Annual Sessions of IESL, pp. [377 - 384], 2019

Determination of Pile capacities for piles socketed on fractured rock, with field test data and empirical formulae

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Abstract: The development of tall structures as an uprising 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. Near the Beira Lake, the authors have noted that the basin consists of highly fractured rock layers and the estimation of the shaft friction and end bearing of piles that rest on these rock conditions are considered to be complex. The capacities are estimated by considering the fracture condition, spacing, weathering profile and also the strength of the intact rock. This envisage that in obtaining the accurate parameters to estimate the pile capacities in such rock conditions, an expert observation is required.

This study shares the authors experience in estimating the geotechnical capacity of piles in such rock conditions, to achieve an economical and stable pile foundation system. The empirical formulae used to estimate the shaft friction and end bearing of these piles are discussed extensively in the literature. The study compares these estimated pile capacities obtained from empirical formulae with the field data, for the verification of the estimated capacities. This method can be effectively used for optimizing the pile design to carry specified loading conditions.

Keywords: Pile capacity, fractured rock, field test data, shaft resistance, end bearing.

1. Introduction

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 cast insitu pile foundations for tall buildings is a common practice in Sri Lanka, as the bedrock is present at relatively shallow depths. These piles are socketed into the bedrock, where the bedrock is the strongest strata as oppose to the overburden. However, the Authors have noted that near the Beira Lake region the upper lithosphere consists of fractured rock masses. The presence of fractures inhibits a complexity to the rock mass, as these joints are considered as weak planes and these joints, and the joint conditions contribute to the overall performance of the rock mass. For this study 3 projects were selected near the Beira Lake, Colombo. The three projects selected for the study contain two residential developments and an office complex. The three projects selected for this study contains; 4 piles subjected to high strain dynamic load test using the PDA method (at the time of carrying out the study) from a residential development (Project 1) ;5 piles subjected to PDA testing for the office complex (Project 2) and, 2 piles subjected to PDA testing from another residential development (Project 3).

2. Estimating the geotechnical capacity of Piles on fractured rock using empirical formula

2.1 Estimating the End bearing using RMR method

There have been many empirical and semi-empirical formulae that have been introduced to estimate the capacity at the base of the pile. Peck et al. [1], suggested a semi empirical method for

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the estimation of the bearing capacity based on the Rock quality designation (RQD) value. However, this method has its limitation since the method can only be applied to rock masses with discontinuous that are tight and not open. The use of simple indexes along with the rock mass weathering classification were found to provide superior results [2]. Such methods that employ simple indices and the weathering and joint conditions include the RMR method [3] and the rock mass quality index i.e. the Q system [4]. This study uses the RMR method for the evaluation of the allowable bearing capacity. The Rock Mass Rating (RMR) method which was introduced by Bieniawski [3] for tunnelling, has been modified to be applied for different engineering uses, with Rock. The RMR method has been used for tunnelling, estimating the modulus of elasticity of the rock mass and to obtain the allowable bearing capacity of piles founded on rock masses. The RMR method is suitable for the piling work as the ratings can be estimated from the samples and details of the borehole investigation. The RMR method considers the strength of the intact rock mass, the joint spacing frequency, joint spacing, joint condition as well as the infilling condition of the joints which are detrimental to the determination of the overall rock mass strength. The Hong Kong guidelines [5] indicates that RMR method is also considered to be applicable to sedimentary and metamorphic, except for rocks that have been affected by dissolution features. Thilakasiri et al. [6] used the results of limited number of instrumented pile load tets to verify the applicability of the RMR method to bedrock found in Sri Lanka.

The Colombo region which belongs to the Highland complex contains primarily of gneisses rocks (metamorphic) and the basin rocks near the Beira lake are found to be highly fractured. This can be considered as an idealised situation to use the RMR method for the estimation of allowable bearing pressure.

