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# An Investigation on actual Soil Skin Friction capacity of CIB Piles- Case study: Proposed Forty Two Storied Building Project, Colombo 03

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**Abstract** – *Cast in-situ Bored (CIB) piles in Sri Lanka are very often designed considering only the end bearing capacity, neglecting the soil and rock skin friction. This causes foundations to become very uneconomical. The study presented here was done as a case study, where the subsurface does not contain any compressible soil layers, which subsequently cause to buildup negative friction forces on piles during its consolidation process. Three different design methodologies were adopted and compared with field load test values to assess, which best simulate the realistic conditions. The Burland method, ICTAD method and O'Neil & Reese method were used to calculate the theoretical soil skin friction levels, whilst Williams and Pells method was used to calculate the skin friction in the rock socket. Both High Strain Dynamic Test (using Pile Dynamic Analyzer (PDA)) and Static Load Test (SLT) results were used to interpret the actual field skin friction values, keeping in mind about the relative merits and demerits of these techniques. It was revealed that the results obtained during field load tests are substantially higher when compared to the theoretical results obtained through all three methods. However, O'Neil & Reese method in combination with Williams and Pells method provides substantially higher values compared to other two, which are the most widely used methods in local pile design practice. Hence the most appropriate method of calculating soil skin friction is O'Neil & Reese method in local context compared to other two methods. The reason behind the large discrepancy between theoretical values and field load test values may be due to two reasons, viz., (a) soil parameters obtained from in-situ test results with the help of standard charts and tables do underestimate local subsurface conditions and (b) the methods used to calculate the rock socket friction highly underestimate the locally available high grade-high strength metamorphic bedrock conditions.*

***Keywords**: Cast In-Situ Bored Piles, High Strain Dynamic Test, Skin Friction, Standard Penetration Test 'N' value, Static Load Test*

## Nomenclature

| | | | |
|---|---|---|---|
| *CIB* | Cast In situ Bored Piles | *PDA* | Pile Dynamic Analyzer |
| *EB* | End Bearing | *SLT* | Static Load Test |
| *SF* | Skin Friction | | |

## 1 INTRODUCTION

Cast In-Situ Bored (CIB) Piles are widely used in Sri Lanka as foundations to support heavily loaded structure like high rise buildings, bridges, flyovers and towers. In most cases design engineers tend to follow the design parameters in the site investigation reports rather than going from the basics, mainly because of lack of access to latest engineering foundation design practices and construction methodologies and lack of confidence in the time effects of bentonite filter cake around the pile (Thilakasiri, 2006). In most cases it has been revealed that these design parameters underestimate the local subsurface conditions. Owing to above factors, CIB piles are very often designed considering only the end bearing (EB) capacity, neglecting the skin friction (SF) capacity.

When the compressive strength of the bedrock in the coastal zone, closer to Colombo is considered, the experimental compressive strength values are very much higher than those used in soil reports to estimate the end bearing capacity. For example, average UCS of moderately weathered rock core samples in CCR project is 45 MPa, OZO Colombo Hotel Project is 60 MPa and Shangri-la Hotel Project is 81 Mpa. In contrast, recommended end bearing capacity for moderately weathered bedrock is around 3-5 MPa as given by most of the soil investigations reports.

As the pile is loaded axially and forced to move downwards, the first type of resistance it has to undergo is the skin friction (Bowles, 1997 and Tomlinson et al, 2008). The skin friction is activated under very small displacement (Tomlinson et al, 2008) and its magnitude depends on the strength properties of the surrounding soil, method of installation of the pile, and the properties of the pile surface etc. (Poulos and Davis, 1980).In addition, the use of the bentonite slurry during the drilling process has a significant impact on the mobilized ultimate skin frictional resistance during loading as a filter cake formed on the drilled wall of the pile bore, which considerably reduces the mobilized skin frictional resistance (Tomlinson et al, 2008).

Skin friction of a pile includes two components, namely frictional resistance and the adhesive resistance. Skin friction is mobilized in both cohesive and cohesionless soils and can be calculated by various methods.

In order to estimate the skin friction, certain engineering parameters must be obtained, such as unit weight, shear strength and consolidation properties in addition to the lateral soil pressure coefficients. In most cases these parameters are obtained from correlations with in-situ test results (in most cases the SPT’N’ value). However, when the bases of these correlations are investigated, it is found that they are developed in countries where more unfavourable geotechnical conditions exist compared to local context. Apart from few low land areas, the subsurface is composed of residual soils in most parts of Sri Lanka. These residual soils are of sub-rounded to angular grains of mostly quartzite in nature, which always generate much higher skin friction than the soils in the subsurface conditions where these correlations have developed.

Secondly the underestimation of local geotechnical conditions is created in the method of calculating the skin friction capacity. As mentioned above these methodologies and related equations (empirical equations related to in-situ test results) have developed for more poor subsurface conditions compared to local context. Therefore, authors believe that, it is worthwhile to look into the possibility of using much higher skin friction capacity levels than the levels currently being used, which will enable the use of reduced pile diameters and hence more economical design.

