COMPARISON BETWEEN EMPIRICAL, NUMERICAL AND PRACTICAL COMPRESSION CAPACITY OF ROCK SOCKETED BORED AND CAST IN-SITU PILE: A CASE STUDY

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Thesis submitted in partial fulfillment of the requirements for the
Degree of Master of Engineering
in Foundation Engineering and Earth Retaining Systems

Supervised by
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DECLARATION

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The above candidate has carried out research for the Master's thesis under my supervision.

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Prof. U. G. A. Puswewala Date

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20th August 2020

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Abstract

The development of tall structures as a rapidly developing trend in Colombo-Sri Lanka is evident during the recent past due to the high land prices. These tall structures require to be founded on strong substrata and piling is the most popular method that has been used as the foundation for these tall buildings. In Colombo area having found bed rock at shallow depth around 15m to 20m, always design engineers tend to specify the rock socketed end bearing piles without much considering the load carrying mechanism of the pile. It is evident that Sri Lankan design engineering community has a tendency to disregard the pile shaft skin friction resistance, mostly due to the existence of bentonite slurry within borehole during concreting. Therefore, load carrying capacity of such piles is determined completely based on the end bearing from the bed rock. In addition to that in most standards and codes of practice, the pile load carrying capacity correlations are given for specific soil types i.e. sand, clay, gravel. However in local context it is hard to find such conditions and almost all the soils are residual soils having both $c, \emptyset$ values.

In this research, different correlations for pile load capacity and its variations are evaluated. A detail comparison is conducted between the compression capacity of piles obtained from different empirical/semi-empirical methods, numerical methods such as FEM and in-situ testing i.e. MLT and HSDLT against the code of practices and local guide lines.

KEY WORDS: Empirical, Semi-empirical, Correlations, FEM, Pile load capacity, Skin friction, End bearing, Rock socket, MLT, HSDLT.

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ABBREVATIONS

c	-	Cohesion or Cohesive Strength (kPa)
E_m	-	Elastic modulus of rock mass
g	-	Gravitational acceleration (~9.81 mS^{-2})
f_s	-	Unit soil skin friction capacity of pile shaft
f_r	-	Unit rock socket skin friction capacity of pile shaft
j	-	Rock mass factor
q_b	-	End bearing capacity of pile
m_i	-	Hoek-Brown constant for intact rock
m_b	-	Hoek-Brown constant for broken rock mass
$N_{corr.}$	-	Corrected SPT N value.
$N_{uncorr.}$ Or $N_{field.}$	-	Uncorrected field SPT N value.
p_a	-	Atmospheric pressure (101 kPa)
σ'_{vm}	-	Vertical effective overburden pressure.
ν	-	Poisson's ratio
ψ	-	Dilatancy angle
ϕ	-	Friction angle
γ'	-	Effective unit weight
γ_w	-	Unit weight of water
CR	-	Core Recovery
DVL	-	Design Verification Load
SWL	-	Specified Working Load
MLT	-	Maintained Load Test
HSDLT	-	High Strain Dynamic Load Test
MSL	-	Mean Sea Level
RQD	-	Rock Quality Designation
RMR	-	Rock Mass Rating
UCS Or q_c Or σ_c	-	Uni-axial Compressive Strength of rock(=Unconfined Compressive Strength of intact rock)
HK	-	Hong Kong
BS	-	British Standards
CIDA	-	Construction Industry Development Authority
GSI	-	Geological Strength Index

1 INTRODUCTION

1.1 CONTEXT AND MOTIVATION

The most prevalent opportunity for a growing city with high land prices and limited space such as Colombo is to develop tall buildings, where the demand of space is created vertically. Structures such as tall buildings require to be founded on strong strata. Employing bored and cast insitu pile foundations for tall buildings is a common practice in Sri Lanka, as the bedrock is present at moderately shallow depths. These piles are socketed into the bed rock, where the bedrock is the strongest strata as opposed to the overburden.

Conversely, top region of the bed rock is in highly fissured and weathered state in most of the locations, and high localize variability of the bedrock is a very common occurrence. Design engineers often tend to highly overdesign by underestimating the pile capacity as a result of lack of data available and lack of competence in estimating of skin friction of rock socket. It can be seen engineers specifying uneconomical rock socket depth i.e. minimum 1.5D or 2D into hard rock and unnecessary larger diameter of piles based on specification proposed in foreign researches without considering local geological properties of rock.

The behavior of single pile under axial loading, to the extent that load distribution and settlement along the pile are concerned, have been evaluated through several approaches. They can be categorized into three main categories:

1. Empirical Method: where empirical and semi-empirical equations are used to calculate capacity of geological aspect.
2. Practical Method: where actual pile is subjected to a load testing and variation of other parameters is obtained. i.e. Load vs. Settlement.
3. Numerical Method: where Finite Element Model based analysis to generate behavior of pile under the loading.

In this research, a study was done to identify the different methods available to obtain pile load capacity and its application in local context and further discuss the suitability of different methods by comparing results.

1.2 OBJECTIVE

The key objective is to compare the validity of available empirical and semi-empirical correlations to compute the pile load capacity of rock socketed bored and cast in-situ piles as applicable to geological conditions of hard crystalline rock as those prevailing in Sri Lanka.

2 LITERATURE REVIEW

2.1 LOAD TRANSFER MECHANISUM AND PILE CAPACITY

The piles may be considered loaded axially and laterally or both at once. The limit states that need to be considered when designing piles are as follows. (EN-1997-1, Section.7.2(1) P):

- Pile foundation failure due to insufficient bearing resistance.
- Inadequate compressive resistance of the pile material (Figure.1a)
- Inadequate tensile resistance or uplift of the pile (Figure. 1d)
- Ground failure due to transverse pile load (Figure. 1f)
- Pile structural failure in excessive compression stress (Figure. 1b), tensile stress (Figure. 1e), bending stress (Figure.1g), buckling (Figure.1c) or shear stress (Figure.1h)
- Collective failure of exceeding geological & pile foundation structural capacity.
- Lateral movement, soil heave or excessive settlements.
- Loosing of structural system overall stability
- Excessive ground vibrations. i.e. tremors, earthquakes.

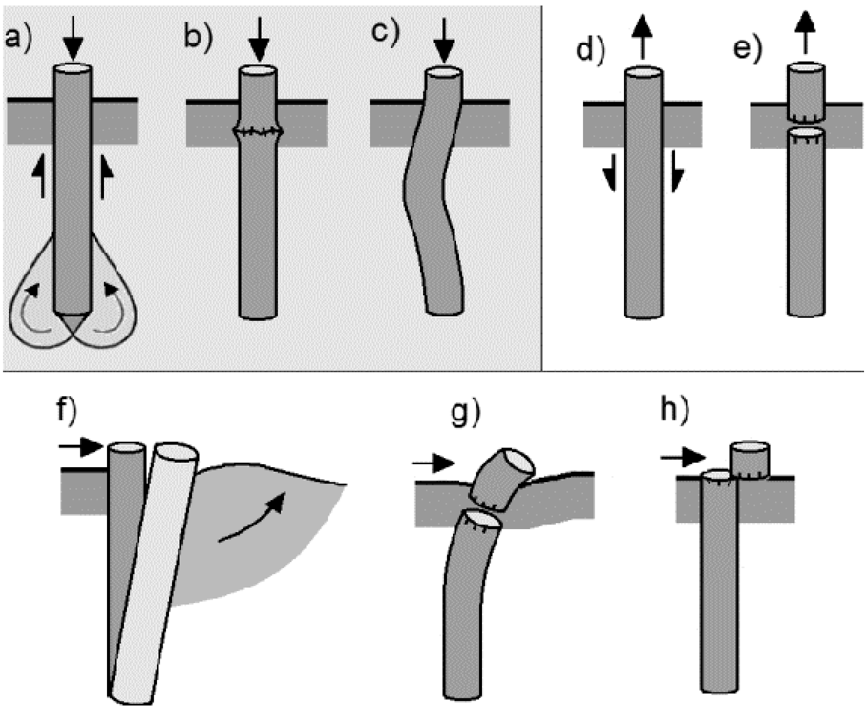


Figure 1. Pile failures on compression: (a) - (c), on tensile stress (d)&(e), on transverse loading (f)-(h).

While the pile is undergone a gradually increasing compressive load at a rapid or moderately rapid application rate, the resulting load-settlement curve is shown in Figure 2. Initially the pile-soil system acts elastically. There is a straight line relationship up to some point A on the curve and if the load is released at any stage up to this point the pile head will recover to its original level. When the load is increased further than point A there is yielding at, or near to, the pile-soil interface and slippage follows until point B is reached, when the maximum skin friction on the pile shaft will have been mobilized. If the load is released at this step the pile head will recover to point C, the amount of 'permanent set' being the distance OC. The movement required to mobilize the maximum skin friction is relatively small and is only of the order of 0.3 to 1% of the pile diameter. The base resistance of the pile requires a greater downward movement for its full mobilization, and the amount of movement depends on the diameter of the pile. It may be in the range of 10 to 20% of the base diameter. When the stage of full mobilization of the base resistance is reached (point D in Figure 2) the pile plunges downwards without any further increase of load, or small increases in load produce increasingly large settlements.

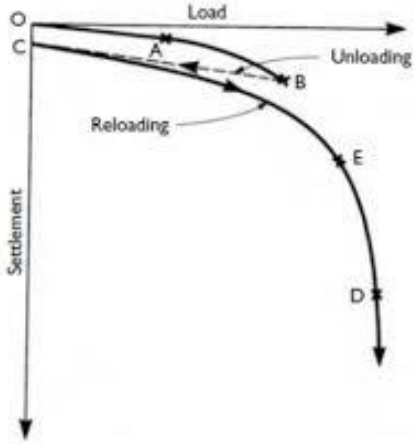


Figure 2. Load-settlement curve for failure under compressive load on pile

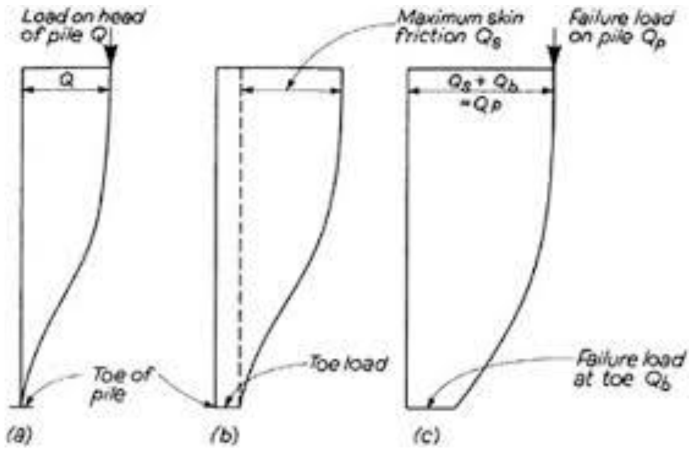


Figure 3. Load transmission from top of pile to end through shaft.

If strain gauges are installed at various points along the axis of the pile, so that the compressive load of the pile can be derived from each point, the graph shown in Figure 3 will be obtained, which shows the movement from pile to soil at each position The load transfer loading stage is shown in Figure 2. Therefore, when the load reaches point A, almost all the load is borne by the skin friction on the pile shaft, and almost no or no load is transferred to the pile toe. (Figure 3(a)). When the load reaches point B the pile shaft is carrying its maximum skin friction and the pile toe will be transmitting some load (Figure 3(b)). At point D there is no further escalation in the load transferred in skin friction but the base load will have reached its extreme value (Figure 3(c)).

The basic concept of pile static bearing capacity is based on separately evaluating end bearing resistance and shaft skin friction. The elementary equation is;

$$\boldsymbol{Q_u = Q_p + Q_s - W_p} \qquad \dots (01)$$

Where; $Q_u - Ultimate\ pile\ capacity$

$Q_p - Ultimate\ pile\ end\ bearing\ resistance$

$Q_s - Ultimate\ shaft\ skin\ friction\ resistance.$

$W_p - Pile\ self\ weight$

Generally the self-weight of the pile (W_p) is insignificant in relative to (Q_u) and this term is usually disregarded in above equation.

$$\boldsymbol{Q_u = Q_p + Q_s} \qquad \dots (02)$$

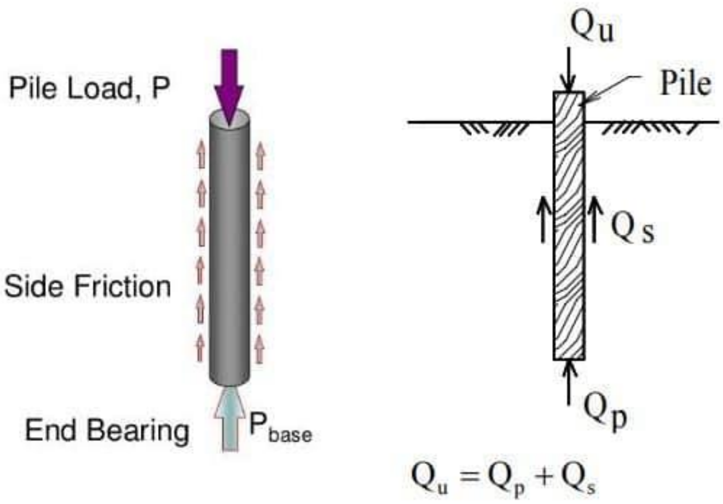


Figure 4. Static pile ultimate capacity equation.

The corresponding loading to point D on the load-settlement curve shown in Figure 2 denotes the ultimate pile capacity or ultimate limit state of the pile. Ultimate limit state represents general shear failure of the soil or socketed rock at pile toe. Conversely, reference to the British Standard (BS8004:1986) outlines that the ultimate pile bearing capacity can be expressed as the corresponding load act on pile, which cause the pile top to settle 10% of the pile diameter. Otherwise ultimate pile capacity obtained from the load-settlement curve, with expert judgement. However elastic shortening of pile material under loading also should consider.

2.2 CALCULATION OF PILE CAPACITY

Pile load carrying capacity subject to on several aspects, comprising pile physical properties (pile length, cross-section/diameter and shaft profile), soil strength parameters and pile installation method.

Approaches to obtain single pile capacity under axial load can be categorized as described previously as follows;

- Empirical Method - Empirical or semi-empirical correlations.
- Numerical Method - FEM based analysis.
- Practical Method - In-situ testing of pile with application of axial load at field.

2.2.1 EMPIRICAL METHOD

Empirical correlations are pure mathematical equations with combination of soil parameters to obtain pile capacity, while semi-empirical correlations have parameters from field testing such as SPT, CPT testing.

There are various correlations developed by researches, based on the different experimental results. Most of correlations are only valid for specific soil conditions, such as pure cohesive soil ($clayey\ soil\ where, \emptyset = 0$), pure cohesion less soil ($sandy\ soil\ where, c = 0$).

However, in local context it is hard to find such soil conditions, and almost all the cases are residual soils with both $c\ and\ \emptyset$ values. Therefore, it was decided to use semi-empirical correlations rather than pure empirical, as real soil test parameters are incorporated in to the calculation.

Approach to calculate pile load capacity can be sub-divided on (1) Soil Skin Friction, (2) Rock Socket Skin Friction and (3) Rock End Bearing.

In this research study, subsequent empirical/semi-empirical approaches for estimating skin friction and end bearing capacity will be discussed in details.

Methods used to estimate the soil skin friction in the shaft (above rock level)

- M.1.1 – Method outlined in ICTAD guidelines
- M.1.2 – O'Neill and Reese Method (1999)

Methods used to estimate the rock socket skin friction.

- M.2.1 – Limiting value given in ICTAD guidelines

- M.2.2 – Rowe and Armitage (1987)
- M.2.3 – Method given in Hong Kong guidelines
- M.2.4 – William and Pells (1981)
- M.2.5 – Meigh and Wolski (1979)
- M.2.6 – Hovarh and Kenny (1987)

Methods used to estimate the rock end bearing capacity

- M.3.1 – Method given in BS8004 (same as in ICTAD)
- M.3.2 – RMR method (Hong Kong guideline)
- M.3.3 – Kulhawy and Goodman
- M.3.4 – Method outlined in M. J. Tomlinson
- M.3.5 – Peck et. al (1986)
- M.3.6 – Bell Solution

2.2.1.1 CALCULATION OF SHAFT SKIN FRICTION

2.2.1.1.1 M.1.1 – Method outlined in ICTAD guidelines

Ref. to ICTAD/DEV/15 (1997) has specified a simplest method 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). The unit ultimate skin friction per unit surface area of shaft (f_s) will be given in kN/m^2

$$\boldsymbol{f_s = 1.3 \times N_{corr}} \qquad \dots (03)$$

Where; $f_s - Ultimate\ unit\ shaft\ skin\ friction\ (kN/m^2)$

$N_{corr} - Corrected\ SPT\ N\ value$

Further it states to limit maximum value $f_s \leq 100\ kN/m^2$, which limits $N_{corr} \leq 76$

2.2.1.1.2 M.1.2 – O'Neill and Reese Method

O'Neill and Reese (1999) is one of the methods that is most commonly used in practise in most parts of the world. Even though it derivate specially for cohesion less and gravelly sands, it had been widely used for other types of soils, by correlating only SPT blow values.

$$\boldsymbol{f_s = \beta . \sigma'_{vm}} \qquad \dots (04)$$

Where; $f_s - Ultimate\ unit\ shaft\ skin\ friction\ (kN/m^2).\ \leq 200\ kN/m^2$

$\sigma'_{vm} - vertical\ effective\ stress\ at\ the\ middle\ of\ each\ layer$

$\beta - Dimensionless\ factor$ where;

$For\ SPT\ N_{uncorr.} \geq 15\ blows/0.3m:\ \beta = 1.5 - 0.245(Z_i)^{0.5}$

$For\ SPT\ N_{uncorr.} < 15\ blows/0.3m:\ \beta = {N_{uncorr}}/{15}\{1.5 - 0.245(Z_i)^{0.5}\}$

$Z_i - Vertical\ distance\ from\ the\ ground\ surface\ (m)\ to\ middle\ of\ i^{th} layer$

2.2.1.2 CALCULATION OF ROCK SOCKET SKIN FRICTION

Rock socketed bored cast-in-situ concrete piles act as both friction and end bearing piles. Factors govern the development of shaft skin friction and toe end bearing within the rock socketed region is estimated by Duncan C. Wyllie (1991) and listed as follows;

- The geometry of the socket as defined by the length to diameter ratio.
- The elastic modulus of the rock mass, presence at sides and below toe.
- Compressive strength of the rock mass at shaft perimeter & below pile toe.
- The state of the side-walls with respect to irregularity and the existence of drill cuttings or bentonite cakes.
- The state of the toe of the pile with respect to the removal of drill cuttings and other loose material from the bottom of the socket.
- Layering in the rock and the presence of seams with differing strengths and elastic modules.
- Pile settlement related to socketed region side-wall shear strength.
- Creep settlements with time within the rock-pile material (concrete) interface.

Wyllie (1991) specified that the rock socket skin friction need to be reduced by 25% related to clean rock socket, if the drilling mud (bentonite slurry) is used during the boring operation. Unless otherwise proposed to verify the friction resistance through practical pile load testing.

Skin friction resistance of rock socketed region, is subject to the interaction among the pile material (in this case concrete) and the socketed rock. Pile-rock interaction depends on the unconfined compression strength (UCS) of the rock, the rock socket bond stress has been developed by a number of researches. i.e. Rosenberg and Journeaux (1976), Horvarth (1978), and Williams and Pells (1981)

Early studies in Australia for the development of rock socket resistance using nonlinear pile design in Melbourne Mudstones were done by Williams, Johnston and Donald (1980) and elastic pile design with Sydney Sandstones and Shales were carried out by Rowe and Pells (1980). Field and laboratory tests were performed by Horvath and Kenny (1979) using Canadian mudstones. Similar studies were carried out by Meigh and Wolshi (1979) in Europe. Rowe and Armitage (1987) contributed to detailing of rock socketed region side slip design work.

Discontinuity in shaft skin friction among clays and several soft rocks (shales, mudstones and limestone) was presented by Kulhawy and Phoon (1993). Seidel and Haberfield (1995) done comprehensive study and specified development of rock socket skin friction is greatly dependent on pile diameter, side-wall roughness.

In general, capacity calculation of rock socket is governed by serviceability load conditions than ultimate load conditions. Load – settlement behaviour of the rock sockets is defined mostly by deformation properties of the rock mass. Zhang (2004) expressed about the theory of estimating the rock mass modulus (E_m) value using the intact rock modulus (E_i) by reduce factoring for the rock discontinuity frequency.

Characteristic compressive rock strength (q_{uc}) value has been frequently used for various pile rock socket design procedures and to calculate the design shaft capacity based on correlations suggested by several researchers. Kulhawy et al. (2005) summarized the shaft shear capacity equations derived by different researches. Comprehensive analysis on selected four methods by Gannon et al. (1999) stated rock socket shear capacity is varying widely, even though consistent properties of rock were used in pile design. In general Carter and Kulhawy (1988) design method results higher (uneconomical) pile socket lengths, despite the fact that design methods proposed by Rowe & Armitage (1987) and Williams et al. (1980) estimates lesser socket lengths by 40-60%. Ng et al. (2001) confirmed through his research studies that the Hovarth et al. (1983) and Rowe & Armitage (1987) correlations can be use on piles socketed in volcanic and sedimentary rocks respectively.

2.2.1.2.1 M.2.1 – Limiting value given in ICTAD guidelines

Ref. to ICTAD/DEV/15 (1997) publication under item 3.1 specify unit skin friction in rock correlated with SPT N value. Even though it correlates SPT N values, assigning SPT N value and accuracy of results are questionable. The unit ultimate skin friction of rock per unit surface area of shaft (f_r) will be given in kN/m^2

$$\boldsymbol{f_r = 2.0 \times N_{corr}} \qquad \ldots (05)$$

Where; $f_r - Ultimate\ unit\ rock\ shaft\ skin\ friction\ (kN/m^2)$

$N_{corr} - Corrected\ SPT\ N\ value$

Further it states not to exceed maximum value $f_r \leq 200\ kN/m^2$

2.2.1.2.2 M.2.2 – Rowe and Armitage (1987)

Ref. to "Foundation on Rock" (2nd edition) by Duncan C. Wyllie, for clean rock sockets, with side wall undulations among 1mm and 10mm deep and less than 10mm wide (0.04-0.4 in deep, < 0.4 in wide) Rowe and Armitage (1987) given side wall shear stress can be related with unconfined compressive rock strength (UCS in MPa) by the expression.

$$\boldsymbol{f_r = 0.60(q_{uc})^{0.5}} \qquad \ldots (06)$$

Where; $f_r - Ultimate\ unit\ rock\ shaft\ skin\ friction\ (MN/m^2)$

$q_{uc} - Unconfined\ compressive\ strenght\ (MN/m^2)$

2.2.1.2.3 M.2.3 – Method given in Hong Kong guidelines

Ref. to Hong Kong Geo Publication No.1/2006 – Foundation Design and Construction states, several empirical correlations were derive the shaft resistance based on the UCS (Uniaxial Compressive Strength) value of intact rocks, $\sigma_c\ (=\ q_{uc})$. It further states that some cases the shaft resistance within the rock socket is more than the concrete bond strength. Its due to confinement within rock socket and pile reinforcement, concrete is behaving stronger than unconfined and unreinforced state.

Serrano & Olalla (2004) used the Hoek & Brown (1980) failure criterion for rock masses to established a method to estimate the rock socket ultimate shaft resistance. (Figure 5)

$$\boldsymbol{f_r = \alpha.(\sigma_c)^{0.5}} \qquad \ldots (07)$$

Where; $f_r - Ultimate\ unit\ rock\ shaft\ skin\ friction\ (kN/m^2)$

$\alpha - Coefficient\ ranges\ from\ 0.1\ to\ 0.8. (Generally\ takes\ 0.2)$

$\sigma_c\ (= q_{uc}) - Unconfined\ compressive\ strenght\ (MN/m^2)$

2.2.1.2.4 M.2.4 – William and Pells (1981)

Ref. to Pile Design and Construction Practice (4th Ed.) by M. J. Tomlinson,

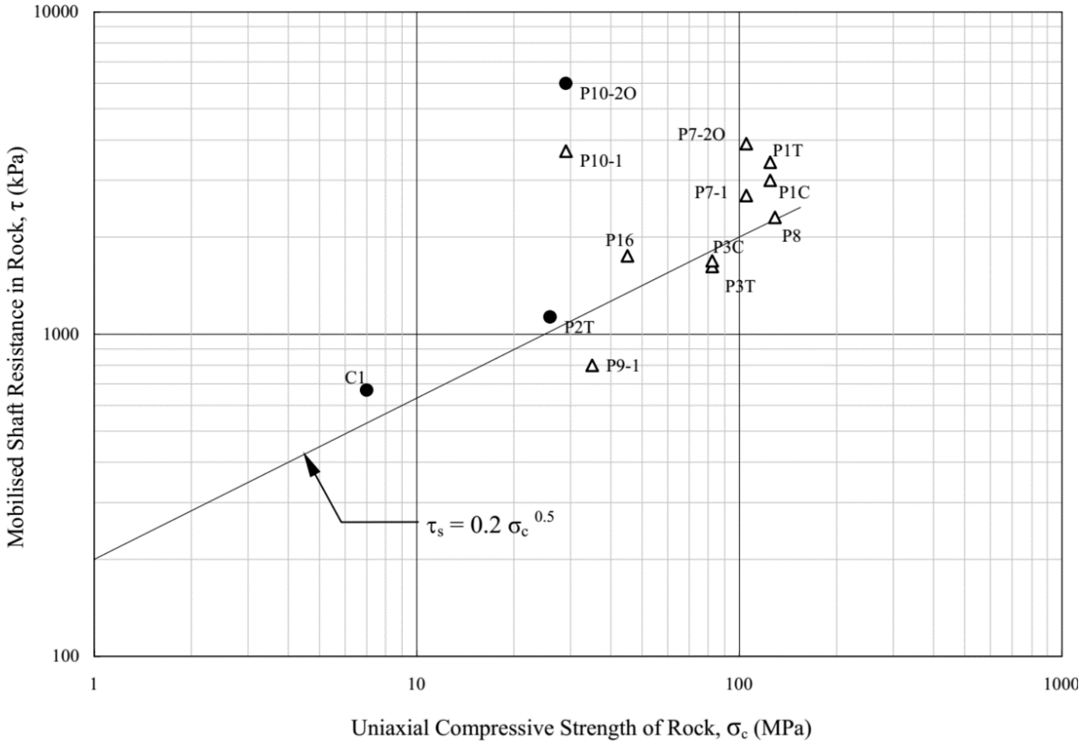
$$\boldsymbol{f_r = \alpha . \beta . q_{uc}} \quad \dots (08)$$

Where; $f_r - Ultimate\ unit\ rock\ shaft\ skin\ friction\ (kN/m^2)$

$\alpha - Reduction\ factor\ related\ to\ q_{uc}\ as\ shown\ in\ (Fig.6)$

$\beta -$
$Correction\ factor\ related\ to\ the\ discontinuity\ spacing\ in\ the\ rock\ massas\ shown\ in\ (Fig.7)$

$\sigma_c\ (= q_{uc}) - Unconfined\ compressive\ strenght\ (MN/m^2)$



Legend :

● = Substantially mobilised

△ = Degree of mobilisation unknown

Notes :

(1) For details of tested materials and pile construction, see Table A4

(2) Pile mark designation: prefix – P for bored piles or minipile and C for hand-dug caisson
suffix – C for compression test, T for tension test and 1 or 2 for stages of pile loading test, O denotes the use of Osterberg cell

Figure 5. Mobilized shaft resistance in piles socketed in rock.
(Hong Kong Geo Publication No.1/2006)

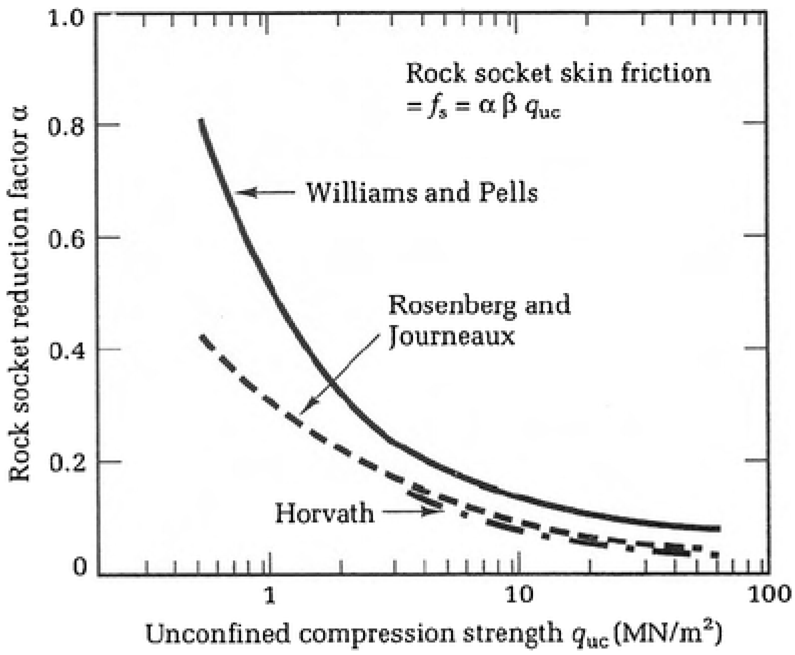


Figure 6. Reduction factors for rock socket shaft friction.
(Tomlinson. M. & Woodward. J., 2006)

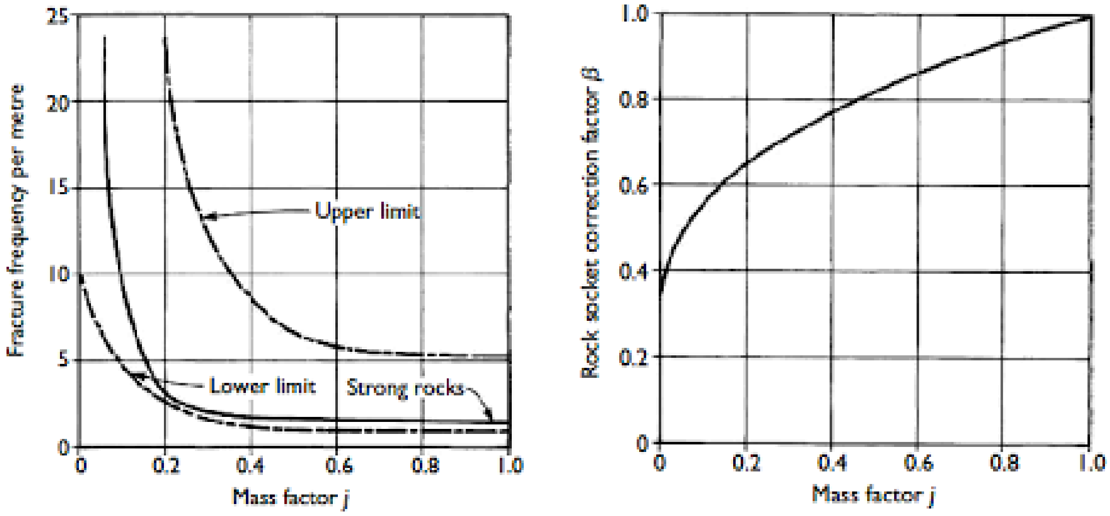


Figure 7. Reduction factors for discontinuities in rock mass.
(Tomlinson. M. & Woodward. J., 2006)

The curve denotes Williams and Pells (1981) in Figure 6 is greater than the Rosenberg & Journeaux and Horvath curves. On the other hand the β factor is same for all curves as it is based on the mass factor, j. Rock mass factor is derived by the ratio of the elastic modulus of rock mass and the intact rock as shown in Figure 7. Hobbs (1975) recommended mass factor, j can be estimated relating to the RQD (Rock Quality Designation) or the discontinuity spacing as per the Table 1.