Figure 1a, shows the correlation of RMR and the allowable end bearing capacity to be used for pile design along with the Ratings assigned for individual parameters of the rock for the estimation of RMR, shown in Figure 1b. Employing the method of RMR, the allowable end bearing capacities of piles, subjected to PDA® testing with CAPWAP® signal matching, were estimated, using the information of the boreholes located closest to the pile. The estimated allowable end bearing capacity of the piles of Project 1, Project 2 and Project 3 are provided in Table 1a, Table 1b and Table 1c respectively.

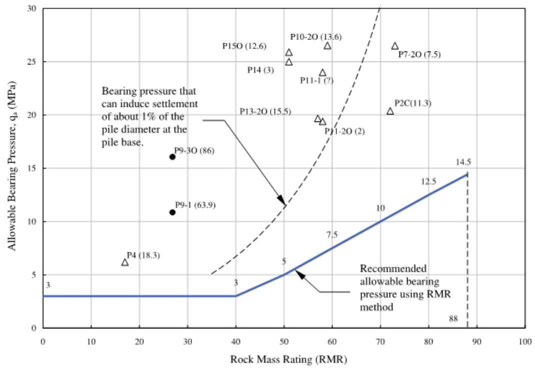

Figure 1a - Estimation of allowable bearing capacity of piles based on calculated RMR values (source: Hong Kong guideline [5])

(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

Figure 1b - Ratings assigned to individual parameters using RMR classification system (source: Hong Kong guideline [5])

Table 1a - Estimated end bearing capacities from calculated RMR values for piles for Project 1

Pile Number		TP-01	TP-02	P-105	P-70
Pile Diameter(m)		0.8	0.8	1.5	1.8
RMR value for piles	*Strength of Intact rock*	4	4	4	4
	RQD Designation	8	13	13	8
	Spacing of Joints	10	10	10	10
	Discontinuity length	2	2	2	2
	Separation rating	5	5	5	5
	Roughness rating	5	3	3	5
	Infilling (Gouge) rating	4	4	4	4
	Weathering Rating	5	5	5	5
	Ground water	7	7	7	7
	Total RMR value	50	53	53	50
Estimated allowable end bearing (MPa)		5	5.5	5.5	5

Table 1b - Estimated end bearing capacities from calculated RMR values for piles for Project 2

Pile Number		P-16	P-28	P-45	P-47	P-01
Pile Diameter		1.5	1.2	1.2	1.0	1.2
RMR value for piles	*Strength of Intact rock*	2	2	2	2	2
	RQD Designation	8	8	8	5	5
	Spacing of Joints	10	10	5	8	8
	Discontinuity length	4	4	2	1	1
	Separation rating	5	5	1	4	5
	Roughness rating	5	5	5	5	5
	Infilling (Gouge) rating	4	4	4	4	4
	Weathering Rating	5	5	5	5	5
	Ground water	10	10	10	10	10
	Total RMR value	53	53	42	44	45
Estimated allowable end bearing (MPa)		5.5	5.5	3.5	3.9	4.0

Table 1b - Estimated end bearing capacities from calculated RMR values for piles for Project 3

Pile Number		P4-21	P4-115
Pile Diameter		1800	1800
RMR value for piles	*Strength of Intact rock*	2	2
	RQD Designation	5	5
	Spacing of Joints	8	10
	Discontinuity length	4	6
	Separation rating	4	4
	Roughness rating	3	3
	Infilling (Gouge) rating	2	2
	Weathering Rating	3	3
	Ground water	10	10
	Total RMR value	41	45
Estimated allowable end bearing (MPa)		3.1	4.2

2.2 Estimating the shaft friction of piles in fractured rock

When considering the shaft friction contribution of the rock mass, correlations between the unconfined compressive strength of the rock and rock socket bond strength have been established by Horvarth [7], Rosenberg and Journeaux [8] and Williams and Pells [9]. Tomlinson and Woodward [10] gives the relationship between the ultimate shaft resistance (f_s) to the average unconfined compressive strength $\bar{q}_{uc}$ of the rock is given in Equation 1.

$$f_s = \alpha\beta\bar{q}_{uc} \qquad ....(1)$$

Where;
α is the reduction factor relating to $\bar{q}_{uc}$ (shown in Figure 2).
β is the correction factor related to discontinuity spacing in the rock mass as shown in Figure 3.