### 1.1 Objectives

This study was carried out to achieve the following objectives.

a. To investigate the generated skin friction distribution along the pile using different theoretical skin friction calculation methods.
b. To analyze the suitability of above theoretical concepts by comparing with the field load test results.
c. To identify the most appropriate method of calculating the skin friction distribution of a pile under local context.

## 2 METHODOLOGY

Following methodology was adopted in this study to achieve the above mentioned objectives.

- Determination of engineering parameters of different soil strata with the use of in-situ test data.
- Calculation of soil skin friction, rock socket skin friction and end bearing capacity for the closest pile to the particular borehole using three (03) different theoretical concepts.
- Determination of actual soil skin friction and end bearing capacity levels with the use of field load test results.
- Comparison of results under (1) and (2) and finding the most appropriate theoretical method of calculating soils skin friction for CIB piles for local context.

### 2.1 Determination of engineering parameters from borehole log data

Engineering parameters were determined from standard correlations between in-situ test results and particular engineering parameters or using standard tables and charts (Tomlinson and Boorman, 1995).

The subsurface under the study are composed of sands/silty sands and do not contain any compressible soils (Geotech, 2003). Hence, in-situ test results of Standard Penetration Test (SPT) '**N**' values (Geotech, 2003) were used to determine the engineering parameters of different soil strata. These SPT'**N**' values were initially corrected using Equation 01 and then these corrected 'N' values (Bowles, 1997) were used in determining engineering parameters.

$$N_{\text{corrected}} = N_{\text{field}} C_N \eta_1 \eta_2 \eta_3 \eta_4 \qquad (01)$$

Where;

$C_N$, $\eta_1$ , $\eta_2$, $\eta_3$ and $\eta_4$ are the correction factors for overburden, hammer energy, rod length, sampler and the borehole diameter respectively.

### 2.2 Calculation of theoretical skin friction

Following three methods were employed in calculation of theoretical soil skin friction, where all three methods are valid only for cohesionless sandy soils.

1. Burland Method
2. ICTAD Method
3. O'Neill and Reese Method

The skin friction capacities in rock socket region were calculated using William and Pells method.

### 2.2.1 Burland Method

For coarse grained soils Burland, 1973, (Bowles, 1997)proposes that the ultimate shaft resistance ($f_s$) on bored piles in coarse grained soils at a point can be expressed in terms of effective stress as depicted in equation 02.

$$f_s = \beta\sigma_v' \tan\delta \quad (02)$$

$$\beta = (1 - \sin\phi) \quad (03)$$

$$\delta = 0.75\phi \quad (04)$$

Where;

$f_s$- ultimate shaft unit side resistance (skin friction) at a point on the pile in kPa.

$\sigma_v'$- effective vertical stress along the pile

$\phi$- angle of internal friction of soil

$\beta$- shaft resistance factor for coarse grained soils.

According the Equation 02, the unit soil skin friction increases with depth. However this is only up to a certain depth called ***critical depth ($z_c$)*** and beyond which the imposed soil skin friction value will be constant. The critical depth was calculated using Equation 05 and chart proposed by Meyerhof, 1976 (Poulos and Davis, 1980), which provides $z_c/d$ ratio for corresponding ($\phi''$) and ('d'), the pile diameter.

$$\emptyset'' = \phi - 3 \quad \text{for CIB piles} \quad (05)$$

Here, $\phi$ and $\emptyset''$ are expressed in degrees.

### 2.2.2 ICTAD Method

This is one of the simplest methods that can be used to evaluate skin friction of bored piles. In this method skin friction totally depends on the SPT 'N' values and hence the variation of skin friction along the pile shaft reflects the variation of SPT 'N' values. This is an extended version of Meyerhof, (1956, 1976) and Shioi and Fukui, (1982)(Bowles, 1997). The unit ultimate shaft resistance of bored piles was estimated using equations (06) and (07) (ICTAD, 1997).

$$f_s = 1.3 * N_{corr} \quad (06)$$

$$f_r = 2.0 * N_{corr} \text{ and } f_r < 200kPa \quad (07)$$

Where;

$f_s$ and $f_r$- unit ultimate skin friction of soils and rock respectively in kPa

$N_{corr}$ - corrected average SPT'**N**' value

### 2.2.3 O'Neill and Reese Method

O'Neill and Reese, 1999(Seavey and Ashford, 2004) is one of the methods that most commonly used in practice in most parts of the world. Here, the skin friction was estimated using equation (08).

$$f_s = \beta_i \sigma'_{vm} \tag{08}$$

(i) For $N_{corr} \geq 15$ ;

$$\beta_i = 1.5 - 0.245 * Z_i \tag{09}$$

(ii) For $N_{corr} < 15$ ;

$$\beta_i = \{(N_{corr}/15) * [1.5 - 0.245 * Z_i^{0.5}]\} \tag{10}$$

Where;