Table 1. Mass Factor j value relating to RQD and Discontinuity Spacing.
(Tomlinson. M. & Woodward. J., 2006)

RQD (%)	***Fracture frequency per meter***	***Mass factor - j***
0 – 25	15	0.2
25 – 50	15 – 18	0.2
50 – 75	8 – 5	0.2 – 0.5
75 – 90	5 – 1	0.5 – 0.8
90 – 100	1	0.8 – 1.0

2.2.1.2.5 M.2.5 - Meigh and Wolski (1979)

$$\boldsymbol{f_r = 0.55 p_a . (q_{uc})^{0.6}} \quad \dots (09)$$

Where; $f_r - Ultimate\ unit\ rock\ shaft\ skin\ friction\ (kN/m^2)$

$p_a - Atmospheric\ pressure\ (101\ kPa)$

$q_{uc} - Unconfined\ compressive\ strength\ (MN/m^2)$

2.2.1.2.6 M.2.6 – Hovarth and Kenny (1987)

$$\boldsymbol{f_r = 0.65 p_a . \left({q_{uc}}/{p_a}\right)^{0.5}} \qquad \dots (10)$$

Where; $f_r - Ultimate\ unit\ rock\ shaft\ skin\ friction\ (kN/m^2)$

$p_a - Atmospheric\ pressure\ (101\ kPa)$

$q_{uc} - Unconfined\ compressive\ strength\ of\ rock\ mass\ (MN/m^2)$

2.2.1.3 CALCULATION OF ROCK SOCKET END BEARING CAPACITY

2.2.1.3.1 M.3.1 – Method outlined in BS8004 (1986)

Ref. to ICATD/DEV/15 (1997) - Guidelines for Interpretation of Site Investigation Data for Estimate the Carrying Capacity of Single Piles for Design of Bored and Cast Insitu Reinforced Concrete Piles, also states the same procedure given in BS8004 (1986) – Code of Practice for Foundations.

Allowable bearing capacity of weathered and fractured rocks subject to the rock mass strength and compressibility. The rock mass compressibility is related to the UCS value of the intact rock, lithology, occurrence discontinuities orientation and frequency in the rock mass. Rocks categorised in groups for the purpose of allowable bearing capacity calculation. Grouping is based on alike modulus ratio, which is the ratio between Young's modulus and compressive strength of the intact rock as given in Table 02. In Sri Lankan context for metamorphic rocks, Group 2 is selected form the rock classification.

Table 2. Grouping of weak and broken rocks.
(Code of Practice for Foundations-BS8004, 1986)

Group	Type of rock
1	Pure limestones and dolomites
	Carbonate sandstones of low porosity
2	Igneous
	Oolitic and marly limestones
	Well cemented sandstones
	Indurated carbonate mudstones
	Metamorphic rocks, including slates and schists (flat cleavage/foliation)
3	Very marly limestones
	Poorly cemented sandstones
	Cemented mudstones and shales
	Slates and schists (steep cleavage/foliation)
4	Uncemented mudstones and shales

Curves has been developed for designated values of allowable bearing pressure opposed to rock strength and discontinuity spacing and bedding in the rock mass as shown in Figure 8.

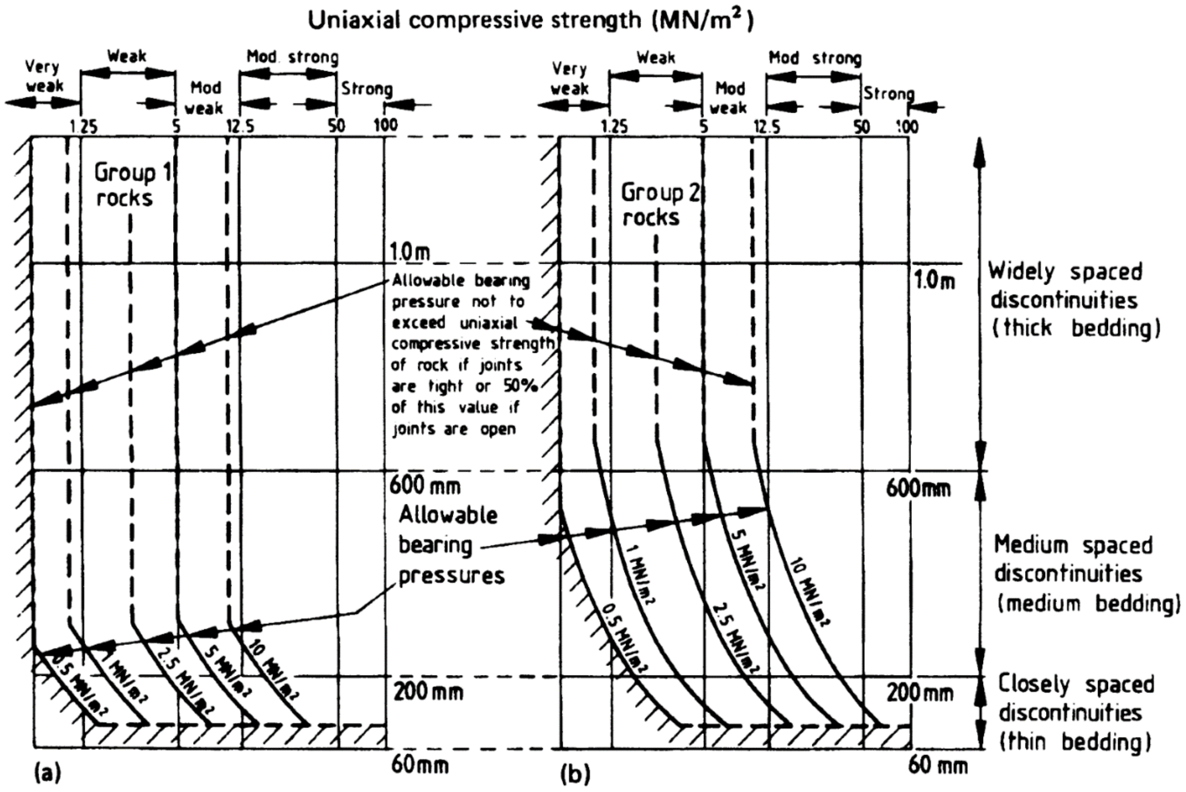


Figure 8. Allowable bearing pressure of rocks relates to UCS and discontinuities. (Code of Practice for Foundations-BS8004, 1986)

2.2.1.3.2 M.3.2 – RMR method (Hong Kong guideline)

Ref. to Hong Kong – Geo Publication No. 1/2006 – Foundation Design and Construction. Irfan & Powell (1985) stated weathering classification system of rock mass, in combination with point load index tests, which is better than use of RQD or total CR (Core Recovery). It allowed limited field data available to apply successfully over a large site area. Rock Mass Rating (RMR) proposed by Bieniawski (1974) and Rock Mass Quality Index (Q-Index) suggested by Barton et al. (1974) can be used to estimate of rock mass strength parameters and allowable bearing pressure.

Several researchers have suggested to use RMR for rock mass classification for engineering requirements. Bieniawski and Orr (1976) suggested that the RMR value can be adapt to represent the influence of rock mass joint orientation on the pile settlement and load capacity. Gannon et al (1999) used to calculate fractured rock modulus based on RMR value

RMR can be estimate from the borehole records form initial site investigation and is more appropriate for piling design & construction work. RMR system reflects in supplementary details of properties of infilled materials and joint characteristics, which are significantly impact to the behaviour of the pile foundations. RMR system applicable to metamorphic rock presence in local context and also for sedimentary rock, apart from rock masses having dissolution features, such as marble formation.

RMR index for the parent rock masses beneath the test pile calculated based on the guidelines given in Figure 9. The allowable bearing pressures can be obtained from the given relationship in Figure 10.

(A) Strength of Intact Rock

Uniaxial compressive strength, σ_c (MPa)	> 250	250 – 100	100 – 50	50 – 25	25 – 5	5 – 1	< 1
Point load strength index, PLI_{50} (MPa)	> 10	10 – 4	4 – 2	2 – 1	σ_c is preferred		
Rating	15	12	7	4	2	1	0

(B) Rock Quality Designation (RQD)

RQD (%)	100 – 90	90 – 75	75 – 50	50 – 25	< 25
Rating	20	17	13	8	3

(C) Spacing of Joints

Spacing	> 2 m	2 m – 0.6 m	0.6 m – 0.2 m	200 – 60 mm	< 60 mm
Rating	20	15	10	8	5

(D) Conditions of Joints

Discontinuity length[(1)]					
Rating	2				
Separation	None	< 0.1 mm	0.1 – 1 mm	1 – 5 mm	> 5 mm
Rating	6	5	4	1	0
Roughness	Very rough	Rough	Slightly rough	Smooth	Slickenside
Rating	6	5	3	1	0
Infilling (gouge)	None	Hard filling < 5 mm	Hard filling > 5 mm	Soft filling < 5 mm	Soft filling > 5 mm
Rating	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
Rating	6	5	3	1	0

(E) Groundwater

Rating[(1)]	7

Notes :

(1) Rating is fixed as the parameter is considered not relevant to the evaluation of allowable bearing pressure of rock mass.

(2) RMR is the sum of individual ratings assigned to parameters (A) to (E).

Figure 9. RMR classification system based on Bieniawski, 1989. (Hong Kong – Geo Publication No. 1/2006)

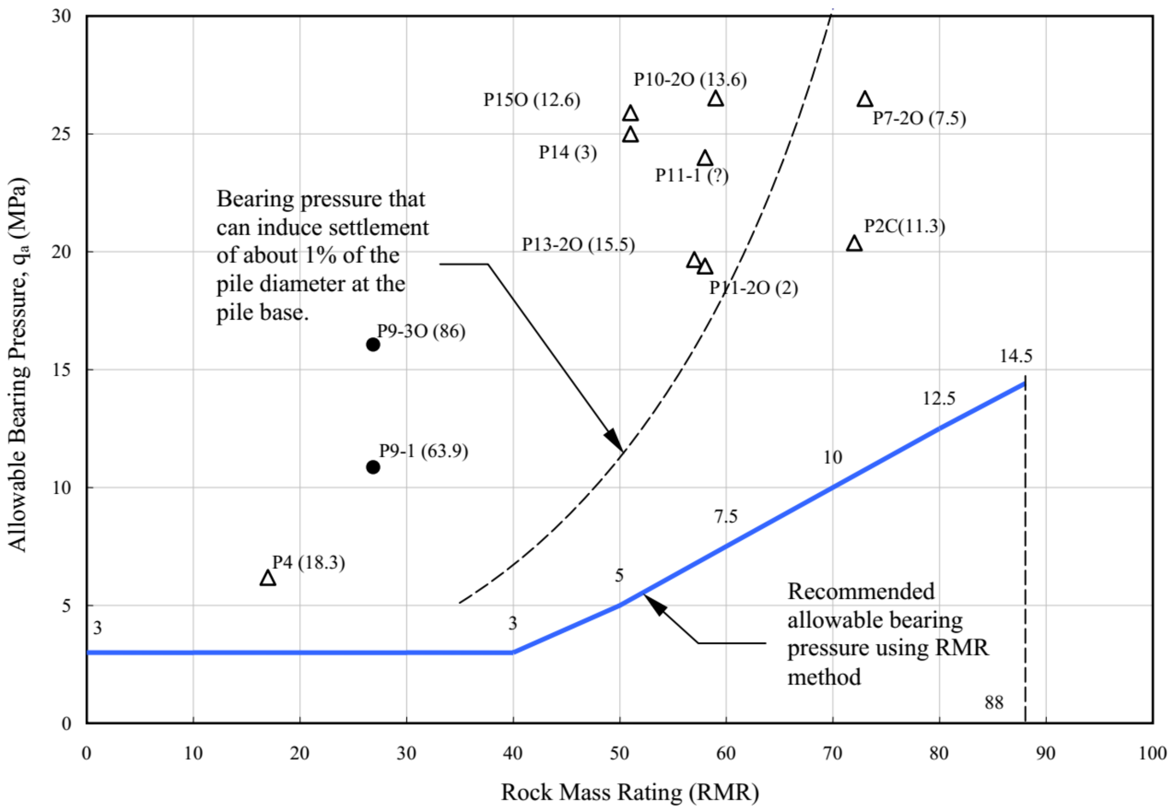


Figure 10. Allowable bearing pressure related to RMR value for a jointed rock mass beneath piles (Hong Kong – Geo Publication No. 1/2006)

2.2.1.3.3 M.3.3 – Kulhawy and Goodman

Reference to Pile design and construction practice (4^{th} Ed.) by M. J. Tomlinson (2006), Kulhawy and Goodman (1980,1987), have shown that ultimate end bearing capacity, q_c be able to correlated with the RQD values of the rock mass and proposed the estimated relationship as given in Table 3.

Table 3. Relation of RQD to rock mass strength parameters and ultimate bearing. (Tomlinson. M. J., 2006)

RQD (%)	***Rock mass properties***		
	q_c	c	$\emptyset$
0 - 70	$0.33q_{uc}$	$0.1q_{uc}$	30°
70 - 100	$0.33q_{uc}\ to\ 0.8q_{uc}$	$0.1q_{uc}$	$30^{\circ} - 60^{\circ}$

Where; $q_c - Ultimate\ end\ bearing\ capacity\ (kN/m^2)$

$q_{uc} - Unconfined\ compressive\ strength\ of\ rock\ mass\ (MN/m^2)$

2.2.1.3.4 M.3.4 - Method outlined in M. J. Tomlinson

Reference to Pile design and construction practice (4th Ed) by M. J. Tomlinson (2006) gives, for both driven and bored piles on rock, the ultimate pile base resistance as follows;

$$\boldsymbol{q_b = 2.N_{\emptyset}.q_{uc}} \quad \ldots (11)$$

$$N_{\emptyset} = \tan\left(45 + {}^{\emptyset}/_{2}\right)^2$$

Where; $q_b - Ultimate\ end\ bearing\ capacity\ (kN/m^2)$

$N_{\emptyset} - Bearing\ capacity\ factor$

$q_{uc} - Unconfined\ compressive\ strenght\ (MN/m^2)$

Duncan C. Wylie (1991) suggested a range of angle of internal friction for intact rock as given in Table 4. In local context we can adopt $\emptyset = 27°\ to\ 34°$ for Medium friction – Gneiss.

Table 4. Suggested friction angles for intact rock. (Duncan C. Wyllie, 1991)

Classification	***Type***	***Friction angle : $\emptyset°$***
Low friction	Schists (high mica content) Shale Marl	20 to 27
Medium friction	Sandstone Siltstone Chalk Gneiss Slate	27 to 34
High friction	Basalt Granite	34 to 40

2.2.1.3.5 M.3.5 – Method Proposed by Peck et. al (1986)

Reference to Hong Kong – Geo Publication No. 1/2006 Foundation Design and Construction, semi-empirical method proposed by Peck et. al (1986) given with direct correlation of RQD and Allowable end bearing and he present with graphically in Figure 11.

Notes:

1. if $q_a > q_{uc}(Uniaxial\ compressive\ strength\ of\ rock)$, use q_{uc} instead of q_a.
2. If RQD is fairly even, use an average RQD within $d_b = D_b$. where d_b= depth below pile toe and D_b=width of foundation (or pile diameter)
3. If RQD within $d_b = 0.25D_b$ is lesser, use the lesser value of RQD.

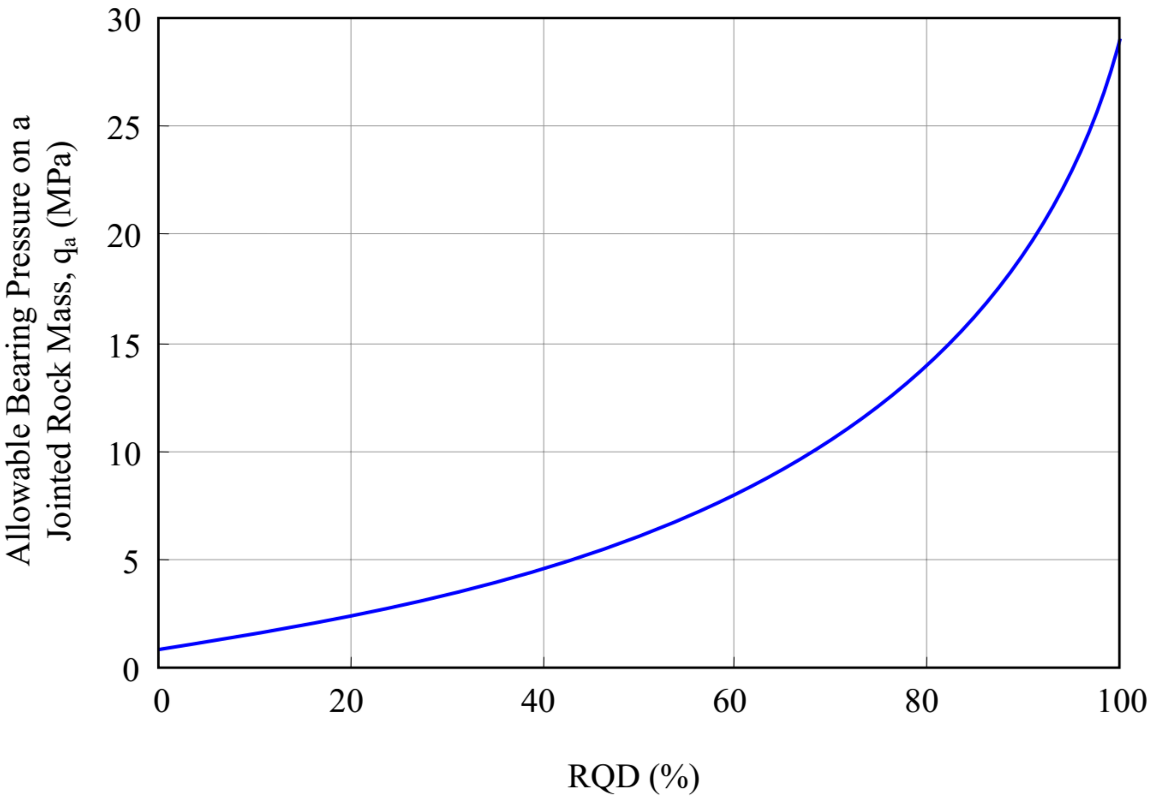


Figure 11. Correlation among RQD and allowable bearing pressure for a Fractured Rock Mass (Peck et al 1974) - (Hong Kong – Geo Publication No. 1/2006)

2.2.1.3.6 M. 3.6 – Bell Solution

The formula of Bell (well known as Bell Solution) is raised by F. G. Bell (1915) which is applied to determination of ultimate end-bearing capacity of closed joints rock-masses.

$$\boldsymbol{q_b = c'(C_{f1}.N_c) + 0.5B_f\gamma_r'(C_{f2}.N_\gamma) + \gamma_r' d_r N_q} \quad \ldots (12)$$

$N_\gamma = 2\sqrt{N_\emptyset}\ (N_\emptyset + 1)$

$N_\gamma = \sqrt{N_\emptyset}\left(N_\emptyset^2 - 1\right)$

$N_q = N_\emptyset^2$

$N_\emptyset = \tan\left(45 + {}^{\emptyset'}/_{2}\right)^2$

Where; $B_f - width\ of\ foundation$

$d_r - foundation\ depth\ below\ rock\ surface$

$\gamma_r' - effective\ unit\ weight\ of\ rock\ mass$

$c'\ and\ \emptyset' - shear\ strenght\ parameters\ of\ rock\ mass$

$C_{f1}\ \&\ C_{f2} - Correction\ factors\ fro\ N_c\ and\ N_c\ respectively.$

Ref. to *Clause 5.2.4 Bearing Capacity Factors (Table 5.4) – Pg. 146*, states in Foundation on Rock – Duncan C. Wyllie. The subsequent modification factors should be applied to N_c and N_γ for different foundation shapes.

Table 5. Correction factors for foundation shapes given on Foundation on Rock. (Duncan C. Wyllie, 1991)

Foundation Shape	$\boldsymbol{C_{f1}}$	$\boldsymbol{C_{f2}}$
Square	1.25	0.85
Rectangular : $L_f/B_f = 2*$	1.12	0.90
Rectangular : $L_f/B_f = 5*$ $* L_f - length\ of\ foundation$	1.05	0.95
Circular	1.20	0.70

2.2.2 PRACTICAL METHOD

Classification of pile compression load tests can be divided in to three categories, which are Static, Dynamic and Statnamic load testing.

- M.4.1 – SLT – Static Load Test
- M.4.2 – HSDLT – High Strain Dynamic Load Test
- M.4.3 – RLT – Rapid (Statnamic) Load Test

Static load test is ideal for pile capacity calculation as its conventionally practiced over a period and it supposed to replicate real behaviour by maintaining load for a long-term condition. Dynamic load tests are commonly carried out as an addition to static load tests and are usually less expensive, compared to static load tests.

Statnamic load test is recently developed method, which generate pressure on pile by burning a solid fuel in a chamber. The developed pressure exerted upward force on reaction masses at the same time equal and opposite force acts downward on pile. The comparative advantages over other testing are load is perfectly axial, eliminates tensile stresses due to relatively slow load application, compression load applied both pile and soil and accurate reading for load-settlement behaviour.

In this research discussed about M.4.1 - SLT and M.4.2 - HSDLT methods, where sleeted test pile was undergone for both SLT and HSDLT.

2.2.2.1 M.4.1 – STATIC LOAD TEST

In this method of loading, load applies on the top of the pile while monitoring the pile top settlement. Depending on the way the load is applied on pile, two types of SLT are practiced namely, constant rate of penetration (CRP) test and maintained load test (MLT). In CRP test, compressive force on the pile is gradually increased to cause the test pile to penetrate in to soil at a constant rate, till failure take place or a specific test load is reached.

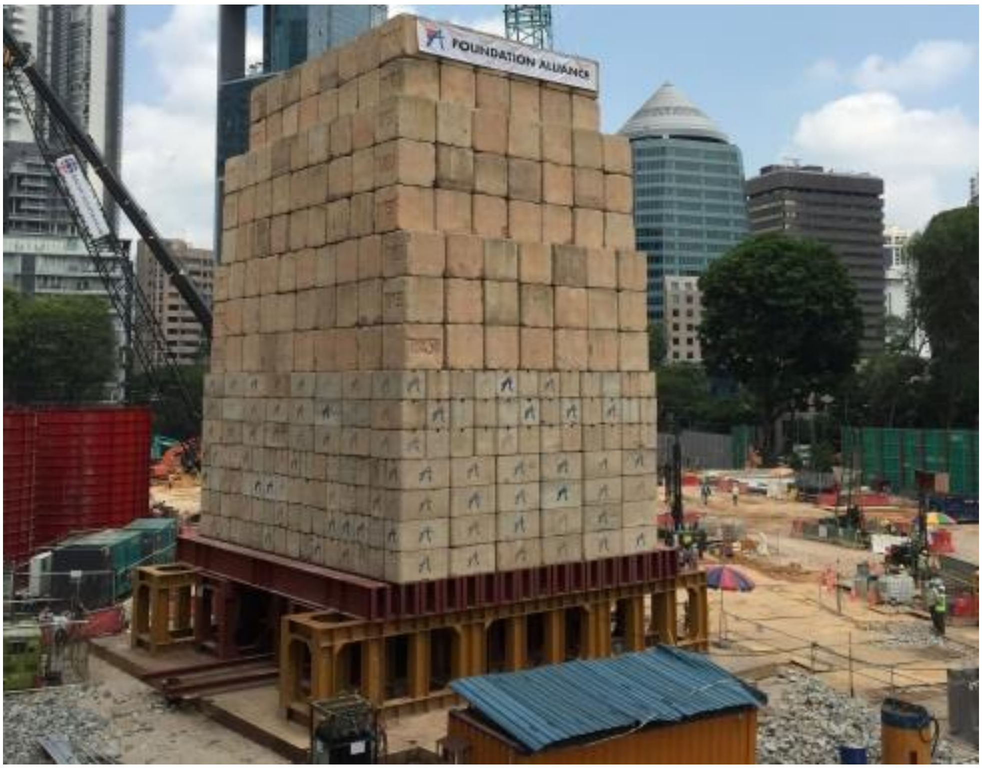


Figure 12. MLT test arrangement with loading blocks

In the MLT, load is escalated in steps up to some multiple, for example, one and one-half or two times the working load, with time-settlement readings logged at each step of loading and unloading. At each loading step, the load is maintained constant until the rate of settlement of the pile is smaller than a specific value, for example 0.25mm/hr, or over a certain specified time period. As it is evident from the testing procedure, the CRP test can finish within a shorter time period than the MLT.

The pile was tested by applying load as specified on the selected pile, where the load consist of Kentledge and concrete blocks (weighing 2.2 tons each) is brought to bear on the pile through 1000 MT hydraulic jacks as per the load to be tested. The test method used was as per CIDA publication No. CIDA/SP/101-Section 5 – Pile Testing by Maintained Load Test.