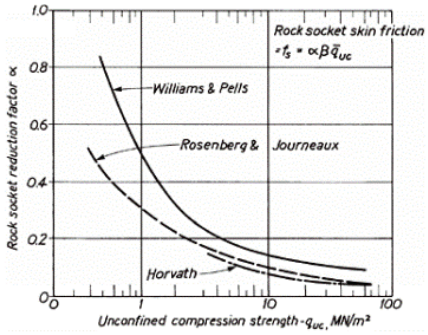

Figure 2 - Reduction factor for shaft friction (after Tomlinson and Woodward [10])

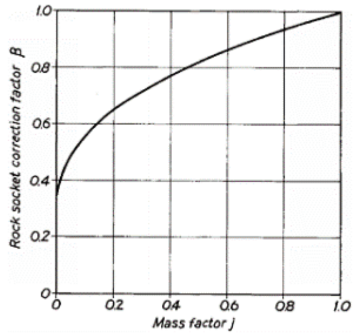

Figure 3 - Reduction factor for discontinuities used for the Williams and Pells curve (after Tomlinson and Woodward [10])

It should be noted that the curves produced by Horvarth[7] and Rosenberg and Journeaux[8] provide less values than that of the curve shown by Williams and Pells [9] for the α values. However, the corresponding β values for the curves of Horvarth[7] and Rosenberg and Journeaux[9] is set at unity and the β values for the Williams and Pells[9] curve is estimated by the mass factor *j*, which is the ratio between of the rock mass to that of the rock mass. The mass factor can be obtained by seismic velocity measurements, loading tests or by approximating them from the RQD values obtained. The correlation between RQD and the mass factor *j* is shown in Table 2, after Hobbs[11].

The bored cast insitu piles constructed in Sri Lanka use bentonite fluid to stabilise the bored hole i.e. to prevent soil collapse from the side walls. Wyllie [12] points out that if bentonite is used as a slurry, the estimated side wall friction has to be reduced to 25% of that of a clean socket to obtain the mobilized shaft friction. However, Thilakasiri et al. [6], based on limited number of instrumented pile load tests, showed that the mobilized shaft resistance is large enough and

25% reduction may not be needed for the bored piles socketed to the bedrock in Sri Lanka. This study used the Williams and Pells[9] curve to obtain α and the corresponding β value is obtained from the RQD values from the corresponding borehole to estimate the shaft resistance of the rock mass. The shaft friction values obtained for the piles used for this study is presented in Table 3a Table 3b and Table 3c for the Project 1, Project 2 and Project 3 respectively. Since it is not pragmatic to conduct UCS (Uniaxial Compressive Strength) tests to all the samples, a reasonable judgment may be made for the strength of the intact rock mass based on the available data. PLI_{50} can be conducted at the site to obtain the equivalent estimate of the UCS values. In the case of borehole investigations, the UCS may not be carried out for the entire core run of the sample and in that case, based on the weathering condition and RQD (%), an estimate of the intact rock mass may be used. However, it is in the best interest of the engineer to use a conservative estimate and or adhere to the geotechnical recommendations. The UCS values presented below are averaged for the core runs considered and reasonable conservative estimates were made to account for the weak rock layers, where UCS test was not possible. In order to obtain the allowable shaft friction, the estimated shaft friction was divided by a factor of safety of 2, which is in accordance to the recommendations of BS Code [13] and standard practices in estimating shaft friction when considering empirical methods of estimation.

