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DEVELOPMENT OF VBA FOR ASSESSMENT OF PRACTICAL ALLOWABLE PILE CAPACITY

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ABSTRACT

The needs for computer-aided tools for fast and accurate engineering computing are increasingly becoming indispensable in the ever-challenging construction industry as well as academia. In this regards, the technical demands and public expectations largely dictate the way the profession are tackled in modern Malaysia. However, these programs are often computationally opaque or complex, not customize to local contexts, expensive due to specialized target market, and potentially requiring hardware-software-specific environment. As an alternative, this paper proposed the use of Visual Basic for Application (VBA) with MS EXCEL as the parent platform and adopting analytical formulations readily familiar to Malaysian practitioners and academicians. The selection is based on the ubiquity of the said platform and established competency among users. The program developed involved determination of allowable pile capacity. Finally, the program was run and tested against a real world pile foundation problem based on geotechnical data borelog. The results were found to be in agreement with manual computational procedures, and thus confirming its veracity and applicability for the prescribed technical determination.

Keywords: pile foundation; allowable pile capacity; ultimate pile capacity; VBA programming; software.

1. INTRODUCTION

Pile foundation is a common construction method in Malaysia used to support critical structural elements. This is backed up with well established research and discoveries in this area by locally grown researchers (Ting, Chan, & Ooi, 2004). These included local contributions in understanding of pile arching mechanism (Gue, Chow, Cheah, Airina, & Zulastri, 2007), the design and construction of micropiles (Chan & Ting, 1996; Gue & Liew, 1998), foundation in limestones (Ting, 1985), piled embankment (Chin, 1985), diagnosis of pile conditions (Chin, 1978), and the method of predicting ultimate capacity of piles (Chin, 1970) among others.

On practical level, piles can be categorized into displacement and non displacement types based on the method of construction. The former constitutes piles driven or jacked into the ground such as spun piles, R.C. piles, prestressed piles and bakau piles. The latter constitues piles casted in situ following soil/rock drilling such as bored piles, micro piles and caissons.

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2. THEORETICAL FRAMEWORK

In essence, the estimation of ultimate pile capacity is difficult due to the variability of loading and site conditions. This is further aggravated by the nature of the construction practice that may inadvertently change the loading and site characteristics to the extent that may not be fully comprehended. Hussein and Goble (2000) exemplified one of the best scenario with pile under a dynamic hammer impact where it is subjected to a complex combination of compressive, tensile, torsional and bending forces.

Notwithstanding these difficulties, the determination of the ultimate pile capacity is usually treated in two parts. The first part consists of skin friction along the surface of the pile side or shaft while the second part consists of end bearing at the base or tip of the pile. Through proper allowance for the factor of safety, a working pile capacity can be determined with measurable degree of certainty against potential failure of the pile while mitigating potential detrimental influences of the indeterminate or irreconcilable factors.

In analytical context, three variants may be contemplated: semi-empirical, simplified soil mechanic and fundamental soil mechanic methods. Selection of any single analytical method or its combination relies upon the quantity and quality of available geotechnical data and information.

2.1 Factor of Safety

The appropriate factor of safety is the moderating element usually employed with the pile capacity to account for any uncertainties inherent in such determination. In principle, the ultimate capacity must exceed the applied load by a sufficient margin lest the foundation may suffer unacceptable settlement or even failure. Furthermore, the required pile capacity depends on the test method adopted for its verification and the frequency of testing.

EC7 employs the partial safety factors γ_b and γ_s for end bearing, R_{bk} and skin friction, R_{sk} resistance, respectively which are based upon the method of pile installation. The term "design bearing resistance, R_{cd}" is thus expressed in this manner:

$$R_{cd} = \frac{R_{bk}}{\gamma_b} + \frac{R_{sk}}{\gamma_s} \text{ where } \begin{cases} \gamma_b = 1.3 \text{ for driven \& bored piles} \\ \gamma_s = \begin{cases} 1.3 \text{ for driven piles} \\ 1.6 \text{ for bored piles} \\ 1.45 \text{ for continuous flight auger piles} \end{cases} \end{cases}$$

The Institution of Civil Engineers' defines "allowable pile capacity" more broadly based upon pile's bearing capacity, its material, required load factors, settlement, pile spacing, down drag, the overall bearing capacity of the underlying soil, etc. as well as the serviceability limit state of the supported structure. This corresponds to "allowable load" in BS 8004.