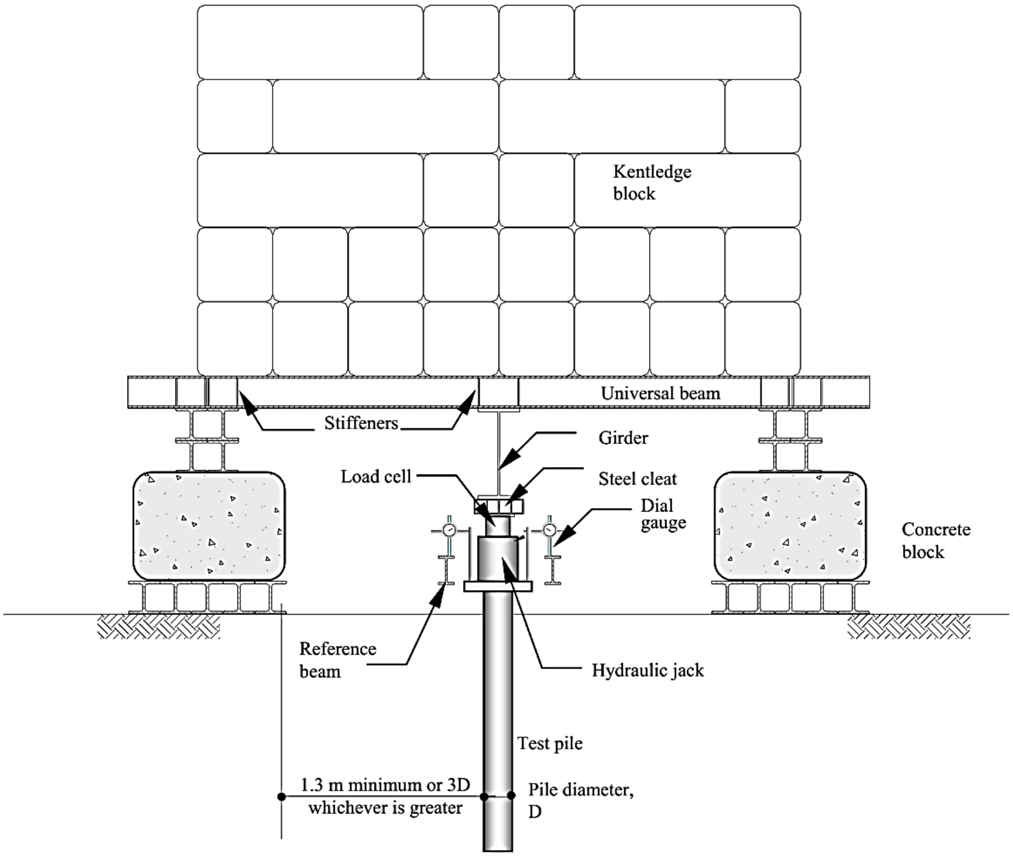


Figure 13. Typical arrangement of compression static load test (Hong Kong-Geo Publication No. 1/2006)

The Kentledge is prepared using steel girders of 300mm x 800mm and 12 m lengths. The main girder consists of two aforesaid girders welded together to form a single beam which is located immediately on top of the pile enclosing the entire pile diameter. A steel plate is placed on top of the test pile which is the base for hydraulic jack. Further steel plates are placed on top of the packing steel plate to lift the Kentledge to ensure the direct transfer of the load to the selected pile.

The test load to be selected two times the working load on any selected test pile and 1.5 times for any working pile according to specification.

The pile head will be trimmed if required and the surrounding ground level reduced to allow the pile head to exposed. The pile head will be thoroughly cleaned and capped with pile build-up concrete if required, to ensure a firm bearing surface perpendicular to the pile axis.

It is essential to ensure that the surrounding area has the capacity to safely carry the total of the Kentledge load once erected. For this purpose, it is proposed that the area identified for the construction of the Kentledge structure shall be compacted by rolling with additional fill material if required and additional support cubes be providing to cater for any initial settlement. A layer of ABC may be used for this purpose.

Surrounding ground surface shall be levelled and prepare for placing concrete blocks for load test.

Load on the test pile shall be applied in equal increments up to the required test load and maintained constant for a specific period of time. Each load increment is normally 25% of the test load. Tomlinson (1994) suggested the loading increments specified in (Table 6) to be used during a maintained load test.

Table 6. Loading increments and maintaining time period (Tomlinson M. J., 1994)

Cycle	***Load***	***Minimum time of holding the load***
1st Cycle	25% DVL	30 min.
	50% DVL	30 min.
	75% DVL	30 min.
	100%	1 hour
	75%	10 min.
	50%	10 min.
	25%	10 min.
	0	1 hour
	100%	6 hours
2nd Cycle	100%DVL + 25%SWL	1 hour
	100%DVL + 50%SWL	1 hour
	100%DVL + 75%SWL	1 hour
	100%DVL + 100%SWL	6 hour
	100%DVL + 75%SWL	10 min.
	100%DVL + 50%SWL	10 min.
	100%DVL + 25%SWL	10 min.
	100%DVL	10 min.
	75%DVL	10 min.
	50%DVL	10 min.
	25%DVL	10 min.
	0	1 hour

DVL – Design Verification Load

SWL – Specified Working Load

The time duration for the test has been selected to ensure that at each point of load increment the load shall be held for the time period in Table above.

The loading shall be terminated if any case is observed as followed;

1. The settlement of maximum loading exceeds to allowable value as specified in Schedule 2 of the CIDA specification (CIDA/SP/101)
2. Until the maximum test load has been applied and maintained.

During the testing at end of holding period of each load increment, dial gauges' readings were recorded against respective load and similarly records during unloading. The relevant readings will be used to produce Load – Settlement curve.

2.2.2.1.1 ULTIMATE FAILURE CAPACITY OF PILES

The true ultimate failure of the pile is defined as the load related to the point in the load – settlement curve, where settlement continues to increase without additional increase in the load (point C in Figure.14). A well-established vertical region beyond point C may be obtained for piles getting a large portion of the capacity from the skin friction resistance (floating pile) However, for rock socketed end bearing piles, load increases continuously with settlement. Therefore, a 'true ultimate condition' shown in Figure 14 is very difficult to achieve especially for end bearing bored piles socketed in to the bed rock. Therefore, there are other definitions of ultimate capacity for such piles as mentioned below;

1. The load equivalent to settlement of the pile equal to 10% (0.1×D) of the pile diameter.
2. The load relates to further increase of gross settlement inconsistently proportionate to the increase in load (point A in Figure 14).
3. The load designated by the intersection of tangent lines drawn through the initial, flatter portion of the gross settlement curve and the steeper portion of the same curve. (point B in Figure 14).

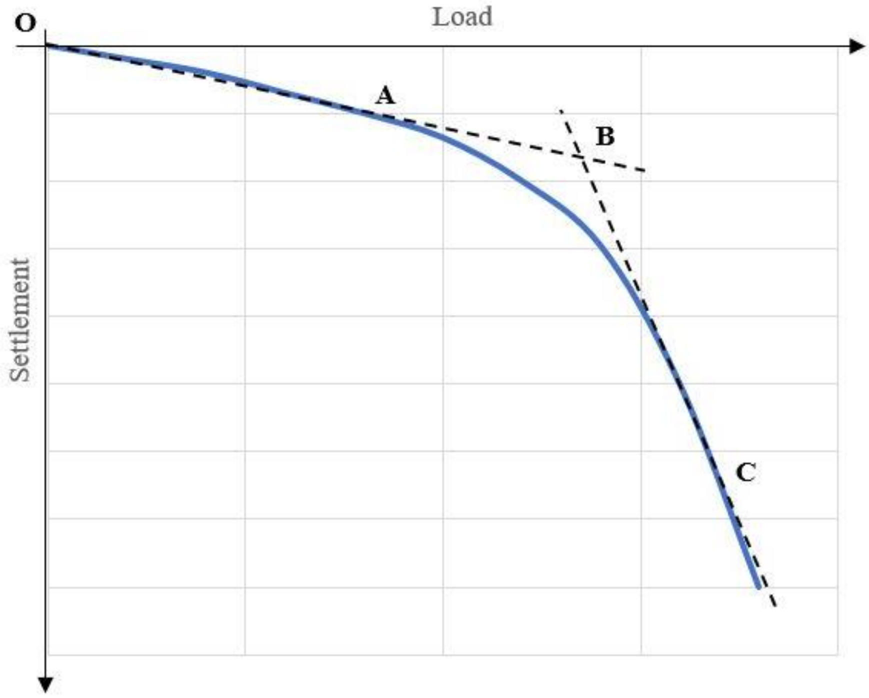


Figure 14. Typical Load - Settlement curve showing ultimate load based on some failure criteria.

The location of points A and B depend on the judgment of the person interpreting the load – settlement curve. The drawing of the initial tangent and the tangent of the flatter portion of the curve also depends on the personal judgement. Moreover, the scale of the graph might have a certain influence on the ultimate capacities determined by these methods. A good method for estimation for the ultimate capacity should be free of the scale effects and the personal judgement of the interpreter.

As loading the pile beyond 'failure' is a practically difficult and uneconomical test. Therefore, proof loading method is mostly adopted, which is estimation of the failure load using a load test results, in which the pile is not loaded to achieve failure and an extrapolation technique is used to estimate the failure load.

The extrapolation techniques generally involve a mathematical relationship representing the variation of the load and settlement during load testing. There are several extrapolation techniques such as;

- Chin-Konder Extrapolation
- Brinch Hansen 80% criterion
- Decourt's Extrapolatioin.

In this research, author has selected Chin-Konder Extrapolation for related study.

2.2.2.1.2 CHIN-KONDER EXTRAPOLATION

The researchers Chin (1971) and Konder (1963) proposed the extrapolation technique, which is called as the Chin-Konder Extrapolation. The extrapolation is based on the observations of the field pile load testing and presumed that the typical Load – Settlement relationship of piles is parabolic.

The equation of the Load – Settlement relationship is assumed to take the form given in Equation (13),

$$\boldsymbol{P} = {}^{\boldsymbol{S}}\!/_{(\boldsymbol{mS} + \boldsymbol{C})} \qquad \ldots (13)$$

Where; $S - Settlement\ at\ load\ 'P'$

m & $C - Constants$

The above equation could be re-arranged to take the form given in Equation (13)

$$^{\boldsymbol{S}}\!/_{\boldsymbol{P}} = \boldsymbol{mS} + \boldsymbol{C} \qquad \ldots (14)$$

$$\boldsymbol{P_u} = {}^{\boldsymbol{1}}\!/_{\boldsymbol{m}} \qquad \ldots (15)$$

Where; $P_u - Ultimate\ failure\ load.$

Therefore, when the ratio between the settlement and the load $(^{S}/_{P})$ is plotted against the settlement (S), the graph should be a straight line and the constants m and C could be obtained from the gradient and the intercept of the line. The load corresponding to a large displacement ($P_u - Ultimate\ failure\ load.$) can be obtained by the inverse of the slope (m), as given in Equation (15).

2.2.2.1.3 ESTIMATION OF MOBILIZED SKIN FRICTION AND END BEARING

Separation of the mobilized capacity, in to skin friction and end bearing is also important when using pile load testing data in designing of piles. For this purpose, the profile of the load – settlement curve could be used. There are methods to separate mobilized skin friction and end bearing based on the profile of the load – settlement curve. Such as;

- Method proposed by Chin (1978)

- Method proposed by Van Weele (1957)

These methods yield only approximate results and the capacities estimated are somewhat subjective to the personal judgement of the interpreter. Here author will consider Van Weele method.

2.2.2.1.4 VAN WEELE METHOD (1957)

Van Weele (1957) proposed that even though a normal load-settlement curve does not directly give the skin friction and end bearing separately, the slope of the load-settlement curve be subject to on the relative magnitude of the shaft skin friction and end bearing and the distribution of the skin friction along the pile shaft. Referred to Van Weele (1957), when a pile is loaded at first the load is carried greatly by skin friction till the limiting skin friction mobilized along the shaft. When the limiting skin friction is mobilized, the point load increases approximately linear until the ultimate end bearing capacity is reached. At the point of the ultimate end bearing, the load settlement curve becomes vertical indicative of large settlement due to any additional load increment on the pile.

Based on the above argument, a typical load settlement curve has three distinct regions as below:

I. Initial straight line segment: within which the capacity is mainly from the skin friction plus small contribution from end bearing (region form point O to A in Figure.15). Point A, needs some visual interpretation as there is rarely have a sharp discontinuity in the curve.
II. Middle curved segment: within which the load capacity is the addition of the limiting skin friction plus the approximately linearly increasing point bearing capacity (region form appoint A to point B in Figure 15).
III. Final segment: often the vertical asymptote is expected and the test is ended before a “vertical” branch is established (region form point B to the point where curve become vertical in Figure 15).

Based on Van Weele (1957) concept, Bowels (1996), states that a line drawn parallel to the point bearing region (AB in Figure 15) through the origin, as shown in Figure 15 can be used to separate the skin friction and end bearing.

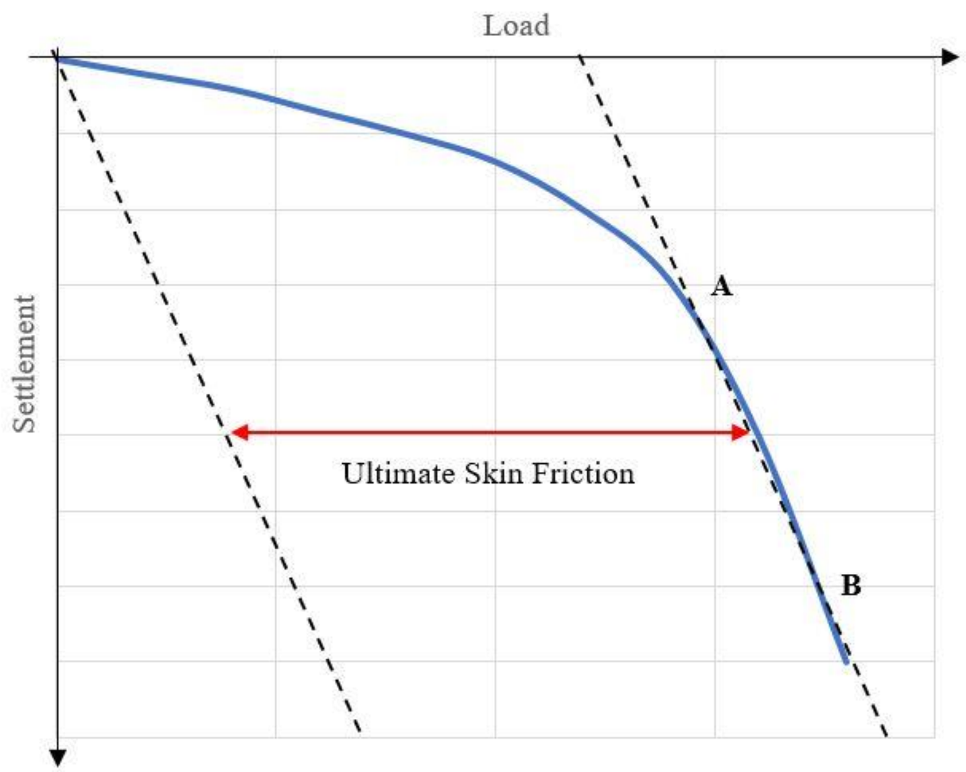


Figure 15. Load-Settlement curve showing different regions from Van Weele (1957) and Bowles (1996)

2.2.2.2 M.4.2 – HIGH STRAIN DYNAMIC LOAD TEST

High Strain Dynamic Load Test (HSDLT) is generally carried out by putting a quick loading on a cast-in-situ pile using a drop weight or hammer. The testing method should be in accordance with at testing standards such as ASTM D4945 (1995b).

HSDLT required a drop weight (weight should select as per test load), strain transducers and accelerometers, accompanied by suitable data logger, processer and measuring equipment.

The drop weight should have a capability large enough to cause adequate pile movement such that the shaft skin resistance of the pile can be entirely mobilized. A steel guide frame arrangement is to make sure that the force will act on the pile true vertically.

Figure 16. HSDLT arrangement with guide frame and 28 ton drop hammer.

Strain transducers includes full-bridge resistive foil gauge. The accelerometer consists of a quartz crystal that generates a voltage that is linearly proportional to acceleration. Two strain sensors and accelerometers are fixed to the opposite side of the pile by drilling and bolting directly on the pile shaft, and are positioned as at least two piles with the longest side of the lower part of the longest side of the diameter or twice the length or less to ensure reasonable and uniform stress field at the measuring height. Variations in the cross-section of the pile due to the connection may affect the signal ratio and thus the quality of the data. Engineer Level or Total Station can be used to monitor the vertical movements of the pile head during testing.

In the test, record the strain and acceleration measured at the pile head for each hit. The signal from the instrument is transmitted to the data recording, filtering and display device to determine the force and velocity changes over time.

2.2.2.2.1 METHOD OF INTERPRETATION

There are two conventional analysis methods based on wave propagation theory, namely direct method and indirect method. The direct analysis method is suitable for the measurement value obtained directly from (one) hit, while the indirect analysis method is based on the signal matching method of the result of one or more hits.

- Direct Methods – CASE®, IMPEDANCE® and TNO®.
- Indirect Methods – CAPWAP®, TNOWAVE® and SIMBAT®.

Test results were analyzed using CAPWAP® (CAse Pile Wave Analysis Program), where The soil is represented by a series of elastoplastic springs in parallel with linear shock absorbers, similar to those used in the wave equation analysis proposed by Smith (1962). When the piles are relatively short, the soil can also be modelled as a continuum. CAPWAP takes acceleration time data as input boundary conditions for measurement. The program calculates the curve of force versus time and compares it with the recorded data. If it does not match, adjust the soil model. This iterative process is repeated until a satisfactory match is obtained between the calculated force-time diagram and the measured force-time diagram.

The dynamic component of penetration resistance is given by;

$$\boldsymbol{R_d = j_s . v_p . R_s} \qquad \ldots (16)$$

Where; $j_s - Smith\ damping\ coefficient$

$v_p - velocity\ of\ pile\ at\ each\ segment$

$R_s - static\ component\ of\ penetration\ resistance$

Input parameters for the analysis include pile dimensions and properties, soil model parameters including the static pile capacity, smith damping coefficient (j_s) and soil quake, and the signals measured in the field. The output form is the resistance of the static unit shaft against depth and foundation response, and the static load-settlement relationship up to about 1.5 times the working load.

2.3 NUMERICAL METHOD

Finite Element Model (FEM) can be used to model the geotechnical arrangement and obtaining required details. Commercially available PLAXIX 2D software was used for modeling and analysis.

3 METHODOLOGY

3.1 SELECTION OF DATA

In this work an ongoing project was selected to obtain research data. The project is ‘Cinnamon Life’ (previously known as ‘Waterfront’) multi-purpose commercial complex project by Jonh Keels Holding PLC at Glennie Street, Colombo 02. A comprehensive site investigation was done by Geotech (Pvt) Ltd. which carried out 43 nos. Bore Hole (BH) investigations, using 75mm diameter rotary wash boring method with NW casing size.

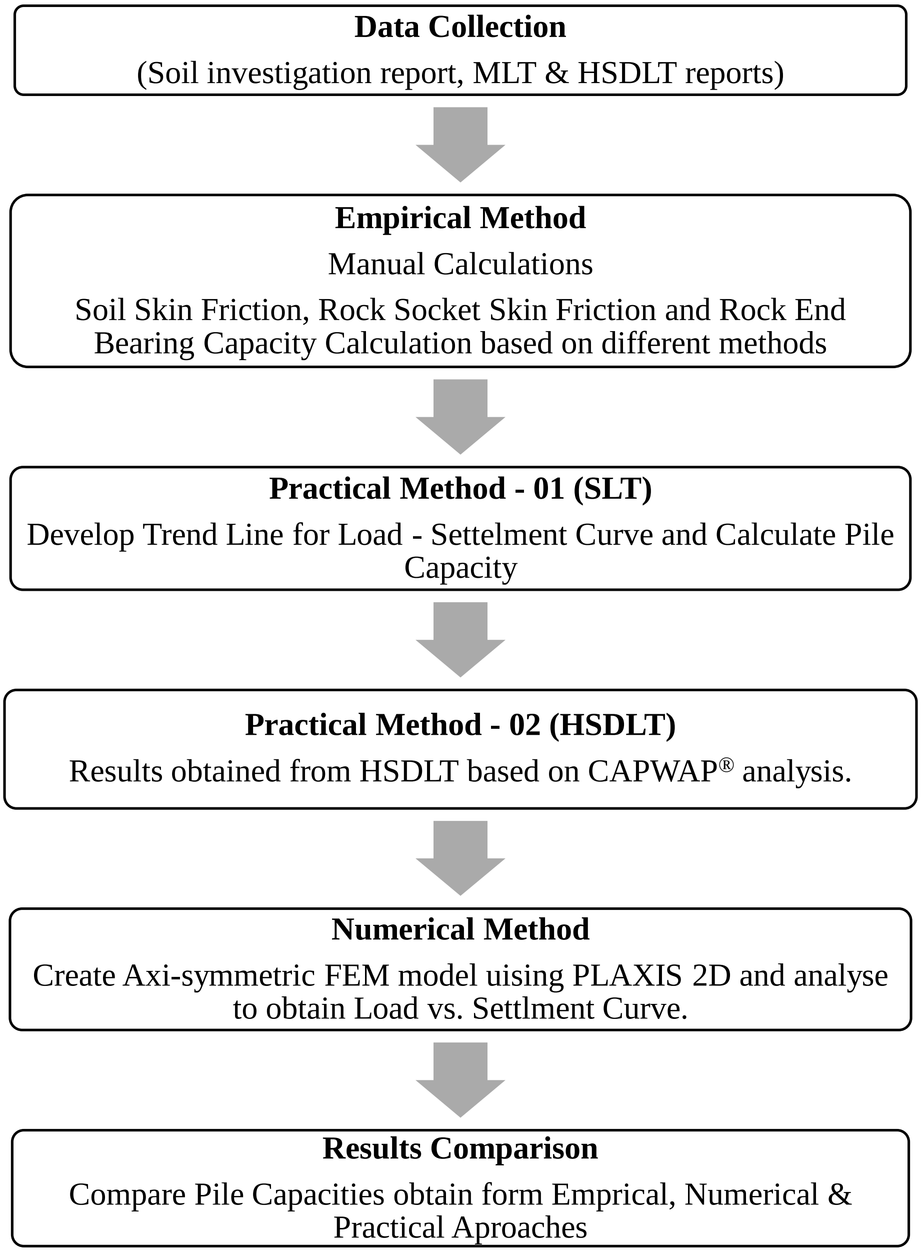
Author selected test pile (TP01) with diameter 1000mm and test report for the MLT load test. Required soil and rock parameters were obtained from soil investigation report and recommendations granted by expert geotechnical specialists. In brief following reports/documents were referred to obtain the research details;

- Geotechnical investigation report (No: G/2952) by Geotech (Pvt) Ltd.
- BH location & Pile locations layout drawing
- Maintain Load Test report by Nawaloka Piling (Pvt) Ltd.
- Pile Dynamic Analyzer (PDA) Test report by Geotech Testing Service (Pvt) Ltd.

The following steps are the used to examine between the theoretical, practical and numerical pile compression capacity.

1. Select one pile (TP01-1000mm diameter) from the case study.
2. Collect all the required data form the geotechnical investigation such as (soil layer’s classifications, strength parameters, rock RQD & CR values, intact rock UCS values and recommendations)
3. For the Empirical Pile Capacity, use empirical and semi-empirical equations to calculate soil skin frication, rock socket skin friction and end bearing separately with different researcher’s correlations.
4. For the Practical Pile Capacity, it will be estimated form the MLT results by using Chin-Konder extrapolation method. Further separate mobilized skin friction and end bearing using Van Weel method.
5. For Numerical Pile Capacity, a finite element model developed by using PLAXIS 2D software to get the pile compression capacity
6. Compare the pile capacities in different cases and critically evaluate the suitability of estimation procedure for pile capacity with standards and local guide lines.

3.2 ALGORITHM OF PROCEDURE



3.3 EMPIRICAL METHOD CALCULATION

3.3.1 CALCULATION OF SHAFT SKIN FRICTION

The soil cross section at TP01 (1000mm Dia.) bored pile location was idealized by interpolating investigation data available at BH-13, BHEX-16 and BHEX-17 in the site as shown in Figure 17.

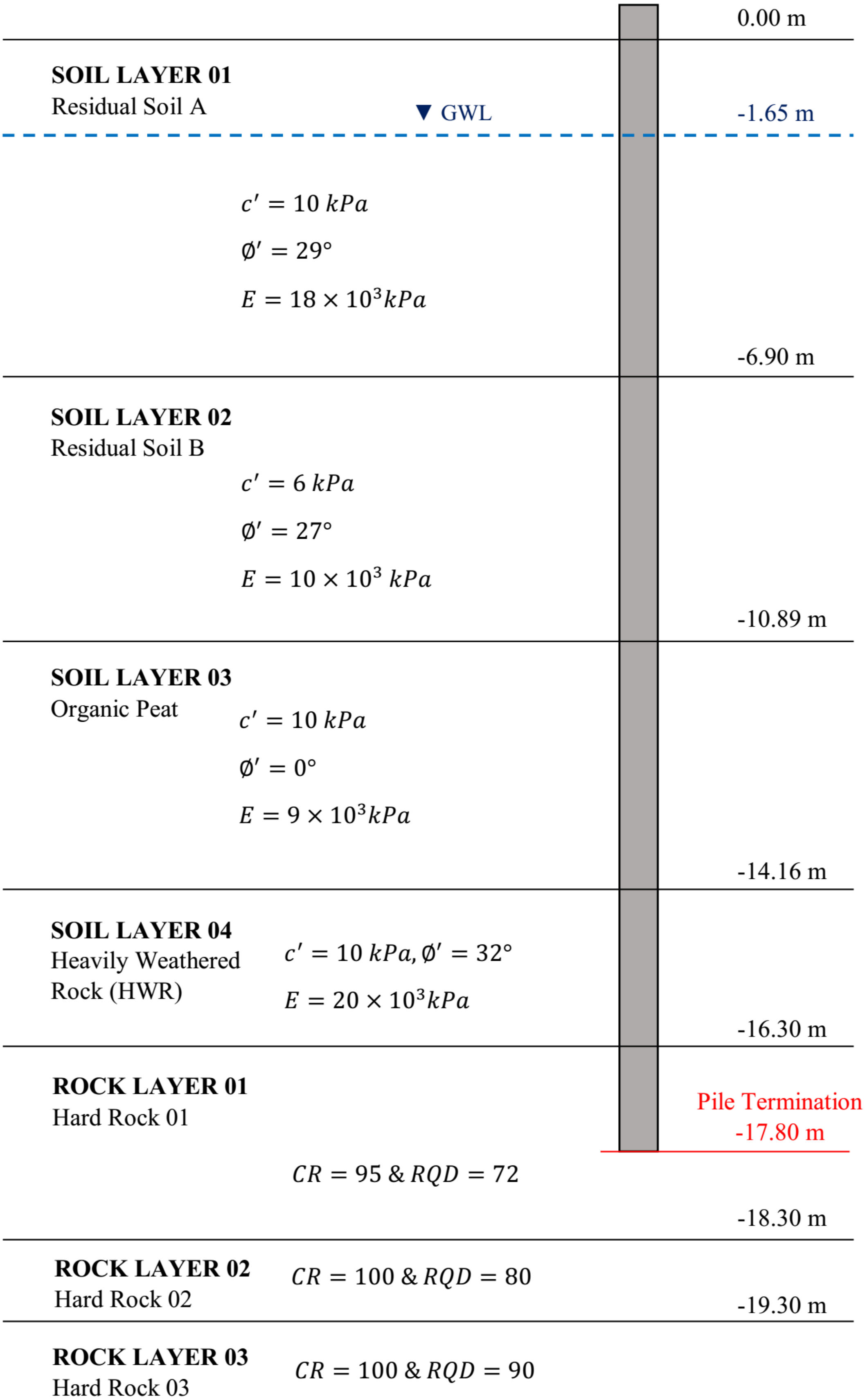


Figure 17. TP01 (1000mm Dia.) Bored Pile Idealized Cross Section as per the BH13, BHEX-16 & BHEX-17

Calculation of saturated unit weights with available $G_s - specific\ gravity$, $\omega - moisture\ content$ and $\gamma_w - unit\ weight\ of\ water$ (9.81 kNm^{-3})

$$\gamma_{sat} = {(G_s.\gamma_w + e.\gamma_w)}/{1 + e} \qquad \dots (16)$$

$$e = \omega. G_s \qquad \dots (17)$$

For Layer 01 – Residual Soil I, $G_s = 2.55$ and $\omega = 27.2\%$;

$$e = 0.272 \times 2.55 = 0.6936$$

$$\gamma_{sat1} = {\{(2.55 + 0.6936) \times 9.81\}}/{(1 + 0.6936)} = 18.79\ kNm^{-3}$$

Similarly;

$$\gamma_{sat2} = 18.33\ kNm^{-3}, \gamma_{sat3} = 18.23\ kNm^{-3}\ and\ \ \gamma_{sat4} = 18.95\ kNm^{-3}$$

3.3.1.1 M.1.1 – Method outlined in ICTAD guidelines

SPT N value correction;

$$N_{corr} = N_{field}.C_N.\eta_1.\eta_2.\eta_3.\eta_4 \qquad \dots (18)$$

$C_N = \sqrt{{95.76}/{P_o'}}$ and $\eta_1 = {E_r}/{E_{rb}}$; ($take\ E_{rb} = 70\ as\ suggested\ by\ J.E.Bowels$)

Correction factors can be obtained from the Table 7 and Table 8 suggested by Bowels (1992)

Table 7. Standard energy ratio suggested. (Bowels J. E., 1992)

E_{rb}	Reference
50 to 55 (use 55)	Schmertmann [in Robertson et al. (1983)]
60	Seed et al. (1985); Skempton (1986)
70 to 80 (use 70)	Riggs (1986)

Table 8. Suggested correction factors. (Bowels J. E., 1992)

Factors η_i For Eq. (3-3)*

Hammer for η_1	Average energy ratio E_r				Remarks
	Donut		Safety		
Country	**R-P**	**Trip**	**R-P**	**Trip/Auto**	
United States/ North America	45	—	70–80	80–100	R-P = Rope-pulley or cathead $\eta_1 = \mathbf{E}_r/\mathbf{E}_{rb} = E_r/70$ For U.S. trip/auto $w/\mathbf{E}_r = 80$ $\eta_1 = 80/70 = 1.14$
Japan	67	78	—	—	
United Kingdom	—	—	50	60	
China	50	60	—	—	

Rod length correction η_2			Remarks
Length	> 10 m	η_2 = 1.00	N is too high for $L < 10$ m
	6–10	= 0.95	
	4–6	= 0.85	
	0–4	= 0.75	

Sampler correction η_3			Remarks
	Without liner	η_3 = 1.00	Base value
With liner:	Dense sand, clay	= 0.80	N is too high with liner
	Loose sand	= 0.90	

Borehole diameter correction η_4			Remarks
Hole diameter:†	60–120 mm	η_4 = 1.00	Base value; N is too small when there is an oversize hole
	150 mm	= 1.05	
	200 mm	= 1.15	

* Data synthesized from Riggs (1986), Skempton (1986), Schmertmann (1978*a*) and Seed et al. (1985).
† η_4 = 1.00 for all diameter hollow-stem augers where SPT is taken through the stem.

