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Allowable carrying capacity estimation of bored piles using low strain pile integrity test

Estimation de la capacité de charge des pieux forés à l'aide d'un test d'intégrité de pieu à faible contrainte

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ABSTRACT: The allowable carrying capacity of a single pile is one of the most important issues in designing of pile foundations. Static Load Testing is considered as the most reliable method to estimate the allowable carrying capacity of piles. However, static load tests are seldom done in Sri Lanka as it is expensive and time consuming. Therefore, more convenient testing methods such as High Strain Dynamic Testing (HSDT) rapidly became a popular alternative method to estimate the carrying capacity of a single pile. The low strain pile integrity test (LSPT) is currently the most widely used method of pile integrity testing to determine the major defects in piles. It has been suggested by several researches, that the dynamic stiffness at low frequencies obtained from the Transient Dynamic Response (TDR) Method of LSPT relates to the static stiffness of a pile. Following the actual field-testing results, Kodithuwakku et al. (2018) suggested a correlation between the static stiffness and the dynamic stiffness of rock-socketed, end-bearing, piles. In this research it is intended to estimate the allowable carrying capacity of a pile using developed equations that derived from the LSPT and HSDT results.

RÉSUMÉ: La capacité de charge d'un pieu unique est l'un des problèmes les plus importants dans la conception des fondations de pieux. Le test de charge statique est considéré comme la méthode la plus fiable pour estimer la capacité de charge admissible des pieux. Cependant, les tests de charge statique sont rarement effectués au Sri Lanka car cela coûte cher et prend du temps. Par conséquent, des méthodes de test plus pratiques telles que le test dynamique haute déformation (HSDT) sont rapidement devenues une méthode alternative populaire pour estimer la capacité de charge d'un seul pieu. Le test d'intégrité des pieux à faible déformation (LSPT) est actuellement la méthode la plus largement utilisée pour tester l'intégrité des pieux afin de déterminer les principaux défauts des pieux. Plusieurs recherches ont suggéré que la rigidité dynamique aux basses fréquences obtenue par la méthode de réponse dynamique transitoire (TDR) de LSPT est liée à la rigidité statique d'un pieu. À la suite des résultats des tests sur le terrain, Kodithuwakku et al. (2018) ont suggéré une corrélation entre la raideur statique et la raideur dynamique des pieux portant des emboîtements rocheux. Dans le cadre de cette recherche, il est prévu d'estimer la capacité de charge admissible d'un pieu à l'aide d'équations développées, dérivées des résultats du LSPT et du HSDT.

Keywords: Allowable carrying capacity, low strain pile integrity testing, transient dynamic response method, high strain dynamic testing, dynamic stiffness

1 INTRODUCTION

Sri Lanka is mainly made up of highly crystalline, non-fossiferous rocks of Precambrian age belonging to one of the most ancient and stable parts of the earth's crust in the Indian Shield (Cooray, P.G., 1984). Due to the presence of a bedrock at about 20m below ground in most parts of the island, rock socketed bored piles are extensively used in civil structures. The allowable carrying capacity of a single pile is regarded as the most important issue in the piling industry. This is significant because 5% of the tested piles in Sri Lanka are defective (Thilakasiri H.S., 2006). In proper termination, soft toe condition and defects in the shaft have the major impact on the carrying capacity of piles. Most superior method to determine the allowable carrying capacity of piles is the static load testing. However, number of static load tests is usually limited to one or two on a particular site as it is expensive and time consuming. Therefore, HSDT has rapidly become a popular tool among the Engineers to determine the carrying capacity of piles. However, use of HSDT is also limited at the site to determine the vertical capacity as it may increase the overall project cost and may decline the project progress. Thus, there is a serious need for an economical Non-Destructive Testing (NDT) method to determine the vertical carrying capacity of piles.

The LSPT is widely used to determine the major defects in piles. It uses a variety of techniques such as pulse echo method (time-domain analysis of pile top velocity) and transient dynamic response method (frequency-domain analysis of velocity and force) for interpretation of force and velocity records taken under the impact of a light weight hammer blow (Rausche, F., Ren - Kung, S. and Likins, G., 1991). In general, only the time-domain analysis of pile top velocity has been widely used in the interpretation because of its direct relationship to the physical condition of the pile and relative ease of interpretation (Liang, L. and Beim, J., 2008).

Reflections of the toe and the pile axis at locations where change of impedance occurs are measured in the pulse – echo method. The pile impedance can be expressed as Z = EA/c (E=elastic modulus of concrete, A=cross section area of pile, c=wave velocity of concrete). One – dimensional wave propagation applies to a linear elastic pile which is long compared to its diameter. A decrease in either area or modulus of elasticity produces a tensile reflection (an increasing produces a compressive reflection). Figure 1 indicates a pile with an increasing impedance at about 6.9m depth. And this pile shows a strong compressive reflection at toe as well.