$f_s$ - ultimate shaft unit side resistance (skin friction) in kPa

$\sigma'_{vm}$ - effective vertical stress at the midpoint of the particular soil layer

$\beta_i$ - dimensionless factor calculated from Equations (09) and (10) for sands

$N_{corr}$ - corrected average SPT'**N**' value

$Z_i$ - vertical distance from the ground surface to the middle of the soil layerin meters

### 2.2.4 Skin Friction in Rock

The skin friction capacity in rock socketed area ($f_{rs}$) was determined using the method proposed by William and Pells (1981) (Tomlinson and Woodward, 2008), using the relationship in Equation (11).

$$f_{rs} = \alpha \beta q_{uc} \tag{11}$$

Where;

$q_{uc}$ - unconfined compressive strength of rock in socketed area in the rock.

$\alpha$ - rock socket reduction factor from the chart $\alpha$Vs$q_{uc}$

$\beta$ - rock socket correction factor from the chart $\beta$Vs$j$

$j$ - mass factor Hobbs, 1975 from the chart fractures/m Vs$j$

These charts are given in Tomlinson and Woodward, 2008.

### 2.2.5 Calculation of end bearing resistance on Pile in Rock

Allowable end bearing capacity was obtained using chart for allowable bearing pressures for metamorphic rocks given in BS 8004:1986 clause 2.2.2.3.1 figure 1 (b) (BSI, 1998).

## 2.3 Field Testing of Piles

Field testing of piles is done with the use of basically two types of Pile load capacity testing in Sri Lanka, viz.(Thilakasiri, 2009).

1. High Strain Dynamic Test(HSDT) using Pile Driving Analyzer (PDA)
2. Static Load Test (SLT)

### 2.3.1 High Strain Dynamic Testing of Bored Piles (PDA)

Both the skin frictional and end bearing components of the developed resistance on a CIB could be estimated using the dynamic testing of piles using Pile Driving Analyzer (PDA). The PDA is both a field data acquisition unit as well as a computer unit for onsite data assessment. The CAPWAP computer software allows full and accurate analysis of the PDA field data. CAPWAP model is a match curve of computed pile top force to the measured pile top force time record. It is capable of providing total computed soil capacity, sum of skin friction and end bearing. Furthermore, it provides the skin friction force for the pile and its distribution along the pile shaft and pile toe bearing capacity can be obtained separately. It also computes Load Vs Settlement curve (Thilalkasiri, et.al. 2006).

In this study, the processed CAPWAP data for the particular pile tests were directly used. The data were constituted of skin friction distribution along the pile and the end bearing component separately (Geotech, 2006).

### 2.3.2 Static Load Test (SLT)

A constant axial load is applied on a pile for a predetermined time interval and the settlement is measured. This load is increased incrementally, generally up to 150% of the working load. The load shall be measured by a load-measuring device and by calibrated pressure gauges included in the hydraulic system (Thilalkasiri, et.al. 2006).

Thevariation of load-settlement with time was obtained from the particular pile test reports (Geotech, 2006) and then this data were processed to determine the skin friction and end bearing components of particular load tests. Van Weele, 1957 method (Bowles, 1997) was used in finding the skin friction and end bearing capacity levels of particular shafts. The applicability of this procedure has been tested for local context (Thilalkasiri, 2006) and has proven to be matching the results with CAPWAP results. The load-settlement curve interpretation is depicted in Fig.1.

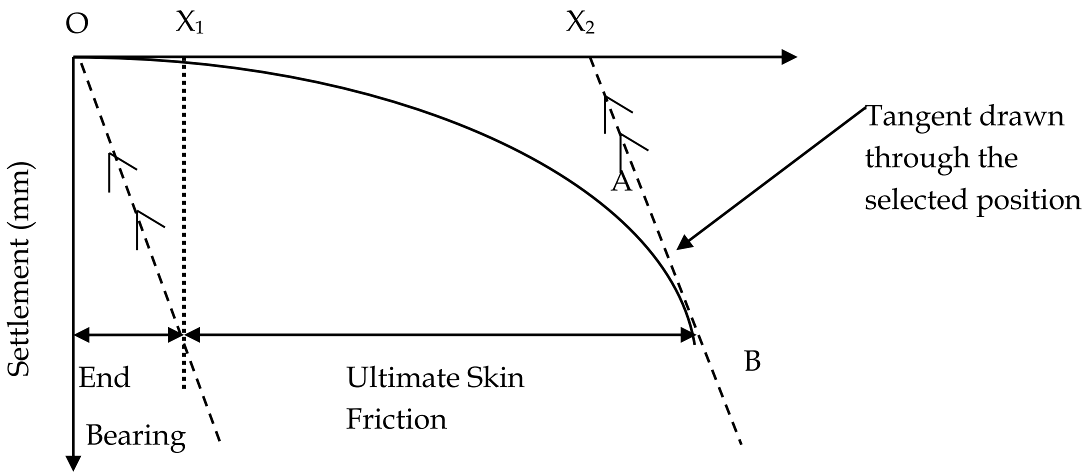


**Fig. 1 Load vs. Settlement behavior proposed by Van Weele**

The ultimate end bearing resistance is equal to $OX_1$.

| | | | | |
|---|---|---|---|---|
| Ultimate Skin Friction Capacity | = | Value of the tangent at B ($OX_2$) | - | Value of the parallel line through (0, 0) to the tangent at B ($OX_1$) |

## 2.4 Data collection

Data for the analysis were collected from pile load test results of the particular project (Geotech, 2006). Details of Pile Numbers are presented in Table 1.