For Layer 01 – Residual Soil I,

$P_o' = 15.18 \times 1.65 + (18.79 - 9.81) \times 1.8 = 25.047 + 16.164 = 41.211$

$C_N = \sqrt{{}^{95.76}/_{41.211}}$ =1.52 $\quad \eta_1 = {}^{70}/_{70} = 1.0 \quad \eta_2 = 0.75 \quad \eta_3 = \eta_4 = 1.0$

Use Eq. (14), $N_{corr} = 25 \times 1.52 \times 1.0 \times 0.75 \times 1.0 \times 1.0 = 29\ (integer\ value)$

Use Eq. (05), $f_{s1} = 1.3 \times 29 = 37.7\ kNm^{-2}$

Therefore, skin friction force due to Layer 01 - Residual Soil I

$$F_{s1} = f_{s1} \times A = 37.7 \times \pi \times 6.9 = 817.22\ kN$$

Similarly, other layers can be calculated and results are tabulated in Table 9 and Table 10

Table 9. SPT correction for soil layers.

Soil Layer	P'_o	C_N	η_1	η_2	η_3	η_4	N_{field}	N_{corr}
Layer 01	41.211	1.52	1.0	0.75	1.0	1.0	25	29
Layer 02	89.232	1.04	1.0	0.95	1.0	1.0	12	12
Layer 03	115.367	0.91	1.0	1.0	1.0	1.0	03	03
Layer 04	134.243	0.84	1.0	1.0	1.0	1.0	36	30

Table 10. Total Skin Friction Calculation as per ICTAD guidelines.

Soil Layer	N_{corr}	$f_s = 1.3 \times N_{corr} \leq 100(kNm^{-2})$	Layer thk. (m)	$F_s\ (kN)$
Layer 01	29	37.7	6.9	817.22
Layer 02	12	15.6	4.0	196.04
Layer 03	03	3.9	3.26	39.942
Layer 04	30	39.0	2.14	262.197
Total Soil Skin Friction : $F_{s.total}$				**1315.40**

3.3.1.2 M.1.2 – O'Neill and Reese Method

For Layer 01 – Residual Soil I;

$\sigma_{vm1} = 41.211\ kNm^{-2}, Z_i = 3.45m$

$For\ SPT\ N_{uncorr.} = 25 \geq 15\ blows/0.3m:\ \beta = 1.5 - 0.245(Z_i)^{0.5}$ =1.045

Use Eq. (04); $f_{s1} = 1.045 \times 41.211 = 43.065\ kNm^{-2}$

Therefore, skin friction force due to Layer 01 - Residual Soil I

$$F_{s1} = f_{s1} \times A = 43.065 \times \pi \times 6.9 = 933.52\ kN$$

Similarly, other layers can be calculated and results are tabulated in (Table 11)

Table 11. Soil skin friction capacity as per O'Neil and Reese Method

Soil Layer	σ_{vm}	β	$f_s = \beta.\sigma'_{vm}. \leq 200(kNm^{-2})$	Layer thk. (m)	$F_s\ (kN)$
Layer 01	41.211	1.045	43.065	6.9	933.52
Layer 02	89.232	0.615	54.878	4.0	689.617
Layer 03	115.367	0.127	14.652	3.26	150.06
Layer 04	134.243	0.544	73.028	2.14	490.968
Total Soil Skin Friction : $F_{s.total}$					**2264.165**

3.3.2 CALCULATION OF ROCK SOCKET SKIN FRICTION

Ref. to soil investigation report:

$UCS = q_{uc} = \sigma_c = 23.22\ MN/m^2$ (For rock at 13.70-15.70m depth at BH-EX17)

RQD = 72 and Fracture Spacing = 4 per meter

Rock socketed length = 1.5 m

3.3.2.1 M.2.1 – Limiting value given in ICTAD guidelines

Use Eq. (05) $SPT\ N > 100$; therefore limiting value - $f_r = 200\ kNm^{-2}$

3.3.2.2 M.2.2 – Rowe and Armitage (1987)

Use Eq. (06) $f_r = 0.60 \times 23.22^{0.5} = 2.89\ MNm^{-2} = 2890\ kNm^{-2}$

3.3.2.3 M.2.3 – Method given in Hong Kong guidelines

Use Eq. (07) $\alpha = 0.2$; therefore $f_r = 0.2 \times 23.22^{0.5}\ MNm^{-2} = 964\ kNm^{-2}$

3.3.2.4 M.2.4 – William and Pells (1981)

For $UCS = q_{uc} = \sigma_c = 23.22\ MN/m^2 \Rightarrow \alpha = 0.1$

For RQD =72 and Fracture Spacing = 4 per m ⇒ $j = 0.5$; therefore $\beta = 0.82$

Use Eq. (08) $f_r = 0.1 \times 0.82 \times 23.22 = 1.904\ MNm^{-2} = 1904\ kNm^{-2}$

3.3.2.5 M.2.5 - Meigh and Wolski (1979)

Use Eq. (09) $f_r = 0.55 \times 101 \times 23.22^{0.6} = 366.61\ kNm^{-2}$

3.3.2.6 M.2.6 – Hovarth and Kenny (1987)

Use Eq. (10) $f_r = 0.65 \times 101 \times (\frac{23.22\times10^3}{101})^{0.5} = 995.42\ kNm^{-2}$

3.3.2.7 Summary of Rock Skin Friction Capacities

Table 12. Summary of Soil Skin Capacity from different methods

Method	f_r (kNm^{-2})	Rock Socket Shaft Area - πDH (m^2)	Capacity (kN)
M.2.1 ICTAD guidelines	200	$\pi \times 1.0 \times 1.5$	942.48
M.2.2 Rowe and Armitage	2890	$\pi \times 1.0 \times 1.5$	13618.80
M.2.3 HK guidelines	964	$\pi \times 1.0 \times 1.5$	4542.74
M.2.4 Williams & Pells	1904	$\pi \times 1.0 \times 1.5$	8972.39
M.2.5 Meigh and Wolski	366.6	$\pi \times 1.0 \times 1.5$	1727.56
M2.6 Hovarth and Kenny	995.42	$\pi \times 1.0 \times 1.5$	4690.81

3.3.3 CALCULATION OF ROCK SOCKET END BEARING CAPACITY

3.3.3.1 M.3.1 – Method outlined in BS8004 (1986)

Rock Group – 02 : Metamorphic Rocks and Fracture Spacing 4 per meter ⇒ Medium spaced discontinuities (250mm)

For $UCS = 23.22\ MN/m^2 \Rightarrow q_a = 15$ MPa

$FOS = 2.5 \ \Rightarrow q_{ULT} = 15 \times 2.5 = 37.5$ MPa

3.3.3.2 M.3.2 – RMR method (Hong Kong guideline)

Rock Layer at Pile Toe (16.3m to 18.3m)

$UCS = 23.22\ MNm^{-2}$

$RQD = 72$ and $CR = 95$

$Avg. discontinuity\ spacing = 239\ mm$

$Discontinuity\ Frequency = 4\ m^{-1}$

Table 13. RMR for rock at pile toe based on HK Guidelines

RMR value for rock at pile tip	***Rating***
Strength of intact rock	2
RQD designation	13
Spacing of joints	10
Discontinuity length	2

Separation rating	4
Roughness rating	3
Infilling (gouge) rating	4
Weathering rating	3
Ground water	7
RMR Value	**48**

Ref. recommended value of HK guidelines; RMR = 48 ⇒ $q_a = 4.6\ Mpa$

$FOS = 2.5\ \Rightarrow q_{ULT} = 4.6 \times 2.5 = 11.5$ MPa

3.3.3.3 M.3.3 – Kulhawy and Goodman

For RQD = 72% ⇒ $q_c = 0.33 q_{uc} = 0.33 \times 23.22 = 7.7\ MPa$

3.3.3.4 M.3.4 - Method outlined in M. J. Tomlinson

For Medium Friction – Gneiss Rock Type ⇒ $\emptyset = 30°$ | $N_{\emptyset} = \tan\left(45 + {}^{\emptyset}/_{2}\right)^2 = 3.0$

Use Eq. (11); $q_b = 2.N_{\emptyset}.q_{uc} = 2 \times 3 \times 23.22 = 139.32\ MPa$

3.3.3.5 M.3.5 - Method Proposed by Peck et. al (1986)

For RQD = 72 ⇒ $q_a = 11\ MPa$

$FOS = 2.5\ \Rightarrow q_{ULT} = 11 \times 2.5 = 27.5$ MPa

3.3.3.6 M.3.6 – Bell Solution

$B_f = 1.0\ m\ (pile\ diameter)$

$d_r = 1.5\ m\ (pile\ rock\ socket\ depth)$

$\gamma_r = 25.51\ kN/m^3$

Ref. (Table 05) shape factors for circular pile toe : $C_{f1} = 1.2$ and $C_{f2} = 0.7$

Ref. (Table 29) rock equivalent Mohr-Coulomb parameters: $c = 1104\ kPa$ and $\emptyset = 39.3°$

$N_{\emptyset} = \tan\left(45 + {}^{39.3}/_{2}\right)^2 = 4.455$

$N_q = 4.455^2 = 19.849$

$N_\gamma = \sqrt{4.455} \times (4.455^2 - 1) = 39.78$

$N_c = 2\sqrt{4.455} \times (4.455 + 1) = 23.027$

Use Eqn. (12): $q_b = (1104 \times 1.2 \times 23.027) + (0.5 \times 1.0 \times 25.51 \times 0.7 \times 39.78) + (25.51 \times 1.5 \times 19.849)$

$$q_b = 30506.17 + 355.176 + 759.522 = 31620.868\ kPa$$

$$q_b = 31.621\ MPa$$

3.3.3.7 Summary of Rock End Bearing Capacities

Table 14. Summary of rock end bearing capacities based on different correlations

Method	$q_b\ (MPa)$	***Pile Toe Area -*** $\pi D^2/4$ ***(***m^2***)***	***End Bearing Capacity (kN)***
M.3.1 Method Outline in BS8004	37.5	$\pi/4$	29452.43
M.3.2 RMR method (Hong Kong guideline)	11.5	$\pi/4$	9032.08
M.3.3 Kulhawy and Goodman	7.7	$\pi/4$	6047.57
M.3.4 Method outlined in M. J. Tomlinson	139.32	$\pi/4$	109421.67
M.3.5 Method Proposed by Peck et. al	27.5	$\pi/4$	21598.45
M.3.6 Bell Solution	31.621	$\pi/4$	24835.08

3.4 PRACTICAL METHOD CALCULATION

3.4.1 M.4.1 – STATIC LOAD TEST RESULTS ANALYSIS

With reference to Piling works for the Waterfront Integrated Resort Project - Maintain Load Test Report – TP01 by Nawaloka Piling (Pvt) Ltd.

The Maintained Load Test was carried out as indicated in the factual report of TP01, which is a 1000mm dia. pile. The objective of the test was indicated as being to confirm the pile carrying capacity as had been used in the design.

3.4.2 ESTIMATION OF THE ULTIMATE CARRYING CAPACITY

The ultimate carrying capacity estimated using Chin-Konder extrapolation technique. The following details are obtained from the construction records of pile borings and sample cutting observations;

Table 15. TP01 pile maintain load test data.

Pile ref. no.	TP01
Pile diameter	1000 mm
Concrete grade	50 Nmm^{-2}
Date of casting of pile	04^{th} Apr. 2014
Date of testing of pile	09^{th} Jun. 2014,15.00hrs to 10^{th} Jun. 2014,00.15hrs.
Age of the pile at load test	65 days
Design Working Load (DWL)	6283 kN
150% of DWL	9425 kN
250% of DWL	15708 kN

Table 16. TP01 pile geological details at construction records.

Ground elev. At top of pile bore	+3.145 m MSL
Depth of commencement of HW Rock	14.60 m
Elev. At commencement of HW Rock	-11.455 m MSL
Depth of commencement of Fresh Rock	16.30 m
Elev. At commencement of Fresh Rock	-13.155 m MSL
Depth of termination	17.80 m
Elev. At pile termination	-14.655 m MSL
Thickness of HWR + Fresh Rock	3.20 m

Thickness of Fresh Rock	1.50 m

Table 17. Maintain Load Test Results.

Load (kN)	**Load WL%**	**Average Settlement (mm)**	***Settlment/Load***
0.000	0	0.18	NA
1570.750	25	0.91	0.000579341
3141.500	50	1.77	0.000563425
4712.250	75	2.59	0.000549631
6283.000	100	3.96	0.000630272
7853.750	125	5.05	0.000643005
9424.500	150	6.70	0.000710913
10995.250	175	8.21	0.000746686
12566.000	200	9.77	0.000777495
14136.750	225	10.97	0.000775992
15707.500	250	14.40	0.00091676

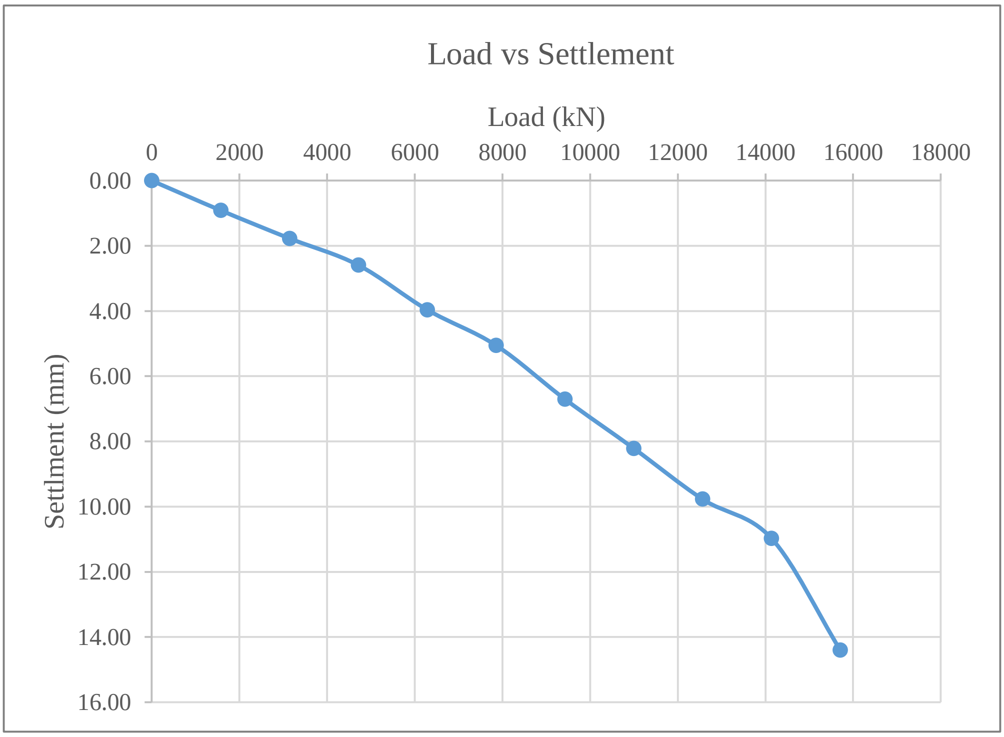


Figure 18. Load vs. Settlement Curve for SLT

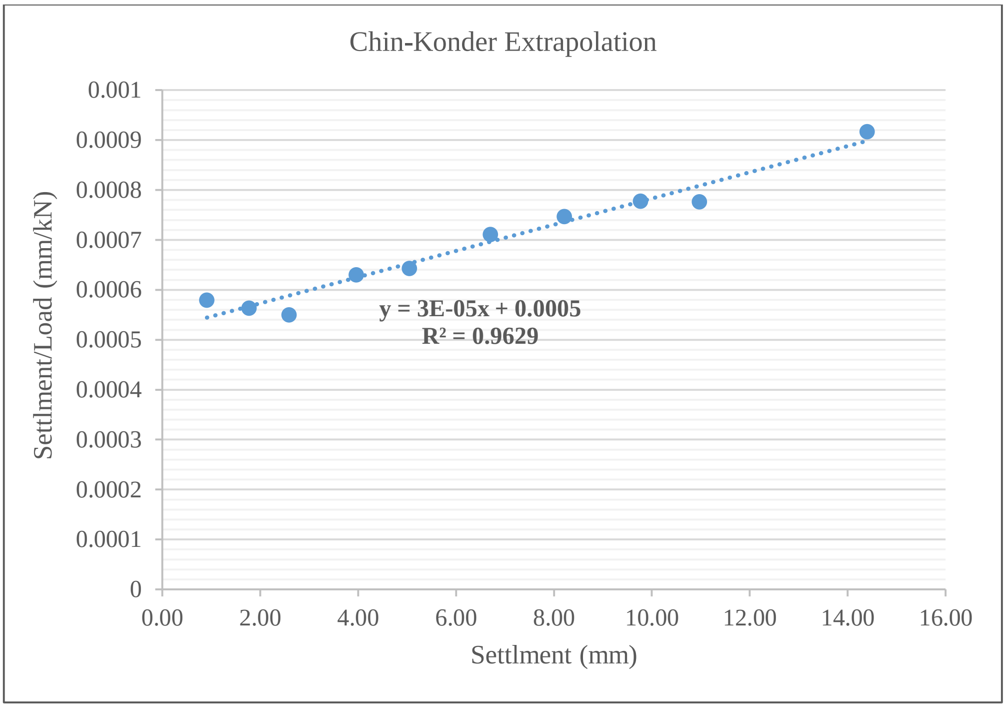


Figure 19. Chin-Konder extrapolation for TP01

Use Eq. (15) $P_u = {}^{1}/_{m} = {}^{1}/_{3 \times 10^{-5}} = 33{,}333.33\ kN$

Therefore Ultimate Pile Capacity : $P_u = 33{,}333\ kN$

3.4.3 ESTIMATION OF MOBILIZED SKIN FRICTION AND END BEARING

Van Weele curve development technique used to separate the mobilized skin friction and end bearing for test pile

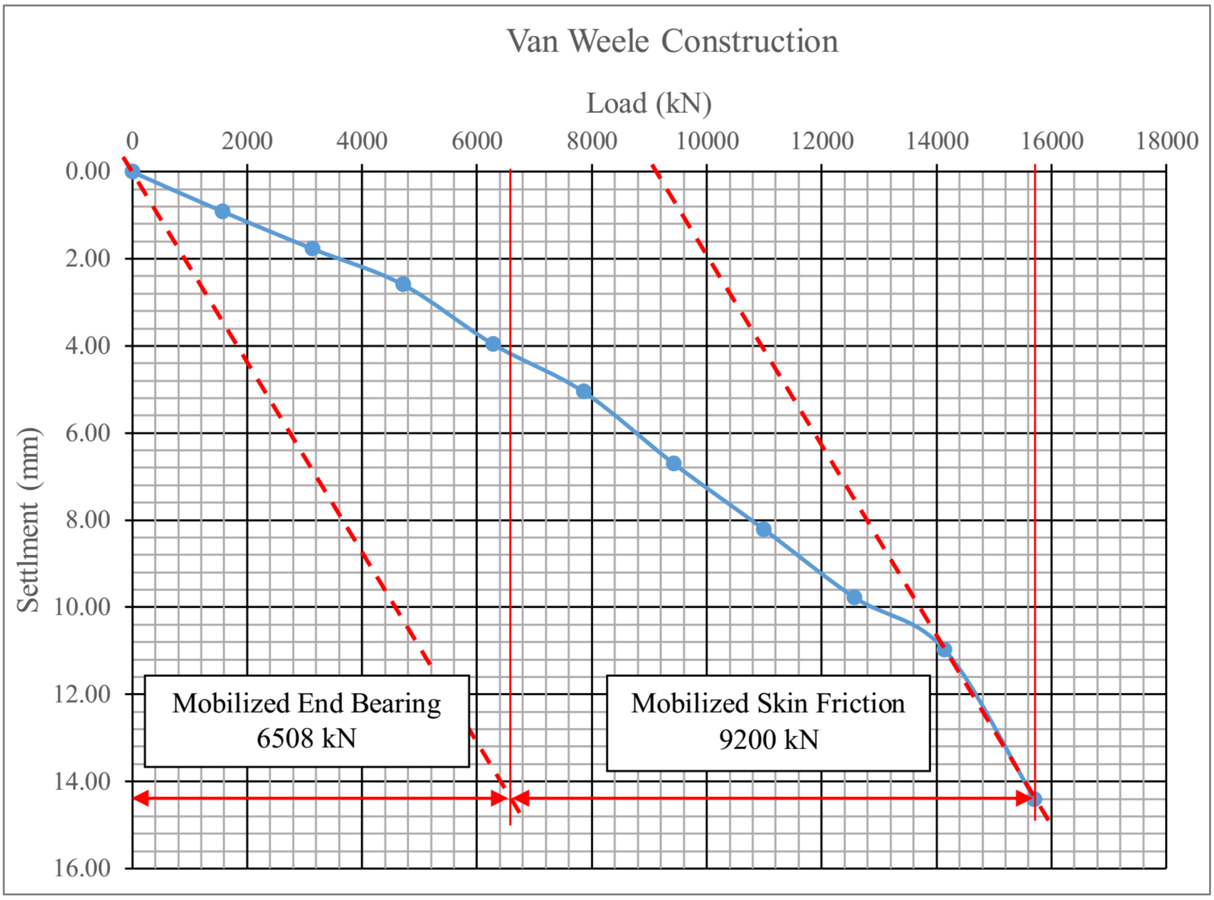


Figure 20. Van Weele construction for estimation of mobilized skin friction and end bearing from SLT

Van Weele construction for the TP01 test pile has results:

- Mobilized Skin Friction (Soil & Rock Socket) = 9200 kN
- Mobilized End Bearing = 6508 kN

3.4.4 M.4.2 – HIGH STRAIN DYNAMIC LOAD TEST RESULTS ANALYSIS

With reference to test report on the HSDLT carried out on Test Pile No. 01 (TP01) by Geotech Testing Services (Pvt) Ltd. and subsequent CAPWAP® analysis results were used for the comparison.

Following are the brief details of the HSDLT arrangements and parameters used for the CAPWAP® analysis.

Table 18. TP01 pile HSDLT data

Pile ref. no.	TP01
Pile diameter	1000 mm
Concrete grade	50 Nmm^{-2}
Date of casting of pile	04th Apr. 2014
Date of testing of pile	18th Jun. 2014 at 18.00 hrs.
Age of the pile at load test	74 days
Design Working Load (DWL)	6283 kN
Test Load	15396 kN
Hammer Type	Drop
Hammer Weight	28 Tons
Hammer Drop	1.15 m

3.4.5 CAPWAP® ANALYSIS AND RESULTS

CAPWAP® analysis results are based on mathematical model simulation, and the reported results come from the best matching model obtained during the analysis process.

In the analysis carried out, a Match Quality of 4.72 is reported. This indicates that the mathematical simulation adopted is acceptable.

In the CAPWAP® analysis, which had been carried out for this pile, the following observations are made.

Table 19. CAPWAP® analysis results and recommendations

Load carried in shaft (soil + rock) friction	9370 kN
Load carried in end bearing	7653 kN
Load carried in total	17023 kN
Mobilized end bearing pressure	9743 kNm^{-2}

Ultimate skin friction coefficient (within rock socketed region)	324 kNm^{-2}

3.5 NUMERICAL METHOD

The pile can be modeled by using Axisymmetric option, where surrounding soil layers can be modeled by using Mohr-Coulomb, pile can be modeled as Elastic material and rock layers can be modeled as Mohr-Coulomb or Elastic material. Prescribed settlement can be applied to pile head and analysis can be done to obtain Load vs. Settlement curve via. PLAXIS 2D

3.5.1 PILE AND SOIL INTERFACE REDUCTIOIN FACTOR

Pile material (concrete) and soil interface condition is an important factor, which has significant impact on the pile skin friction resistance. Interface elements simulate the interaction between the pile and the soil, between smooth and completely rough. The surface roughness of the interaction is modeled by selecting an appropriate value for the strength reduction factor in the inter face (R_{inter}).

Therefore, R_{inter} – reduction factor is based on following factors, which is considered in modeling of pile;

- Soil strata classification
- The pile material. i.e. concrete.
- The installation method, as use of bentonite slurry in the pile installation will have a negative impact on the frictional resistance of the skin because it creates a smooth surface among the pile material and the adjacent soil. As a result, compared with other installation methods, the reduction factor in this case is very small.

In general, the reduction factor of skin friction resistance due to interface condition has a value between 1.0 to 0.5.

3.5.2 GRAPHICAL BOUNDARIES

Graphical boundaries were selected as shown in (Figure 20) to avoid any disturbance for the analysis and without reducing the resolution of the required element behavior.

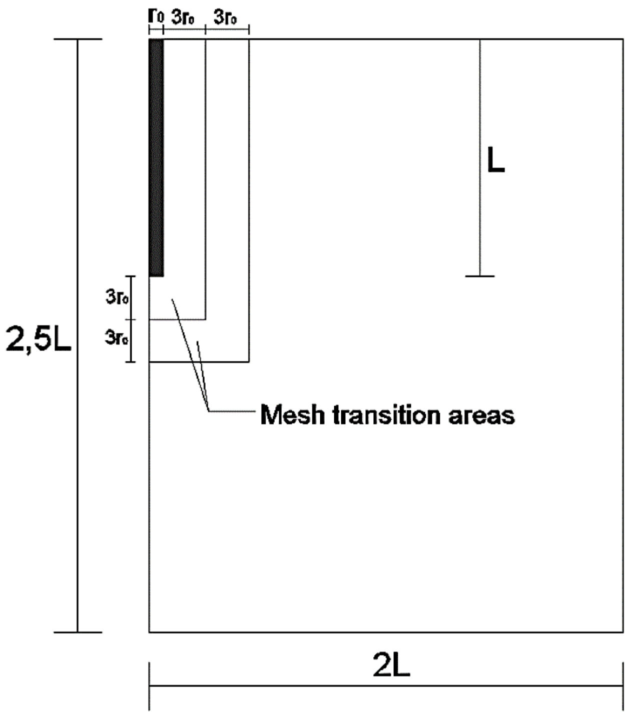


Figure 21. Geometrical boundaries of the model.

3.5.3 MATERIAL MODELS

3.5.3.1 Linear Elastic (LE) Model

The most basic LE model is one of the models available in PLAXIS 2D software package, which was used to perform linear elastic analysis of the materials used in this study. The LE model is based on Hooke's law of isotropic elasticity. It comprises two basic elastic parameters, i.e. Young's modulus (E) and Poisson's ratio (ν). LE model is not suitable for modelling the soil, even though it can be used to model the rigid volume of the soil or the rigid formation in the soil.

In this study concrete pile material is considered as LE material and model using LE modeling.

3.5.3.2 Mohr – Coulomb (MC) Model

The MC model is one of the nonlinear models used in this study. The MC model is a simple non-linear model based on the known soil parameters in most practical situations. It comprises five input parameters, Elastic modulus (E), Poisson's ratio (ν), Friction angle (ϕ), Cohesion (c) and Dilatancy angle (ψ).

Here soil layers and rock layers are considered as MC behavior and modeled using MC model.

3.5.4 ROCK LAYER CLASSIFICATION AND ROCK PARAMETERS

3.5.4.1 Elastic Modulus of Rock Layers

Elastic modulus of pile founding rock mass (E_m) which is at confined state can be obtained only if the pile toe response is measured using pile toe instrumented load test i.e. Osterburg Cell test. Pile instrumented load tests are not practiced in local context and E_m value are determined based on correlations. i.e. RMR & UCS values.

There are several researches proposing different correlations based on UCS values of intact rock and different rock mass classification systems. i.e. RMR, Q-System, GSI & RMi.

