the maximum relative in the order of about 10 %.

$$error\ \% = \frac{q_{u,upper\ bound} - q_{u,lower\ bound}}{q_{u\ ,upper\ bound} + q_{u,lower\ bound}} \times 100 \qquad (7)$$

## 3.2 BENCHMARKING AND COMPARISON WITH PUBLISHED DATA

There is limited data available to compare the current study with, however, analyses were undertaken for various cases and benchmarked against available published solutions for a square footing resting on a uniform sand layer (Martin 2005, Meyerhof 1951, Meyerhof 1963), for a strip footing resting on sand overlying clay (Michalowski & Shi 1995, Burd & Frydman 1997, Shiau et al 2003 and Salimi Eshkevari et al 2019a), and centrifuge data of strip and circular footings resting on sand overlying clay (Okamura et al 1997). The results from the benchmark / validation exercises are presented in Figure 3 (a) to (d).

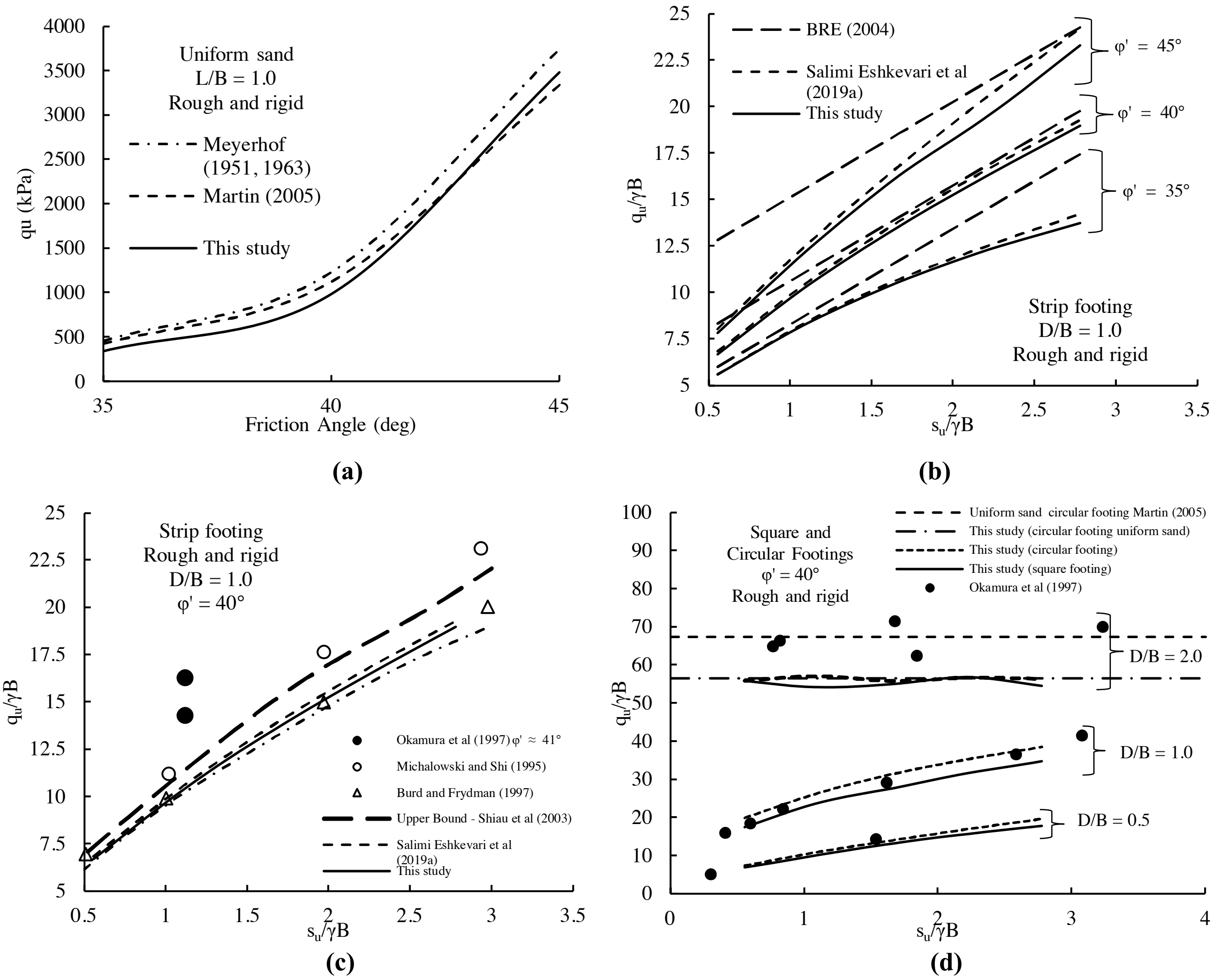



**Figure 3: Results of benchmarking and comparisons with published data:**
**(a) Square footing on uniform sand – This study, Meyerhof (1951), Meyerhof (1963) and Martin (2005).**
**(b) Strip footing *D/B* = 1.0 – This study, BRE (2004) and Salimi Eshkevari et al (2019a)**
**(c) Strip footing *D/B* = 1 φ' = 40° – This study, Michalowski & Shi (1995), Burd & Frydman (1997), Okamura et al (1997), Shiau et al (2003) and Salimi Eshkevari et al (2019a).**
**(d) Square and circular footing φ' = 40° – This study, Okamura et al (1997) and Martin (2005).**

The bearing capacity estimated from this study for the various cases considered can be seen to compare reasonably well with other available published results and data.

Although not presented here, the benchmarking exercises included comparisons of the resulting limit loads from lower, upper and mixed element types in the three-dimensional analyses for selected cases. The relative error between the mixed element and those obtained from the averaging of the upper and lower bounds was found to be generally less than about 3 %, with the maximum relative error in the order of about 10 %.

### 3.3 RESULTS AND DISCUSSION

#### 3.3.1 Parametric study and ultimate bearing capacity

A parametric study based on the Table 1 parameters was undertaken to estimate the limit loads for rough and rigid, strip, square (*L/B* = 1) and rectangular (*L/B* = 2 and 5) footings resting on sand overlying clay. The results of the analysis have been presented in Figure 4 to Figure 8 and Table 2 to Table 4. It is acknowledged some results exhibit some minor oscillating / variation, albeit very small, and is likely to be attributed to minor variations in numerical convergence.

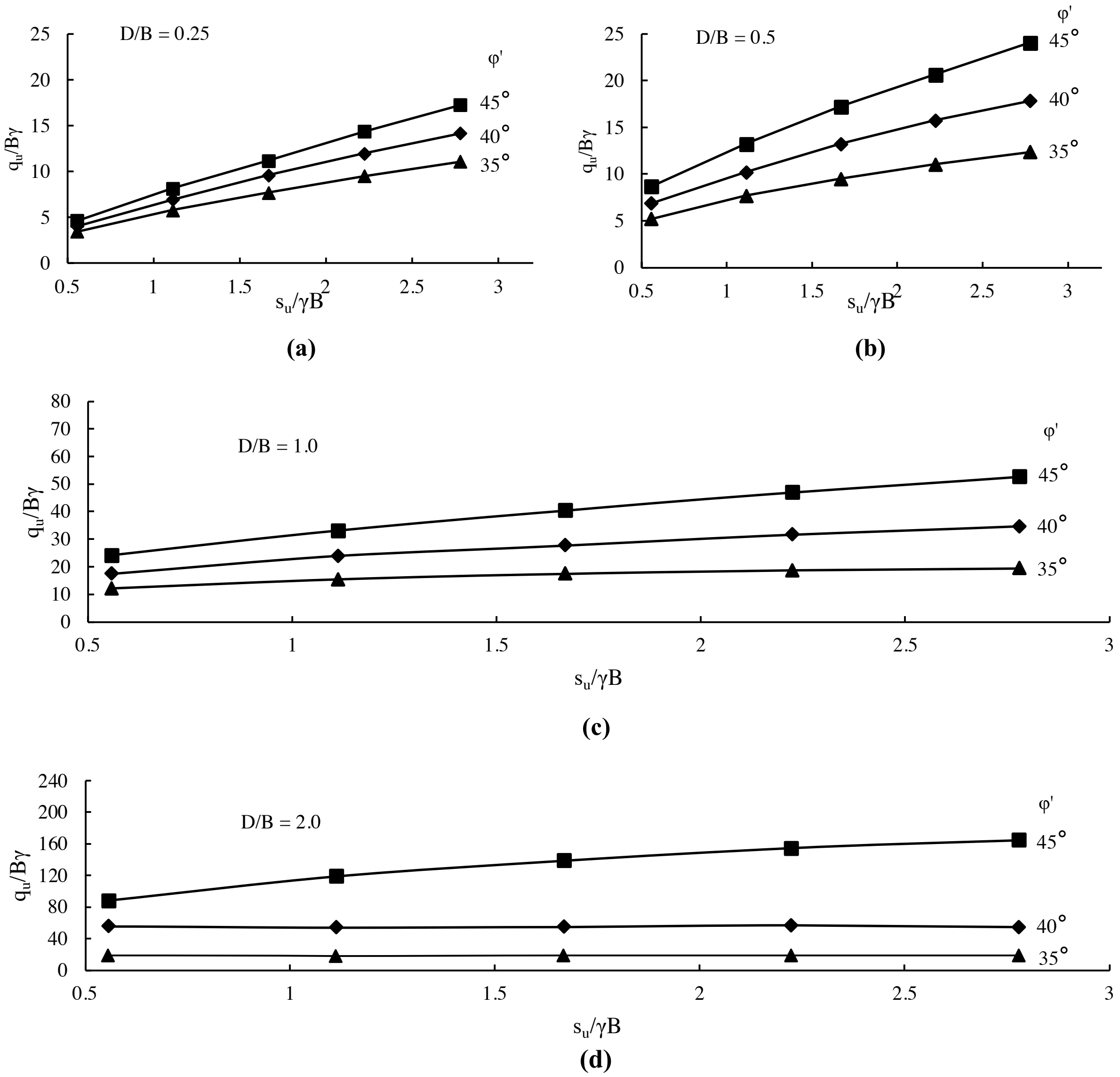



**Figure 4: Ultimate bearing capacity ($q_u/B\gamma$) for rough rigid square footings (*L/B* = 1.0) on sand overlying clay: (a) *D/B* = 0.25. (b) *D/B* = 0.5. (c) *D/B* = 1.0. (d) *D/B* = 2.0.**