Table 2 - RQD and Mass factor *j* relationship (after Hobbs [11])

RQD (%)	Fracture frequency per m	Mass factor j
0-25	15	0.2
25-50	15-8	0.2
50-75	8-5	0.2-0.5
75-90	5-1	0.5-0.8
90-100	1	0.8-0.1

Table 3a - Estimated shaft friction for piles from empirical methods for Project 1.

Pile	TP-01	TP-02	P-105	P-70
α	0.12	0.11	0.12	0.12
β	0.7	0.73	0.65	0.65
UCS (Mpa)	12.6	10.21	12.86	12*
Mobilized ultimate shaft resistance (kPa)	264.6	205.0	250.8	234.0
Allowable shaft resistance (kPa)	132.3	102.5	125.4	117.0

Table 3b - Estimated shaft friction for piles from empirical methods for Project 2.

Pile	P-16	P-28	P-45	P-47	P-01
α	0.12	0.12	0.15	0.13	0.14
β	0.65	0.65	0.6	0.65	0.62
UCS (Mpa)	12*	12*	17.43	15.98	16.5
Mobilized ultimate shaft resistance (kPa)	234.0	234.0	392.2	337.6	358.1
Allowable shaft resistance (kPa)	117.0	117.0	196.1	168.8	179.0

Table 3c - Estimated shaft friction for piles from empirical methods for Project 3.

Pile	P4-21	P4-115
α	0.14	0.14
β	0.65	0.65
UCS (Mpa)	15	15
Mobilized ultimate shaft resistance (kPa))	341.3	341.3
Allowable shaft resistance (kPa)	170.6	170.6

3. Estimating the geotechnical capacity of Piles on fractured rock using PDA test results

High strain dynamic testing is carried out by applying a high energy impacts on the pile head to mobilize the soil resistance. The most widely used high strain dynamic testing are Pile Dynamic Incorporated (PDI) procedure, TNO Procedure, and the SIMBAT method, out of which the PDI procedure is the most common method that has been used in Sri Lanka [14]. The PDI procedure is carried out by applying several blows at different drop heights and the dynamic results are collected from the accelerometers and strain gauges that are fixed at the pile top. The acceleration and velocity data gathered from the transducers are displayed in real time and are stored in the PDA (Pile Driving Analyzer®) unit. Figures 4 and 5 illustrate a test setup for a high strain dynamic test procedure carried out for a large diameter pile and the real time PDA test results shown on the unit's screen. There are two main methods by which the velocity and force waves are analysed which are;

i. Case method – assuming the pile material to be linear elastic and soil properties to be dynamic and uses the wave propagation theory

ii. CAPWAP® method – an iterative numerical process that uses the field recorded velocity and force measurement to match and determine the unknown soil parameters, with a combined wave equation soil model and a continuous pile model.

Figure 4 -A field preparation for the PDI testing procedure.

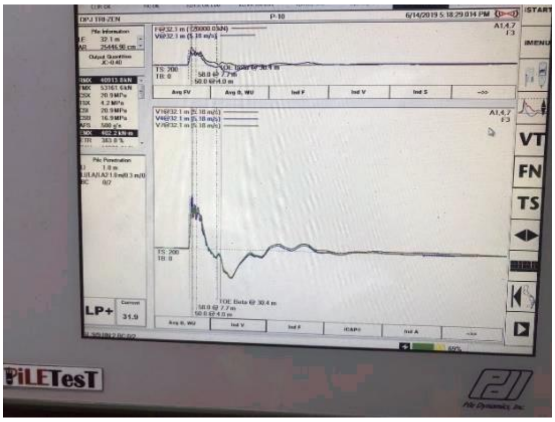

Figure 5 -Results of the high strain dynamic test shown in the PDA unit.