**Table 1 PDA tested piles against total number of piles and respective pile diameters.**

| **Pile Dia.(mm)** | **1800** | **1500** | **1200** | **900** | **600** |
|---|---|---|---|---|---|
| **Number of piles** | 90 | 55 | 25 | 7 | 5 |
| **Test piles** | 35 | 7 | 3 | - | - |

Altogether data of 15 boreholes were available for the particular site and there were 182 CIB piles of diameters ranging from 600mm to 1800mm (Table 1). Out of those 182 piles, 41 piles were tested using high strain dynamic tests (HSDT) using PDA (Pile dynamic Analyzer). Three (03) SLTs (Static Load Test) have been performed on three selected piles. The details of the tested piles in reference to borehole locations are given in Table 2, which were collected from soil investigation reports (Geotech, 2003)

**Table 2 Details of load tested piles with respect to the nearest boreholes**

| **BH-1** | **BH-2** | **BH-3** | **BH-4** | **BH-5** | **BH-6** | **BH-A** | **BH-B** | **BH-G** | **BH-I** | **BH-J** |
|---|---|---|---|---|---|---|---|---|---|---|
| P012 | P050 | P101 | P132 | P040 | P072 | P015 | P010 | P131 | P088 | P042 |
| P020 | P049 | | P151 | P042 | | | | P142 | P093 | P045 |
| P014 | P046 | P114 | P133 | P061 | | | | P148 | P087 | P033 |
| P003 | | P123 | P149 | P074 | | | | | P78b | P032 |
| P004 | | P126 | P150 | | | | | | P094 | |
| P087 | | P124 | | | | | | | P78A | |
| P013 | | P130 | | | | | | | P064 | |
| P023 | | P121 | | | | | | | | |

# 3 RESULTS AND ANALYSIS

## 3.1 Theoretical skin friction capacities

The theoretical ultimate skin friction on each pile was calculated using the methods mentioned under section 2.3, with the help of filed in-situ test values (SPT'**N**') mentioned under section 2.2. Later safe skin friction values, as shown in Table 3, were obtained by factoring the ultimate soils skin friction by a factor of safety of 3.0 and ultimate rock socket skin friction by a factor of safety of 2.5. Higher factor of safety in obtaining safe soil skin friction was used mainly because of uncertainties involve in the adjacent smear zone and the bentonite cake that forms around the pile borehole.

**Table 3 Skin friction (SF) capacity levels acting on piles using theoretical methods**

| Pile | Reference Borehole | Depth of Pile (m) | Safe soil skin friction (kN) | | | Safe Rock socket friction (kN) |
|---|---|---|---|---|---|---|
| | | | Burland | ICTAD | O'Neil &Reese | |
| P020 | BH-01 | 29.10 | 2250 | 2198 | 6508 | 791 |
| P013 | BH-01 | 28.35 | 1438 | 1319 | 4652 | 528 |
| P014 | BH-01 | 27.54 | 1377 | 1198 | 4354 | 528 |
| P004 | BH-01 | 30.00 | 2342 | 2347 | 7110 | 791 |
| P023 | BH-01 | 28.19 | 1520 | 1281 | 4672 | 528 |
| P003 | BH-01 | 29.70 | 2348 | 2387 | 7357 | 791 |
| P012 | BH-01 | 28.90 | 1875 | 1802 | 5900 | 659 |
| P050 | BH-02 | 28.19 | 2074 | 1834 | 6941 | 371 |
| P049 | BH-02 | 29.63 | 2485 | 2316 | 8453 | 371 |
| P046 | BH-02 | 31.05 | 2444 | 2565 | 8526 | 371 |
| P124 | BH-03 | 28.55 | 1943 | 2356 | 8069 | 661 |
| P101 | BH-03 | 28.80 | 3017 | 2615 | 10032 | 661 |
| P121 | BH-03 | 30.05 | 3275 | 2715 | 10317 | 661 |
| P114 | BH-03 | 26.20 | 2873 | 2263 | 10687 | 992 |
| P140 | BH-03 | 27.80 | 3007 | 2413 | 9928 | 827 |
| P126 | BH-03 | 27.85 | 3141 | 2564 | 10317 | 992 |
| P109 | BH-03 | 31.23 | 3543 | 2564 | 10388 | 661 |
| P151 | BH-04 | 27.90 | 1959 | 1681 | 5783 | 930 |
| P130 | BH-04 | 28.50 | 1952 | 1681 | 5975 | 1116 |
| P132 | BH-04 | 27.87 | 2342 | 2000 | 5673 | 1116 |
| P133 | BH-04 | 27.20 | 1875 | 1522 | 4728 | 1116 |
| P150 | BH-04 | 29.35 | 1963 | 1681 | 5975 | 930 |
| P074 | BH-05 | 30.15 | 1678 | 1636 | 5236 | 715 |
| P040 | BH-05 | 29.60 | 2308 | 2039 | 6779 | 858 |
| P061 | BH-05 | 30.83 | 2411 | 2209 | 6525 | 858 |