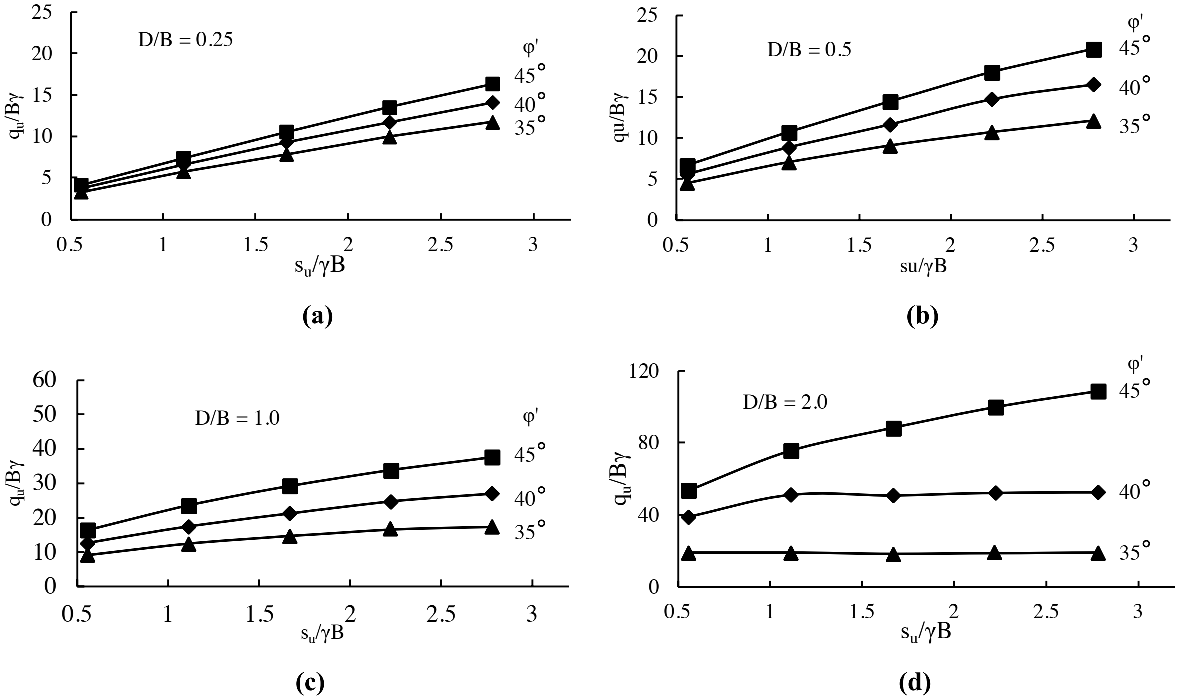



**Figure 5: Ultimate bearing capacity ($q_u$/B γ) for rough rigid rectangular footings (*L/B* = 2.0) on sand overlying clay:**

**(a). *D/B* = 0.25. (b) *D/B* = 0.5.**

**(c) *D/B* = 1.0. (d) *D/B* = 2.0.**

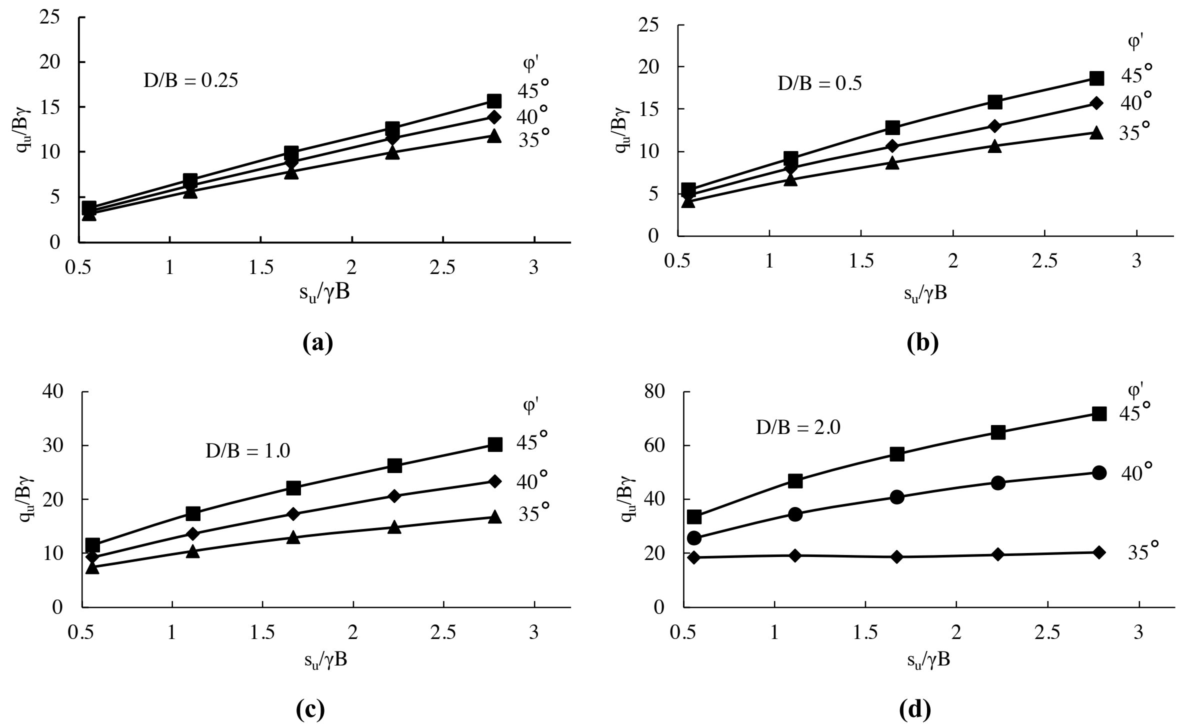


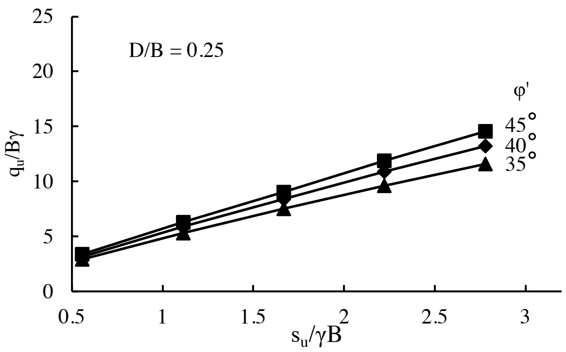

**Figure 6: Ultimate bearing capacity (*$q_u$/B γ*) for rough rigid rectangular footings (*L/B* = 5.0) on sand overlying clay:**

**(a) *D/B* = 0.25. (b) *D/B* = 0.5.**

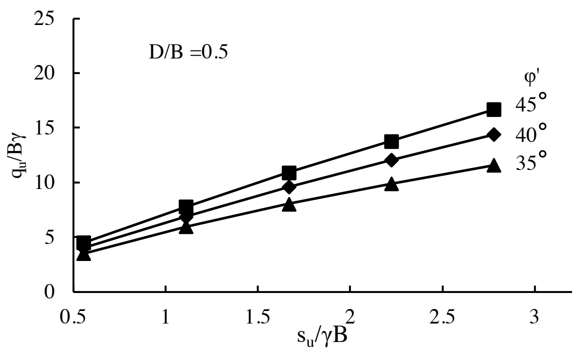
**(c) *D/B* = 1.0. (d) *D/B* = 2.0**

**Table 2: Values of ultimate bearing capacity ($q_u/B\gamma$) rough rigid footings φ' = 35 °**

| *D/B* | $s_u/\gamma B$ | Ultimate bearing capacity ($q_u/B\gamma$) | | | |
|---|---|---|---|---|---|
| | | **Square Footing (*L/B* = 1.0)** | **Rectangular Footing (*L/B* = 2.0)** | **Rectangular Footing (*L/B* =5.0)** | **Strip Footing (*L/B* = ∞)** |
| 0.25 | 0.56 | 3.43 | 3.28 | 3.17 | 2.90 |
| | 1.11 | 5.78 | 5.75 | 5.64 | 5.31 |
| | 1.67 | 7.68 | 7.83 | 7.80 | 7.51 |
| | 2.22 | 9.48 | 9.97 | 9.97 | 9.60 |
| | 2.78 | 11.10 | 11.74 | 11.84 | 11.59 |
| 0.5 | 0.56 | 5.24 | 4.54 | 4.11 | 3.53 |
| | 1.11 | 7.71 | 7.06 | 6.68 | 5.96 |
| | 1.67 | 9.50 | 9.09 | 8.71 | 8.04 |
| | 2.22 | 11.03 | 10.76 | 10.65 | 9.87 |
| | 2.78 | 12.41 | 12.14 | 12.25 | 11.57 |
| 1.0 | 0.56 | 12.21 | 9.27 | 7.41 | 5.61 |
| | 1.11 | 15.43 | 12.50 | 10.40 | 8.46 |
| | 1.67 | 17.53 | 14.72 | 12.97 | 10.72 |
| | 2.22 | 18.73 | 16.63 | 14.88 | 12.55 |
| | 2.78 | 19.46 | 17.47 | 16.75 | 14.22 |
| 2.0 | 0.56 | 18.93 | 19.11 | 18.44 | 11.84 |
| | 1.11 | 18.53 | 19.11 | 19.10 | 15.99 |
| | 1.67 | 18.64 | 18.37 | 18.62 | 16.67 |
| | 2.22 | 18.67 | 18.81 | 19.39 | 16.57 |
| | 2.78 | 18.65 | 19.13 | 20.27 | 16.55 |
| Uniform Sand | - | 18.98 | 19.06 | 18.92 | 16.53 |