Detailed description on the theoretical aspects of the Case method and CAPWAP® analysis as well as the procedures of testing for the PDI test are described by Thilakasiri [14]. This study focuses on the results obtained from the CAPWAP® analysis since the shaft friction for soil segments can be obtained separately from CAPWAP® simulation and will be focusing on the rock segments of the pile to ascertain the mobilised shaft friction of the rock and will neglect the contribution of soil friction contribution in order to qualitatively compare the results yielded from the empirical estimation. Table 4a and 4b provides the shaft friction and toe resistance obtained from the CAPWAP® analysis respectively. It should be noted that for the shaft friction, the contribution from the rock mass is considered as an average and the length in the rock is based on the segmentation of the CAPWAP® analysis. The allowable resistance is taken as the mobilised capacity divided by 1.5 (The factor by which the working load is multiplied for the test load). The allowable resistance is measured as a stress to compare with the empirical methods used for the analysis. In this study, it is assumed that the CAPWAP® signal matching is carried out by an experience engineer considering the subsurface conditions.

Table 4a - Shaft resistance for the rock mass based on the CAPWAP® Analysis.

Project	Pile No.	Pile Dia. (m)	Pile length in rock taken from PDA (m)	Average shaft resistance of Rock		Allowable resistance
				(kN)	(kPa)	(kPa)
Project 1	TP-01	0.8	14.1	7,300	206.0	137.33
	TP-02	0.8	16.2	6,458	158.6	105.75
	P-70	1.8	12.3	19,697	283.2	188.79
	P-105	1.5	10.5	7,994	161.6	107.70
Project 2	P-16	1.5	8	5,930	157.3	104.86
	P-28	1.2	6.3	5,520	232.4	154.94
	P-45	1.2	8.3	4,725	151.0	100.67
	P-47	1	6.6	3,178	153.3	102.18
	P-1	1.2	6.5	4,215	172.0	114.67
Project 3	P4-21	1.8	8.1	10,326	225.4	150.30
	P4-115	1.8	8.5	9,069	188.7	125.79

Table 4b - Toe resistance for the rock mass based on the CAPWAP® Analysis.

Project	Pile No.	Pile Dia. (m)	Ultimate Toe resistance		Allowable Toe resistance
			(kN)	(kPa)	(kPa)
Project 1	TP-01	0.8	4,707.1	9,364.48	6,242.99
	TP-02	0.8	4,899.9	9,748.04	6,498.69
	P-70	1.8	21,143.0	8,308.67	5,539.12
	P-105	1.5	19,579.0	11,079.45	7,386.30
Project 2	P-16	1.5	19,209.0	10,870.07	7,246.71
	P-28	1.2	11,686.0	10,332.69	6,888.46
	P-45	1.2	7,471.0	6,605.81	4,403.88
	P-47	1	5,614.0	7,147.97	4,765.31
	P-1	1.2	8,599.0	7,603.19	5,068.79
Project 3	P4-21	1.8	15,208.0	5,976.37	3,984.24
	P4-115	1.8	18,259.0	7,175.33	4,783.56

4. Comparison of Estimated capacities from empirical methods and PDA testing

The toe resistance of the piles estimated from the RMR method and the CAPWAP® analysis are represented graphically in Figure 6 while the comparison of the estimated and measured shaft resistance values are shown in Figure 7. The estimated values from the RMR indicate a lower value than the analysis shown in the CAPWAP® analysis and this difference is not very significant as the coefficient of determination, R^2 is found to be 0.7552. One reason for this variation could be that the lower mobilization of the end bearing capacities at the applied energy during the PDA test. Since the data show the estimated RMR values are lower than the tested estimate, the RMR method to estimate the allowable bearing capacity can be considered as a reasonable method to be used to estimate the capacity while ensuring the safety of the pile.

The comparison of the shaft resistance using empirical methods and PDA testing is shown in Figure 7 while the allowable shaft resistance values from empirical and PDA testing are shown in Table 5b. In the case of estimating the shaft resistance, four piles indicated a marginal estimation greater than the average shaft resistance estimated using the CAPWAP® analysis. The data comparison shows a coefficient of determination, R^2 of 0.7883. This indicates that the estimated shaft resistance from the empirical method can be used to reasonably estimate the shaft resistance, however, measures have to be made to ensure that the shaft resistance is not overestimated. The design engineer should ensure that a conservative approach is made in the estimation of the average UCS for the entire core run.