With due observance to the above principles, a more general expression involving allowable geotechnical pile capacity, Q_{all}, ultimate pile capacity, Q_u, ultimate end bearing, Q_b, ultimate skin friction, Q_s, global factor of safety, F, partial factor of safety for end bearing, γ_b, partial factor of safety for skin friction, γ_s and structural pile capacity, Q_{str} is provided below:

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$$Q_{all} = \text{the lesser of} \left[\frac{Q_u}{F}, \frac{Q_b}{F_b} + \frac{Q_s}{F_{s\,1}} \right] \le Q_{str}$$

2.2 Geotechnical Pile Capacity in Soils

In soils, the theoretical static Q_u may be expressed in this form (Tomlinson, 2001):

$$Q_u = Q_b + Q_s - NSF + W_2 - W_1 \Rightarrow Q_u = Q_b + Q_s \text{ - NSF if } W_2 = W_1$$

where :

$$Q_b = f_b A_b$$
$$Q_s = f_s A_s$$
$$NSF = \text{negative skin friction}$$
$$\mathrm{W}_1, \mathrm{W}_2 = \text{weight of pile \& soil displaced by pile respectively}$$
$$f_b, f_s = \text{end bearing \& skin friction resistance respectively}$$
$$A_b, A_s = \text{cross sectional area of pile base \& surface area of pile shaft respectively}$$

Based on the simplified soil mechanic method for cohesionless soils, the end bearing and skin friction resistance can be determined from the following equations which were developed by Berezantsev (1961) and Broms (1966):

$$f_b = \sigma'_{vo} N_q \le 11000\text{kN/m}^2$$
$$f_s = \sum \overline{K}_s \overline{\sigma}'_{vo} \tan\delta \le 110\text{kN/m}^2$$

where :

$$\sigma'_{vo}, \overline{\sigma}'_{vo} = \text{effective and average overburden pressure at pile base and along pile shaft respectively}$$
$$N_q = \text{bearing capacity factor (see Figure 1)}$$
$$\overline{K}_s = \text{average coefficient of lateral earth pressure (see Figure 2)}$$
$$\delta = \text{angle of friction at the pile/wall interface (see Figure 2)}$$

For cohesive soils, the end bearing and skin friction resistance can be determined as follows (developed by Skempton (1951) and Tomlinson (2001)):

$$f_b = c_u N_c$$
$$f_s = \sum \alpha \overline{c}_u$$

where :

$$c_u, \overline{c}_u = \text{specific and average undrained shear strength of soil below pile base and along pile shaft respectively}$$
$$N_c = \text{bearing capacity factor (see Figure 3)}$$
$$\alpha = \text{adhesion factor (see Figure 4)}$$

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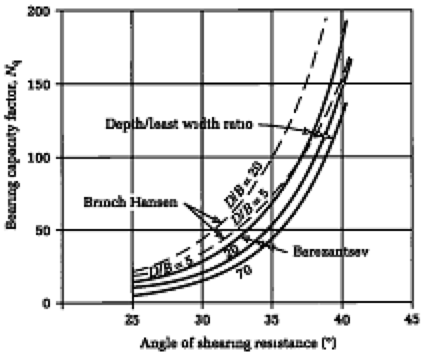
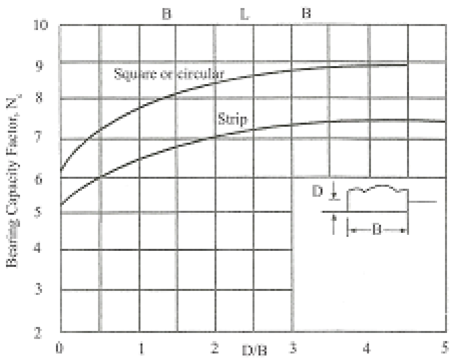


$$N_q = e^{\pi \tan\phi} \tan^2\left(45 + \phi/2\right)$$

Figure 1: Berezantsev's end-bearing capacity factors (after Tomlinson (2001)).