**(a)**




**(b)**

**Figure 7: Ultimate bearing capacity ($q_u/B\gamma$) for rough rigid strip footings on sand overlying clay:**

**(a) *D/B* = 0.25. (b) *D/B* = 0.5.**

**Table 3: Values of ultimate bearing capacity ($q_u/B\gamma$) rough rigid footings φ' = 40 °**

| *D/B* | $s_u/\gamma B$ | Ultimate bearing capacity ($q_u/B\gamma$) | | | |
|---|---|---|---|---|---|
| | | Square Footing (*L/B* = 1.0) | Rectangular Footing (*L/B* = 2.0) | Rectangular Footing (*L/B* =5.0) | Strip Footing (*L/B* = ∞) |
| 0.25 | 0.56 | 4.03 | 3.73 | 3.44 | 3.17 |
| | 1.11 | 6.92 | 6.60 | 6.28 | 5.85 |
| | 1.67 | 9.57 | 9.29 | 8.88 | 8.38 |
| | 2.22 | 11.95 | 11.71 | 11.55 | 10.88 |
| | 2.78 | 14.18 | 14.11 | 13.91 | 13.21 |
| 0.5 | 0.56 | 6.93 | 5.59 | 4.82 | 4.01 |
| | 1.11 | 10.22 | 8.88 | 8.03 | 6.85 |
| | 1.67 | 13.26 | 11.67 | 10.62 | 9.59 |
| | 2.22 | 15.74 | 14.77 | 13.01 | 12.04 |
| | 2.78 | 17.87 | 16.57 | 15.69 | 14.40 |
| 1.0 | 0.56 | 17.55 | 12.74 | 9.28 | 6.69 |
| | 1.11 | 23.92 | 17.53 | 13.61 | 10.39 |
| | 1.67 | 27.79 | 21.31 | 17.27 | 13.55 |
| | 2.22 | 31.68 | 24.77 | 20.60 | 16.35 |
| | 2.78 | 34.73 | 27.16 | 23.35 | 18.99 |
| 2.0 | 0.56 | 55.80 | 39.00 | 25.58 | 14.45 |
| | 1.11 | 54.17 | 51.31 | 34.55 | 20.97 |
| | 1.67 | 55.06 | 50.87 | 40.84 | 26.21 |
| | 2.22 | 56.79 | 52.35 | 46.20 | 29.81 |
| | 2.78 | 54.60 | 52.66 | 49.93 | 33.51 |
| Uniform Sand | - | 54.81 | 50.12 | 49.05 | 40.91 |

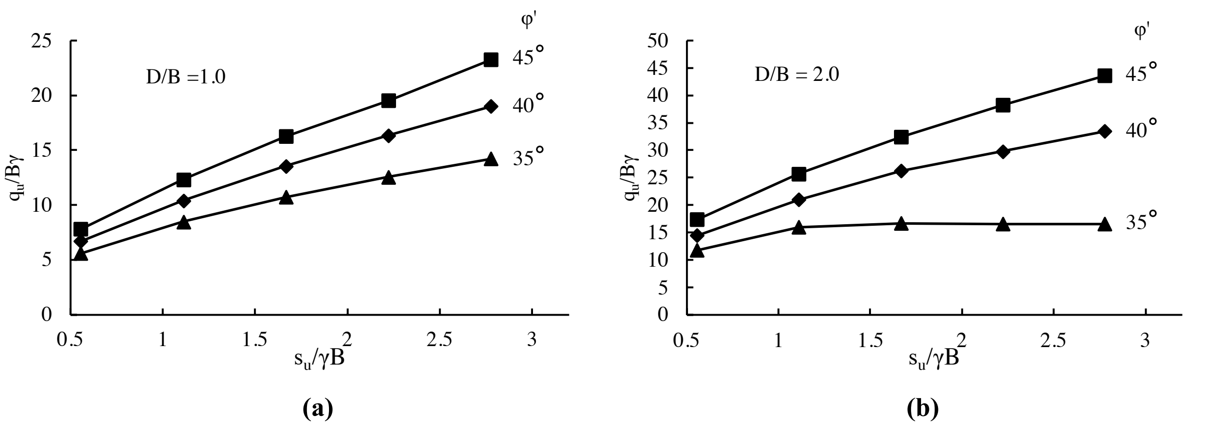



**Figure 8: Ultimate bearing capacity ($q_u/B\gamma$) for rough rigid strip footings on sand overlying clay:**

**(a) *D/B* = 1.0. (b) *D/B* = 2.0.**

**Table 4: Values of ultimate bearing capacity ($q_u/B\gamma$) rough rigid footings φ' = 45 °**

| *D/B* | $s_u/\gamma B$ | Ultimate bearing capacity ($q_u/B\gamma$) Square Footing (*L/B* = 1.0) | Rectangular Footing (*L/B* = 2.0) | Rectangular Footing (*L/B* =5.0) | Strip Footing (*L/B* = ∞) |
|---|---|---|---|---|---|
| 0.25 | 0.56 | 4.63 | 4.12 | 3.78 | 3.37 |
| | 1.11 | 8.12 | 7.36 | 6.90 | 6.29 |
| | 1.67 | 11.17 | 10.55 | 9.92 | 9.04 |
| | 2.22 | 14.35 | 13.48 | 12.64 | 11.88 |
| | 2.78 | 17.26 | 16.35 | 15.68 | 14.58 |
| 0.5 | 0.56 | 8.71 | 6.66 | 5.48 | 4.50 |
| | 1.11 | 13.24 | 10.73 | 9.18 | 7.78 |
| | 1.67 | 17.24 | 14.49 | 12.81 | 10.89 |
| | 2.22 | 20.65 | 18.08 | 15.90 | 13.76 |
| | 2.78 | 24.10 | 20.90 | 18.68 | 16.64 |
| 1.0 | 0.56 | 24.23 | 16.56 | 11.56 | 7.83 |
| | 1.11 | 33.18 | 23.66 | 17.43 | 12.32 |
| | 1.67 | 40.43 | 29.34 | 22.13 | 16.27 |
| | 2.22 | 46.99 | 33.88 | 26.22 | 19.54 |
| | 2.78 | 52.69 | 37.76 | 30.15 | 23.29 |
| 2.0 | 0.56 | 88.12 | 53.79 | 33.58 | 17.44 |
| | 1.11 | 119.00 | 75.77 | 46.82 | 25.71 |
| | 1.67 | 138.97 | 88.43 | 56.74 | 32.40 |
| | 2.22 | 154.82 | 99.95 | 64.86 | 38.26 |
| | 2.78 | 164.78 | 108.82 | 71.78 | 43.62 |
| Uniform Sand | - | 193.57 | 160.25 | 141.75 | 112.51 |

The results shown in Figure 4 to Figure 8 indicate that the ultimate bearing capacity from the FELA have an almost linear variation with the undrained shear strength of the clay for low *D/B* ratios. However, as *D/B* increases, this tends to a power law, particularly for higher friction angles of the sand. A similar behaviour for strip footings was observed by Shiau et al (2003) and Salimi Eshkevari et al (2019a).

For *D/B* = 2.0, the FELA analysis indicates for φ' < 40°, the failure mechanism is generally constrained to the sand (ie the constant ratio of $q_u/B\gamma$), refer Figure 4 (d), Figure 5 (d) and Figure 6 (d). However, for the same *D/B* and when φ' = 45° (refer Figure 4 (d), Figure 5 (d) and Figure 6 (d)) the lower clay layer is still having an appreciable effect on the ultimate bearing capacity. This indicates that there is a critical ratio of *D/B* and that there is a likely a transitional failure mechanism (eg from a general bearing failure within the sand to a punching failure into the clay) that is influenced by the relative shear strength of the upper and lower layers, and the geometrical aspects of the loaded area. Similar behaviours have been discussed by Michalowski & Shi (1995), Shiau et al (2003), Merifield & Nguyen (2006) and Salimi Eshkevari et al (2019b). It is also of interest to highlight that the limiting values of the ultimate bearing capacity observed are similar to $N_\gamma/2$ obtained from Martin (2005) ($N_\gamma/2$ = 17.23 to 42.78 for φ' = 35° and 40° respectively) and Meyerhof (1951, 1963) ($N_\gamma/2$ = 18.58 to 46.85 for φ' = 35° and 40° respectively), albeit with some slight variation which would be attributed to the footings three dimensional geometry.

### 3.3.2 Comparisons with BRE (2004)

The results of the analysis in Section 3.3.1. have been compared to an equivalent BRE (2004) assessment. The BRE (2004) assessment has been based on Equations 4 to 6. Where relevant, the BRE (2004) assessment has considered a limiting ultimate bearing capacity based on a uniform sand layer and the procedures in BRE (2004) (eg assuming uniform granular layer with a load applied at the surface). The comparisons are presented in Figure 9 and Figure 10 for the selected cases of a square footing ($L/B$ = 1.0) and rectangular footing ($L/B$ = 5) respectively.

