Optimization of the Designing of Bored and Cast in-situ Piles in Sri Lanka

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ABSTRACT: End bearing bored and cast in-situ piles are very often used in Sri Lanka to support large structures such as high-rise buildings. Due to the rapid growth in the construction industry in the recent time, high capacity piles are required to support large high-rise structures. However, designs of such piles are carried out using empirical methods which are not verified to the ground conditions and the construction methodology adopted in Sri Lanka. As a result, highly conservative designs are used in practice. In this paper, results of the few instrumented pile load tests done in Sri Lanka are presented and compared with the same obtained from the commonly used design approaches. Based on the comparison study, the shortcomings of the currently used design methods are identified and the tentative proposals are made regarding the design methods that should be adopted to suite the ground condition and the construction methodology adopted in Sri Lanka.

1 INTRODUCTION

The ground condition across major cities in Sri Lanka indicates that solid bedrock of granitic origin is present at relatively shallow depths. In the capital city Colombo, the bedrock is present on average at about 20m depth below the ground surface level. However, top region of the bedrock is in a highly fractured and weathered state in most places, and high spatial variability of the bedrock is a very common occurrence. It is often seen that the pile foundations are highly overdesigned causing significant additional cost to the client.

The designers tend to be on the conservative side in designing the rock socketed end bearing pile foundations due to the following reasons:

i. High special variability of the bedrock profile at most of the locations;
ii. Uncertainty regarding the quality of the constructed pile foundations by different piling contractors;
iii. Lack of data available regarding the skin friction mobilized in the socketed region of the piles and the mobilized end bearing; and
iv. Lack of design methods verified to the bedrock present and the construction methodology adopted.

Even though large number of static load test results are available, there is no reliable way of estimation of the carrying capacity of the socketed region of the pile from the available static load test results. The high strain dynamic load test results can be used to determine the mobilized skin friction along the pile shaft and the end bearing. Even though the total carrying capacity of the pile, load settlement behaviour of the piles, location and the magnitude of defects are accurately determined from high strain dynamic load test, it is an accepted fact that the skin friction distribution along the pile shaft and the end bearing given in the high strain dynamic load testing is very approximate.

In this research paper, measured mobilized skin friction and end bearing in piles using instrumented pile load tests are presented and the commonly used analytical methods are used to propose reliable methods to estimate the skin friction and end bearing in the bedrock.

2 COMMONLY USED METHODS TO ESTIMATE THE CARRYING CAPACITY IN THE SOCKETED REGION

Rock mass is a combination of solid rock, partially weathered rock, fracture surfaces weathered to different degrees, and the presence of weathered products in the fractures. Due to the complex nature of the rock mass, most of the design methods are empirical in nature and use some combination of parameters such as rock quality designate (RQD), core recovery (CR), and uniaxial compression strength (UCS). However, there are other more elaborate methods using estimated parameters such as rock mass rating (RMR) and Q-index, which take into account the weathered state of the fractures present in the rock mass and/or the infill material present in the fractures.

In this research, following commonly used methods will be used in the estimation of skin friction and end bearing capacity of the bedrock.

Methods used to estimate the skin friction in the socketed region:

M 1*a* - Limiting value given in ICTAD guidelines;
M 2*a* - Method given in Hong Kong guidelines; and
M 3*a* - Method proposed by William et al. (1980)

Methods used to estimate end bearing capacity of piles:

M 1*b* - Method outlined in ICTAD guidelines;
M 2*b* - Method given in the Hong Kong guidelines; and
M 3*b* - Method proposed by Kulhawy

Methods used to estimate elastic modulus of the bedrock at the pile toe:

M 1*c* - Method given in BS 8004;
M 2*c* - Method given in the Hong Kong guidelines; and
M 3*c* - Method proposed by Serafim and Pereira (1983).

3 DATA ANALYSIS

Results from three instrumented pile load tests are available from three sites in Colombo for the analysis.

3.1 Specimen data analysis for case 1

3.1.1 Quality of the bedrock in the vicinity of the tested pile

An instrumented pile load test carried out on a 1200mm diameter 13.30 m long bored and cast in-situ pile is used for the specimen calculations. The RQD and the CR of the nearby boreholes are given in Table 1 together with the RMR estimated according to the method proposed by Bieniawski (1989).

Table 1. Quality of the bedrock in the nearby boreholes

Depth (m)		CR (%)	RQD (%)	RMR
From	To			
BH 05 (closest borehole)				
-9.75	-10.50	86	27	43
-10.50	-11.50	100	55	57
-11.50	-12.50	100	30	52
BH 09				
-8.65	-9.65	100.0	100.0	76
-9.65	-10.65	100.0	100.0	76
BH 9_1				
-12.00	-13.50	75.0	50.0	43
-13.50	-15.00	100.0	67.0	57
-15.00	-16.00	100.0	100.0	76
BH 10				
-7.50	-8.50	100.0	100.0	79
-8.50	-9.50	90.0	85.0	59

3.1.2 Observed results from the test pile

The pile top load – settlement curve for the pile is given in Fig. 1 and the Pile bottom settlement Vs pile bottom mobilized end bearing curve is shown in Fig. 2. Similarly, skin friction mobilized vs avg. pile settlement in the pile socket is shown in Fig. 3.