Pile material	δ	Values of $\bar{K}_s$ Low D_r	High D_r
Steel	20°	0.5	1.0
Concrete	3/4ϕ	1.0	2.0
Wood	2/3ϕ	1.5	4.0

Figure 2: Values of $\bar{K}_s$ and δ (after Tomlinson (2001)).



$$N_c = 5.14\left(1 + 0.2\frac{B}{L}\right)\left(1 + \sqrt{0.0053\frac{D}{B}}\right) \leq 9$$

Figure 3: Skempton's bearing capacity factors (after Tomlinson (2001)).

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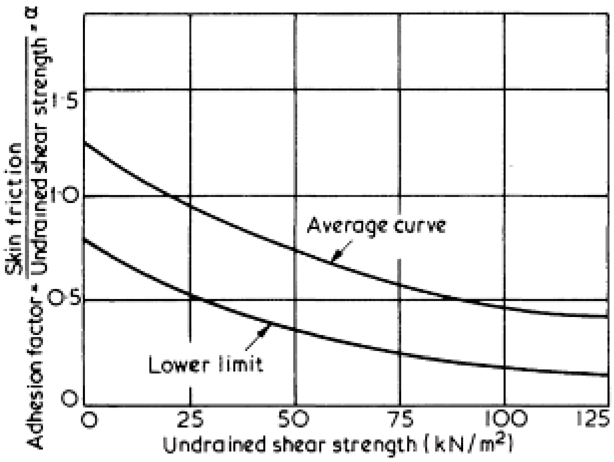

Figure 4: Relationship between adhesion factor and undrained strength of clay (after Tomlinson (2001)).

Based on semi-empirical method for cohesionless soils, the end bearing and skin friction resistance can be determined from Meyerhof (1976)'s correlations to SPT's N:

$$
\begin{aligned}
f_b &= \frac{40ND}{B} \leq 400N \text{ for } N > 10 \\
&= \frac{40ND}{B} \leq 300N \text{ for } N \leq 10 \\
f_s &= \overline{N} \text{ for average driven pile diameter } < 300 \text{ mm} \\
&= 2\overline{N} \text{ for average driven pile diameter } \geq 300 \text{ mm}
\end{aligned}
$$

where :

$N, \overline{N}$ = specific and average SPT value below pile base and along pile shaft respectively

D = embedded length of pile

B = diameter of pile

In addition for cohesive soils, the end bearing and skin friction resistance can be determined from their correlations as follows:

$$
\begin{aligned}
f_b &= N_c c_u \\
f_s &= \alpha \overline{c}_u
\end{aligned}
$$

where :

N_c = bearing capacity factor = 9

$c_u = 6N$

$\alpha = 1.00$ when $\overline{N} < 8$

$= 0.45$ when $\overline{N} \geq 8$

$\overline{c}_u = 6\overline{N}$

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The same semi-empirical method can also be used when JKR probe test was conducted (instead of the SPT) by adopting the well-known correlations between the two as published by Jabatan Kerja Raya (JKR):

Table 1: Relationship between SPT, JKR probe and unconfined compressive strength of clay.

SPT's N (Blow/ft)	Consistency	Unconfined Compressive Strength (ton/ft ^{2})	JKR Probe (Blow/ft)
0-2	Very soft	0-0.25	0-10
2-4	Soft	0.25-0.50	10-20
4-8	Medium (Firm)	0.50-1.00	20-40
8-15	Stiff	1.00-2.00	40-70
15-30	Very stiff	2.00-4.00	70-100
> 30	Hard	4.00	100

Note: 1 ton/ft ^{2} = 100 kN/m ^{2}

Table 2: Relationship between SPT, JKR probe and allowable soil pressure for sand.