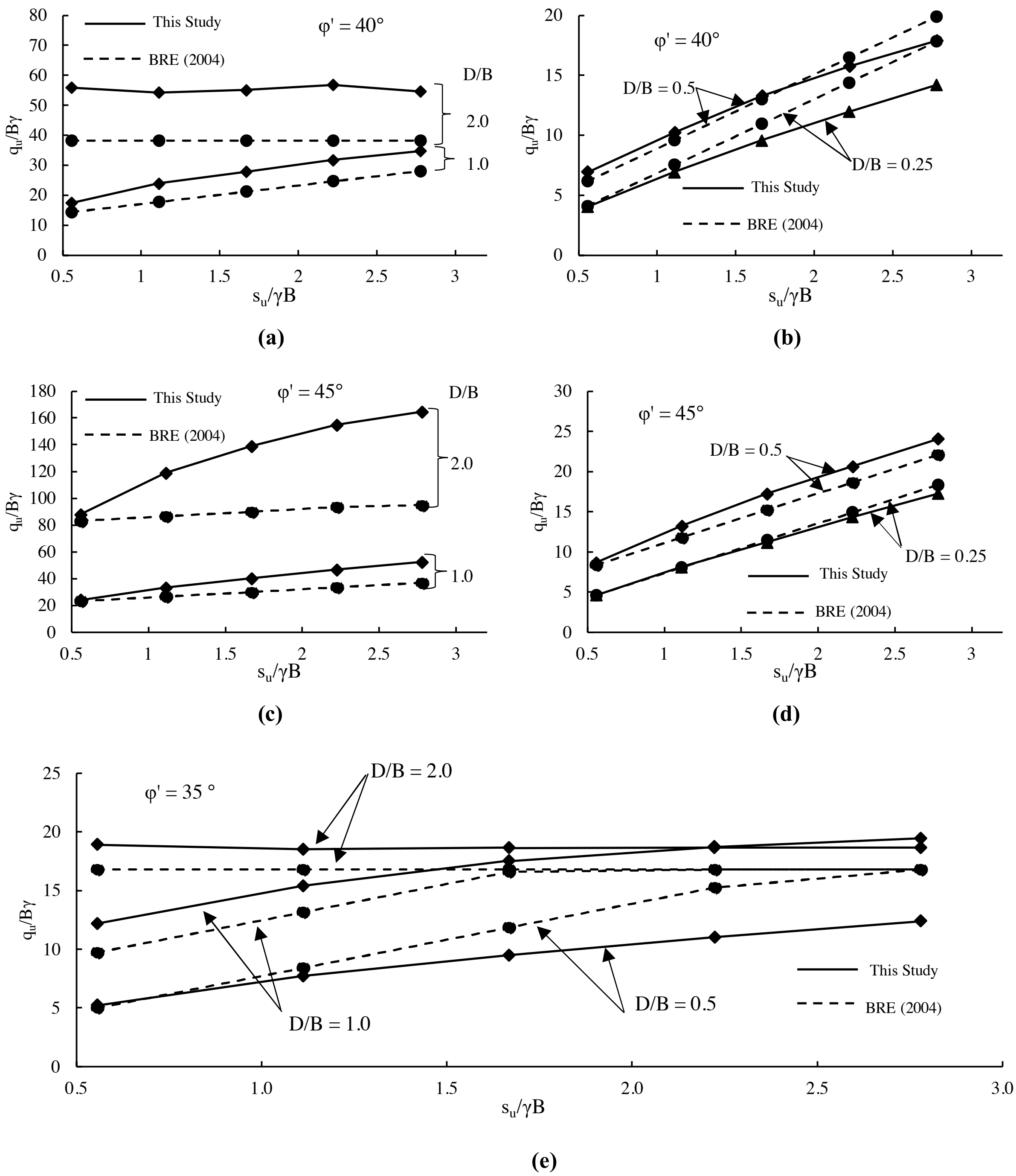



**Figure 9: Comparisons with BRE (2004) and the effect of sand thickness for a rigid rough square footing ($L/B$ = 1.0):**

**(a) φ' = 40° with $D/B$ = 1.0 and $D/B$ = 2.0.** **(b) φ' = 40° with $D/B$ = 0.25 and $D/B$ = 0.5.**

**(c) φ' = 45° with $D/B$ = 1.0 and $D/B$ = 2.0.** **(d) φ' = 45° with $D/B$ = 0.25 and $D/B$ = 0.5.**

**(e) φ' = 35° with D/B = 0.5, $D/B$ = 1.0 and $D/B$ = 2.0.**

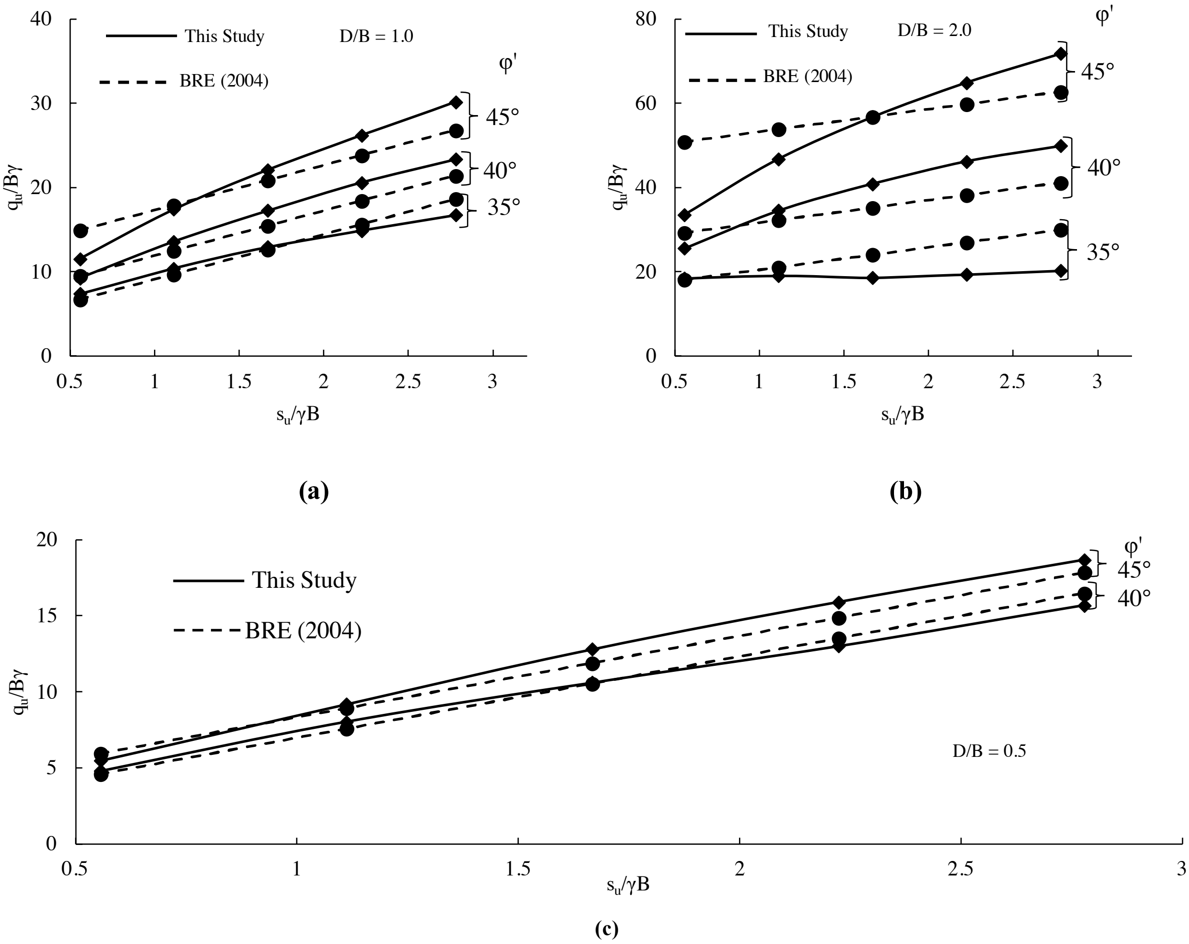



**Figure 10: Comparisons with BRE (2004) and the effect of friction angle for rigid rough footing (*L/B* = 5)**

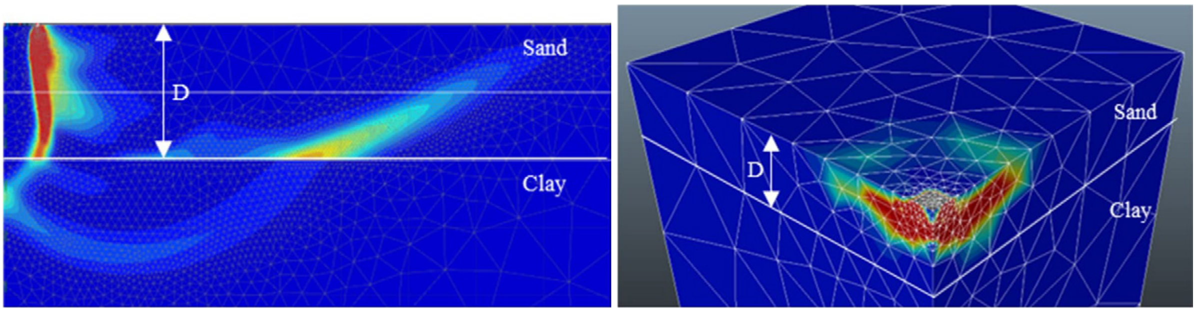
**(a) *D/B* = 1.0. (b) *D/B* = 2.0. (c) *D/B* = 0.5**

Figure 9 (b) and (d) and Figure 10 (c) show that the ultimate bearing capacity from BRE (2004) is generally comparable to or slightly less than the FELA results for φ' $\geq$ 40° for *D/B* $\leq$ 0.5 for the *L/B* ratios considered in this study. Although, as the strength of the clay increases, BRE (2004) tends to overestimate the bearing capacity for the case of φ' = 40° (refer Figure 9 (b)).

As the working platform thickness increases, Figure 9 (a) and (c) and Figure 10 (a) and (b) show that the ultimate bearing capacity from BRE (2004) tends to be underestimated compared to the FELA (in the order of about 10% to 30% for *D/B* = 1.0 and 50% to 70% for *D/B* = 2.0). Whilst not all presented here, the results from the FELA for *L/B* $\leq$ 2.0 are different to the those from Salimi Eshkevari et al (2019a), who indicated that for a strip footing, the BRE (2004) method tends to overestimate the ultimate bearing capacity of clay subgrades of strengths lower than 55 kPa, with high friction angles (φ' > 40°). However, as *L/B* increases to 5.0 (refer Figure 10 (a) and (b)) it is clear that BRE (2004) tends to overestimate the ultimate bearing capacity for $s_u/\gamma B$ less than about 1.11 to 1.67 and *D/B* $\geq$ 1.0. This is similar to the results obtained by Salimi Eshkevari et al (2019a) (refer to Figure 3 (b)) which shows BRE (2004) strip footing results compared with the strip footing of this study and Salimi Eshkevari et al (2019a)).

Where thin platforms are constructed out of lower frictional material (φ' =35°), it can be seen in Figure 9 (e) that BRE (2004) tends to overestimate the ultimate bearing capacity compared to the FELA, and is generally only comparable for the case of *L/B* = 5.0 and *D/B* = 1.0. Such a situation where a working platform is constructed out of a low frictional material is arguably not within the scope of the BRE (2004) guideline. However, depending on location and material availability, crushed rock from quarries may not be available and designers may need to consider a poorer quality source (eg river won gravel that may be rounded to sub rounded).

Whilst a direct comparison with BRE (2004) is not shown here, it is of interest to point out a special case where *D/B* = 2.0 and φ' = 40°, where the cases for *L/B* $\leq$ 2.0 (refer Figure 4 (d) and Figure 5 (d)) are compared with the strip footing results in Figure 8 (b). It is clear from the analysis that the failure mechanism from the three-dimensional FELA is constrained to the upper sand layer (ie the FELA has converged to a constant ratio of $q_u/B\gamma$), albeit with a minor amount of numerical variation. However, the values of $q_u/B\gamma$ derived from the analysis of a strip footing indicate that the clay layer is still having an appreciable effect on the resulting bearing capacity. Such a situation highlights the caution required when considering applying shape factors to a layered bearing capacity problem, as is done in the BRE (2004) method. Inspection of the shear dissipation plots from the FELA further emphasises this, where a punching mechanism is observed in the plain strain model (refer Figure 11 (a)), whereas the square and rectangular footings indicate the failure mechanism is a general bearing failure constrained to the sand (refer Figure 11 (b)). This is likely to be attributed to the greater influence depth of a strip footing compared to that of a square or rectangular footing. Further discussion on the effect of footing geometry is provided in Section 3.3.3.