**Table 3 (Cont.)**

| Pile | Reference Borehole | Depth of Pile (m) | Safe soil skin friction (kN) | | | Safe Rock socket friction (kN) |
|---|---|---|---|---|---|---|
| | | | Burland | ICTAD | O'Neil &Reese | |
| P015 | BH-A | 23.75 | 1532 | 1402 | 5482 | 848 |
| P010 | BH-B | 29.40 | 2241 | 2515 | 6807 | 936 |
| P165 | BH-B | 29.35 | 1620 | 1622 | 5265 | 520 |
| P166 | BH-B | 31.45 | 1498 | 1442 | 5392 | 520 |
| P131 | BH-G | 27.10 | 2242 | 1999 | 7837 | 936 |
| P148 | BH-G | 27.50 | 2346 | 2109 | 7595 | 936 |
| P088 | BH-I | 29.70 | 2525 | 2368 | 7860 | 834 |
| P093 | BH-I | 27.30 | 2219 | 1969 | 7338 | 834 |
| P078 | BH-I | 28.82 | 2421 | 2194 | 7343 | 834 |
| P094 | BH-I | 30.20 | 2522 | 2368 | 7856 | 834 |
| P064 | BH-I | 29.65 | 2624 | 2495 | 6808 | 834 |
| P042 | BH-J | 29.98 | 2624 | 2084 | 6808 | 780 |
| P032 | BH-J | 28.40 | 2168 | 1798 | 5227 | 780 |
| P033 | BH-J | 26.60 | 2159 | 1472 | 5086 | 936 |
| P045 | BH-J | 30.50 | 2440 | 1876 | 7413 | 936 |
| P043 | BH-J | 30.70 | 2527 | 2021 | 5550 | 936 |

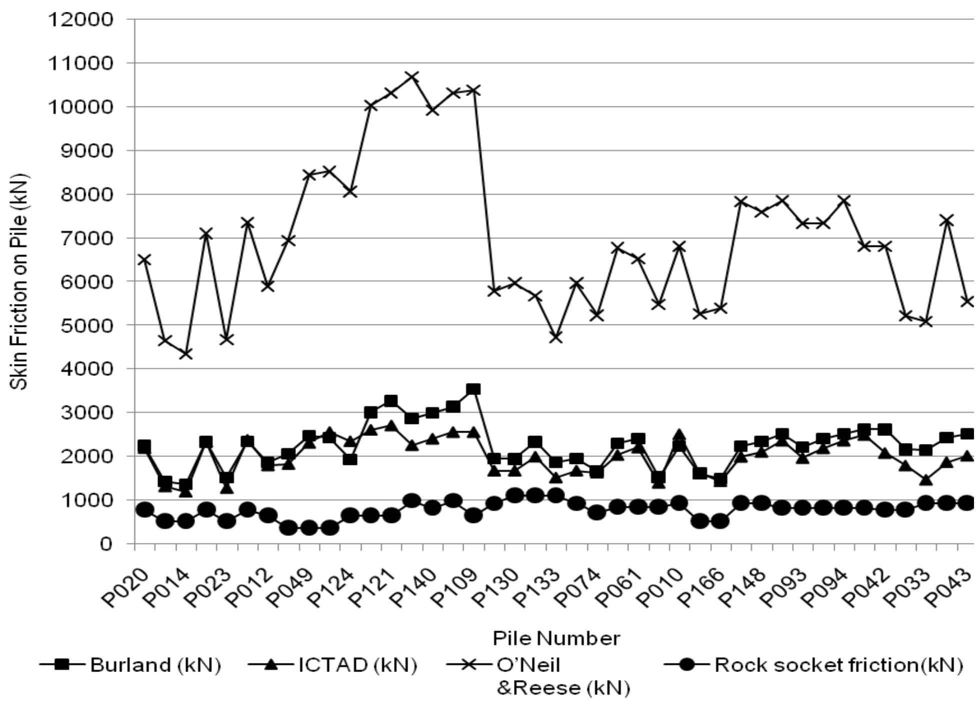


**Fig. 2 Skin Friction on Piles vs. Pile Number**

### 3.2 Allowable End bearing Resistance of Pile

The allowable end bearing capacities were calculated using the method mentioned under section 2.3.5 and given in Table 4. The * marked piles have been terminated after encountering much competent bedrock profiles (which depends on quality of cores recovered in pile boring process)compared to reference boreholes and below the termination depth of respective boreholes. Hence the UCS values and RQD values determined for upper layers during soil investigation stage had to be ignored in these cases.