SPT's N (Blow/ft)	Relative Density	Allowable Soil Pressure (ton/ft ^{2})	JKR Probe (Blow/ft)
0-4	Very loose	-	0-10
4-10	Loose	0-0.8	10-30
10-30	Medium	0.8-2.8	30-80
30-50	Dense	2.8-4.7	80-110
> 50	Very dense	4.7	110

2.3 Negative Skin Friction

Negative skin friction (NSF) occurs on pile installed through or adjacent to compressible fill or soil showing appreciable consolidation e.g. soft, lightly over consolidated or sensitive clay leading to down drag of the pile. The following semi-empirical expression can be used to estimate this value:

$$NSF = (1 - \sin\phi)\tan\phi\sqrt{OCR}$$

where :

OCR = overconsolidation ratio

2.4 Geotechnical Pile Capacity in Rocks

In Malaysia, the determination of rock socket capacity is heavily based on semi-empirical method. Generally, the movement to mobilize the base resistance is a few folds higher than that of socket resistance despite the former having greater magnitude. As such, it would be prudent to apply appropriate mobilising factors to both terms after verification through fully instrumented pile load test with the latter having the greater value. Hence, the determination

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of the base and socket resistance values can be carried out using the following expressions which were developed by Tan (2003), Williams (1981), Horvath (1978) and Rosenberg (1976):

$$f_b = cN_c + \frac{\gamma B N_\gamma}{2} + \gamma D N_q$$

$$f_s = \alpha \beta q_{uc}$$

where :

f_b, f_s = base and socket resistance respectively

c = cohesion

$N_\phi = \tan^2\left(45 + \phi/2\right)$

$N_c = 2N_\phi^{1/2}\left(N_\phi + 1\right)$

$N_\gamma = N_\phi^{1/2}\left(N_\phi^2 - 1\right)$

$N_q = N_\phi^2$

α = reduction factor with respect to q_{uc} (see Figure 5)

β = reduction factor with respect to the rock mass effect (see Figure 6)

q_{uc} = unconfined compressive strength of intact rock

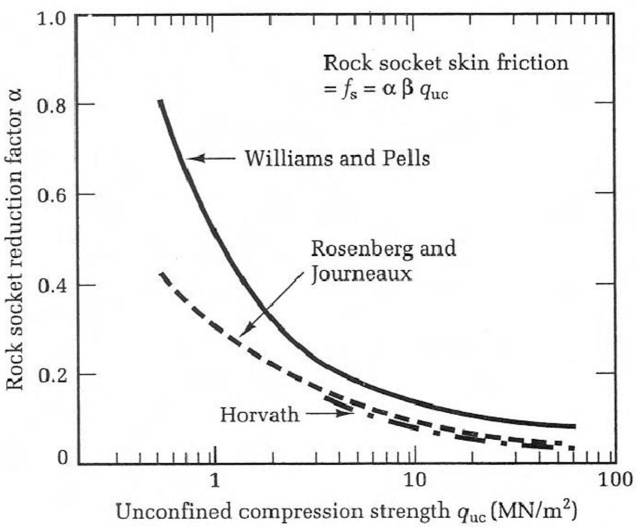

Figure 5: Rock socket reduction factor, α with respect to unconfined compressive strength (after Tomlinson (2001)).

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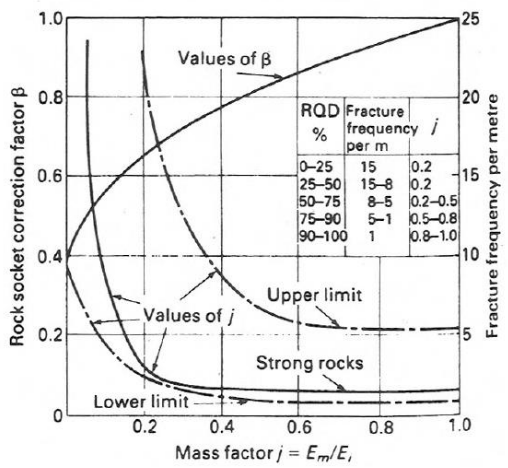

Figure 6: Rock socket reduction factor, β with respect to rock mass discontinuity (after Tomlinson (2001)).

Alternatively, Tan (2003) and Thome (1977) recommended the following values of f_s for various types of rock formation and grade:

Table 3: Summary of rock friction design values.