**(a)** **(b)**

**Figure 11: Shear dissipation plots for $D/B$ = 2.0, φ' = 40° and $s_u/\gamma B$ = 1.11.**

**(a) Plain strain with B = 1.0 m. (b) Square footing ($L/B$ = 1.0)**

The FELA has indicated that for many practical situations where thin working platforms, eg $D/B \leq 0.5$, are proposed to be constructed out of good quality fine crushed rock with φ' > 40° (preferably 45°) over a soft to firm clay, the BRE (2004) guideline generally provides estimates of the ultimate bearing capacity within about 10 % of the FELA. However, for thicker working platforms, BRE (2004) may not yield reliable estimates of the ultimate bearing capacity and may potentially be either overly conservative or unconservative. Some circumstances where the undrained shear strength was low ($s_u/\gamma B$ < 1.11), the FELA indicated that the BRE (2004) method can provide reasonable estimates of the ultimate bearing capacity. Although, the FELA has also indicated that there are possible combinations of platform material strength, thickness and load geometry where BRE (2004) yielded conservative and unconservative ultimate bearing capacities. BRE (2004) states that the ultimate bearing capacity using a punching shear model is not appropriate when the ratio of $D/B$ is larger than 1.5, however, based on the results of the FELA, it is suggested to be less, and no larger than $D/B$ of about 0.5 to 1.0, although this depends on the ratio of $L/B$ and the relative shear strength of the sand and clay.

### 3.3.3 Effect of footing dimensions

As noted in Section 3.3.2, caution should be exercised when considering the modification of the ultimate bearing capacity of a strip footing using shape factors for a layered bearing capacity problem. To further investigate the effect of footing dimensions on the ultimate bearing capacity, Figure 12 presents the ultimate bearing capacity ($q_u/B\gamma$) as a function of the aspect ratio $L/B$ for various ratios of $D/B$, $s_u/\gamma B$, and friction angles. The results have also been extrapolated out to the ultimate bearing capacity from the plain strain analysis. For illustrative purposes, an $L/B$ of 1000 has been adopted for the plain strain case (ie ideally plain strain).

Figure 12 indicates for cases where $D/B \leq 1.0$ and $L/B > 5.0$, the ultimate bearing capacity is tending to that of the plain strain solution. This indicates that in situations where $D/B \leq 1.0$ and $L/B > 5.0$, the use of shape factors to modify a plain strain solution may not be justified for a layered soil problem. Similar behaviours have been observed by Merifield (2002) for plate anchors and Salgado et al (2004) for uniform clays. Where $D/B$ = 2.0 it can be seen the aspect ratio is still having an appreciable effect on the ultimate bearing capacity ($q_u/B\gamma$). It is relevant to highlight that the shape factor $s_c$ used in BRE (2004) derived from Equation 5 for $L/B$ = 5.0 is only 1.04, indicating that the overall geometrical contribution from the clay layer is small in this instance (but only if the shape factor is valid). However, the shape factor $s_p$ from Equation 6 is 1.20 for $L/B$ = 5.0, indicating that BRE (2004) assumes that there is still an appreciable contribution from the footing geometry within the granular layer.

Finally, it can also be seen that as the relative thickness of the upper sand reduces, the influence the aspect ratio has on the ultimate bearing capacity reduces.

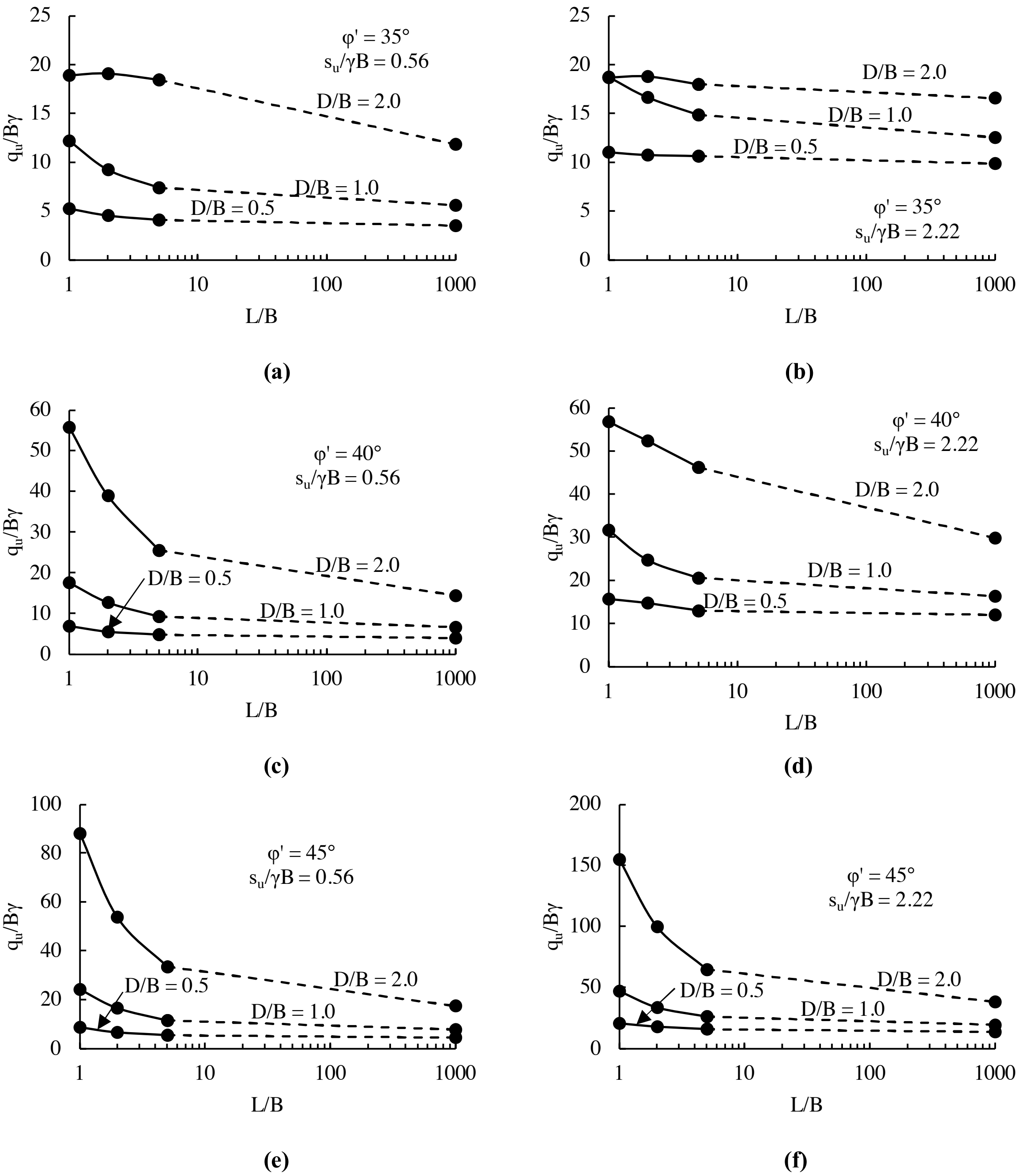



**Figure 12: Effect of footing dimensions:**
**(a) φ' = 35° and $s_u/\gamma B$ = 0.56. (b) φ' = 35° and $s_u/\gamma B$ = 2.22.**
**(c) φ' = 40° and $s_u/\gamma B$ = 0.56. (d) φ' = 40° and $s_u/\gamma B$ = 2.22.**
**(e) φ' = 45° and $s_u/\gamma B$ = 0.56. (f) φ' = 45° and $s_u/\gamma B$ = 2.22.**

An alternative way to examine the geometrical contributions and aspect ratio of a footing resting on sand over clay is to use the FELA results to derive a shape factor. Historically, shape factors have been derived by dividing the ultimate bearing capacity of a circular, square or rectangular footing by the ultimate bearing capacity of the strip footing for uniform soil problems, as defined in the equation below. A similar approach was adopted by Salgado et al (2004) for evaluating the shape factors $s_c$ for rectangular, square, and circular footings resting on uniform clay soils.

$$s^* = \frac{q_{u,square\ or\ rectangle}}{q_{u,strip}} \quad (8)$$

For the current study, Equation 8 is adopted since the contribution of the sand and clay layers to the overall bearing capacity cannot be separated in the FELA. It should be noted that this implies that the geometrical contributions to the bearing capacity from each layer are the same, however, in the authors opinion this is unlikely.