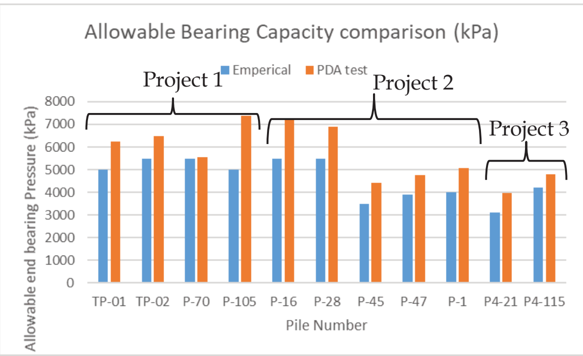

Figure 6: Comparison of empirical (RMR) method and PDA testing for end bearing capacity.

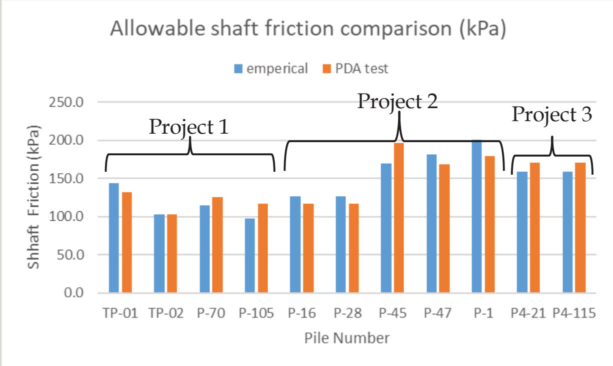

Figure 7 - Comparison of empirical method and PDA testing for the shaft resistance.

Table 5a - Comparison of end bearing capacity obtained by empirical estimation and PDA testing

Project	Pile	Allowable end bearing pressure (kPa)	
		Empirical	PDA
Project 1	TP-01	5000	6242.99
	TP-02	5500	6498.69
	P-70	5500	5539.12
	P-105	5000	7386.30
Project 2	P-16	5500	7246.71
	P-28	5500	6888.46
	P-45	3500	4403.88
	P-47	3900	4765.31
	P-1	4000	5068.79
Project 3	P4-21	3100	3984.24
	P4-115	4200	4783.56

Table 5b - Comparison of shaft friction capacity obtained by empirical estimation and PDA testing

Project	Pile	Allowable shaft friction (kPa)	
		Empirical	PDA
Project 1	TP-01	143.3	132.30
	TP-02	102.5	102.48
	P-70	114.9	125.39
	P-105	97.5	117.00
Project 2	P-16	126.75	117.00
	P-28	126.75	117.00
	P-45	169.94	196.09
	P-47	181.77	168.79
	P-1	201.09	179.03
Project 3	P4-21	158.44	170.63
	P4-115	158.44	170.63