Rock Formation	Working Rock Socket Friction	Source
Limestone	$\left.\begin{array}{l}300\,\text{kN/m}^2 \text{ for RQD} < 30\% \\ 400\,\text{kN/m}^2 \text{ for RQD} = 30\% \\ 500\,\text{kN/m}^2 \text{ for RQD} = 40\% \\ 600\,\text{kN/m}^2 \text{ for RQD} = 55\% \\ 700\,\text{kN/m}^2 \text{ for RQD} = 70\% \\ 800\,\text{kN/m}^2 \text{ for RQD} > 85\%\end{array}\right\} \leq \text{the lesser of } [0.05q_{uc}, 0.05f_{cu}]$	Tan et al. (2003)
Sandstone	$0.01q_{uc}$	Thorne (1980)
Shale	$0.05q_{uc}$	Thorne (1980)
Granite	$1000-1500\,\text{kN/m}^2 \text{ for } q_{uc} > 300\,\text{N/mm}^2$	Tan et al. (2003)

3. SOFTWARE DEVELOPMENT

The program for the determination of the allowable pile capacity was developed using VBA scripting hosted on the standard workbook of the MS Excel application. In essence, the former provides the required computational processes on the background through subroutines and functions while the latter serves as input-output interfaces, and data and information storage. Input or request to the program is established through Excel cell entry, drop-down list selection or button click as deemed appropriate. On the other hand, output from the program is made through Excel cell entry and textbox. The program is saved into a single file with the name of PileCap7.xlsm.

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The programming procedures are illustrated below on Figure 7 via a logical flow chart. In general, the pile properties are established first through a singular selection from a catalogue of locally produced pile products contained in the program database. Then, the soil/rock depth and technical description are entered as prescribed in technical bore logs in accordance to British Soil Classification System for Engineering Purposes and typical empirical scale. At this juncture, if the profile constitutes soil material, compressibility condition of the layer needs to be indicated in order to arrive at either negative or positive skin friction values. Next input includes the measure for material weight in terms of bulk and saturated unit weights. Following this the soil, cohesion and internal friction angle values or SPT values or JKR probe test values would be entered which would dictate either semi-empirical or simplified soil mechanic analytical method shall be applied by the program in subsequent computations. In the case of rock, cohesion, internal friction angle, RQD and unconfined compressive strength values are necessary for all prescribed semi-empirical methods used by the program. The respective analytical routines would then produce the ultimate skin friction, base resistance and/or negative skin friction (if applicable) which would then be summed up to produce the geotechnical capacity of the pile with due allowance for prescribed factor of safety. The same process may be repeated for other soil/rock strata. The actual pile capacity constitutes the lesser of the said geotechnical capacity to the tip of the pile and the structural capacity of the pile material. Through a cell/range select and a button click, the detailed calculations referenced by the selection address would be produced.