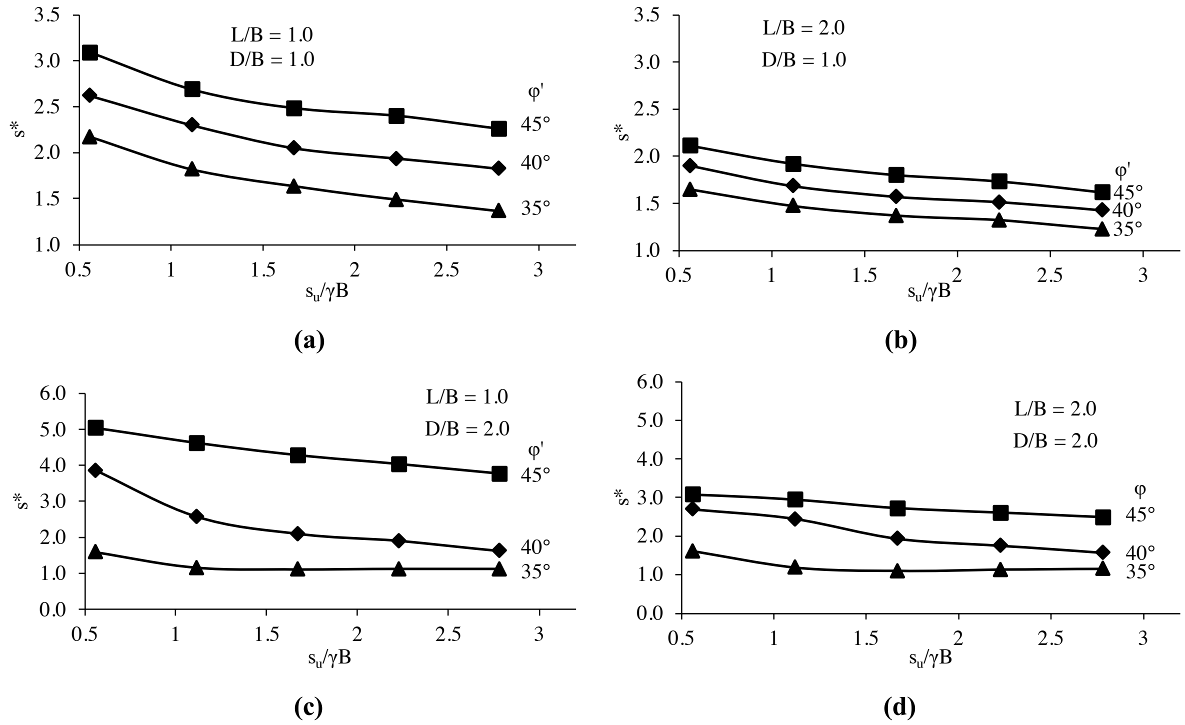



**Figure 13: Preliminary shape factors for sand over clay:**

**(a) *L/B* = *D/B* = 1.0. (b) *L/B* = 2.0 and *D/B* = 1.0.**

**(c) *L/B* = 1.0 and *D/B* = 2.0. (d) *L/B* = *D/B* = 2.0.**

It is apparent from the FELA results in Figure 13 that the shape factor s* for square and rectangular footings depends not only on the aspect ratio *L/B* and relative shear strength of the upper and lower layers, but also the relative thickness *D/B*. These results are not unexpected since many previous studies have shown for the simplest case of a footing resting on a uniform cohesionless soil, the shape factor varies according to the geometry and friction angle (Meyerhof 1963, De Beer 1970 and Zhu et al 2005). Furthermore, Salgado et al (2004) demonstrated for a uniform clay layer, the shape factor for square, circular, and rectangular footings was non-linear and varied according to footing embedment. The results of Merifield & Nguyen (2006) have also indicated the calculated bearing capacity factors for square and circular footings vary according to the relative thickness of the upper clay layer and the relative shear strength of the clay layers.

It must be noted, however, the validity of shape factors derived using Equation 8 and as presented in Figure 13 may depend on the prevailing failure mechanisms in the respective plain strain and three-dimensional analyses. Review of the results for $L/B = 2.0$, $D/B = 2.0$ and $\varphi' = 40°$ in Figure 5 (d) and Figure 13 (d) provide one example of where s* may not be valid. Figure 5 (d) shows that $q_u/B\gamma$ has converged to a constant value which indicates the underlying clay is having no appreciable effect on the ultimate bearing capacity, whereas Figure 13 (d) still indicates the shape factor is reducing. For this same circumstance, review of the plain stain results shown in Figure 8 (b) also clearly show the underlying clay is influencing the ultimate bearing capacity. In this instance, it would only seem reasonable to compare the results to a strip footing resting on a uniform layer of sand, rather than the results for the layered plain strain problem, to derive the true geometrical contribution from the footing. It is postulated that some degree of similarity should be maintained between the observed failure mechanisms from both plain strain and three-dimensional analyses in order for Equation 8 to be valid. Future studies are proposed to further investigate and attempt to better establish the importance of the prevailing failure mechanism and resulting shape factors. Consequently, it is not recommended the shape factors presented in Figure 13 are used in design.

### 3.3.4 Effect of sand thickness

As noted in Section 3.3.1, there is a critical ratio of *D/B* where the failure mechanism is constrained to the upper sand. To further investigate this critical ratio, Figure 14 presents the ultimate bearing capacity ($q_u/B\gamma$) as a function of *D/B* for various ratios of *L/B* and $s_u/\gamma B$ and friction angles. The results have also been extrapolated out to the ultimate bearing capacity for the case of a uniform sand. For illustrative purposes, a *D/B* of 1000 has been adopted for the uniform sand case. Figure 15 reproduces Figure 14 for D/*B* up to 2.0.