Bieniawski (1978)	$E_m = 2RMR - 100$	GPa	For RMR > 50
Serafim & Pereira (1983)	$E_m = 10^{(RMR-10)/40}$	GPa	For RMR = 20 ~ 85
Rowe & Armitage (1984)	$E_m = 0.1\left(\frac{RMR}{10}\right)^3$	GPa	
Grimstand & Barton (1993)	$E_m = 25\log_{10} Q$	GPa	For Q > 1
Clerici (1993)	$E_m = E_{r.stat} \times \frac{E_{m.dyn}}{E_{r.dyn}}$	GPa	
Palmström (1995)	$E_m = 5.6RMi^{0.375}$	GPa	For RMi > 0.1
Hoek & Brown (1998)	$E_m = \sqrt{\sigma_c/100} \times 10^{\left(\frac{GSI-10}{40}\right)}$	GPa	For σ_c < 100 MPa
Reed at. el. (1999)	$E_m = 215\sqrt{\sigma_c}$	MPa	

Where; $E_m - Modulus\ of\ deformation\ of\ rock\ mass$

$RMR - Rock\ Mass\ Rating\ system\ (Bieniawski, 1973)$

$Q - Q\ system\ (Barton\ et\ al., 1974)$

$RMi - Rock\ Mass\ Index\ (Palmst\ddot{o}m, 1995)$

$GSI - Geological\ Strenght\ Index\ (Hoek\ \&\ Brown, 1998)$

$\sigma_c - Uniaxial\ compressive\ strengh\ (in\ MPa)\ of\ intact\ rock$

$E_{r.dyn} - Dynamic\ elastic\ modulus\ of\ intact\ rock$

$E_{r.stat} - Static\ elastic\ modulus\ of\ intact\ rock$

$E_{m.dyn} - Dynamic\ in\ situ\ deformation\ modulus$

The two prevailing equations for estimating E_m by Bieniawski and Serafim & Pereira with use of the RMR system seem pertinent for fractured rock within their suggested range. (Palmstöm. A. & Singh R., 2001).

Here author has selected Serafim & Perira (1983) correlation;

$$\boldsymbol{E_m = 10^{(RMR-10)/40}} \quad \text{... (19)}$$

Where; $E_m - Elastic\ modulus\ of\ rock\ mass\ (in\ GPa)$

$RMR - Bieniawski\ rock\ mass\ rating$

Ref. Geotechnical Investigation Report (Ref No. G/2952) on Oct. 2012 by GeoTech (Pvt) Ltd.

Rock Layer 01 (16.3m to 18.3m)

$UCS = 23.22\ MNm^{-2}$

$RQD = 72$ and $CR = 95$

$Avg.\ discontinuity\ spacing = 239\ mm$

$Discontinuity\ Frequency = 4\ m^{-1}$

Table 20. RMR calculation for Rock Layer 01

RMR value for rock at pile tip	***Rating***
Strength of intact rock	2
RQD designation	13
Spacing of joints	10
Discontinuity length	2
Separation rating	4
Roughness rating	3
Infilling (gouge) rating	4
Weathering rating	3
Ground water	7
RMR Value	**48**
Serafim & Perira (1983) **Elastic Modulus of Rock Mass : E_m (GPa)**	**8.91**

Rock Layer 02 (18.3m to 19.3m)

$UCS = 72.49\ MNm^{-2}$

$RQD = 80$ and $CR = 100$

$Avg.\ discontinuity\ spacing = 194\ mm$

$Discontinuity\ Frequency = 5\ m^{-1}$

Table 21. RMR calculation for Rock Layer 02

RMR value for rock at pile tip	***Rating***
Strength of intact rock	7
RQD designation	17
Spacing of joints	8
Discontinuity length	2
Separation rating	4
Roughness rating	3
Infilling (gouge) rating	4
Weathering rating	5
Ground water	7
RMR Value	**57**
Serafim & Perira (1983) **Elastic Modulus of Rock Mass : E_m (GPa)**	**14.96**

Rock Layer 03 (19.3m to 21.3m)

$UCS = 103.05\ MNm^{-2}$

$RQD = 90$ and $CR = 100$

$Avg.\ discontinuity\ spacing = 425\ mm$

$Discontinuity\ Frequency = 2\ m^{-1}$

Table 22. RMR calculation for Rock Layer 03

RMR value for rock at pile tip	***Rating***
Strength of intact rock	12
RQD designation	20
Spacing of joints	10
Discontinuity length	2
Separation rating	5
Roughness rating	3
Infilling (gouge) rating	4
Weathering rating	6

Ground water	7
RMR Value	**69**
Serafim & Perira (1983) **Elastic Modulus of Rock Mass : E_m (GPa)**	**29.85**

The obtained modulus values were justifiable by referring to the research publication on International Society for Soil Mechanics and Geotechnical Engineering by Ekanayake et al. (2015)

Table 23. Summary of results (samples of Charnockitie Gneiss (CHG), Garnet-Biotite Gneiss (CBG) and Crystalline Limestone (MBL). (Ekanayake et al., 2015)

Rock Type	Weathering condition	Density/ (g/cm^3)	Young's Modulus		Poisson's Ratio (μ)	(σ_i) / MPa	M_R (Modules Ratio)	Bulk Modulus (K)/ GPa	Shear Modulus (G)/ GPa
			Average Modulus (E_{av})/ GPa	Secant Modulus (E_{70})/ GPa					
CHG	FR	2.32-3.07	18.4-70.7	14.6-49.5	0.14-0.39	63.7-167.7	175-972	7.36-25.43	10.59-107.12
	MWR	2.42-3.30	7.5-57.9	8.6-59.6	0.14-0.4	35.4-176.2	116-643	3.1- 20.68	4.07-96.5
GBG	FR	2.59-3.30	11.3-48.8	11.1-52.7	0.11-0.34	33.1-131.0	126-1081	5.04-19.37	4.96-33.89
	MWR	2.50-3.40	8.5-62.8	6.9-40.5	0.13-0.33	29.2-154.7	74-1291	3.37-26.84	5.9-31.72
MBL	FR	2.57-3.01	3.8-35.7	10.0-35.1	0.11-0.30	19.2-145.0	149-782	1.64-14.28	1.86-24.44
	MWR	2.63-3.02	4.0-35.7	4.0-36.2	0.10-0.37	20.6-93.39	81-741	1.82-14.40	1.67-27.18

The rock type encountered are classified under Garnet-Biotite Gneiss (GBG) and the test results are in the range as shown in Table 20.

Elastic Modulus of Rock (E_m) = 8.5 to 62.8 GPa

Density = 2.59 to 3.30 gm^{-3}

Poisson's Ratio (ν) = 0.11 to 0.34

3.5.4.2 Mohr - Coulomb Parameters of Rock Layers

Fractured rock has been idealized as non-linear Mohr-Coulomb model and to be obtain strength parameters of friction angle and cohesion, rock samples should be subjected to Tri-axial tests. As rock Tri-axial tests were not carried out for the selected samples, author has selected empirical correlations to obtain equivalent Mohr-Coulomb parameters through Hoek-Brown criteria.

Generalized Hoek-Brown Criterion

$$\boldsymbol{\sigma_1' = \sigma_3' + \sigma_{ci}\left[m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right]^a} \qquad \dots (20)$$

Where; σ_1' and σ_3' $-$ $the\ maxmium\ and\ minimum\ effective\ stresses\ at\ failure$

$m_b - Hoek - Brown\ constant\ for\ fractured(broken)\ rock\ mass$

s & $a - rock\ mass\ material\ constants$

$\sigma_{ci} - Uniaxial\ compressive\ strenght\ of\ intact\ rock\ pieces$

Blamer (1952) suggested normal and shear stresses can be correlate in terms of the corresponding principal effective stresses.

$$\sigma_n' = \sigma_3' + \frac{\sigma_1' - \sigma_3'}{\left(\partial\sigma_1' / \partial\sigma_3'\right) + 1} \qquad \dots (20.1)$$

$$\tau = (\sigma_1' - \sigma_3')\sqrt{\partial\sigma_1' / \partial\sigma_3'} \qquad \dots (20.2)$$

$For\ the\ GSI > 25, when\ a = 0.5$:

$$\frac{\partial\sigma_1'}{\partial\sigma_3'} = 1 + \frac{m_b\sigma_{ci}}{2(\sigma_1' - \sigma_3')} \qquad \dots (20.3)$$

$For\ the\ GSI < 25, when\ s = 0$:

$$\frac{\partial\sigma_1'}{\partial\sigma_3'} = 1 + \mathrm{a}{m_b}^a\left(\frac{\sigma_3'}{\sigma_{ci}}\right)^{a-1} \qquad \dots (20.4)$$

The tensile strength of the rock mass is calculated from:

$$\sigma_{tm} = \frac{\sigma_{ci}}{2}\left(m_b - \sqrt{{m_b}^2 + 4s}\right) \qquad \dots (20.5)$$

For the intact rock fragments that make up the rock mass Eqn. (20) simplifies to:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left[m_i\frac{\sigma_3'}{\sigma_{ci}} + 1\right]^{0.5} \qquad \dots (20.6)$$

The equivalent Mohr envelop, define by above equation, may be written in the form:

$$Y = \log A + BX \qquad \dots (20.7)$$

where; $$Y = \log {}^{\tau}/_{\sigma_{ci}} \text{ and } X = \log\left(\frac{\sigma_n' - \sigma_{tm}}{\sigma_{ci}}\right) \qquad \ldots (20.8)$$

using the value of σ_{tm} calculated from Eqn. (20.5) and a range of values of τ and σ_n' calculated from Eqn. (20.1) and (20.2), the values of A and B can be calculated by linear regression where;

$$B = \frac{\sum XY - {}^{(\sum X \sum Y)}/_{T}}{\sum X^2 - {}^{(\sum X)^2}/_{T}} \qquad \ldots (20.9)$$

$$A = 10\text{\^{}}\left\{{}^{\sum Y}/_{T} - B\left({}^{\sum X}/_{T}\right)\right\} \qquad \ldots (20.10)$$

where; $T - Total\ number\ of\ data\ pairs\ included\ in\ the\ regression\ analysis$

The most critical step in this procedure is the choice of the range of σ_3' values. Hoek & Brown (1998) stated that there are no theoretically precise approaches for choosing this range and a trial and error method, based upon practical conciliation, has been used for selecting the range included in the spreadsheet calculation.

For a Mohr envelope defined by Eqn.(20.6), the friction angle $\emptyset_i'$ for a specified normal stress σ_{ni}' is given by;

$$\emptyset_i' = \tan^{-1}\left[AB\left(\frac{\sigma_n' - \sigma_{tm}}{\sigma_{ci}}\right)^{B-1}\right] \qquad \ldots (20.11)$$

The corresponding cohesive strength c_i' is given by;

$$c_i' = \tau - \sigma_{ni}' \tan \emptyset_i' \qquad \ldots (20.12)$$

and the corresponding uniaxial compressive strength of the rock mass can be expressed as;

$$\sigma_{cmi} = \frac{2c_i' \cos \emptyset_i'}{1 - \sin \emptyset_i'} \qquad \ldots (20.13)$$

Fitting a tangent to the curved Mohr envelope gives in Eq. (20.12) an upper bound value for the cohesive strength c_i'. It is suggested that this value be reduced by about 25% so as to elude over-estimation of the rock mass strength.

In order to use the Hoek-Brown principle to approximation the strength and deformability of a jointed rock mass, three properties of the rock mass must be estimated. These are;

1. The uniaxial compressive strength of the intact rock fragments in the rock mass (σ_{ci}) – values can be obtained from site investigation report.

2. Hoek-Brown constant (m_i)
3. The value of the Geological Strength Index for the rock mass (GSI)

Hoek-Brown Constant (m_i)

It's recommended conduct set of Tri-axial tests to obtain Hoek-Brown constant (m_i). Even though, when laboratory tests are not possible and for preliminary design purposes researcher's had proposed tabular formats to estimate m_i values (Table 24).

Geological Strength Index (GSI)

Hoek (1991) introduced the Geological Strength Index (GSI) and Hoek, Kaiser and Bawden provides a method for estimating the reduction in rock mass strength based on different geological conditions. The GSI system is represented in Table 25 and Table 26. Experience has shown that Table 25 is sufficient for field interpretations as it is only essential to note the letter code which identifies each rock mass grouping. These codes can then be used to obtain the GSI value from Table 26.

Once the GSI and Hoek-Brown constant for intact rock (m_i) were obtained, Hoek-Brown parameters which describe the rock mass strength characteristics are calculated as follows;

$$m_b = m_i . exp\left(\frac{GSI - 100}{28}\right) \qquad \ldots (20.14)$$

For GSI < 25, i.e. rock masses of very poor quality, the modified Hoek-Brown method applies with;

$$s = 0 \ \ \& \ a = 0.65 - {}^{GSI}/_{200} \qquad \ldots (20.15)$$

For GSI > 25, i.e. rock masses of good to reasonable quality, the original Hoek-Brown method is applicable with;

$$s = exp\left(\frac{GSI-100}{9}\right) \ \& \ a = 0.5 \qquad \ldots (20.16)$$

Table 24. Values of the constant m_i for intact rock, by rock group. Note that values in parenthesis are estimates. (Hoek & Brown, 1998)

Rock type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very fine
SEDIMENTARY	Clastic		Conglomerate (22)	Sandstone 19	Siltstone 9	Claystone 4
SEDIMENTARY	Clastic			Greywacke (18)		
SEDIMENTARY	Non-Clastic	Organic		Chalk 7		
SEDIMENTARY	Non-Clastic	Organic		Coal (8-21)		
SEDIMENTARY	Non-Clastic	Carbonate	Breccia (20)	Sparitic Limestone (10)	Micritic Limestone 8	
SEDIMENTARY	Non-Clastic	Chemical		Gypstone 16	Anhydrite 13	
METAMORPHIC	Non Foliated		Marble 9	Hornfels (19)	Quartzite 24	
METAMORPHIC	Slightly foliated		Migmatite (30)	Amphibolite 25 - 31	Mylonites (6)	
METAMORPHIC	Foliated*		Gneiss 33	Schists 4 - 8	Phyllites (10)	Slate 9
IGNEOUS	Light		Granite 33		Rhyolite (16)	Obsidian (19)
IGNEOUS	Light		Granodiorite (30)		Dacite (17)	
IGNEOUS	Dark		Diorite (28)		Andesite 19	
IGNEOUS	Dark		Gabbro 27	Dolerite (19)	Basalt (17)	
IGNEOUS	Dark		Norite 22			
IGNEOUS	Extrusive pyroclastic type		Agglomerate (20)	Breccia (18)	Tuff (15)	

Table 25. Characterization of rock masses on the basis of interlocking and joint alteration. (Hoek & Brown, 1998)

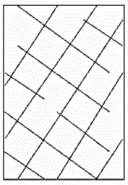
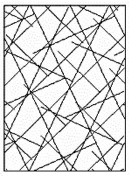
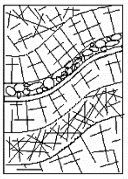
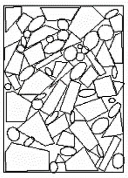
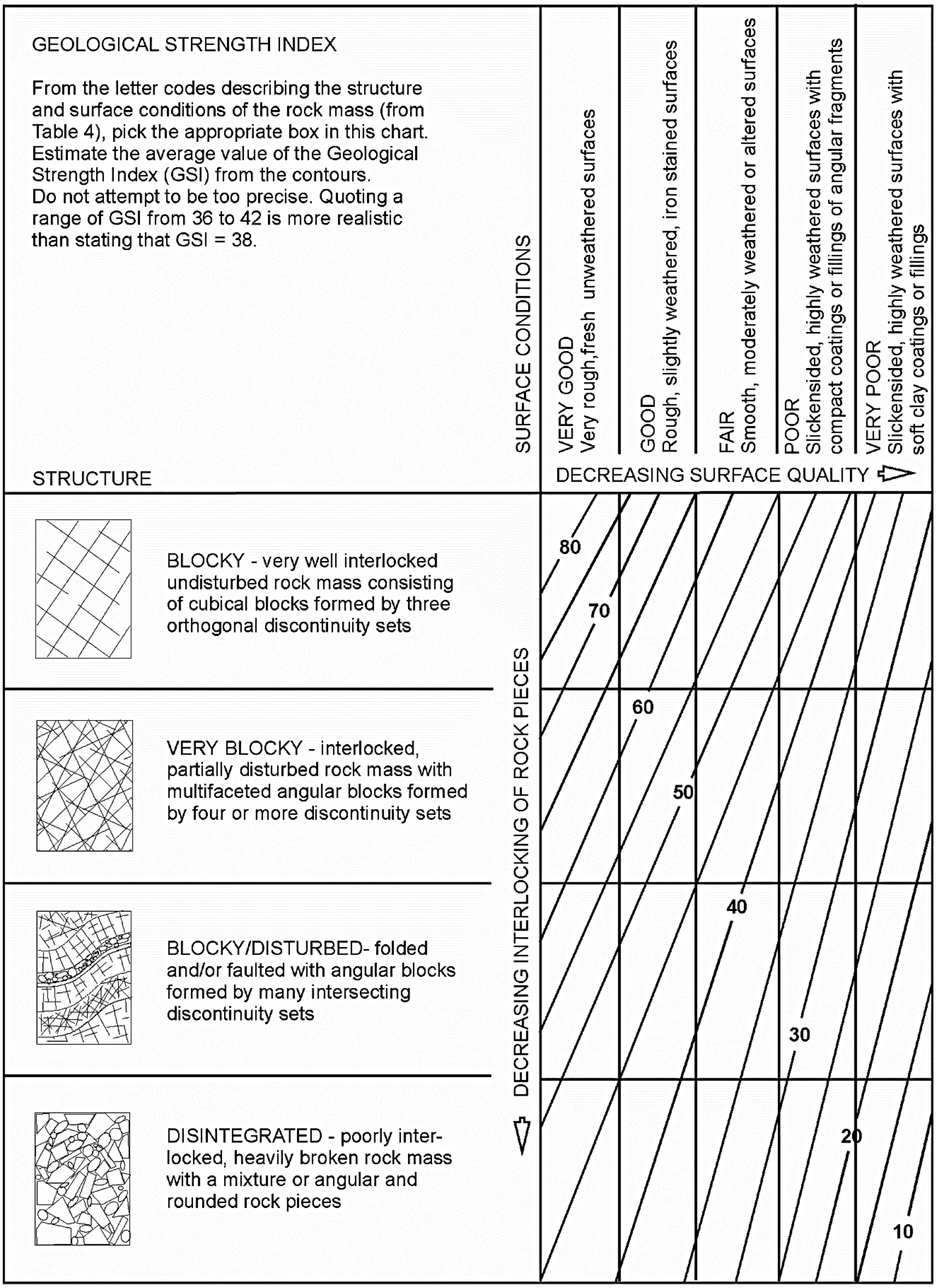
ROCK MASS CHARACTERISTICS FOR STRENGTH ESTIMATES Based upon the appearance of the rock, choose the category that you think gives the best description of the 'average' undisturbed in situ conditions. Note that exposed rock faces that have been created by blasting may give a misleading impression of the quality of the underlying rock. Some adjustment for blast damage may be necessary and examination of diamond drill core or of faces created by pre-split or smooth blasting may be helpful in making these adjustments. It is also important to recognize that the Hoek-Brown criterion should only be applied to rock masses where the size of individual blocks is small compared with the size of the excavation under consideration. STRUCTURE		SURFACE CONDITIONS DECREASING SURFACE QUALITY ⇨	VERY GOOD Very rough, fresh unweathered surfaces	GOOD Rough, slightly weathered, iron stained surfaces	FAIR Smooth, moderately weathered or altered surfaces	POOR Slickensided, highly weathered surfaces with compact coatings or fillings of angular fragments	VERY POOR Slickensided, highly weathered surfaces with soft clay coatings or fillings
	BLOCKY - very well interlocked undisturbed rock mass consisting of cubical blocks formed by three orthogonal discontinuity sets	DECREASING INTERLOCKING OF ROCK PIECES ⇩	B/VG	B/G	B/F	B/P	B/VP
	VERY BLOCKY - interlocked, partially disturbed rock mass with multifaceted angular blocks formed by four or more discontinuity sets		VB/VG	VB/G	VB/F	VB/P	VB/VP
	BLOCKY/DISTURBED- folded and/or faulted with angular blocks formed by many intersecting discontinuity sets		BD/VG	BD/G	BD/F	BD/P	BD/VP
	DISINTEGRATED - poorly inter-locked, heavily broken rock mass with a mixture or angular and rounded rock pieces		D/VG	D/G	D/F	D/P	D/VP

Table 26. Estimation of Geological Strength Index GSI based on geological conditions (Hoek & Brown, 1998)



Reference to the borehole data (BH-Ex16, BH-Ex17 & BH13) and core logs, GSI for the rock layers were decided as follows:

Details of Core Logs of BH-13

Table 27. Rock coring details of BH-13

Rock Level Depth	13.20m
Rock Layer 01 (13.20 to 14.80)	CR = 100%, RQD = 100%, UCS = 15 MPa
Rock Layer 02 (14.80 to 16.45)	CR = 100%, RQD = 81%
Rock Layer 03 (16.45 to 18.46)	CR = 82%, RQD = 82%
BH Termination Depth	18.46m

Figure 22. Core Logs (Core Boxes) of BH-13

Details of Core Logs of BH-Ex16

Table 28. Rock coring details of BH-Ex16

Rock Level Depth	14.20m
Rock Layer 01 (14.20 to 15.40)	CR = 92%, RQD = 77%, UCS = 26.63 MPa Dis. Spacing = 166 mm, Dis. Freq. = 6 m^{-1}
Rock Layer 02 (15.40 to 16.70)	CR = 100%, RQD = 54%, UCS = 117.02 MPa Dis. Spacing = 108 mm, Dis. Freq. = 9 m^{-1}
Rock Layer 03 (16.70 to 18.50)	CR = 100%, RQD = 89% Dis. Spacing = 274 mm, Dis. Freq. = 4 m^{-1}
Rock Layer 04 (18.50 to 19.60)	CR = 95%, RQD = 90, UCS = 57.07 MPa Dis. Spacing = 327 mm, Dis. Freq. = 3 m^{-1}
BH Termination Depth	19.60m

Figure 23. Core Logs (Core Boxes) of BH-Ex16

Details of Core Logs of BH-Ex17

Table 29. Rock coring details of BH-Ex17

Rock Level Depth	13.70 m
Rock Layer 01 (13.70 to 15.70)	CR = 95%, RQD = 72%, UCS = 23.22 MPa Dis. Spacing = 239 mm, Dis. Freq. = 4 m^{-1}
Rock Layer 02 (15.70 to 16.70)	CR = 100%, RQD = 80%, UCS = 72.49 MPa Dis. Spacing = 194 mm, Dis. Freq. = 5 m^{-1}
Rock Layer 03 (16.70 to 17.70)	CR = 100%, RQD = 90% Dis. Spacing = 425 mm, Dis. Freq. = 2 m^{-1}
Rock Layer 04 (17.70 to 18.70)	CR = 100%, RQD = 90, UCS = 103.05 MPa Dis. Spacing = 178 mm, Dis. Freq. = 6 m^{-1}
BH Termination Depth	18.70 m

Figure 24. Core Logs (Core Boxes) of BH-17

Table 30. Estimated GSI values for idealized rock layers in FEM model.

Idealized Rock Layer for TP01 pile	***UCS (MPa)***	***Classification (Table 22)***	***Estimated GSI (Table 23)***	***Hoek-Brown Constant (m_b)***
Rock Layer 01	23.22	VB/P	40	3.87
Rock Layer 02	72.49	VB/F	50	5.53
Rock Layer 03	103.05	B/G	65	9.45

The calculation of obtaining equivalent Mohr-Coulomb parameters from Hoek-Brown criteria involves linear regression analysis and iteration process. Therefore, author has developed the MS Excel spread sheet based on steps given by Hoek and Brown (1998).

Table 31. Equivalent friction angle & cohesion values calculated on MS Excel Spreadsheet.

Idealized Rock Layer for TP01 pile	***Layer Depth***	***Equivalent Friction Angle***	***Equivalent Cohesion***
Rock Layer 01	16.3 to 18.3	39.3 deg.	1.104 MPa
Rock Layer 02	18.3 to 19.3	42.1 deg.	4.01 MPa
Rock Layer 03	19.3 to 21.3	46.2 deg.	7.421 MPa

The obtained values were compared with published literature (Kulhawy, F. H. and Goodman, 1987) and justification were made the values for fractured rocks are acceptable and accurate for purpose.

Table 32. Typical strength values for rock (Kulhawy F. H. and Goodman, 1987)

Rock	***Cohesion (MPa)***	***Friction Angle (degrees)***	***Range of Confining Pressure (MPa)***
Berea Sandstone	27.2	27.8	0 – 200
Muddy Shale	38.4	14.4	0 – 200
Sioux Quartzite	70.6	48.0	0 – 203
Georgia Marble	21.2	25.3	6 – 69
Chalk	0	31.5	10 – 90
Granite & Gneisses	55.1	51.0	0 – 69
Indiana Limestone	6.7	42.0	0 – 10

The material properties of soil layers, rock layers and concrete pile materials used to FEM analysis were summarized in Table 33.

Table 33. Material properties for the soil, rock and pile

Parameter	***Name***	***Residual Soil I***	***Residual Soil II***	***Peat***	***HWR***	***Rock 01***	***Rock 02***	***Rock 03***	***Concrete Pile***	***Unit***
Material model	$Model$	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Linear-Elastic	-
Material behavior	$Type$	Drained	Drained	Drained	Drained	Non-Porous	Non-Porous	Non-Porous	Non-Porous	-
Unsaturated soil weight	γ_{unsat}	15.18	17.47	17.48	15.92	22.75	22.75	22.75	-	kNm^{-3}
Saturated soil weight	γ_{sat}	18.79	18.33	18.23	18.95	-	-	-	-	kNm^{-3}
Permeability	$K_x = K_y$	0.001	0.001	0.001	0.01	-	-	-	-	$^{m}/_{day}$
Young's modulus	E_{ref}	18×10^3	10×10^3	9×10^3	20×10^3	8.91×10^6	14.96×10^6	29.85×10^6	25.7×10^6	kNm^{-2}
Poisson's ratio	ν	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	-
Cohesion	c_{ref}	10	6	10	10	1104	4010	7421	-	kNm^{-2}
Friction angle	ϕ	29	27	0	32	39.3	42.1	46.2	-	$^\circ$
Dilatancy angel	ψ	0	0	0	2	9.3	12.1	16.2	-	$^\circ$
Interface reduction factor	R_{inter}	0.85	0.85	0.85	0.85	0.95	0.95	0.95	0.95	-

3.5.5 PRESCRIBED LOADING

It has been selected prescribed loading arrangement to obtain the load required for generate prescribed settlement on pile head. Following 2 cases of loading conditions were studied in this research.

I. Ultimate Criteria – referred to the BS8004: 1986 describes that the ultimate pile capacity as the load cause soil to mobilized its full resistance against the applied load. In general, takes loading causing the settlement of pile head by 10% of the pile width or diameter.

II. Ultimate Criteria – with reference to the ICTAD/DEV/16 (1997) given its performance specification, as the maximum allowable gross settlement of 25mm for load cycle up to 1.5 × working load.

3.5.6 PLAXIS 2D MODEL

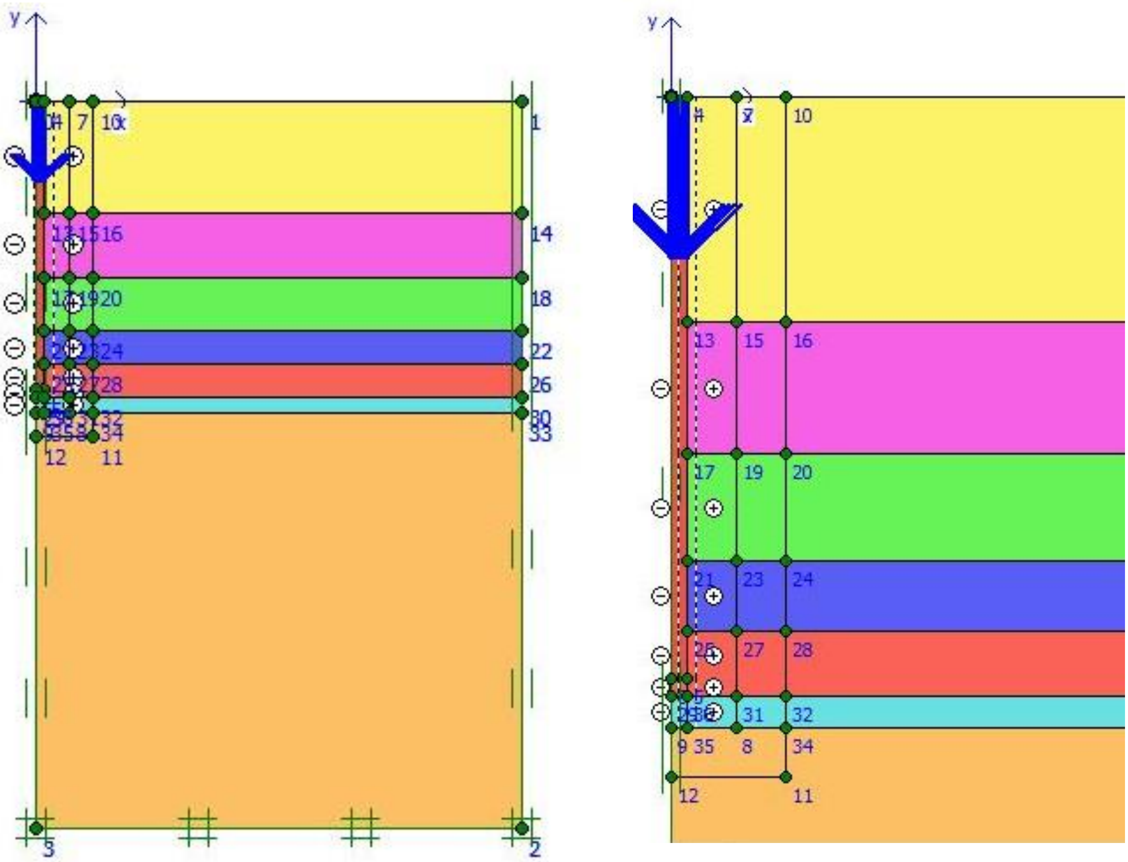


Figure 25. PLAXIS 2D FEM – Asymmetrical (15 nodes) with prescribed loading of 10% Pile Dia. on pile head

PLAXIS 2D allows spontaneous generation of finite element mesh based on robust triangulation procedure, which results in "Unstructured" meshes. It might appear disorderly manner, but the numerical behavior of the meshes possibly yield better results rather than for regular structured meshes.