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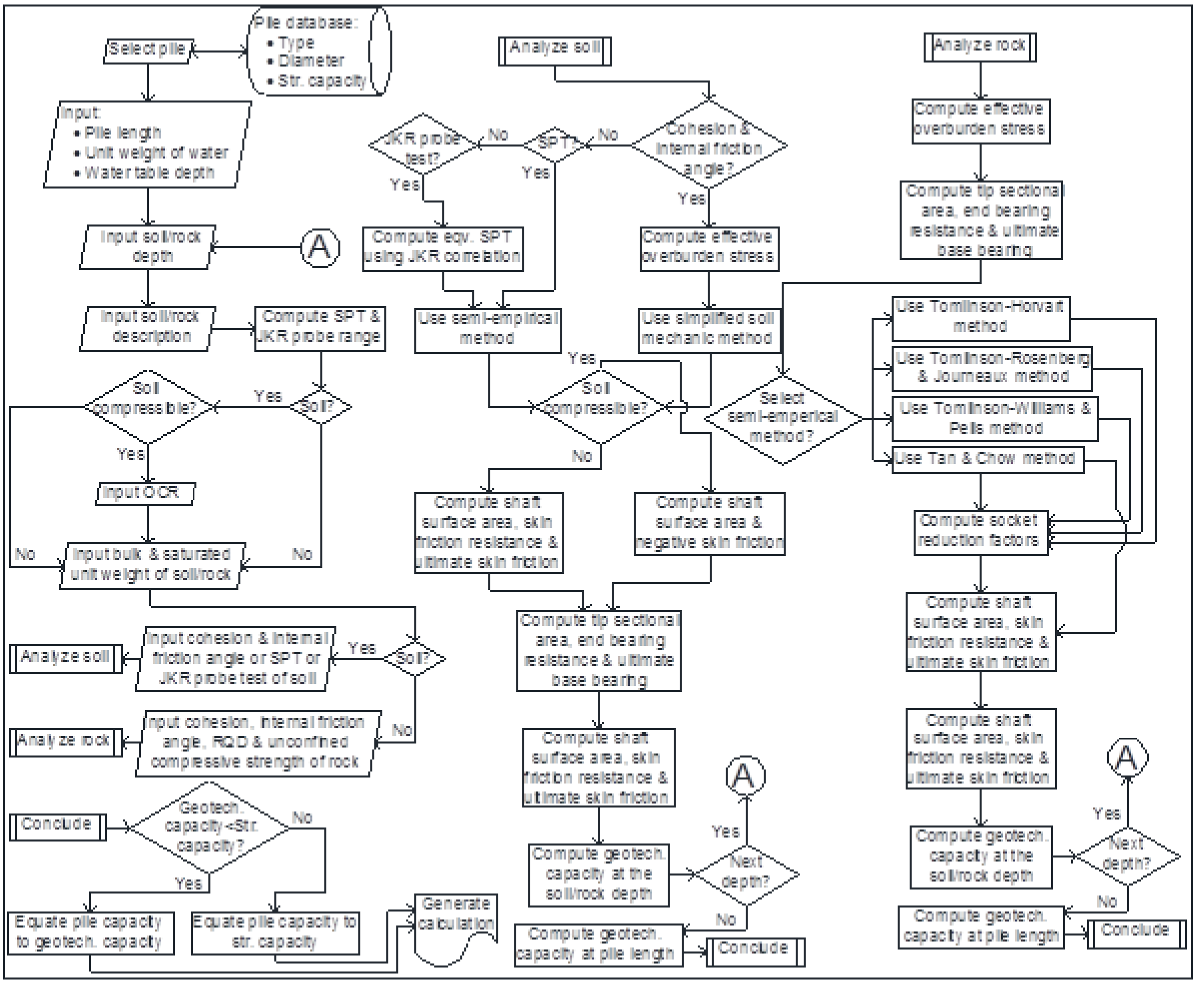

Figure 7: Flow chart of PileCap7 programming procedures.

4. CASE STUDY

A 4-storey shop/office was proposed at Burma Road, Penang and a specialist contractor was engaged to conduct soil investigation works for the said project in order to determine the subsoil conditions through soil/rock sampling and testing, and ground water regime. The land was relatively flat and accessible on the surface, and two exploratory boreholes referenced BH-1 and BH-2 were advanced using rotary wash boring technique. Standard Penetration Test (SPT) was conducted at regular interval of 1.5 m in conjunction with disturbed sampling using a split-spoon sampler. The resulting samples were primarily used for visual examinations and classification tests in the laboratory. Undisturbed samples were also randomly taken using thin wall sampler primarily for the determination of the shear strength and compressibility of soil in the laboratory. Figure 8 illustrated some of the findings.

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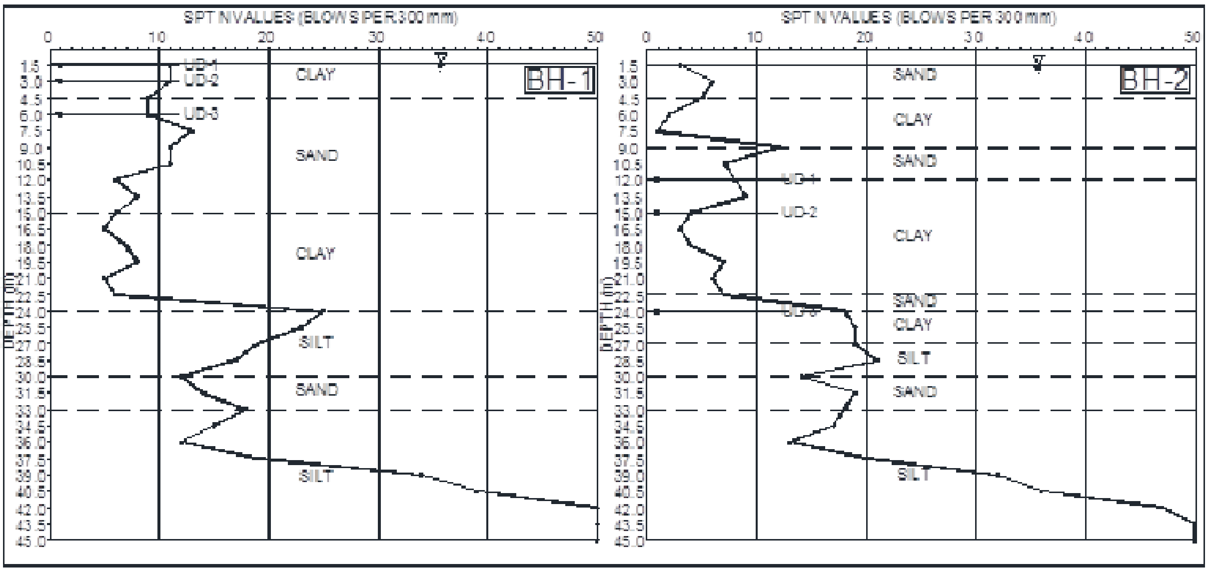