**Table 4 End bearing (EB) capacity levels acting on piles using theoretical methods**

| Pile | Reference Borehole | Depth of Pile (m) | Weathering grade | RQD % | UCS (MPa) | End bearing capacity (kPa) |
|---|---|---|---|---|---|---|
| P020* | BH-01 | 29.10 | Fresh | 52 | | 12500 |
| P013* | BH-01 | 28.35 | Fresh | 52 | | 12500 |
| P014* | BH-01 | 27.54 | Fresh | 52 | | 12500 |
| P004* | BH-01 | 30.00 | Fresh | 52 | | 12500 |
| P023* | BH-01 | 28.19 | Fresh | 52 | | 12500 |
| P003* | BH-01 | 29.70 | Fresh | 52 | | 12500 |
| P012* | BH-01 | 28.90 | Fresh | 52 | | 12500 |
| P050 | BH-02 | 28.19 | Fresh | 33 | | 5500 |
| P049* | BH-02 | 29.63 | Fresh | 33 | | 6000 |
| P046* | BH-02 | 31.05 | Fresh | 33 | | 6500 |
| P124 | BH-03 | 28.55 | Fresh | 28 | | 5000 |
| P101 | BH-03 | 28.8 | Fresh | 28 | | 5000 |
| P121 | BH-03 | 30.05 | Fresh | 28 | | 5000 |
| P114 | BH-03 | 26.28 | Fresh | Nil | | 275** |
| P140 | BH-03 | 27.80 | Fresh | 10 | 7.13 | 330** |
| P126 | BH-03 | 27.85 | Fresh | 10 | 7.13 | 330** |
| P109* | BH-03 | 31.23 | Fresh | 28 | | 5500 |
| P151* | BH-04 | 27.90 | Fresh | 80 | | 10500 |
| P130* | BH-04 | 28.50 | Fresh | 80 | | 10500 |
| P132* | BH-04 | 27.87 | Fresh | 80 | | 10500 |
| P133 | BH-04 | 27.20 | Fresh | 80 | | 10000 |
| P150* | BH-04 | 29.35 | Fresh | 80 | | 10500 |
| P074 | BH-05 | 30.15 | Fresh | 89 | | 11500 |
| P040 | BH-05 | 29.60 | Fresh | 89 | | 11500 |
| P061* | BH-05 | 30.83 | Fresh | 89 | | 12000 |
| P015* | BH-A | 23.75 | Fresh | 100 | | 12500 |
| P010* | BH-B | 29.40 | Fresh | 93 | | 12000 |

**Table 4 (Cont.)**

| Pile | Reference Borehole | Depth of Pile (m) | Weathering grade | RQD % | UCS (MPa) | End bearing capacity (kPa) |
|---|---|---|---|---|---|---|
| P165* | BH-B | 29.35 | Fresh | 93 | | 12000 |
| P166* | BH-B | 31.45 | Fresh | 93 | | 12000 |
| P131* | BH-G | 27.10 | Fresh | 91 | | 11500 |
| P148* | BH-G | 27.50 | Fresh | 91 | | 11500 |
| P088* | BH-I | 29.70 | Fresh | 70 | | 10000 |
| P093* | BH-I | 27.30 | Fresh | 70 | | 10000 |
| P078* | BH-I | 28.82 | Fresh | 70 | | 10000 |
| P094* | BH-I | 30.20 | Fresh | 70 | | 10000 |
| P064* | BH-I | 29.65 | Fresh | 70 | | 10000 |
| P042* | BH-J | 29.98 | Fresh | 90 | | 11500 |
| P032* | BH-J | 28.40 | Fresh | 90 | | 11500 |
| P033* | BH-J | 26.60 | Fresh | 90 | | 11500 |
| P045* | BH-J | 30.50 | Fresh | 90 | | 11500 |
| P043* | BH-J | 30.70 | Fresh | 90 | | 11500 |

Note *
The end bearing capacity of piles which have been terminated below the depth of reference boreholes was determined using standard tables (Tomlinson and Boorman, 1995), which gives general end bearing capacity levels for different RQD and degree of weathering levels.
Note **
In actual practice the Piles P114, P126 and P140 have been terminated much highly competent layer compared to the reference borehole level and the discrepancy in the borehole results may be due to variation of bedrock profile.

## 3.3 Practical (Field) pile capacities

### 3.3.1 High Strain Dynamic Testing (PDA) results

The Table 5 depicts the PDA data From CAPWAP analysis results of piles, providing the skin friction, end bearing and total capacity forces respectively.