General meshing parameter is required for mesh generator, which represents the average element size, l_e, calculated based on the external geometry dimensions ($X_{min}, X_{max}, Y_{min}$ and Y_{max}) using following relationship:

$$l_e = \sqrt{\frac{(X_{max} - X_{min}) \times (Y_{max} - Y_{min})}{n_c}}$$

Where; $n_c = 25$: Very Coarse Mesh

$n_c = 50$: Coarse Mesh

$n_c = 100$: Medium Mesh

$n_c = 200$: Fine Mesh

$n_c = 400$: Very Fine Mesh

Here it has been used Very Fine Mesh for higher resolution and increase the resolution of the results.

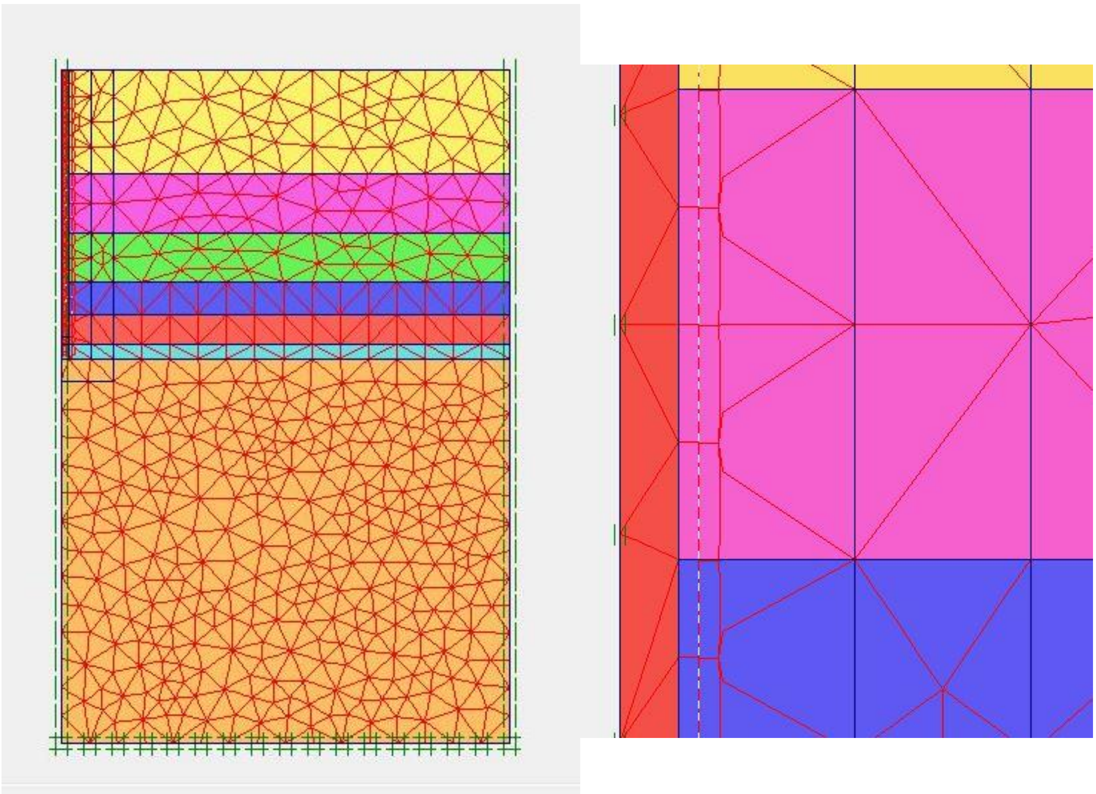


Figure 26. A. Meshing with very fine coarseness for higher resolution of results. B. Very Fine Mesh closer to the pile element.

Point “A” was selected on extreme end of the pile head from symmetrical axis to obtain curve for Load vs. Settlement behavior.

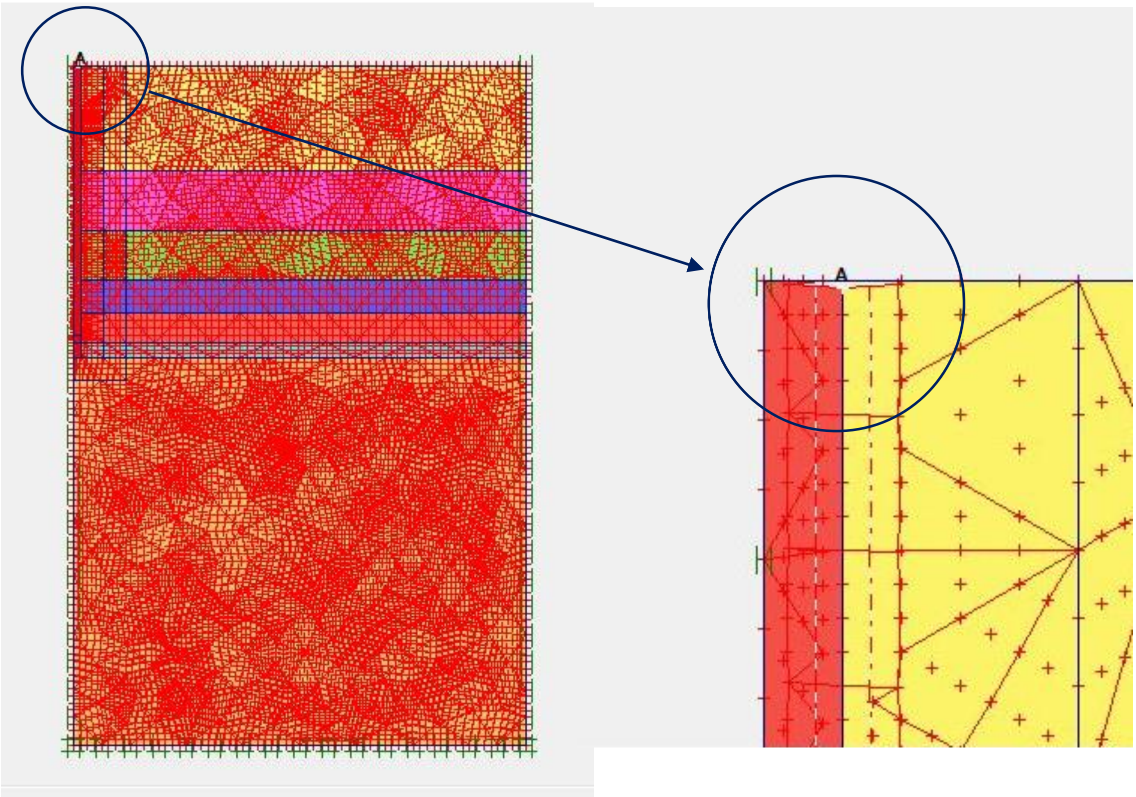


Figure 27. Select point A on pile top extreme end from axis to generate Load vs. Displacement

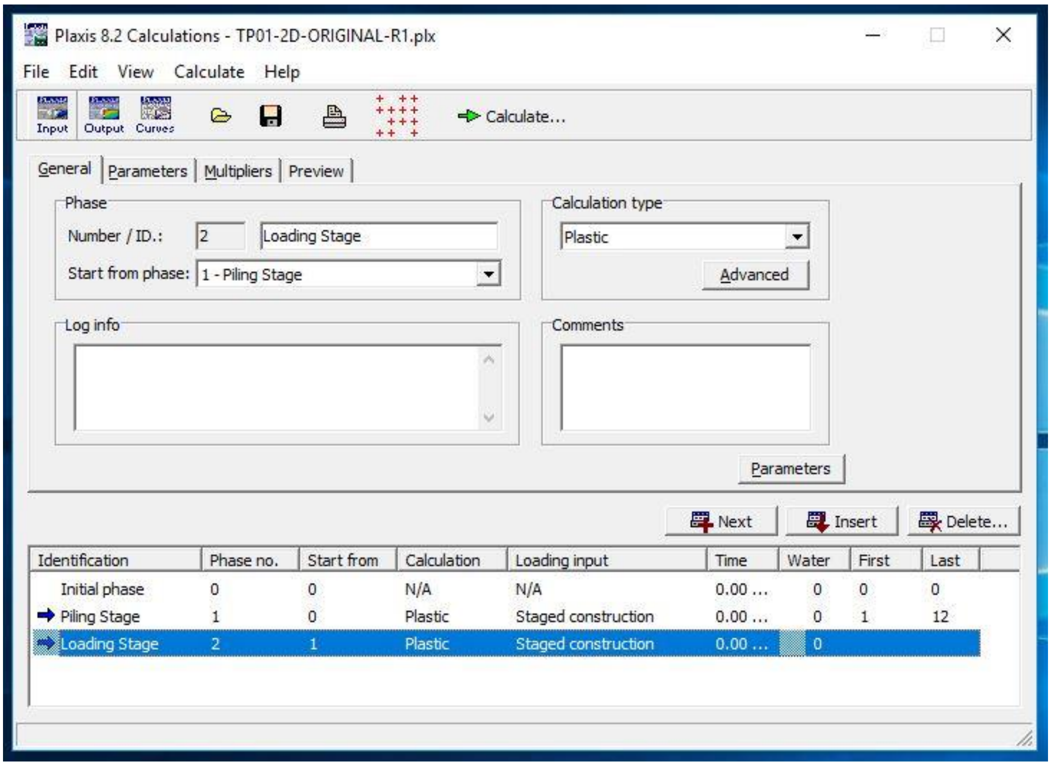


Figure 28. Calculation for single stage for with initial displacement reset to Zero.

The load vs. pile head vertical settlement obtained from the FEM calculations shown in Figure 24.

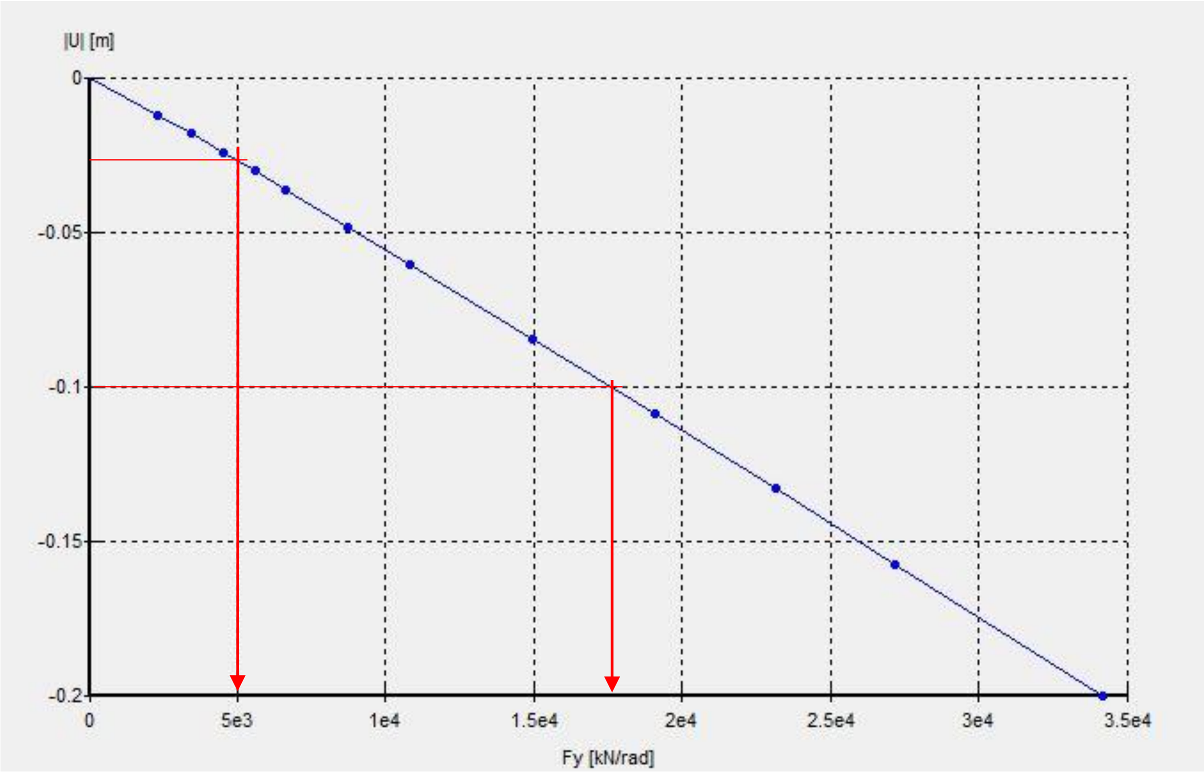


Figure 29. Figure 24. Load vs. Settlement curve obtained from PLAXIS 2D

I. Ultimate Criterion – Reference to BS 8004:1986 the capacity of the pile can be calculated from the Load – Settlement curve as follows:

$$10\% \times 1000mm\ (pile\ diamter) = 100mm$$

From Figure 24, $F_y = 17500\ {}^{kN}/_{rad.}$ at settlement equal to 100mm.

$$Q_{ult.} = F_y \times 2\pi = 17500 \times 2\pi = 109955\ kN$$

II. Ultimate Criterion – Reference to ICTAD/DEV/16 (1997) for load corresponded to 25mm gross settlement on pile head can be obtained from the cure as follows:

From Figure 24, $25mm\ settlement\ \Rightarrow F_y = 5000\ {}^{kN}/_{rad.}$

$$Q_{ult.} = F_y \times 2\pi = 5000 \times 2\pi = 31416\ kN$$

4 RESULTS ANALYSIS AND DISCUSSION

4.1 COMPARISON OF PILE CAPACITY BASED ON APPROACH

Pile capacities calculated from the empirical, practical and numerical approach has been compared and tabulated in Table 34 and Figure 30.

Table 34. Tabulated pile capacity obtained from different approaches.

Approach	*Pile Capacity (kN)*	*Percentage (%)*
Empirical (average)	40936.79	123
Practical – SLT	33,333.33	100
Practical – HSDLT (maximum mobilized)	17023.36	51
Numerical - 01 (ICTAD:1997 criteria)	31,416.00	94
Numerical - 02 (BS8004:1986 criteria)	109955.00	330

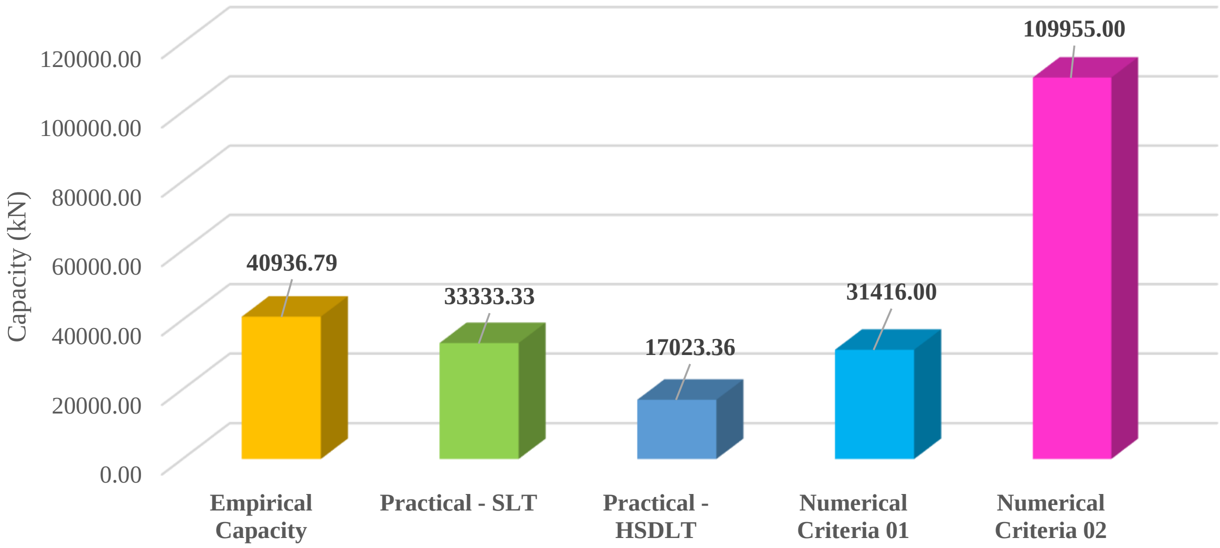


Figure 30. Pile capacity graphical representation based on different approach.

As per the results numerical capacity calculated based on 10% pile diameter criteria (BS8004:1986) is the highest value and 25mm settlement criteria (ICTAD:1997) has resulted reasonable matching with practical capacity obtained using SLT. In broad comparison of the different approach had resulted different values, except 10% pile diameter criteria, all are within realistic range of 30000 ~ 40000 kN. It has been observed that numerical pile capacity based on 25mm settlement criteria and practical pile capacity has a reasonable matching within difference of 2000 kN.

4.2 COMPARISON OF SOIL SKIN FRICTION CAPACITY

Resultant skin friction developed in soil layers were semi-empirically calculated based on two different methods and comparison of values are tabulated in Table 35 and Figure 31.

Table 35. Tabulated soil skin friction capacities calculated based on semi-empirical

Method	***Soil Skin Friction Capacity (kN)***
M.1.1 - Method Outlined in ICTAD	1315.40
M.1.2 - O'Neill and Reese Method	2264.17

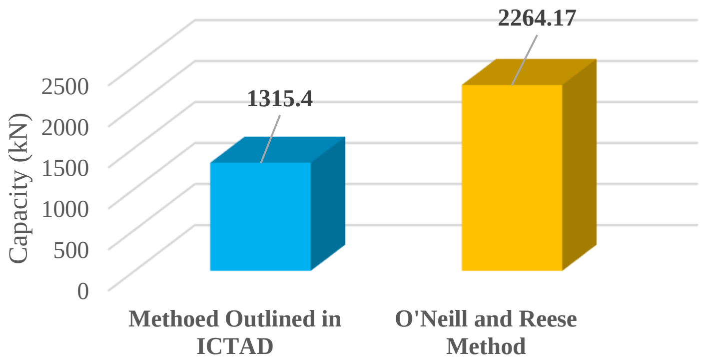


Figure 31. Graphical representation of soil skin friction capacities based on different semi-empirical formulae.

It can be evident that M.1.1 results are very low value (nearly half of the M.1.2) and agrees with the fact that M.1.1 is underestimate the soil skin friction value with reference to Thilakasiri et al (2015).

4.3 COMPARISON OF ROCK SOCKET SKIN FRICTION CAPACITY

Rock socket skin friction developments were calculated based on six different semi-empirical methods and compared with practically obtained HSDLT results as shown in Table 36 and Figure 32.

Table 36. Tabulated results of unit rock socket skin friction values

Method	***Unit Rock Skin Friction (kN/m^2)***	***Total Rock Skin Friction (kN)***
M.2.1 – ICTAD guidelines	200	942.48
M.2.2 – Rowe & Armitage	2890	13618.80
M.2.3 – Hong Kong guidelines	964	4542.74
M.2.4 – Williams & Pells	1904	8972.39
M.2.5 – Meigh & Wolski	366.6	1727.56
M.2.6 – Hovarth & Keny	995.42	4690.81
M.4.2 – HSDLT	324	1526.81

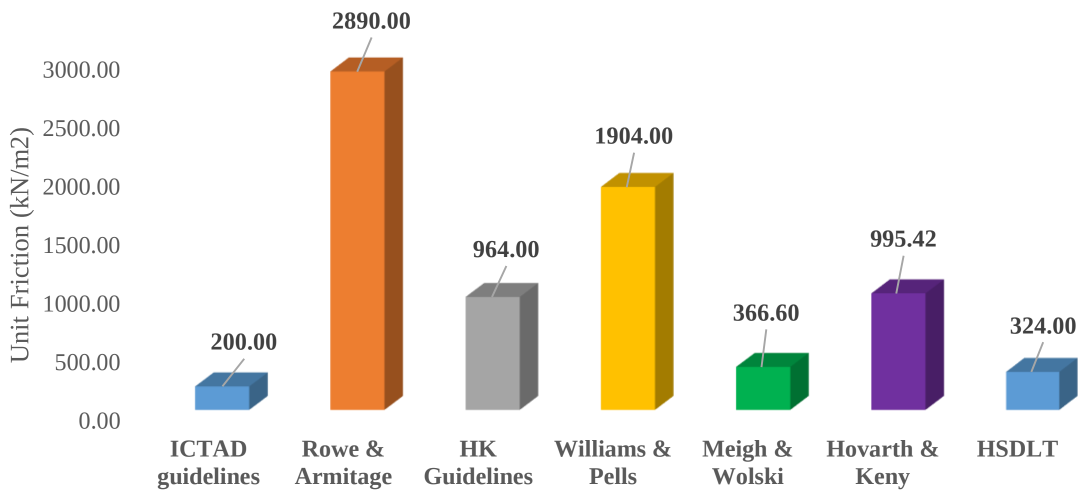


Figure 32. Graphical representation of unit rock socket skin friction values.

Reference to the results, it can be identified that the values given by each equations are highly variable and none of the value is within the close range. Even though it can be identified that the limiting values specified in ICTAD is underestimates the rock socket skin friction value, while Rowe & Armitage is overestimate.

It can be state that values estimated using HK guidelines and Hovarth & Keny are seems to be reasonable.

4.4 COMPARISON OF ROCK END BEARING CAPACITY

Rock end bearing values were calculated based on six different methods and results were compared with practically obtained SLT and HSDLT results shown in Table 37 and Figure 33.

Table 37. Tabulated rock end bearing capacities based on different methods.

Method	***Unit Rock End Bearing Capacity (MPa)***	***Rock End Bearing Capacity (kN)***
M.3.1 – BS 8004	37.5	29452.43
M.3.2 – RMR method (HK guidelines)	11.5	9032.08
M.3.3 – Kulhawy & Goodman	7.7	6047.57
M.3.4 – M. J. Tomlinson	139.32	109421.67
M.3.5 – Peck et. al.	27.5	21598.45
M.3.6 – Bell solution	31.62	24835.08
M.4.1 – SLT	8.286	6508
M.4.2 – HSDLT	9.747	7653.11

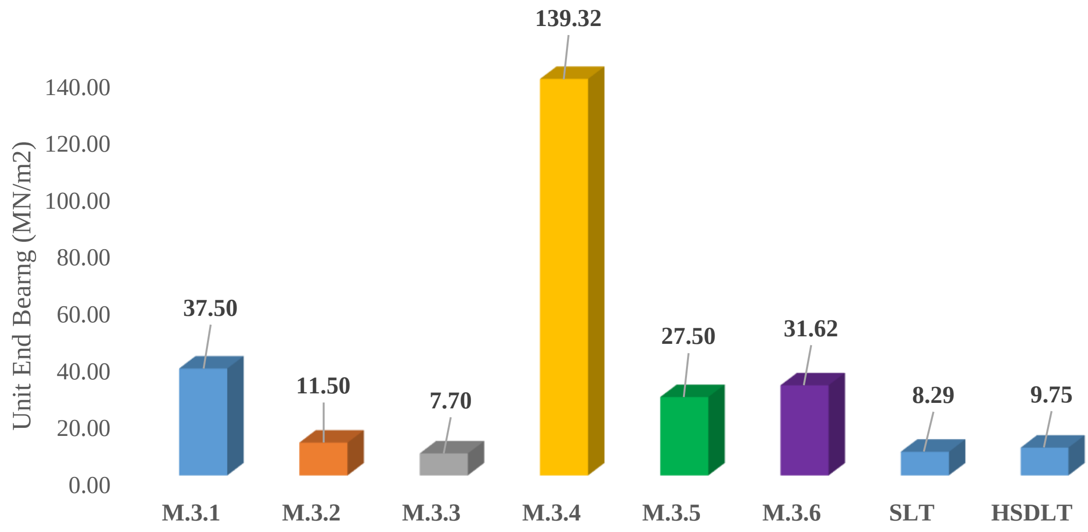


Figure 33. Graphically represented rock end bearing capacities based on different methods

It can be seen that most of the method estimates reasonable value for the rock end bearing values except M. J. Tomlinson method which is heavily overestimates the bearing capacity of fractured rock mass.

Excluding the M. J. Tomlinson method due to its high deviation from other results, mean will be 23 MPa. Rock end bearing capacity estimated using Kulhawy & Goodman and RMR Method (HK Guideline) are seems reasonable.

4.5 COMPARISON OF MOBILIZED SKIN FRICTION

It has been selected following combinations of empirical capacity calculation method for compare with practical mobilized skin friction capacity obtained from Van Weele method and HSDLT.

Table 38. Combinations selected for shaft skin friction comparison.

Combination	***Soil Skin Friction***	***Rock Socket Skin Friction***
COM-01	M.1.2 : O'Neill & Reese	M.2.3 : HK Guidelines
COM-02	M.1.2 : O'Neill & Reese	M.2.4 : Williams & Pells
COM-03	M.1.2 : O'Neill & Reese	M.2.6 : Hovarth & Keny

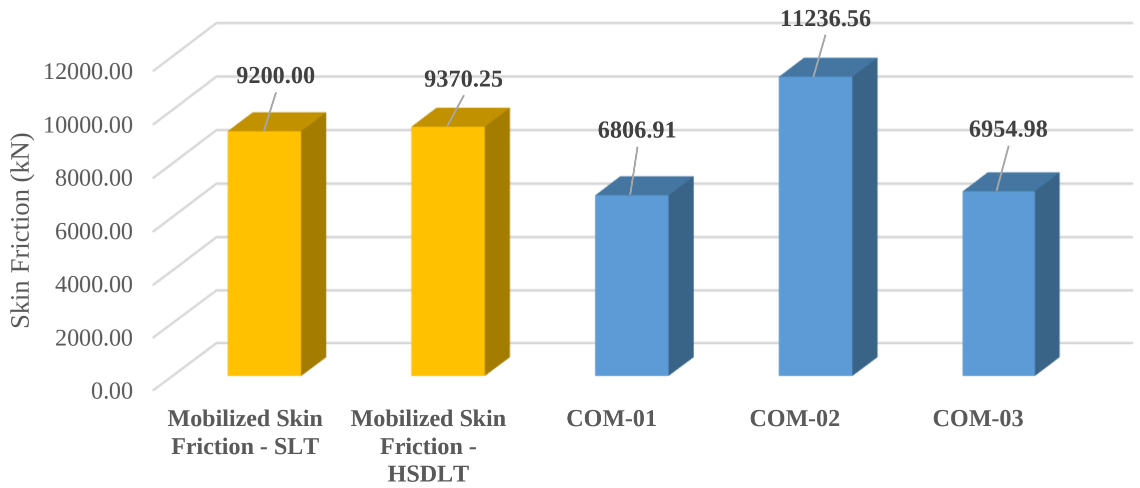


Figure 34. Graphical representation of Actual Mobilized Skin Friction & Empirical ultimate values

Table 39. Comparison of Theoretical Skin Friction & Mobilized Skin Friction

Combination	***Empirical Total Skin Friction (kN)***	***SLT – Mobilized (Max) Skin Friction (kN)***	***HSDLT – Mobilized (Max) Skin Friction (kN)***
COM-01	6806.91	9200	9370.25
COM-02	11236.56		
COM-03	6954.98		

Observation of Load vs. Settlement curve indicates that even though pile capacity not reached its ultimate (where curve has not reached its vertical asymptote), it has already reached its ultimate skin friction value at the loading of 250% of working load (where initial straight-line behavior was over and curve has tends to reach curve region).

Therefore, mobilized skin friction of 9200 kN can be considered as Maximum Mobilized Skin Friction or Ultimate Skin Friction of pile shaft.

According to the comparison given in Figure 34 and Table 39, it can be noted that combination 02 (COM-02 = M.1.2 + M.2.4) gives a better relation than the other combinations. Therefore, combination of M.1.2: O'Neill & Reese method and M.2.4: Williams & Pells method can be recommended for the pile skin friction capacity calculation.