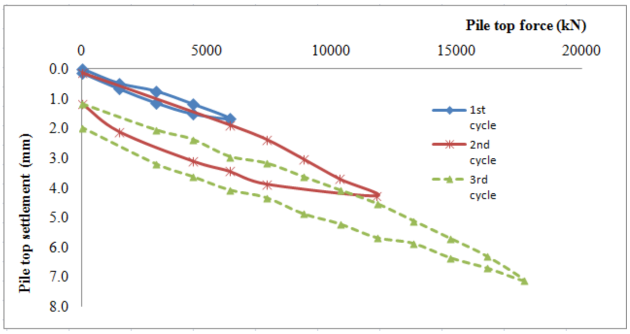

Fig. 1 Pile top settlement vs Pile top force

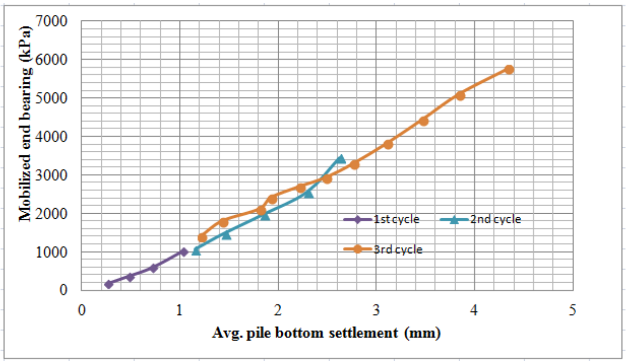

Fig. 2 Pile bottom settlement Vs pile bottom mobilized end bearing curve

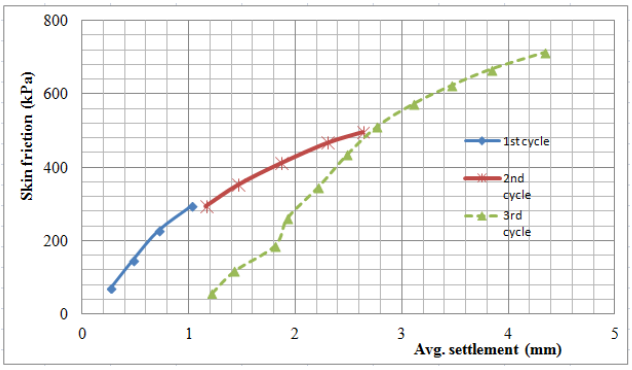

Fig. 3 Skin friction mobilized vs avg. pile settlement in the pile socket

3.1.3 Analysis of the skin friction in the socketed region in the bedrock

The mobilized skin friction in the socketed region of the pile is 712 kPa. If one uses the relationship given in the Hong Kong guidelines, the mobilized maximum skin friction is estimated to be 1260 kPa. If one uses the method specified by William et al. (1980), the limiting skin friction is estimated as 1400 kPa. As the observed skin friction has not reached the ultimate condition, there is a possibility the limiting value may reach the maximum mobilized skin friction given in the Hong Kong guidelines or the William et al (1980). However, the limiting value of 200 kPa specified in the ICTAD guidelines highly underestimates the mobilized skin friction in strong rocks present within the site.

3.1.4 Analysis of the allowable end bearing capacity

The method described in Hoek (1990) is used to estimate the shear strength parameters c' and ϕ' of the rock mass assuming a specified normal stress of 120 kPa at the top of the bedrock level. c' and ϕ' are estimated to be 130 kPa and 61^0 respectively. The ultimate bearing capacity of the bedrock is estimated using the estimated rock strength parameters and found to be 50.9 MPa. However, the applied stress at the bedrock level is about 6 MPa.

Corresponding to an average RMR of 55, the allowable carrying capacity of the pile for less than 0.05% pile diameter is 6.25 MPa. The settlement of the pile base is about 4.5mm for about 5.8MPa. As the 0.05% of the pile diameter is 6mm, the observed settlement satisfies the limit specified in the Hong Kong guidelines (HK guidelines).

3.1.5 Analysis of elastic modulus of the bedrock at the pile bottom

It is clearly seen from Fig. 2 that the pile bottom is behaving more or less in a linear manner. Based on the method proposed by Tomlinson (1994), pile bottom settlement, ρ_b, and pile bottom force, P_b, are related by the following relationship.

Based on the method proposed by Tomlinson (1994), pile bottom settlement, ρ_b, and pile bottom force, P_b, are related by Eq. [1].

$$\rho_b = \frac{\pi}{4}\frac{P_b}{A_b}\frac{B(1-\nu^2)I_P}{E_b} \qquad [1]$$

Where,

E_b- Elastic modulus of the material at the pile base

ν- Poisson ratio of the material at the pile base

B- Width of the pile

I_p- Settlement influence factor that depends on L/B and the Poisson ratio of the material at the pile base, generally taken as 0.5 for large L/B.