**Table 5 SF and EB capacity levels acting on piles given by Field PDA Test Results**

| Pile | Total skin friction (kN) | End bearing capacity (kN) | Total capacity (kN) |
|---|---|---|---|
| P020 | 8456 | 4483 | 12939 |
| P013 | 5945 | 6602 | 12547 |
| P014 | 9476 | 7112 | 16589 |
| P004 | 15078 | 10281 | 25359 |
| P023 | 6180 | 8211 | 14391 |

**Table 5 (cont.)**

| Pile | Total skin friction (kN) | End bearing capacity (kN) | Total capacity (kN) |
|---|---|---|---|
| P003 | 18227 | 7299 | 25526 |
| P012 | 13783 | 5641 | 19424 |
| P050 | 19080 | 12125 | 31206 |
| P049 | 5997 | 6871 | 12868 |
| P046 | 14401 | 16589 | 30990 |
| P124 | 23681 | 8731 | 32412 |
| P101 | 17746 | 7819 | 25565 |
| P121 | 19561 | 7534 | 27095 |
| P114 | 17766 | 8260 | 26026 |
| P140 | 9967 | 9163 | 19130 |
| P126 | 21072 | 4199 | 25271 |
| P109 | 16000 | 9339 | 25339 |
| P151 | 14568 | 9251 | 23819 |
| P130 | 13351 | 13606 | 26958 |
| P132 | 23691 | 13165 | 36856 |
| P133 | 15206 | 14529 | 29734 |
| P150 | 13312 | 6573 | 19885 |
| P074 | 6720 | 2727 | 9447 |
| P040 | 13234 | 17383 | 30617 |
| P061 | 18070 | 7161 | 25231 |
| P015 | 11252 | 7495 | 18747 |
| P010 | 18158 | 7269 | 25428 |
| P165 | 5189 | 4189 | 9378 |
| P166 | 5474 | 3571 | 9045 |
| P131 | 20238 | 10899 | 31137 |
| P148 | 20022 | 5935 | 25957 |
| P088 | 18080 | 8780 | 26860 |
| P093 | 16245 | 9006 | 25251 |
| P078 | 13371 | 13705 | 27076 |
| P094 | 14735 | 11223 | 25957 |
| P064 | 18335 | 8348 | 26683 |
| P042 | 15078 | 13508 | 28586 |
| P032 | 8466 | 11811 | 20277 |
| P033 | 17256 | 14558 | 31814 |
| P045 | 11919 | 17658 | 29577 |
| P043 | 15441 | 10556 | 25997 |

### 3.3.2 Static Load Test (SLT) Results

As the second field testing method, the field skin friction and end bearing capacity levels were estimated using the method described under 2.4.2. The values are presented in Table 6.

**Table 6 SF and EB capacity levels acting on piles given by Field SLT Test Results**

| Pile | Total skin friction (kN) | End bearing capacity (kN) | Total capacity (kN) |
|---|---|---|---|
| P050 | 15696 | 9810 | 25506 |
| P014 | 7995 | 4513 | 12508 |
| P042 | 11772 | 6867 | 18639 |

## 3.4 Comparison of Theoretical Results with Field Test Results

The Table 7 summarizes the total skin friction levels (both soil as well as rock socket) acting on respective piles, estimated using theoretical methods against actual field observations.

**Table 7 Comparison of theoretical SF capacity levels against field SF capacity levels**

| Pile | Total safe theoretical skin friction (kN) | | | Total Skin Friction from PDA (kN) | Total Skin Friction from SLT (kN) |
|---|---|---|---|---|---|
| | Burland | ICTAD | O'Neil & Reese | | |
| P020 | 3041 | 2990 | 7300 | 8456 | |
| P013 | 1966 | 1846 | 5179 | 5945 | |
| P014 | 1904 | 1726 | 4881 | 9476 | 7995 |
| P004 | 3134 | 3139 | 7901 | 15078 | |
| P023 | 2047 | 1809 | 5199 | 6180 | |
| P003 | 3139 | 3178 | 8148 | 18227 | |
| P012 | 2534 | 2462 | 6559 | 13783 | |
| P050 | 2445 | 2205 | 7312 | 19080 | 15696 |
| P049 | 2856 | 2687 | 8824 | 5997 | |
| P046 | 2815 | 2936 | 8897 | 14401 | |
| P124 | 2604 | 3017 | 8730 | 23681 | |
| P101 | 3678 | 3277 | 10693 | 17746 | |
| P121 | 3936 | 3376 | 10978 | 19561 | |
| P114 | 3865 | 3255 | 11679 | 17766 | |
| P140 | 3834 | 3240 | 10754 | 9967 | |
| P126 | 4133 | 3556 | 11309 | 21072 | |