Figure 8: Deduced sub-soil profiles from the soil investigation works.

The results of the tests indicated that BH-2 had a more critical condition than BH-1. In general, alluvium layer of clay or silt was present close to the surface (N<10), followed by residual soil of silt derived from the weathering of parental granitic rock (10≤N≤50). The thickness of the former ranged from 23 m to 32 m while the latter ranged from 12 m to 21 m. However, the holes were not advanced much further beyond this point in order to determine the location of bedrock with reasonable certainty, but can be postulated to reside at the depth of which was too great to be mobilized effectively and thus provided minimal contribution. The ground water table stabilized at about 1.30 m to 1.50 m below the ground level. In the laboratory, the consolidated undrained triaxial test which was conducted on an undisturbed sample obtained from BH-2 at 15 m deep indicated a modest value of cohesion, c' and internal friction angle, ø' of 7 kN/m² and 20° respectively. Furthermore, the oedometer test that was conducted on three-undisturbed samples from BH-1 at 3 m deep, and BH-2 at 12 m and 24 m deep indicated a high value of compression index, C_c at 0.397, 0.378 and 0.214 respectively at relatively low imposed stresses. Lastly, the indicated bulk unit weights for soil showed loosely structured disparity probably based on soil type and densification. For simplification, the following values shall be adopted for bulk and saturated unit weights respectively: with clays and silts, 15 kN/m³ and 15.5 kN/m³ for N ≤4, 16 kN/m³ and 16.5 kN/m³ for 4<N≤8, 17 kN/m³ and 17.5 kN/m³ for 8<N≤15 and 18 kN/m³ and 18.5 kN/m³ for N>15; with sands and gravels, 19 kN/m³ and 19.5 kN/m³ for N ≤10 and 20 kN/m³ and 20.5 kN/m³ for N >10.

Based on these findings, it can be deduced that the upper alluvium layer is highly compressible with low bearing capacity and potentially excessive settlement for any structural elements resting on it. The high water table and low permeability would also suggest a costly and prolonged ground improvement exercise should shallow foundation be attempted. The only practical foundation solution for the moderate-scale project would involve the use of pile specifically of driven type which derives its load-carrying capacity from the skin friction and partly through the end bearing. However, allowance should be made for the highly probable negative skin friction build up due to the excess ground-to-pile settlement.

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Due to the scarcity of laboratory data, the only reasonably representative information for entries into the program would involve the use of SPT's N values. The 250 mm diameter spun pile was specified with structural capacity of about 458 kN. Excessive compressibility resulting in negative skin friction was stipulated for silts and clays with very soft to firm conditions (i.e. N≤8) or sands & gravels with very loose to loose conditions (i.e. N≤10); in both cases OCR value of 1.0 was used. The required length of pile based on data from BH-1 was 42 m with the pile capacity of about 319 kN. However, data from BH-2 did not provide satisfactory solution for all available piles due to inadequate exploratory depth. These conclusions were verified through manual computations.

5. CONSLUSION AND SUGGESTION FOR FUTURE RESEARCH

The research has shown that Excel-based VBA is a suitable platform for repetitive and mentally challenging computations when performed unaided. This work should be continued to incorporate more pile analytical options as well as technical correlations and models to make data entry and application usage as a seamless and intuitive process as possible.

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