Based on the above relationship and the measured pile bottom response, the elastic modulus of the pile bottom material is determined to be 5.36 GPa. Based on the relationship between the elastic modulus and the RMR of the bedrock given in the HK guidelines, the estimated RMR of the rock is 89.9, which is slightly higher than the maximum possible value of 88 by Bieniawski (1989). By observation of the quality of the rock in the surrounding area, it is possible that the rock quality is very high in the vicinity of the test pile. However, if one considers the nearest borehole BH 05, the average RMR near the toe is about 55 and hence, the estimated elastic modulus according to the HK guidelines is 0.94 GPa, which is less than the observed value of 5.36. Therefore, according to these results, the method proposed in the Hong Kong guidelines to estimate the elastic modulus from the RMR of the bedrock underestimates the elastic modulus of the bedrock. Similarly, the method proposed in BS 8004 (1986) indicates that the estimated elastic modulus is about 1.2 GPa, which is less than the observed value. The modulus of elasticity based on the method proposed by Serafim and Pereira (1983) is 13.33 GPa and it overestimates the elastic modulus of the rock.

3.2 *Summary of the results of the case studies*

Two other case studies were considered and the summaries of the results of all three case studies are given below. Methods referred in this section M 1*a* to M 3*c* are given in section 2.0.

Table 2. Quality of the bedrock in the nearby boreholes of Cases 1, 2 & 3.

Case	CR	RQD	UCS (MPa)	RMR
1	100	20 - 50	39.8	52 - 57
2	30 – 50	< 10	< 20	< 25
3	30 - 50	< 10	< 20	< 25

(Extracted from NBRO soil investigation report table 5.1)

Table 3. Estimated ultimate SF and mobilized SF (kPa) in the socketed region in the bedrock

Case	M 1*a*	M 2*a*	M 3*a*	Observed
1	200	1260	1400	712*
2	200	890	910	710*
3	200	890	910	334*

Table 4. Estimated allowable end bearing and mobilized end bearing (kPa) in the socketed region in the bedrock

Case	M 1*b*	M 2*b*	M 3*b*	Observed
1	4000	6250	3490	5769*
2	-	≤ 3000	-	10800
3	-	≤ 3000	-	3025*

* NOT reached ultimate condition

Table 5. Estimated and observed elastic modulus (GPa) of the bedrock from different methods

Case	M 1*c*	M 2*c*	M 3*c*	Observed
Case 1	1.2	0.94	13.33	5.36
Case 2	0.36	0.16	1.78	0.17
Case 3	1.2	0.15	1.58	0.06

4 CONCLUSIONS

Results from only three case studies are not sufficient enough to make solid conclusions. However, some of the observations made from the three case studies are listed below. It is seen from the mobilized skin friction in the socketed region of the piles that the upper limit of 200 kPa is too conservative even for very weak rock conditions. The reduction of the mobilized skin friction due to the use of bentonite slurry is also not shown through the observations. Overall, the method adopted in the HK guidelines and the method proposed by William et al. (1980) to estimate the ultimate skin friction mobilized in the rock socket seems to be reasonable. However, use of a factor of safety of about 3 on the estimated ultimate skin friction may be sufficient to arrive at a reasonable allowable skin friction in the socketed region of the bored and cast in-situ piles. Further, the use of the method proposed in the HK guidelines seems to be reasonable to estimate the allowable end bearing capacity of the bedrock to have a settlement less than 0.5% of the pile diameter. The method proposed in the HK guidelines to estimate the elastic modulus from the estimated RMR value of the bedrock seems to be reasonable but for good quality rocks this method underestimates the elastic modulus of the bedrock.

REFERENCES

Bieniawski, Z.T. (1989). *Engineering Rock Mass Classification.* John Wiley, New York.

Code of Practice for Foundations, BS 8004, 1996, British Standard Institute.

HK Guidelines (2006) *Foundation design and construction, Geo Publication no. 1/2006*, Geotechnical Engineering Office, Civil Engineering and Development Department, The Government of the Hong Kong Special Administrative Region.

Hoek, E., (1990), Estimating Mohr-Coulomb Friction and Cohesion Values from the Hoek-Brown Failure Criterion (Technical note*), Int. J. Rock Mech.I, Min. Sci & Geomech*. Abstr. Vol. 27. No, 3. pp. 227-229.

Serafim, J. L., and Pereira, J. P., 1983, Consideration of the Geomechanics Classification of Bieniawski, *Proc. Int. Symp Engng Geol. and underground construction*, Lisbon, Protugal, pp 1133 – 44.

Tomlinson, M. J., 1994, Pile design and construction practices, 4th Edition, E & FN Spoon.

Williams, A. F., Johnstons, I. W., and Donald, I. B., 1980, The design of socketed piles in weak rock, *Int. Conf. structural foundations on rock*, Sydney Balkema.