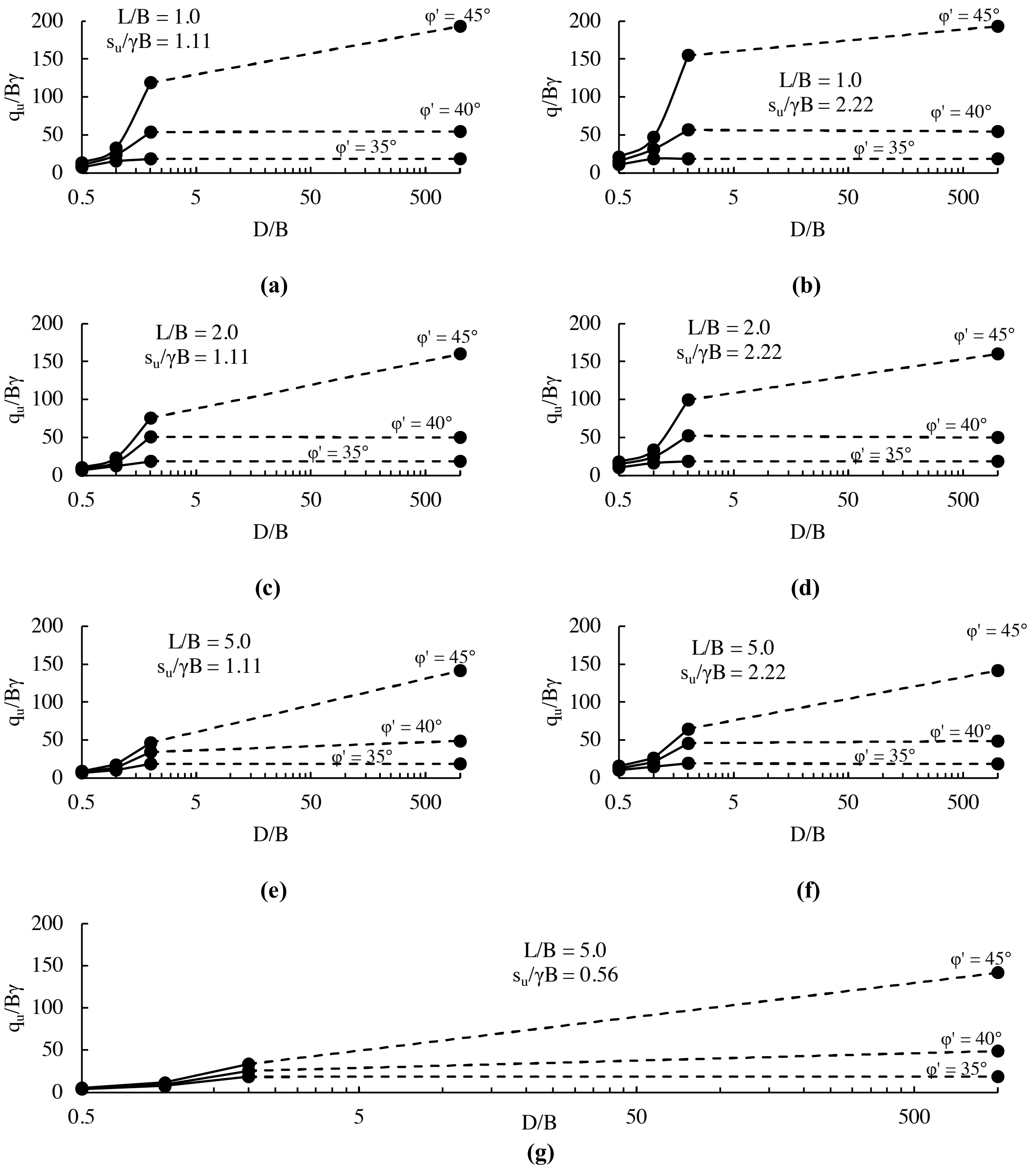



**Figure 14: Effect of sand thickness:**

**(a) $s_u/\gamma B$ = 1.11 and *L/B* = 1.0. (b) $s_u/\gamma B$ = 2.22 and *L/B* = 1.0.**

**(c) $s_u/\gamma B$ = 1.11 and *L/B* = 2.0. (d) $s_u/\gamma B$ = 2.22 and *L/B* = 2.0.**

**(e) $s_u/\gamma B$ = 1.11 and *L/B* = 5.0. (f) $s_u/\gamma B$ = 2.22 and *L/B* = 5.0.**

**(g) $s_u/\gamma B$ = 0.56 and *L/B* = 5.0.**

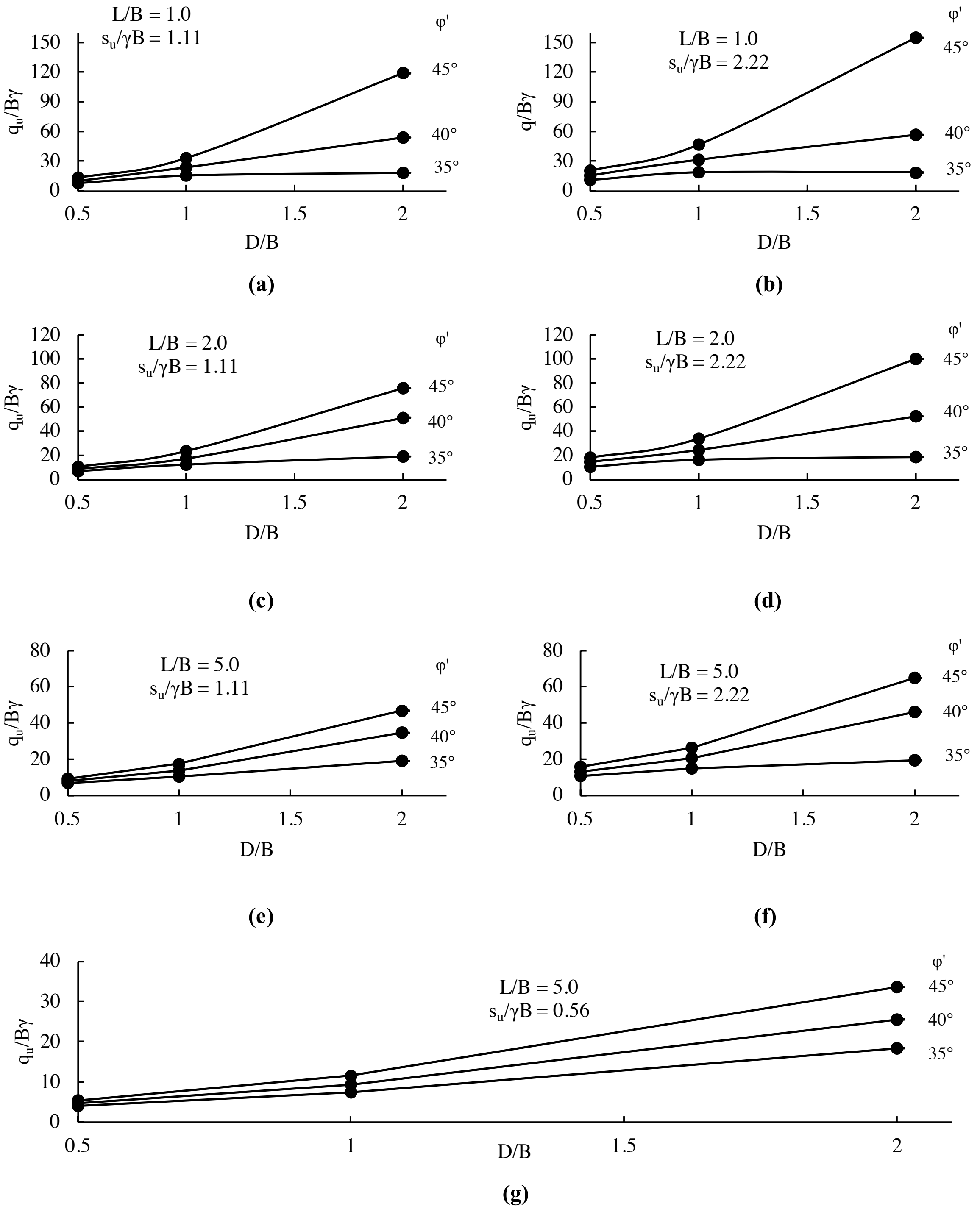



**Figure 15: Effect of sand thickness for up to D/B = 2.0**

**(a) $s_u/\gamma B$ = 1.11 and $L/B$ = 1.0. (b) $s_u/\gamma B$ = 2.22 and $L/B$ = 1.0.**

**(c) $s_u/\gamma B$ = 1.11 and $L/B$ = 2.0. (d) $s_u/\gamma B$ = 2.22 and $L/B$ = 2.0.**

**(e) $s_u/\gamma B$ = 1.11 and $L/B$ = 5.0. (f) $s_u/\gamma B$ = 2.22 and $L/B$ = 5.0.**

**(g) $s_u/\gamma B$ = 0.56 and $L/B$ = 5.0.**

Figure 14 generally indicates that when φ' ≤ 40°, the critical thickness ratio for the cases presented here is around $D/B$ = 1.5 to 2.0. This is indicated by the asymptotic value of $q_u/B\gamma$. Comparison to a Prandtl type mechanism indicates for φ' = 35° to 40°, the critical $D/B$ is in the order of about 1.52 to 1.71. However, when φ' = 45° a critical thickness has not been identified in the current study, and Figure 14 clearly indicates that the underlying clay is still having an appreciable influence on the ultimate bearing capacity. For the case of φ' = 45°, the critical thickness based on a Prandtl mechanism would be about 1.93, which is clearly much less then what would be obtained from the FELA. Where a working platform is being considered, it is obvious that the critical thickness for φ' = 45° far exceeds what would be encountered in practice.

Whilst not a clear or direct comparison, it is of interest to highlight similar behaviours have been observed for pile groups founded in a competent upper stratum underlain by a weaker layer (see Merifield et al (2021)). It can be seen that where the upper stratum is appreciably stiffer, the underlying weak layer has a greater influence on the settlement performance of the pile group.

### 3.3.5 Effect of footing roughness

To investigate the effect of footing roughness on the ultimate bearing capacity, Figure 16 presents the ultimate bearing capacity (*$q_u/B\gamma$*) as a function *$s_u/\gamma B$* for various friction angles and ratios of *D/B* and *L/B*.

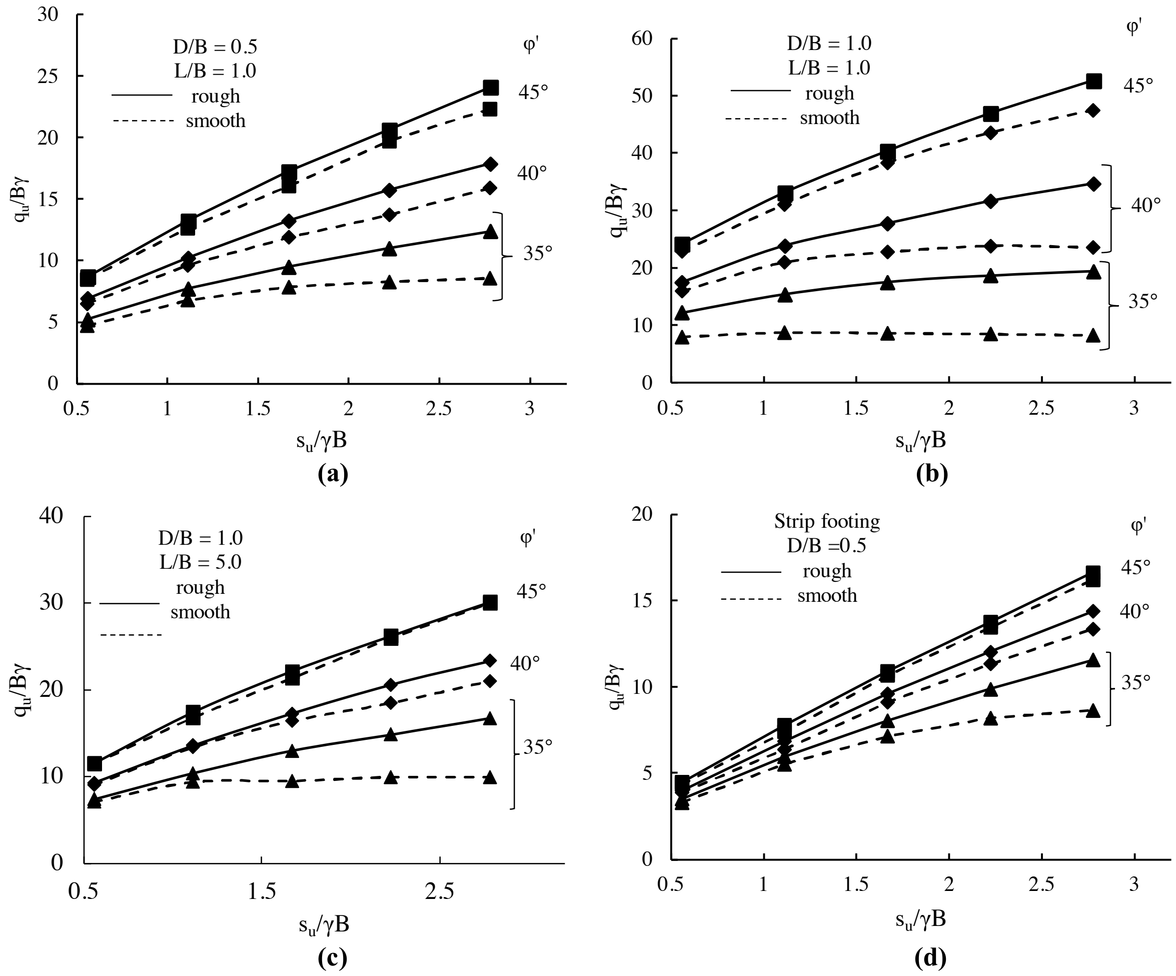



**Figure 16: Effect of footing roughness for selected cases:**

**(a) *D/B* = 0.5 and *L/B* = 1.0. (b) *D/B* = *L/B* = 1.0.**

**(c) *D/B* = 1.0 and *L/B* = 5.0. (d) *D/B* = 0.5 Strip Footing.**

Figure 16 indicates that generally when φ' ≤ 40°, the footing roughness has the most influence on the ultimate bearing capacity, although the overall impact reduces with increasing *L/B*. This result is not unexpected since the cases where the overall failure mechanism is more likely to be constrained to the upper sand layer the roughness is expected to have the largest influence (eg *L/B* = 1, *D/B*>0.5 and φ' ≤ 40°). This is consistent with previous studies after Chen (1975) and Martin (2005), where it was demonstrated that the bearing capacity factor $N_\gamma$ for a perfectly smooth footing is about half of the perfectly rough case for the case of a uniform cohesionless soil. Similar behaviours for strip footings on sand overlying clay were also observed by Shiau et al (2004).

# 4 APPLICATION OF THE SOLUTIONS TO PRACTICAL PROBLEMS

## 4.1 EXAMPLE 1 – 600 T CRAWLER CRANE USED TO INSTALL BRIDGE GIRDERS

A 600-tonne crawler crane was proposed to lift bridge girders into place. The maximum working ground bearing pressure for the crane was estimated to be a peak of 670 kPa at the toe of the 1.35 m wide tracks, which linearly reduced to zero over a length of 6 m. This load was advised to be during the crane lifting a girder and is considered commensurate with Load Case 1 (eg handling a lift) as per BRE (2004). Due to the appreciable loads, it was proposed to use 3.8 m wide load spreading mats. Accordingly, based on the methods of Meyerhof (1953) and the proposed load spread mats, the design equivalent uniform bearing pressure to consider for the working platform thickness was estimated to be about 180 kPa over a design length of 4 m. Cone penetration testing had indicated that the subgrade comprised firm clay with a typical $s_u$ of about 30 to 40 kPa. The working platform was proposed to be constructed mostly from a non-spec ballast with a particle size ranging from 20 mm to 75 mm with a skim layer of DGB20 to provide a level / trafficable surface. Placement and compaction of the ballast would comprise end dumping by truck and then track rolling with excavators.

BRE (2004) is proposed to be used to obtain a first pass of a minimum platform thickness, which can then be checked using the FELA charts for $q_u/B\gamma$. The material and geometrical parameters input to the assessment have been summarised in Table 5. Based on the non-dimensional parameters in Table 5, the results of Figure 4 (a) and (b) may be used. For convenience in assessing the bearing capacity, power laws have been fit to the relevant data from Figure 4, as shown in Figure 17. The results are summarised in Table 5.