5 CONCLUSION

Results from only one case study is not sufficient enough to make solid conclusions. However, following important observations can be made from the case study are listed below.

1. It’s an essential fact that, proper soil investigation should have carried out from specialist soil test laboratory prior to design stage, to ascertain a necessary information about the soil layers’ classification and soil parameters such as soil unit weight, internal angel of friction, cohesion and the modulus of elasticity of each soil layer. It’s a mandatory requirement for develop FEM, by using any geotechnical software and to achieve results near from the practical condition.

2. This research provides a comparison between the empirical, practical and numerical pile capacities for one case study where the pile has been installed and tested in Sri Lanka. The research result indicates that numerical pile capacity is showing good matching with the practical pile capacity. It shows, estimation of pile capacity can be done using FEM and can be extended to minimize the number of static load test piles requirement for the preliminary design.

3. The best combination for estimating the empirical pile capacity can be suggested as; O’Neill & Reese method for soil skin friction, HK guidelines and Hovarth & Keny methods for rock socket skin friction and rock end bearing estimates from Kulhawy & Goodman and RMR Method (HK Guideline).

4. The best combination for estimating the pile shaft skin friction (soil & rock socket) can be suggested as O’Neill & Reese method and Williams & Pells.

6 RECOMMENDATIONS

The research can be extended to the study of the development of the rock socket skin friction, by monitoring pile toe movement with instrumented SLT i.e. O-cell (Osterburg Cell) can be model with FE for better understanding of rock socket behavior.

Considering design of piles according to Eurocode 7 allows utilizing the pile capacities obtained from field test data i.e. static load tests, ground investigation tests and dynamic tests. The geotechnical capacities can be ascertained as design capacities based on the frequency of tests conducted. However, EC7 does not contain a clear guideline for the design of piles on rock. This study could be further developed to incorporate a method by which partial factors can be assigned for the empirical methods to utilize and develop these methods in the context of using EC7 for the local content of designing pile founded on rock as a national annexure.

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ANNEXURE – A

Geotechnical Investigation Report – December, 2011

G/2952

GEOTECHNICAL INVESTIGATION FOR PROPOSED KEELLS CITY PROJECT AT COLOMBO 02

Client: John Keells Holdings PLC

December, 2011

Geotech (Pvt) Ltd
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E-mail: Geotechlanka@gmail.com
Web: www.geotechlanka.com

BOREHOLE LOG		Job No: G-2952	BH No: 13
Date of Start: 02-11-2011	Date of End: 02-11-2011	Casing Diameter	MSL:
Drilling Method: Rotary Wash Boring		75 mm	
Equipment: Acker, Drilling Machine, NW Casings, T/C Casing Shoe etc:		Borehole Diameter	Ground Level
		75 mm	Sheet 1 of 2

Date and Time	Casing depth (m)	Depth to water (m)	Sample details: Depth (m) from - to	Type	No	SPT Blow/N	SPT Drive (mm)	Description of strata	Depth (tks) (m)	Level m OD	Legend
02/11			0.00-0.50 (Bulk)					Blackish brown very fine to medium CLAYEY SAND & Rubbles -fill-	0.60		
			0.50-0.95	D	1	04	450	Loose ,Black fine to coarse SAND with Rubbles -Fill			
		1.60m	1.50-1.95	D	2	04	450				
			2.50-2.95	D	3	03	450		3.20		
			3.50-3.95	D	4	01	450	Very soft Blackish dark brown very fine to medium PEATY CLAY	4.15		
			4.50-4.95	D	5	01	450	Very soft Black very fine PEATY CLAY	5.80		
			6.00-6.45	D	6	30	450	Hard Dark Brown PEAT with decomposed Timber Matter	7.40		
			7.50-7.95	D	7	17	450	Medium dense Dark Grey very fine to fine CLAYEY SILTY SAND	9.00		
			9.50-10.00	D	8	01	450	Very soft Brownish Grey very fine to medium PETLY silty CLAY			

Remarks:	BH Terminated at 18.46m Ground Water Level :- 1.60m Weather :- Hot & Rainy	Logged By	A.V.V.De Alwis
Project:	Soil Investigation for Proposed Keells City Project at Glennie Street Colombo 02	Compiled By	- R.Madhushika
Client:	JOHN KEELS HOLDING PLC - PROPERTY GROUP	Checked By	-

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BOREHOLE LOG		Job No: G-2952	BH No: 13
Date of Start: 02-11-2011	Date of End: 02-11-2011	Casing Diameter	MSL:
Drilling Method: Rotary Wash Boring		75 mm	
Equipment: Acker, Drilling Machine, NW Casings, T/C Casing Shoe etc:		Borehole Diameter	Ground Level
		75 mm	Sheet: 2 of 2

Date and Time	Casing depth (m)	Depth to water (m)	Sample Depth (m) from - to	Type	No	SPT Blow/N	SPT Drive (mm)	Description of strata	Depth (tks) (m)	Level m OD	Legend
								-DO-			
			10.50-10.95	D	9	24	450		10.80		
								Very stiff, Brownish Grey very fine to medium SILTY CLAY	11.80		
			12.00-12.45	D	10	06	450	Yellowish brown to whitish pale gray completely WEATHERED ROCK stiff fine to medium SANDY SILT			
			13.20	C/R(%)		RQD(%)		Rock Level	13.20		
			UCS15								
			13.20 to 14.80	100		100		Blackish gray banded pale gray BIOTITE GRANITIC GNEISS fresh ,very slightly fractured			
			14.80 to 16.45	100		81					
			16.45 to 18.46	82		82			18.46		
			18.46					BH Terminated			

Remarks:	BH Terminated at 18.46m Ground Water Level :- 1.60m Weather :- Hot & Rainy	Logged By	A.V.V.De Alwis
Project:	Soil Investigation for Proposed Keells City Project at Glennie Street Colombo 02	Compiled By -	R.Madhushika
Client:	JOHN KEELS HOLDING PLC - PROPERTY GROUP	Checked By -	

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Lab: 465/1, Sunethradevi Rd, Pepiliyana, Boralesgamuva.
Tel/Fax : 2769828

UNCONFINED COMPRESSIVE STRENGTH TEST RESULTS OF ROCK

TEST METHOD : ASTM : D 2938 - 86

PROJECT : Proposed Keels City Project, Grannie Street, Colombo - 02

Sheet 02 of 02

CLIENT : M/S Geotech (Pvt) Ltd.

JOB NO : GL/0382/001

BH No.	BH - 04	BH - 05	BH - 06	BH - 06	BH - 13	BH - 13
Depth (m)	24.55-24.85	20.65-20.95	18.60-18.90	20.80-21.05	14.80-15.09	18.13-18.28
Diameter of Core (mm)	54	54	54	54	54	54
Length of Core (mm)	108	108	108	108	105	95
Load at Failure (kN)	121	112	127	190	131	113
Unconfined Compressive Strength (MN/m^2)	**52.83**	**48.90**	**55.45**	**82.96**	**57.00**	**48.54**
Failure Sketch						
Rock Strength **	Strong	Moderately Strong	Strong	Strong	Strong	Moderately Strong

*** With reference to BS 5930 : 1999*

TESTED BY
G.G.W.H. Randunu

CHECKED BY
Eng. K.V.S.D. Jayamali

GROUP ENGINEERING LABORATORIES (PVT) LTD.
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16/11/11
DATE

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SUMMARY OF LABORATORY TEST RESULTS OF BOREHOLE SAMPLES

CLIENT : M/s Geotech (Pvt) Ltd.

PROJECT : Proposed Keels City Project, Glannie Street, Colombo - 02

Sheet 05 of 05

JOB NO : GL/0382/005

BH No.	Depth (m)	Soil Description	Unified Soil Classification	Atterburg Limits			Sieve Analysis (%)			NMC (%)	Wet Density (kg/m3)	Dry Density (kg/m3)	Specific Gravity
				LL %	PL %	PI %	>4.75 mm	4.75mm to 0.075mm	<0.075 mm				
BH - 13	3.50-3.95	Silty Sand	SM	NP	NP	NP	0.0	88.0	12.0	27.2	1.548	1.217	2.55
	7.50-7.95	Silty Sand	SM	17	16	1	0.0	82.3	17.7	15.1	1.781	1.548	2.15
	9.00-9.45	Peaty Clay	-		***		0.0	27.9	72.1	33.8	1.782	1.332	2.62
	12.00-12.45	Silty Sand	SM	24	21	3	0.0	51.3	48.7	27.6	1.623	1.272	2.60

*** Not enough sample to perform the test

Laboratory Engineer
K.V.S.D. Jayamali

12/12/2011
Date

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TEST RESULTS OF WET DENSITY & DRY DENSITY OF SOIL

TEST METHOD - BS 1377 : Part 2 : 1990

Project :- Proposed Keels City Project, Glannie Street, Colombo - 02

Client :- M/S Geotech (Pvt) Ltd — Sheet 01 of 02

Date :- 07/12/2011 — **Job No** : GL/0382/005

BH No	Depth (m)	Weight of Mould (g)	Volume of Mould (cm^3)	Weight of Dry Soil + Mould (g)	Weight of Dry Soil (g)	Dry Density (g/cm^3)	Moisture Content (%)	Wet Density (Mg/m^3)
BH - 08	2.50-2.95	39.07	25.0	80.43	41.36	1.654	19.9	1.984
	7.50-7.95	14.53	45.0	89.28	74.75	1.661	10.7	1.839
	10.50-10.95	39.07	25.0	69.46	30.39	1.216	32.6	1.612
BH - 09	2.50-2.95	39.07	25.0	76.78	37.71	1.508	17.5	1.772
	9.00-9.45	39.07	25.0	80.85	41.78	1.671	13.2	1.891
	10.50-10.95	39.07	25.0	79.07	40.00	1.600	11.6	1.786
	12.00-12.45	39.07	25.0	74.28	35.21	1.408	21.8	1.716
BH - 10	4.50-4.95	39.07	25.0	80.14	41.07	1.643	12.2	1.843
	6.00-6.45	39.07	25.0	73.04	33.97	1.359	22.5	1.664
	10.50-10.95	39.07	25.0	65.89	26.82	1.073	55.7	1.670
	12.00-12.45	39.07	25.0	79.10	40.03	1.601	70.5	2.731
BH - 11	6.00-6.45	39.07	25.0	69.50	30.43	1.217	18.9	1.447
	7.50-7.95	39.07	25.0	73.81	34.74	1.390	24.7	1.733
	9.00-9.45	39.07	25.0	69.56	30.49	1.220	57.2	1.918
	12.00-12.45	39.07	25.0	68.66	29.59	1.184	34.1	1.587
BH - 12	1.50-1.95	14.53	35.0	72.43	57.90	1.654	16.6	1.929
	4.50-4.95	39.07	25.0	65.71	26.64	1.066	17.0	1.246
	9.00-9.45	39.07	25.0	79.38	40.31	1.612	11.9	1.804
	10.50-10.95	39.07	25.0	67.23	28.16	1.126	46.1	1.646
BH - 13	3.50-3.95	39.07	25.0	69.50	30.43	1.217	27.2	1.548
	7.50-7.95	39.07	25.0	77.76	38.69	1.548	15.1	1.781
	9.00-9.45	39.07	25.0	72.38	33.31	1.332	33.8	1.782
	12.00-12.45	39.07	25.0	70.87	31.80	1.272	27.6	1.623

TESTED BY
G.G.W.H. RANDUNU

CHECKED BY
ENG. K.V.S.D. JAYAMALI

12/12/2011
DATE

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ANNEXURE – B

Geotechnical Investigation Supplementary Report – October 2012

G/3061

GEOTECHNICAL INVESTIGATION FOR PROPOSED KEELLS CITY PROJECT AT GLENNIE STREET, COLOMBO 02

(Supplementary Report)

CLIENT – John Keells Holdings PLC

October
2012

Geotech (Pvt) Ltd
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BOREHOLE LOG

Project :	ADDITIONAL SOIL INVESTIGATION FOR PROPOSED KEELS CITY DEVELOPMENT PROJECT AT 130, GLENNIE ST, COLOMBO 02	Job No :	G/ 3061 — BH No: EX-16
		Drilling Rig :	Y.W.E. D - 90
		Duration :	18-07-2012 to 20-07-2012
Client :	JOHN KEELS HOLDINGS PLC	Coordinates:	397686.799 E 491799.323 N
Consultant :	BALMOND STUDIO -LONDON	Logged By :	IFHAM SARUDEEN

Depth (m)	Water (m)	Casing (m)	In situ and Laboratory Test: Depth (m)	Type	00 to 15 (cm)	15 to 30 (cm)	30 to 45 (cm)	SPT No	Lithological Description	Legend (m)
0.10										0.10m
0.10–1.60	GWL 1.60								Fill with Building debris & tar Mixtures Blackish	1.60m
03			03.00-03.45	SPT(D)	00	00	01	01	Blackish brown ORGANIC CLAY	5.30m
06			Water loss at 5.80m 06.00 - 06.45	SPT(D)	05	17	20	37	Greyish CLAYEY SAND CLAY - 320% SAND - 80% Sand fine to medium	
09			09.00 - 09.45	No SPT was done Due Unchanged Layer (Wash Sample)						

Penetration Resistance (Based on SPT Values): 10 20 30 40

Key:

PP	- Physical Property Test	RQD	- Rock Quality Designation (%)
MP	- Mechanical Property Test	CR	- Core Recovery (%)
CP	- Chemical Property Test	NR	- No Recovery
UD	- Undisturbed Samples	SPT	- Standard Penetration Test
HB	- Hammer Bouncing	D	- Disturbed Samples

Rock Level - 14.20m
BH Terminated at - 19.60m
Weather - HOT
Ground Water Level - 1.60m

Sheet 1 of 2

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E-Mail : geotech@eureka.lk , geotech@sltnet.lk Web : www.geotechlanka.lk

BOREHOLE LOG		Job No :	G/ 3061	BH No: FX-16
Project :	ADDITIONAL SOIL INVESTIGATION FOR PROPOSED KEELS CITY DEVELOPMENT PROJECT AT 130, GLENNIE ST, COLOMBO 02	Drilling Rig :	Y.W.E. D - 90	
		Duration :	04-07-2012 to 05-07-2012	
Client :	JOHN KEELS HOLDINGS PLC	Coordinates:	397716.301 E 491879.774 N	
Consultant :	BALMOND STUDIO -LONDON	Logged By :	IFHAM SARUDEEN	

Depth (m)	Water (m)	Casing (m)	In situ and Laboratory Test: Depth (m)	Type	00 to 15 (cm)	15 to 30 (cm)	30 to 45 (cm)	S.P.T No	Lithological Description	Legend (m)	Penetration Resistance (Based on SPT Values) 10 20 30 40	Depth (m)
11									-Do-			11
12			12.00-12.45	SPT(D)	15	29	--	50<		12.30m		12
13									Greyish Slightly WEATHERED ROCK			13
14					C/R(%)		R.Q.D(%)		Rock Level	14.20m		14
15			14.20 to 15.40		92		77					15
16			15.40 to 16.70		100		54					16
17			16.70 to 18.50		100		89		BIOTITE GNEISS			17
18												18
19			18.50 to 19.60		95		90			19.60m		19
20									BH Terminated			20

Key:				Rock Level - 14.20m
PP	- Physical Property Test	RQD	- Rock Quality Designation (%)	BH Terminated at - 19.60m
MP	- Mechanical Property Test	CR	- Core Recovery (%)	Weather - HOT
CP	- Chemical Property Test	NR	- No Recovery	Ground Water Level - 1.60m
UD	- Undisturbed Samples	SPT	- Standard Penetration Test	
HB	- Hammer Bouncing	D	- Disturbed Samples	Sheet 2 of 2

GEOTECH (PVT) LIMITED
Foundation Specialist

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<u>BH EX 16</u>


BH-16 KEELLS CITY PROJECT AT COLOMBO-02 18-07-2012
15.40
M
16.70
M


BH-16 KEELLS CITY PROJECT AT COLOMBO-02
18.50
M
BH-16
SAMPLE
REMOVED
FOR-U.C.S
BORE HOLE
TERMINATED
AT - 19.60
M

B O R E H O L E   L O G

Project :	ADDITIONAL SOIL INVESTIGATION FOR PROPOSED KEELS CITY DEVELOPMENT PROJECT AT 130, GLENNIE ST, COLOMBO 02	Job No :	G/ 3061 — BH No: EX-17
		Drilling Rig :	ACKER
		Duration :	20-07-2012 to 21-07-2012
Client :	JOHN KEELS HOLDINGS PLC	Coordinates:	397692.496 E 491774.991 N
Consultant :	BALMOND STUDIO -LONDON	Logged By :	A.V.V.De Alwis

Depth (m)	Water (m)	Casing (m)	Test Depth (m)	Type	00 to 15 (cm)	15 to 30 (cm)	30 to 45 (cm)	SPT No	Lithological Description	Legend (m)	Depth (m)
			00.00 -00.15						Aspoil Layer	0.15m	
01	GWL								Yellowish brown very fine to medium CLAYEY SAND		01
			01.60							1.60m	
02											02
03			03.00-03.45	SPT(D) N/R	17	13 (Wash Sample)	12	25			03
04									Greyish very fine to medium CLAYEY SAND		04
05										5.80m	05
06			06.00 - 06.45	SPT(D)	03	05	07	12			06
07									Greyish very fine CLAYEY silty SAND		07
08											08
09			09.00 - 09.45	SPT(D)	04	01	02	03		9.15m	09
10									Blackish dark brown medium SANDY ORGANIC CLAY		10

Penetration Resistance (Based on SPT Values): 10 20 30 40

Key:
PP - Physical Property Test
MP - Mechanical Property Test
CP - Chemical Property Test
UD - Undisturbed Samples
HB - Hammer Bouncing
RQD - Rock Quality Designation (%)
CR - Core Recovery (%)
NR - No Recovery
SPT - Standard Penetration Test
D - Disturbed Samples

Rock Level - 13.70m
BH Terminated at - 18.70m
Weather - HOT
Ground Water Level - 1.50m

Sheet 1 of 2

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BOREHOLE LOG		Job No :	G/ 3061	BH No:	EX-17
Project :	ADDITIONAL SOIL INVESTIGATION FOR PROPOSED KEELS CITY DEVELOPMENT PROJECT AT 130, GLENNIE ST, COLOMBO 02	Drilling Rig :	Y.W.E. D - 90		
		Duration :	20-07-2012 to 21-07-2012		
Client :	JOHN KEELS HOLDINGS PLC	Coordinates:	397692.496 E 491774.991 N		
Consultant :	BALMOND STUDIO - LONDON	Logged By :	A.V.V.De Alwis		

Depth (m)	Water (m)	Casing (m)	In situ and Laboratory Test: Depth (m)	Type	00 to 15 (cm)	15 to 30 (cm)	30 to 45 (cm)	S.P.T No	Lithological Description	Legend (m)	Penetration Resistance (Based on SPT Values) 10 20 30 40
11									-Do-	11.90m	
12			12.00-12.45	SPT(D)	10	17	19	36	Greyish very fine to medium stiff silty SANDY CLAY Highly WEATHERED GNEISS		
13									Rock Level	13.70m	
					C/R(%)		R.Q.D(%)				
14–15			13.70 to 15.70		95		72		Blackish grey banded pale grey BIOTITE GRNITIC GNEISS		
16			15.70 to 16.70		100		80				
17			16.70 to 17.70		100		90				
18			17.70 to 18.70		100		90			18.70m	
19–20									BH Terminated		

Key:

PP	- Physical Property Test	RQD	- Rock Quality Designation (%)
MP	- Mechanical Property Test	CR	- Core Recovery (%)
CP	- Chemical Property Test	NR	- No Recovery
UD	- Undisturbed Samples	SPT	- Standard Penetration Test
HB	- Hammer Bouncing	D	- Disturbed Samples

Rock Level	- 13.70m
BH Terminated at	- 18.70m
Weather	- HOT
Ground Water Level	- 1.50m

Sheet 2 of 2

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BH EX 17


BH-17 - KEELLS CITY PROJECT - COLOMBO - 02
20.07.2012
15.70
M
16.70
M
BH-17
SAMPLE
RETRIVED
FOR - U.C.S


BH-17 - KEELLS CITY PROJECT - COLOMBO - 02
20.07.2012
15.70
M
16.70
M
BH-17
SAMPLE
RETRIVED
FOR - U.C.S

GROUP ENGINEERING LABORATORIES (PVT) LIMITED

A Member of Geotech Group

Head office : 13/1, Pepiliyana Mw, Kohuwala, Nugegoda
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Lab 465/1, Sunethradevi Rd, Pepiliyana, Boralesgamuva
Tel/Fax : 2769828

UNCONFINED COMPRESSIVE STRENGTH TEST RESULTS OF ROCK

TEST METHOD : ASTM : D 2938 - 86

PROJECT : Proposed Keells City Project, Glannie Street, Colombo - 02

Sheet 08 of 15

CLIENT : M/S John Keells Holdings PLC

JOB NO : GL/0757

BH No.	BH EX -15	BH EX -15	BH EX -15	BH EX -16	BH EX -16	BH EX -16
Depth (m)	13.70 - 14.90	15.90-16.90	17.90 - 18.90	14.20-15.40	15.40-16.70	18.50-19.60
Diameter of Core (mm)	54	54	54	54	54	54
Length of Core (mm)	71	80	108	108	108	106
Load at Failure (kN)	283	100.7	66.7	61	268	131
Unconfined Compressive Strength (MN/m^2)	**116.30**	**42.20**	**29.12**	**26.63**	**117.02**	**57.07**
Failure Sketch						

TESTED BY
D.R.K. Buddhika

CHECKED BY
Eng. K.V.S.D. Jayamali

GROUP ENGINEERING LABORATORIES (PVT) LTD.
No. 465/1, Sunethradevi Road,
Pepiliyana, Boralesgamuwa.
Tel/Fax : 0112-769828

29/8/12
DATE

GEOTECH (PVT) LIMITED

GROUP ENGINEERING LABORATORIES (PVT) LIMITED

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Tel/Fax : 2769828

UNCONFINED COMPRESSIVE STRENGTH TEST RESULTS OF ROCK

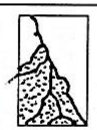
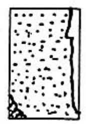
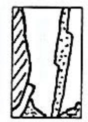
TEST METHOD : ASTM : D 2938 - 86

PROJECT : Proposed Keells City Project, Glannie Street, Colombo - 02

CLIENT : M/S John Keells Holdings PLC

Sheet 09 of 15

JOB NO : GL/0757

BH No.	BH EX - 17	BH EX - 17	BH EX - 17	BH EX - 18	BH EX - 18	BH EX - 18
Depth (m)	13.70-15.70	15.70-16.70	17.70-18.70	14.25-15.25	15.25-16.85	18.25-19.60
Diameter of Core (mm)	54	54	54	54	54.93	54
Length of Core (mm)	106	90	108	102	184	108
Load at Failure (kN)	53.3	170	236	168	196	196
Unconfined Compressive Strength (MN/m^2)	23.22	72.49	103.05	72.84	86.91	85.58
Failure Sketch						

TESTED BY
D.R.K. Buddhika

CHECKED BY
Eng. K.V.S.D. Jayamali

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29/8/12
DATE

GEOTECH (PVT) LIMITED

Borehole No.	Depth (m)	Elevation (m) MSL	CR (%)	RQD (%)
BH-09	14.80-17.30	-11.93 to -14.43	100	88
	17.30-17.90	-14.43 to -15.03	100	100
	17.90-18.90	-15.03 to -16.03	95	73
	18.90-20.80	-16.03 to -17.93	100	100
BH-10	13.50-15.25	-9.86 to -11.61	46	20
	15.25-16.55	-11.61 to -12.91	85	11
	16.55-17.25	-12.91 to -13.61	86	50
	17.25-17.80	-13.61 to -14.16	100	18
	17.80-18.85	-14.16 to -15.21	86	52
	18.85-19.85	-15.21 to -16.21	100	52
BH-11	15.30-16.80	-12.00 to -13.50	87	00
	16.80-19.05	-13.50 to -15.75	52	05
	19.05-20.05	-15.75 to -16.75	90	00
	20.05-21.30	-16.75 to -18.00	60	15
BH-12	15.00-16.60	-11.17 to -12.77	97	60
	16.60-18.20	-12.77 to -14.37	92	70
	18.20-19.90	-14.37 to -16.07	82	47
	19.90-21.00	-16.07 to -17.17	73	36
	21.00-21.80	-17.17 to -17.97	75	25
	21.80-23.00	-17.97 to -19.17	33	00
BH-13	13.20-14.80	-10.59 to -12.19	100	100
	14.80-16.45	-12.19 to -13.84	100	81
	16.45-18.46	-13.84 to -15.85	82	82
BH EX-01	8.90-10.60	-6.12 to -7.82	23	16
	10.60-12.00	-7.82 to -9.22	100	100
	12.00-12.67	-9.22 to -9.89	100	100
	12.67-14.18	-9.89 to -11.40	100	100
BH EX-02	11.70-12.60	-9.12 to -10.02	100	90
	12.60-14.36	-10.02 to -11.78	100	85
	14.36-15.71	-11.78 to -13.13	100	81
	15.71-16.71	-13.13 to -14.13	95	95
BH EX-03	12.90-15.00	-10.25 to -12.35	43	07
	15.00-16.60	-12.35 to -13.95	100	53
	16.60-18.04	-13.95 to -15.39	100	100
BH EX-04	13.60-15.65	-11.11 to -13.16	95	54
	15.65-17.40	-13.16 to -14.91	100	91
	17.40-18.85	-14.91 to -16.36	100	86
BH EX-05	11.50-13.00	-9.08 to -10.58	97	71
	13.00-14.50	-10.58 to -12.08	100	100
	14.50-15.82	-12.08 to -13.40	100	100
	15.82-17.05	-13.40 to -14.63	100	97
BH EX-06	14.70-16.20	-12.41 to -13.91	100	83
	16.20-17.70	-13.91 to -15.41	100	73
	17.70-19.20	-15.41 to -16.91	98	89
	19.20-19.85	-16.91 to -17.56	100	100
BH EX-07	14.00-15.00	-11.41 to -12.41	100	00
	15.00-15.90	-12.41 to -13.31	100	44
	15.90-16.80	-13.31 to -14.21	100	00
	16.80-17.20	-14.21 to -14.61	100	100
	17.20-18.27	-14.61 to -15.68	100	100
	18.27-19.09	-15.68 to -16.50	100	100

Borehole No.	Depth (m)	Elevation (m) MSL	CR (%)	RQD (%)
BH EX-08	12.80-14.10	-10.16 to -11.46	100	85
	14.10-15.64	-11.46 to -13.00	100	78
	15.64-16.78	-13.00 to -14.14	100	100
	16.78-17.88	-14.14 to -15.24	100	100
BH EX-09	13.20-14.80	-9.45 to -11.05	94	19
	14.80-16.35	-11.05 to -12.60	100	55
	16.35-17.35	-12.60 to -13.60	100	75
	17.35-18.25	-13.60 to -14.50	100	89
BH EX-10	12.50-14.00	-8.76 to -10.26	87	87
	14.00-15.00	-10.26 to -11.26	100	100
	15.00-16.00	-11.26 to -12.26	100	100
	16.00-17.50	-12.26 to -13.76	97	97
BH EX-11	13.00-14.00	-9.26 to -10.26	100	90
	14.00-15.00	-10.26 to -11.26	100	85
	15.00-16.45	-11.26 to -12.71	100	83
	16.45-18.35	-12.71 to -14.61	100	95
BH EX-12	14.70-16.20	-10.88 to -12.38	83	76
	16.20-17.00	-12.38 to -13.18	100	87
	17.00-18.00	-13.18 to -14.18	100	75
	18.00-19.50	-14.18 to -15.68	100	100
BH EX-13	15.50-16.30	-11.65 to -12.45	100	16
	16.30-16.90	-12.45 to -13.05	100	23
	16.90-17.90	-13.05 to -14.05	95	35
	17.90-18.95	-14.05 to -15.10	100	52
	18.95-19.95	-15.10 to -16.10	100	70
	19.95-20.77	-16.10 to -16.92	100	85
BH EX-14	13.40-14.60	-9.58 to -10.78	97	25
	14.60-15.45	-10.78 to -11.63	100	23
	15.45-16.95	-11.63 to -13.13	100	93
	16.95-18.65	-13.13 to -14.83	100	53
BH EX-15	13.70-14.90	-10.68 to -11.88	92	11
	14.90-15.90	-11.88 to -12.88	100	46
	15.90-16.90	-12.88 to -13.88	100	48
	16.90-17.90	-13.88 to -14.88	100	55
	17.90-18.90	-14.88 to -15.88	100	49
BH EX-16	14.20-15.40	-11.20 to -12.40	92	77
	15.40-16.70	-12.40 to -13.70	100	54
	16.70-18.50	-13.70 to -15.50	100	89
	18.50-19.60	-15.50 to -16.60	95	90
BH EX-17	13.70-15.70	-10.60 to -12.60	95	72
	15.70-16.70	-12.60 to -13.60	100	80
	16.70-17.70	-13.60 to -14.60	100	90
	17.70-18.70	-14.60 to -15.60	100	90
BH EX-18	14.25-15.25	-10.91 to -11.91	90	55
	15.25-16.85	-11.91 to -13.51	100	78
	16.85-18.25	-13.51 to -14.91	100	96
	18.25-19.60	-14.91 to -16.26	100	100
BH EX-19	14.50-16.20	-11.42 to -13.12	94	100
	16.20-17.40	-13.12 to -14.32	100	83
	17.40-18.50	-14.32 to -15.42	100	73
	18.50-19.50	-15.42 to -16.42	90	100