5. Conclusions

This study was carried out using 10 number of piles that were socketed in fractured gneissic rock in Colombo near the Beira Lake. The piles selected for this study were considered from two residential development projects and an office development project. The piles those were subjected to high strain dynamic testing using PDA method were selected for this study. The piles were individually evaluated to ascertain the geotechnical capacity of the pile using empirical methods described in section 2.0 of this paper. The piles were individually assessed to ascertain the relationship between the empirical methods and the performance of the pile obtained from PDA testing. The contribution of the geotechnical capacity of the piles were identified separately as the shaft friction and toe resistance, from the results of the CAPWAP® simulations. The allowable toe resistance from the empirical formulae yielded results that were marginally lesser than that shown in the PDA test and the coefficient of determination was found to be 0.7752. This indicate that the empirical method i.e. the RMR method can be adopted for the calculation of the toe resistance and it can be concluded that the values provided for the end bearing capacity, using RMR method could provide an optimised empirical estimation of the toe performance of the pile. Considering the shaft friction, the empirical method presented in the text of Tomlinson and Woodward [10] yielded results that had a coefficient of determination of 0.7883, which would indicate a reasonable correlation. However, four piles indicated a higher value from the empirical formulae when compared to the results yielded from the PDA results. This is because the UCS values are not estimated to the sample of the complete core run and even within a core run of a specified length the intact rock particles may have varied strength index depending on factors such as infilling and weathering condition. Thus, it is recommended when designing to adhere to the geotechnical recommendation provided in the investigation report and to use this empirical formula to ascertain and justify the recommended values.
Eurocode 7 allows for the design of piles by utilising the pile capacities obtained from field test data i.e. static lead tests, ground investigation tests and dynamic tests [15]. 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 [10]. This study could be further developed to incorporate a method by which partial factors can be assigned for the empirical methods to utilise and develop these methods in the context of using EC 7 for the local content of designing pile founded on rock as a national annexure. The study can be further improved by incorporating static load tests and to configure the empirical methods to be used for the design.

Acknowledgements

The Authors of this study would like to acknowledge the Engineers of Civil and Structural Engineering Consultants (Pvt) Ltd., Eng. Bhathiya Waduge, Eng. Rihan Ranees, Eng. Jeshuran Mirron and Eng. Kalpana Senadheera for the assistance extended in providing the required information.

References

1. Peck, R. B., Hanson, W.E.& Thornburn, T.H. *Foundation Engineering*. 2nd Ed. John Wiley and Sons, New York. 1974.

2. Irfan, T.Y. & Powell, G.E. (1985). Engineering geological investigation for pile foundations on a deeply weathered grantic rock in Hong Kong. *Bulletin of the International Association of Engineering Geology.*

3. Bieniawski, Z.T. (1974). Geomechanics classification of rock masses and its application in tunnelling. *Proceeding of the Third International Congress of the International Society of Rock Mechanics, Denver, Vol. 2.*

4. Barton, N.R., Lien, R., & Lunde, J. (1974). Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics, Vol. 6.*

5. GEO PUBLICATION No.1/2006, *Foundation design and Construction.* (First Edition). Geotechnical Engineering Office, Civil Engineering and Development Department, The Government of the Special Administrative Region

6. Thilakasiri, H. S., Abeyasinghe, R. M., and Peiris, T. A (2015). Optimization of the designing of bored and cast in-situ piles in Sri Lanka, *Proc. of the Int. Conf. on Geotechnical Engineering*, Sri Lanka Geotechnical Society, Colombo.

7. Horvarth, R. G (1978). Field load test data on-concrete-to-rock bond strength for drilled pier foundations, University of Toronto, pp. 78-07.

8. Rosernberg, P., & Journeaux, N. L. (1976). Friction and end bearing of tests on bedrock for high capacity socket design, *Canadian Geotechnical Journal,* Vol. 13.

9. Williams, A. F., & Pells, P. J. N. (1981). Side resistance rock socket in sandstone, mudstone and shale, *Canadian Geotechnical Journal,* Vol. 18.
10. Tomlinson, M. & Woodward, J. (2006). *Pile Design and Construction Practice.* (Fifth Edition), Taylor & Francis Group, United Kingdom.

11. Hobbs, N. B. Review Paper – Rocks, *proceedings of the Conference on settlement of structures,* British Geotechnical Society, Penetech Press, 1975, pp. 579-610.

12. Wyllie, D. C. (1991), *Foundation on Rock,* 1st Ed., E & FN Spon, London.

13. BS 8004:1986 – British Standards- *Code of practice for foundation*.

14. Thilakasiri, H. S. (2009). *Construction & Testing of Piles*, 1st Ed., Sarasavi Publishers, Nugegoda, Sri Lanka.

15. BS EN 1997-1:2004 - *Eurocode 7: Geotechnical Design – Part 1: General rules*. BSI.