**Table 7 (Cont.)**

| Pile | Total safe theoretical skin friction (kN) | | | Total Skin Friction from PDA (kN) | Total Skin Friction from SLT (kN) |
|---|---|---|---|---|---|
| | Burland | ICTAD | O'Neil & Reese | | |
| P109 | 4204 | 3225 | 11049 | 16000 | |
| P151 | 2889 | 2611 | 6713 | 14568 | |
| P130 | 3068 | 2797 | 7091 | 13351 | |
| P132 | 3458 | 3116 | 6789 | 23691 | |
| P133 | 2990 | 2638 | 5844 | 15206 | |
| P150 | 2893 | 2611 | 6905 | 13312 | |
| P074 | 2393 | 2351 | 5951 | 6720 | |
| P040 | 3165 | 2897 | 7637 | 13234 | |
| P061 | 3269 | 3067 | 7382 | 18070 | |
| P015 | 2380 | 2250 | 6330 | 11252 | |
| P010 | 3177 | 3451 | 7743 | 18158 | |
| P165 | 2140 | 2142 | 5785 | 5189 | |
| P166 | 2018 | 1962 | 5912 | 5474 | |
| P131 | 3178 | 2935 | 8773 | 20238 | |
| P148 | 3282 | 3045 | 8531 | 20022 | |
| P088 | 3359 | 3202 | 8695 | 18080 | |
| P093 | 3053 | 2803 | 8172 | 16245 | |
| P078 | 3255 | 3029 | 8178 | 13371 | |
| P094 | 3357 | 3202 | 8691 | 14735 | |
| P064 | 3458 | 3329 | 7643 | 18335 | |
| P042 | 3404 | 2864 | 7588 | 15078 | 7995 |
| P032 | 2948 | 2578 | 6007 | 8466 | |
| P033 | 3095 | 2408 | 6022 | 17256 | |
| P045 | 3376 | 2812 | 8349 | 11919 | |
| P043 | 3463 | 2957 | 6486 | 15441 | |

## 4 DISCUSSION

When comparing the results of theoretical concepts with field test results, the Burland method found to provide most conservative values compared to other two. The field PDA results are higher as much as 210% to about 910% of the skin friction values given by Burland method. Same trend was seen with SLT results and these values are 235% to 642% of Burland skin friction levels.

Even though ICTAD method shows a lesser conservativeness compared to Burland method, the values are unacceptably lower compared to field values. PDA and SLT values are of 223% to 865% and 280% to 712% respectively of the ICTAD method generated skin friction capacity levels.

O'Neil & Reese method provides the least conservative friction levels compared to other two. Apart from pile P049 (where field results are less than theoretical O'Neil &Reese values), the PDA results are of 90% to 350% and SLT results are of 105% to 214% of the O'Neil & Reese method generated skin friction values. Hence it is evident that O'Neil &Reese method provides the least conservative estimate compared to other two methods, with reasonable margin with ultimate skin friction. Skin friction values from this method are 185% to 335% of Burland and 220% to 360% of ICTAD methods. Comparatively lower values of Field test (PDA) results in piles P049, P140, P165 and P166 may be due to other associated quality factors during the casting of piles.

When considering the end bearing capacities, it is very difficult to compare field load test results with theoretical results, mainly due to two reasons.

1. The bedrock at the particular site is fractured and weathered to a considerable depth and thickness of this incompetent zone is highly variable within shorter distances. Hence the bedrock profile of the pile location may be completely different from that of the nearest reference boreholes. Therefore, it is unreasonable to compare the theoretical results with field test results.
2. In field tests, loading are carried out generally only up to 150% of the working load and behaviour of pile is only studied up to this limit only. The bearing component reflects only to this limit and to have an idea about the ultimate level of end bearing, it will be necessary to impose much higher percentage of the working load.

## 5 CONCLUSIONS AND RECOMMENDATIONS

Following conclusions and recommendations can be made based on the outcomes of this study.

1. When considering the skin friction distribution along the pile, even though theoretically Burland method initially considers the overburden effective stress, later it ignores this effect by the critical depth factor. Again, when estimating the shear strength parameters for the same method using correlations with in-situ test data, it underestimates local soil shear strength parameter levels.
2. In ICTAD method, relies only on SPT'**N**' values and does not consider the confinement effects of overburden effective stress as in Burland method. Even though field SPT'**N**' values reflect this overburden effect later these values are corrected for overburden effect during the determination of $N_{corrected}$. However, unlike in Burland method, a second type underestimation of local shear parameter conditions does not occur in this method and thus slightly better results are produced by this method.
3. As mentioned above the application of O'Neil &Reese method produces least conservative and most practical results compared to Burland and ICTAD methods. It may be mainly because it directly considers the depth from surface to particular layer

and thus the confinement effects of overburden effective stress levels.

4. The application of Burland and ICTAD methods will be useful after carrying out detailed studies on applicability of empirical relationships between in-situ test values and shear strength parameters for local conditions. Even the applicability of critical depth factor on locally available highly permeable coarse grained residual soils should be investigated.
5. Even though it has been proven that the O'Neil &Reese method best suits the local conditions, it should be emphasised that adoption of high quality construction techniques and monitoring is essential as it creates only marginal space for errors compared to other two methods.
6. To have a better understanding about merit and demerits of these methods, a detailed study with an instrumented pile testing program is essential. In addition, this study was conducted for a case where subsurface composed only of residual sands, without compressible clays. Therefore, it is recommended for future studies to investigate the applicability of O'Neil &Reese method for complex geotechnical conditions, where negative skin friction comes into the picture.

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