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Tel/Fax : 2769828

PROJECT :- Proposed Keels City Project, Glannie Street, Colombo - 02

CLIENT :- M/S John Keells Holdings PLC

Sheet 03 of 04

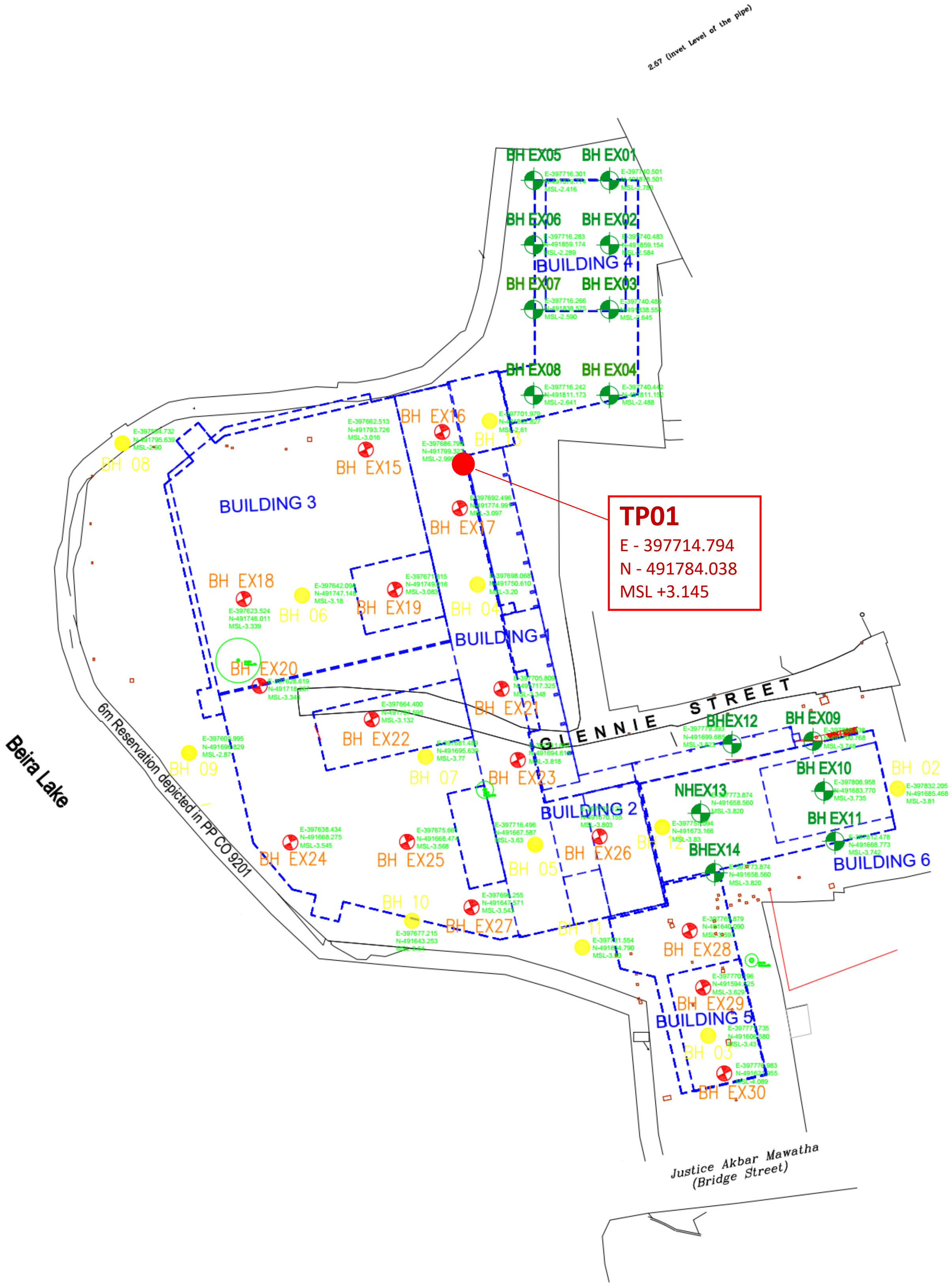
JOB NO :- GL/0757/004

BH No	Depth (m)	RQD (%)	Avg. Discontinuity Spacing (mm)	Discontinuity Frequency (m^{-1})
BH EX - 16	14.20-15.40	77	166	6
	15.40-16.70	54	108	9
	16.70-18.50	89	274	4
	18.50-19.60	90	327	3
BH EX - 17	13.70-15.70	72	239	4
	15.70-16.70	80	194	5
	16.70-17.70	90	425	2
	17.70-18.70	90	178	6
BH EX - 18	14.25-15.25	55	203	5
	15.25-16.85	78	221	5
	16.85-18.25	96	467	2
	18.25-19.60	100	443	2
BH EX - 19	14.50-16.20	100	277	4
	16.20-17.40	83	169	6
	17.40-18.50	73	190	5
	18.50-19.50	100	445	2
BH EX - 20	17.80-19.40	12	234	4
	19.40-20.70	69	167	6
	20.70-21.60	39	108	9
	21.60-23.20	84	260	4
BH EX - 21	17.70-18.90	00	72	14
	18.90-20.10	00	114	9
	20.10-21.70	44	153	7
	21.70-23.30	19	158	6
	23.30-24.70	86	183	5
BH EX - 22	14.50-15.50	50	111	9
	15.50-16.95	79	625	2
	16.95-18.60	88	324	3
	18.60-20.10	97	304	3
BH EX - 23	16.60-17.60	00	48	21
	17.60-18.70	11	90	11
	18.70-19.60	00	81	12
	19.60-21.10	00	76	13
	21.10-21.75	60	150	7

GEOTECH (PVT) LIMITED

ANNEXURE – C

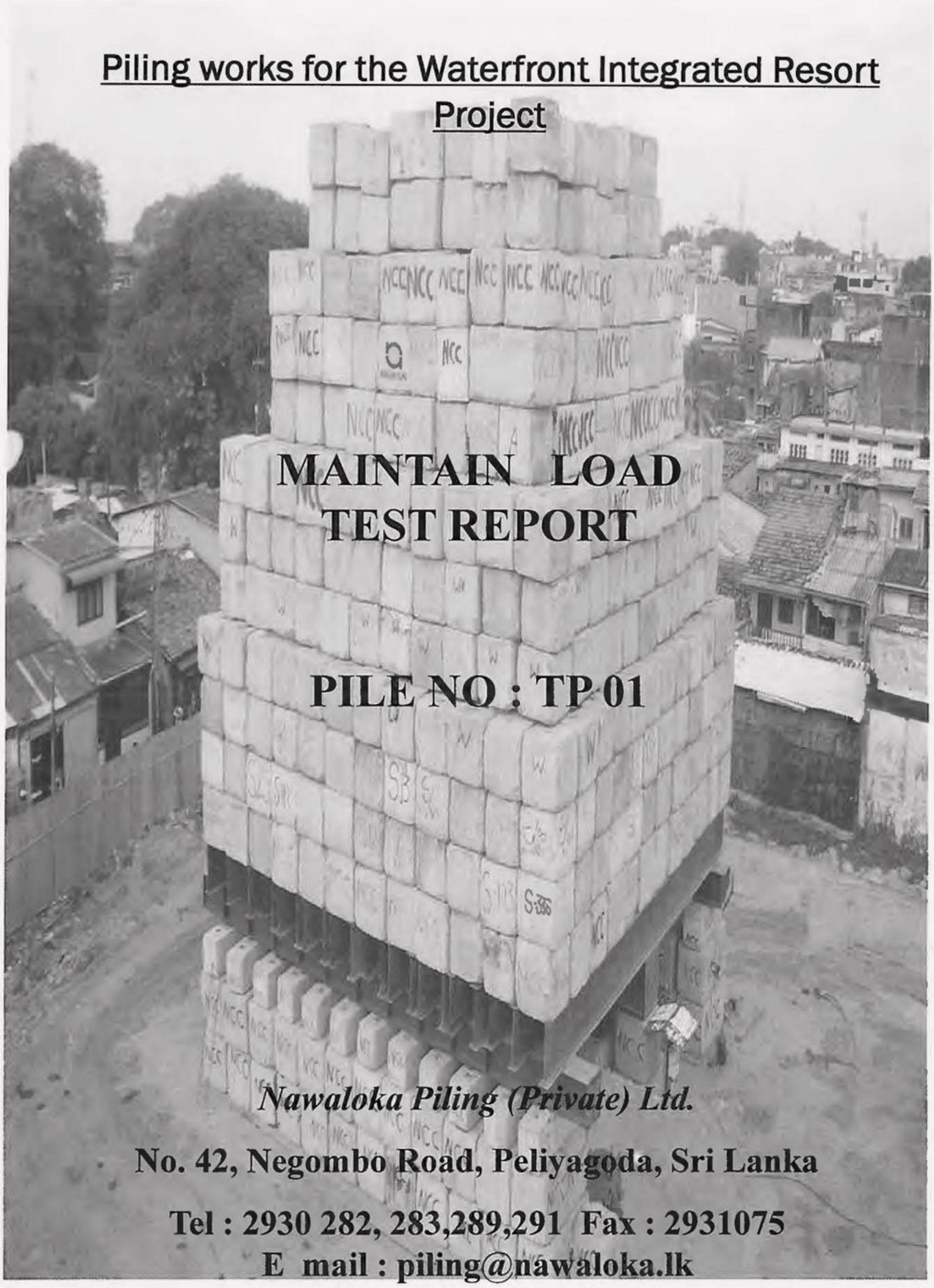
Bore Hole and Test Pile Locations Layout Plan


TP01
E - 397714.794
N - 491784.038
MSL +3.145
BUILDING 1
BUILDING 2
BUILDING 3
BUILDING 4
BUILDING 5
BUILDING 6
GLENNIE STREET
Beira Lake
6m Reservation depicted in PP CO 9201
Justice Akbar Mawatha
(Bridge Street)
2.57 (Invet Level of the pipe)

ANNEXURE – D

Maintain Load Test Report – TP01 Test Pile

Piling works for the Waterfront Integrated Resort Project



MAINTAIN LOAD TEST REPORT

PILE NO : TP 01

Nawaloka Piling (Private) Ltd.

No. 42, Negombo Road, Peliyagoda, Sri Lanka

Tel : 2930 282, 283,289,291 Fax : 2931075

E mail : piling@nawaloka.lk

PILING WORKS FOR THE PROPOSED WATERFRONT INTEGRATED RESORT PROJECT.

1. INTRODUCTION

Proposed Water Front Integrated Project was awarded to Nawaloka Piling (pvt) ltd for bored piling work. At the initial stage five preliminary test piles were installed. This report covers the test pile no TP 01 which was tested on 09th June 2014 to 10th June 2014.

2. DATA OF TEST PILE

Pile Ref. No.	TP 01
Pile diameter	1000mm
Concrete Grade	Grade 50
Date of casting of pile	04/04/2013
Date of testing of pile	09/06/ 2014, 15.00 hrs to 10/06/2014, 00.15 hrs
Age of the pile at load test	65 days
Test Load	15707.5 KN

3. OBJECTIVES

The objective of the test is to verify the design parameters and assumptions which assumed in design.

4. SITE LOCATION

The site is located at Glennie street Colombo 02

5. DETAILS OF TEST.

According to the design the working load of the pile had been estimated as 6283KN and the test was carried out carried out in three cycles of loading and unloading up to 100%, 150% and 250% respectively.

Nawaloka Piling (Pvt) Ltd

6. LOADING ARRANGEMENT

The reaction for the loading was obtained by jacking against kentledge of concrete blocks placed on a steel platform. The load was applied to the test pile using hydraulic jacks (refer annexure 2)

7. CALIBRATION OF PRESSURE GAUGE & DIAL GAUGES.

Details are attached here with in Annexure 02

8. TEST PROCEDURE.

The Maintained Load (M/L) test was carried out in 3 cycles of loading and unloading as indicated in the table in this section.
Displacements were measured using four dial gauges immediately on applying the load at each stage, and at time intervals as follows. (Reference: ICTAD piling specification)

Cycle	Load (%)	Load (kN)	time(min)
1st	25	1570.75	60
	50	3141.50	60
	75	4712.25	60
	100	6283.0	360
	75	4712.25	15
	50	3141.50	15
	25	1570.75	15
	0	0	60
2nd	25	1570.75	15
	50	3141.50	15
	75	4712.25	15
	100	6283.0	15
	125	7853.75	60
	150	9424.50	360
	125	7853.75	15
	100	6283.0	15
	75	4712.25	15
	50	3141.50	15
	25	1570.75	15
	0	0	60

Nawaloka Piling (Pvt) Ltd

3rd	25	1570.75	15
	50	3141.50	15
	75	4712.25	15
	100	6283.0	15
	125	7853.75	15
	150	9424.50	15
	175	10995.25	15
	200	12566.0	15
	225	14136.75	60
	250	15707.50	360
	225	14136.75	15
	200	12566.0	15
	175	10995.25	15
	150	9424.50	15
	125	7853.75	15
	100	6283.0	15
	75	4712.25	15
	50	3141.50	15
	25	1570.75	15
	0	0	60

9. TEST RESULTS.

The results of the readings obtained of dial gauge readings for each increment of load are given in Annexure 1.

Using these, a graph of (Load vs. Settlement) has been constructed and this is also given in the same Annexure.

Applied Load(%)	Applied Load(KN)	Maximum settlement/(mm)					
		1st circle		2nd circle		3rd circle	
		On loading	After unloading	On loading	After unloading	On loading	After unloading
100	6283	5.80	-	4.57	6.99	3.96	9.23
150	9424.5			8.25	-	6.7	12.04
250	15707.5					14.4	

……………………………

Sunil Dharmasena,

Deputy General Manager,

Nawaloka Piling (Pvt) Ltd

PILING WORK FOR THE PROPOSED WATERFRONT INTERGRATED RESORT PROJECT -MLT TEST

Pile No : TP 01
Diameter : 1000mm
Casting Date : 04.04.2014
MaximumTest Load : 15707.5 KN

Load / (KN)	Average Settlement / (mm)
0	-
25	1.33
50	2.71
75	3.92
100	5.80
75	5.45
50	4.29
25	2.49
0	0.32
25	1.11
50	2.14
75	3.18
100	4.57
125	5.89
150	8.25
125	7.94
100	6.99
75	5.59
50	4.18
25	2.29
0	0.18
25	0.91
50	1.77
75	2.59
100	3.96
125	5.05
150	6.70
175	8.21
200	9.77
225	10.97
250	14.40
225	14.40
200	13.99
175	13.30
150	12.04
125	10.49
100	9.23
75	7.68
50	5.48
25	3.00
0	0.20

Report on the Maintained Load Test carried out on Pile No. TP 01 in piling works for proposed Waterfront Integrated Resort Project at Glennie Street, Colombo 02

1. Details of Pile Constructed

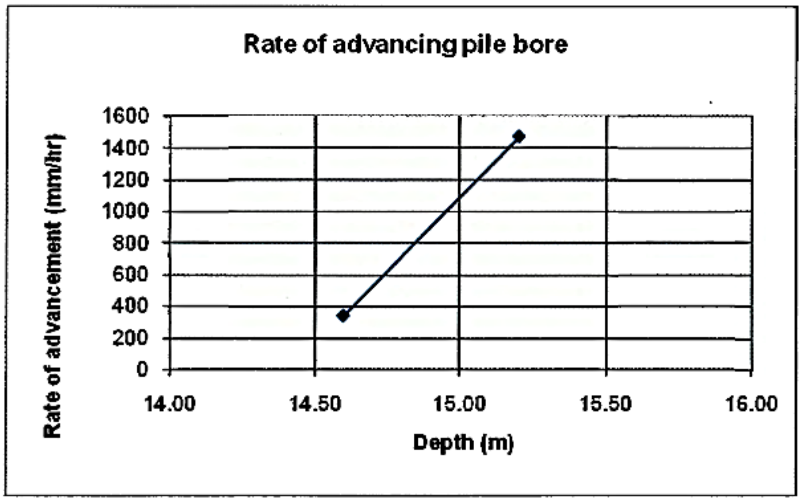
The Maintained Load Test was carried out as indicated in the Factual Report of TP 01, which is a 1000 mm dia. pile. The objective of the test was indicated as being to confirm the pile carrying capacity as had been used in the design.

The following details are observed from the construction records of pile borings and sample cutting observations:

Ground Elev. at top of pile bore	3.145 m MSL
Depth of commencement of HW Rock =	14.60 m
Elev. at commencement of HWR =	-11.455 m MSL
Depth of commencement of Fresh Rock =	16.30 m
Elev. at commencement of Fresh Rock =	-13.155 m MSL
Depth of termination =	17.80 m
Elev. at pile termination =	-14.655 m MSL
Thickness of HWR + Fresh rock =	3.20 m
Thickness of Fresh rock =	1.50 m

2. Analysis of pile boring records

The rate of advancement of the pile bore with depth is indicated below. As per these records, it is not possible to identify clearly the change from 'highly weathered rock' to 'fresh rock'.



3. Details of Maintained Load Test

As per the design, the Working Loads on TP 01 had been estimated as 6283 kN. The test was carried out in three cycles of loading and unloading up to 100%, 150% and 250% respectively.

3.1 Pile No. TP 01

Pile diameter (mm) =	1000
Age at load test (days) =	65
Design Working Load (kN) =	6,283
150% of Design WL (kN) =	9,425
250% of Design WL (kN) =	15,708

3.2 Results of the Pile Load Test

The results of the Pile Load Test are summarized below.

Applied Load (kN)	Average Settlement (mm)	
	On loading	After unloading
6,283	5.80	0.32
9,425	8.25	0.21
15,708	14.40	0.20

These results lie within the acceptability criteria in the ICTAD Guidelines.

4. Conclusion

It is concluded that:

a) the pile is able to safely carry the design Working Load; and

b) the ultimate load carrying capacity of the pile exceeds 15,708 kN.

It is also concluded that from pile boring records for this 1000 mm dia. pile:

a) the penetration rates observed during rock socketing lie in the range of (400-1500) mm/hr; i.e. (100-375) mm per 15 min..

B. L. Tennekoon

Prof. B.L. Tennekoon
Emeritus Professor,
University of Moratuwa

21st June 2014

Prof. B. L. Tennekoon
B.Sc. (Engineering), Ceylon: Ph.D. (Cantab):
C. Eng., F.I.E. (Sri Lanka)
Emeritus Professor,
University of Moratuwa, Sri Lanka.

ANNEXURE – E

Pile Dynamic Analyzer (PDA) Test Report – TP01 Test Pile

Report on the Pile Dynamic Analyzer (PDA) Test carried out on Pile No. TP 01 in piling works for proposed Waterfront Integrated Resort Project at Glennie Street, Colombo 02

Client: Nawaloka Piling (Pvt) Ltd

Prepared by: Prof. B.L. Tennekoon
Emeritus Professor,
University of Moratuwa

Report on the Pile Dynamic Analyzer (PDA) Test carried out on Pile No. TP 01 in piling works for proposed Waterfront Integrated Resort Project at Glennie Street, Colombo 02

1. Details of Pile Constructed

The Pile Dynamic Analyzer (PDA) Test was carried out as indicated in the PDA Contractor's Report of TP 01, which is a 1000 mm dia. pile. The objective of the test was indicated as being to

a) confirm the pile carrying capacity as had been used in the design; and

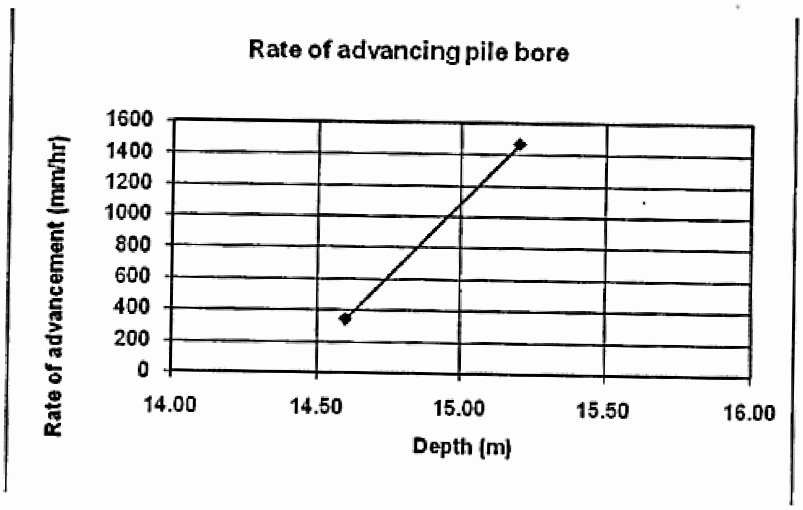
b) Verify pile design parameters.

The following details are observed from the construction records of pile borings and sample observations:

Ground Elev. at top of pile bore =	3.145 m MSL
Depth of commencement of HW Rock =	14.60 m
Elev. at commencement of HWR =	-11.455 m MSL
Depth of commencement of Fresh Rock =	16.30 m
Elev. at commencement of Fresh Rock =	-13.155 m MSL
Depth of termination =	17.80 m
Elev. at pile termination =	-14.655 m MSL
Thickness of HWR + Fresh rock =	3.20 m
Thickness of Fresh rock =	1.50 m

2. Analysis of pile boring records

The rate of advancement of the pile bore with depth is indicated below. As per these records, it is not possible to identify clearly the change from 'highly weathered rock' to 'fresh rock'.



3. Details of PDA Test and CAPWAP Analysis

3.1 Pile details

Pile diameter (mm) =	1000
Total Pile length below GL (m) =	16.25
Pile length below cut-off (m) =	16.5
Design Working Load (tons) =	628
Test Load (tons) =	1,570

3.2 Hammer Details

Hammer Type	Drop
Hammer Weight	28 tons
Hammer Drop	1.15m

3.3 Discussion

3.3.1 Method of testing

The method of testing is to increase the hammer drop starting from a minimum drop of around 0.3m. An immediate field assessment of the Carrying Capacity is available from the Case Static Capacity. The test is terminated if the required Test Load has been achieved.

3.3.2 CAPWAP Analysis

The analysis results from CAPWAP are based on a mathematical model simulation and the results reported are from the best-matched model attained during the analysis.

In the analysis carried out, a Match Quality of 4.72 is reported. This indicates that the mathematical simulation adopted is acceptable.

In the CAPWAP analysis, which had been carried out for this pile, the following observations are made:

- *CAPWAP Analysis*

Load carried in shaft friction (tons) =	*955.5*
Load carried in end bearing (tons) =	*780.4*
Load carried in total (tons) =	*1,735.9*
End bearing pressure (tons/m^2) =	*993.60*

This corresponds to a mobilized end bearing pressure of 9,747 kN/m^2

- The ultimate end bearing capacity exceeds 9.75 N/mm^2.

- *Design recommendations for ultimate skin friction coefficient (f_u)*

Average f_u in last 4.1m of pile (kN/m^2) = *324*

(As per the pile bore construction records, the last 4.1m of pile consists of 3.2m in 'fresh rock' or 'highly fractured rock' +0.9m in 'completely weathered rock'.

- *Settlement Analysis*

Settlement at Working Load (WL) of 628 tons (mm)= *4.3*
Settlement at Test Load of 2.5 times WL (mm) = *11.3*

All these settlements are within acceptable limits.

4. Conclusions

It is concluded that

a) the pile is able to safely carry the design Working Load of 628 tons;

b) the ultimate end bearing capacity of the rock exceeds 9.75 N/mm^2; and

c) the ultimate skin friction coefficient in the 3.2m of 'highly weathered rock' and 'fresh rock' is 324 kN/m^2.

Prof. B.L. Tennekoon
Emeritus Professor,
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24th June 2014

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Emeritus Professor,
University of Moratuwa, Sri Lanka.

3.0 SUMMARY OF TEST RESULTS

Pile Number	TP-01					
Date of Testing	18/06/2014					
Pile Type	BP					
Pile Size (mm)	1000					
Cross Sectional Area (cm^2)	7853					
Wave Speed (m/s)	3800					
Total Length (m)	17.80					
Length Below Gauges (m)	16.50					
Driven Penetration (m)	16.25					
Working Load, WL (tons)	628					
Test Load, TL (tons)	1570					
Hammer Model	Drop					
Ram Weight (tons)	28					
Observed Stroke (m)	1.15					
Blow No.	4					
RMX (tons)	1747					
FMX (tons)	1151					
CSX (MPa)	14.5					
EMX (ton-m)	10.2					
SET (mm/ blow)	0					
Pile Integrity	Satisfactory					
Total Blows Delivered	3					
CAPWAP ANALYSIS						
Skin (tons)	955					
Toe (tons)	780					
Mobilised Capacity (tons)	1735					
Settlement at WL (mm)	4.27					
Settlement at TL (mm)	11.26					

DEFINITIONS

RMX/RSU	Case Static Capacity field estimate (J = 0.50)
FMX	Maximum measured pile top force
CSX	Maximum compressive stress at pile top
ETR	Energy Transfer Ratio
SET	Pile permanent displacement
Blow No.	This blow used for CAPWAP Analysis

PILE TYPE

T	Timber Pile
RC	Reinforced Concrete Pile
PSC	Prestressed Spun Concrete Pile
H	Steel H Pile
SP	Steel Pile Pile
BP	Bored Pile

UNCERTAINTY OF MEASUREMENT

The *Accelerometers* and *Strain Gauges* used in the measurement are subject to *allowable* calibration errors ±3%. The PDA Test has been calibrated using a PDA *Calibration Box* to not exceeding 2% of the maximum signal expected. The combined error should not exeed 5% of the maximum results expected. This is in compliance with *ASTM D4945*.

NOTES

Standard Test Method: *ASTM D4945*

The pile and hammer details are furnished by Client's Representative and / or agent are strictly treated to be correct and accurate

SET is the permanent displacement of the test piles provided by the Client's representative or the piling personnel.

ETR is the calculated energy transfer ratio based on the net measured energy transfer (EMX) over the hammer potential energy.

(Ram Weight x Observed Stroke)

GEOTECH TESTING SERVICES (PVT) LIMITED

ANNEXURE – F

Rock Layers Equivalent Mohr – Coulomb Parameters Calculation

(Microsoft Excel™ Spreadsheet)

CALCULATION OF EQUIVALENT MOHR-COULOMB PARAMETERS									
Source : E. Hoek & E. T. Brown, Practical Estimate of Rock Mass Strenght (Appendix C)									
Input	**sigci=**	85	Mpa	**mi=**	10		**GSI=**	45	
Output	**mb=**	1.403		**s=**	0.0022		**a=**	0.500	
	sigtm=	-0.134	Mpa	**A=**	0.503		**B=**	0.698	
	k=	3.015		**phi=**	30.120	degrees	**coh=**	3.270	Mpa
	sigcm=	11.356	Mpa	**E=**	6913.683	Mpa			
Tangent	**signt=**	15.970	Mpa	**phit=**	30.122	degrees	**coht**	4.116	Mpa
Calculation									
									Sums
sig3	1.00E-10	3.04	6.07	9.11	12.14	15.18	18.21	21.25	85.00
sig1	4.003	22.48	33.27	42.30	50.40	57.91	64.98	71.74	347.08
ds1ds3	15.890	4.07	3.19	2.80	2.56	2.40	2.27	2.18	35.35
sign	0.237	6.87	12.56	17.85	22.90	27.76	32.50	37.13	157.80
tau	0.945	7.74	11.59	14.62	17.20	19.48	21.54	23.44	116.55
x	-2.360	-1.08	-0.83	-0.67	-0.57	-0.48	-0.42	-0.36	-6.77
y	-1.954	-1.04	-0.87	-0.76	-0.69	-0.64	-0.60	-0.56	-7.11
xy	4.611	1.13	0.71	0.52	0.39	0.31	0.25	0.20	8.12
xsq	5.568	1.17	0.68	0.45	0.32	0.23	0.17	0.13	8.74
sig3sig1	0.000	68.23	202.01	385.23	612.01	878.92	1183.65	1524.51	4854.56
sig3sq	0.000	9.22	36.86	82.94	147.45	230.39	331.76	451.56	1290.18
taucalc	0.962	7.48	11.33	14.45	17.18	19.64	21.91	24.04	
sig1sig3fit	11.356	20.51	29.66	38.81	47.96	57.11	66.26	75.42	
signtaufit	3.408	7.26	10.56	13.63	16.55	19.38	22.12	24.81	
tangent	4.253	8.10	11.40	14.47	17.40	20.22	22.97	25.66	