**Table 5: Summary of input and results for 600t crane example**

| Inputs | Input Values |
|---|---|
| **Length of Load, L** | 4 m |
| **Width of Load, B** | 3.8 m |
| **Thickness of Platform, D from BRE (2004)** | First pass check ~ 1.3 m for Load Case 1 |
| ***L/B*** | 1.05 (check *L/B* = 1.0) |
| ***D/B*** | 0.34 (check *D/B* = 0.25 to 0.5) |
| **φ'** | 40° |
| **γ** | 16 kN/m$^3$ |
| $s_u$ | 35 kPa |
| $s_u/\gamma B$ | 0.58 |
| $q_u/B\gamma$ | ~ 4.1 to 7.1 (refer Figure 17) |
| $q_u$ | ~ 250 kPa to 430 kPa. Noting that *D/B* is 0.34, using linear interpolation, indicates an ultimate bearing capacity of about 315 kPa. This gives a FoS of about 1.9, which would be considered acceptable if using the BRE (2004) loading factors as a guide. |
| **Summary** | Indicating the 1.3 m thick platform is adequate, albeit slightly thicker than needed (depending on the factor of safety adopted). |

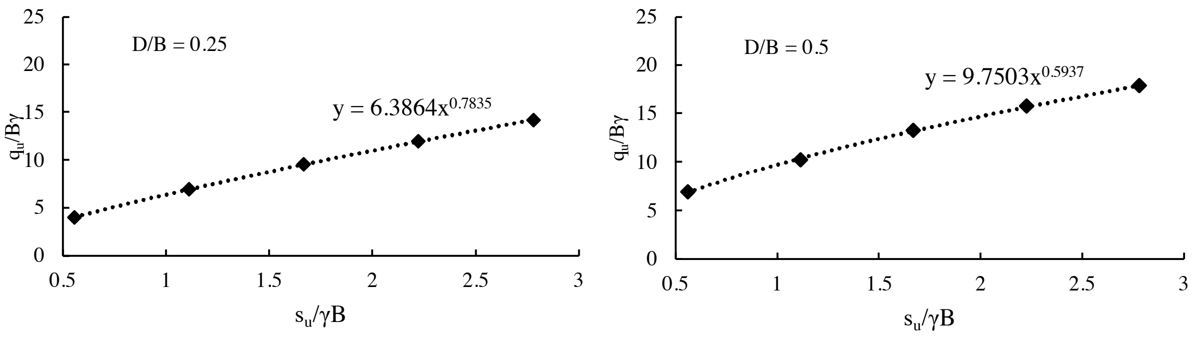



**Figure 17: Charts for Example 1**

### 4.2 EXAMPLE 2 – FUNDEX 3500 PILING RIG

A Fundex 3500 piling rig has maximum design ground bearing pressures as summarised in Table 6.

Shear vane testing indicated that the subgrade at the site comprised a soft clay with a characteristic $s_u$ of about 15 kPa. The working platform was proposed to be constructed from a well graded crushed rock (eg DGB20). Placement and compaction of the working platform would comprise end dumping by truck, spreading with excavators and compaction with a 10 tonne roller. Even though the current problem does not fall within the BRE (2004) guideline it is proposed to be used to obtain a first pass of a minimum platform thickness, which can then be checked using the FELA charts for $q_u/B\gamma$. The material and geometrical parameters input to the assessment have been summarised in Table 6.

Based on the non-dimensional parameters in Table 6, the results of Figure 5 (b) and (c) and Figure 6 (c) may be used. For convenience in assessing the bearing capacity, power laws have been fit to the relevant data, as shown in Figure 18. The results are summarised in Table 6.

**Table 6: Summary of input and results for Fundex 3500 piling rig example**

| **Inputs** | **Input Values** | |
|---|---|---|
| | **Load Case 1** | **Load Case 2** |
| **Applied Pressure** | 261 kPa | 270 kPa |
| **Design Length, L** | 3.5 m | 2.9 m |
| **Width of Load, B** | 0.9 m track width | 1.3 m padfoot width |
| **Thickness of Platform, D from BRE (2004)** | First pass check ~ 1 m (based on BRE (2004)) | First pass check ~ 1 m (based on BRE (2004)) |
| ***L/B*** | 3.9<br>(check *L/B* = 2.0 to 5.0) | 2.2<br>(check *L/B* = 2.0) |
| ***D/B*** | 1.1<br>(check *D/B* = 1.0) | 0.77<br>(check *D/B* = 0.5 to 1.0) |
| **φ'** | 45° | |
| **γ** | 21 kN/m$^3$ | |
| **$s_u$** | 15 kPa | |
| **$s_u/\gamma B$** | 0.79 | 0.55 |
| **$q_u/B\gamma$** | ~ 14.2 to 19.8 (refer Figure 18) | ~ 6.6 to 16.5 (refer Figure 18) |
| **$q_u$** | ~ 270 kPa to 370 kPa<br>This is noting that *L/B* is 3.9, using linear interpolation indicates an ultimate bearing capacity of about 330 kPa.<br>This gives a FoS of about 1.26 which would not be considered acceptable for Load Case 1 if using the BRE (2004) loading factors as a guide. | ~ 180 kPa to 450 kPa<br>This is noting that *D/B* = 0.77, using linear interpolation indicates an ultimate bearing capacity of about 325 kPa.<br>This gives a FoS of 1.20 which would be considered acceptable for Load Case 2 if using the BRE (2004) loading factors as a guide. |
| **Summary** | The proposed thickness of 1 m is not considered suitable for Load Case 1, and consideration should be given to increasing the thickness. By trial and error it can be shown that a working platform of about 1.2 m thickness would satisfy a minimum FoS of 1.6, which would normally be accepted for Load Case 1 (if using BRE (2004) load factors as a guide). | |

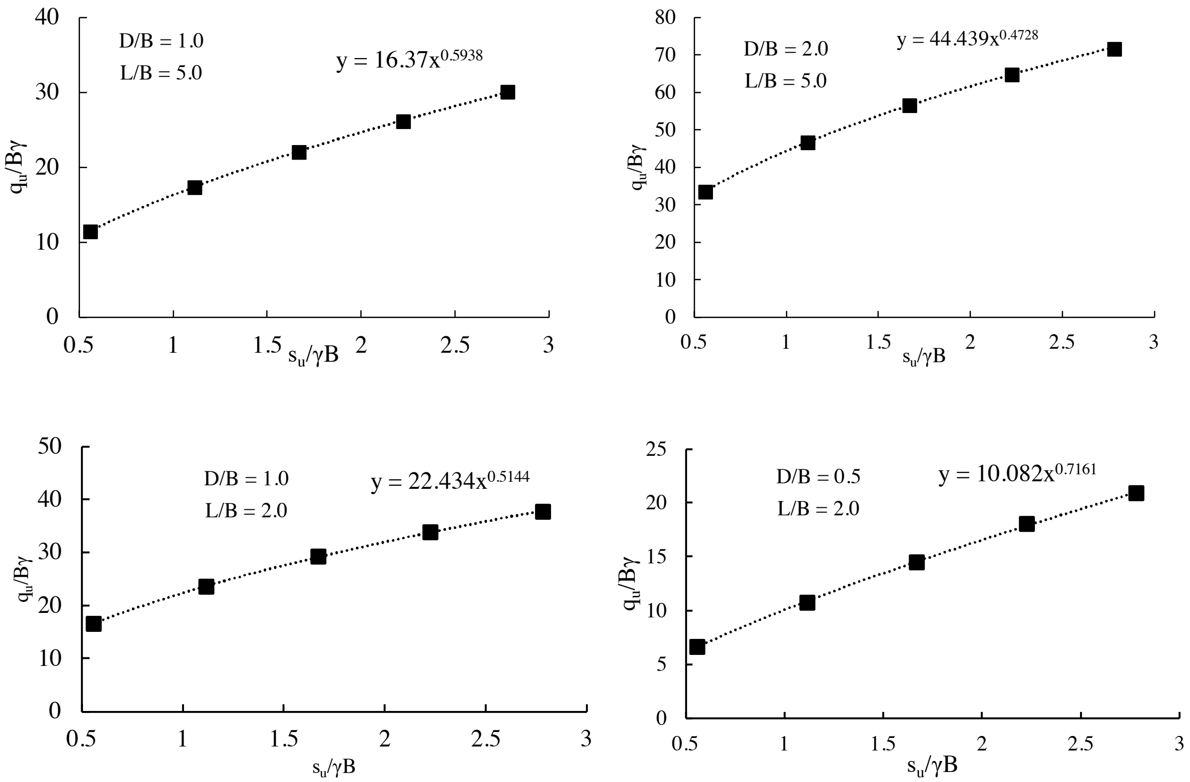



**Figure 18: Charts for Example 2**

# 5 CONCLUSIONS

This paper presents the results of a study using FELA to investigate the ultimate bearing capacity of a footing resting on sand overlying clay. Solutions for strips, square and rectangular footings were developed and, where possible, compared with other available published data and case studies. Comparisons with the available published data and case studies show good agreement and provides confidence in the results of the study.

A range of geometric and material parameters were considered in the study. The results of the parametric study were used to develop design charts where it is hoped that many of the more commonly encountered cases of working platforms in practice can be readily assessed using the design charts. The study also included sensitivity analyses to assess the influence of footing dimension, sand thickness and footing roughness.

Comparisons were made with the commonly used design guideline BRE-470 (BRE 2004). The FELA indicated that for many practical situations where a thin working platform, eg *D/B* of $\leq$ 0.5, is proposed to be constructed out of good quality fine crushed rock with φ' $\geq$ 40° over a soft to firm clay, BRE (2004) generally provides estimates of the ultimate bearing capacity within about 10 % of the FELA. However, numerous situations were presented where BRE (2004) may yield estimates of the ultimate bearing capacity that may either be potentially conservative or unconservative. BRE (2004) also indicates that the ultimate bearing capacity using a punching shear model is not appropriate when the ratio of *D/B* is larger than 1.5, however, the FELA indicates this may be less, and may more appropriately be no larger than *D/B* of about 0.5 to 1.0, although this depends on the ratio of *L/B* and the relative shear strength of the upper and lower layers. An attempt was made to derive shape factors from the FELA, however this study has indicated that it is unlikely to be a straightforward ratio between the ultimate bearing capacity from the respective two and three-dimensional analyses (ie plain strain and true rectangular loading). The FELA has indicated that in situations where *D/B* $\leq$ 1.0 and *L/B* > 5.0, the use of shape factors to modify a plain strain solution may not be justified for a layered soil problem. This highlights that caution is required when applying shape factors to layered bearing capacity problems, as is done in the BRE (2004) method.

Some simple examples of how the design charts could be used were presented. The examples included a situation where the FELA charts may be used as a cross check against BRE (2004), and a situation where the FELA charts indicated that the working platform thickness from BRE (2004) was inadequate. The FELA charts are also suited to situations where

BRE (2004) leads to an overly conservative working platform thickness, and a more efficient / sustainable thickness needs to be justified.

## 6 ACKNOWLEDGEMENTS

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