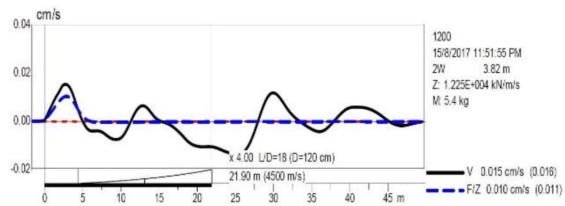

Figure 1. Time domain reflectogram of pile P9

The TDR method requires an instrumented hammer to measure the impact force and hence, can better quantify the response signal. The TDR method analyses both the force and the velocity signals in the frequency domain. The velocity spectrum divided by the force spectrum to determine the mobility spectrum (Pile Dynamic Inc., 2009.). Spectrum of velocity and force in frequency domain can be obtained by performing Fourier Transform on the measured velocity and force signals in the time domain. An idealised graph in the frequency domain is shown in Figure 2. Pile length information or distance to major damage may be obtained from frequency differences between major mobility peaks and/ or the patterns of peak locations. As visible in the Figure 2, the pile length and the depth to bulging location are visible in the peak patterns of the mobility plot. The dynamic stiffness is the initial slope of the mobility spectrum at low frequencies. The dynamic stiffness at low frequencies corresponds the initial tangent modulus of a load-displacement graph obtained from a static load test on the pile (Davis, A.G. and Dunn., 1974). This value is responsive to the stiffness of the pile shaft under compression.

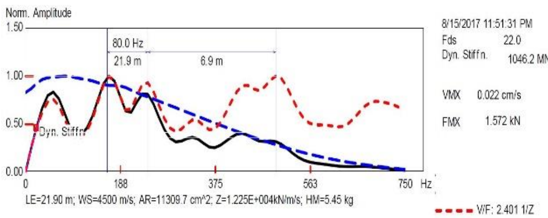

Figure 2. Mobility spectrum of pile P9

In the case of a smaller anomaly, the position of anomaly and length can be estimated from the frequency interval pattern as shown in Figures 1 and 2 for pile P9. A small bulging was observed in the pile P9 at the depth of 6.9m in the corresponding time domain reflectogram. The total length of pile can also be identified from both domains as 21.9m as observed in Figures 1 and 2.

Kodithuwakku et al. suggested a correlation between the static stiffness and the dynamic stiffness of a pile in 2018. The static stiffness determines the load – settlement behaviour of a pile in the initial linear region of the load – settlement curve. Authors recommend referring the technical paper "Case Studies – Use of Low Strain Transient Dynamic Response Method for Rock Socketed End Bearing Bored Piles" (Kodithuwakku et al., 2018) to aware about the limitations of static stiffness calculation from the dynamic stiffness.

Following the actual field load testing results, this paper suggests two settlement criteria, one for the linear region of load-displacement curve and another one for the non-linear region of the load-displacement curve. Derivation of the settlement criteria includes the pile elastic shortening and the pile toe displacement. The allowable carrying capacity is limited either due to exceeding geotechnical capacity, exceeding structural capacity or exceeding allowable settlement limits. Usually, most of the defect-free, socketed piles fail by exceeding structural capacity or exceeding allowable settlement limits. Generally, those piles have higher geotechnical capacities than allowable structural capacities. Usually maximum allowable structural capacity of a pile is limited to the one fourth of the concrete grade times the cross-sectional area of the pile.

This paper presents an approach to determine the allowable carrying capacity of a rock-socketed, end-bearing, bored pile using LSPT results. Maximum allowable carrying capacity is limited either by exceeding allowable settlement limits or by exceeding allowable structural capacity. And in this paper, it is not intended to determine the ultimate carrying capacity of a pile.

2 TEST METHOD

Systematic field tests were performed on bored piles at different parts in Sri Lanka. The piles were tested using both high-strain dynamic load testing and low-strain pile integrity testing. Most of the piles have been bored mostly through the soft peaty soils in low lands. The test piles range from 7m to 50m in length, 600mm to 1800mm in diameter and with 1D to 5D socket length. The Low-strain pile integrity testing was performed according to the ASTM D 5882 – 07 and the High-strain dynamic testing was performed according to the ASTM D 4945. The static load – settlement responses of piles were simulated using the signal- matching technique of the CAPWAP™ software.

The measured elastic shortening is calculated by subtracting the pile toe settlement from the pile top settlement under a particular load. The measured shortening and the interpreted shortening, PL/AE (P= load at pile top, L=Pile length, A=Cross section of pile, E=Elastic modulus of concrete) were determined using two sets of piles, namely one set of piles where piles are in the linear region of the load-displacement graph under the mobilized load, and another set of piles where piles are in the non-linear region of the load-displacement graph. Later measured elastic shortening and interpreted shortening plotted against each other in both regions to determine their relationships.

Thereafter a correlation is developed to determine the pile toe stiffness, K_{toe}. Later the pile static stiffness (K) is calculated using the dynamic stiffness (K_d) as proposed by Kodithuwakku et al. In 2018. This proposed correlation between static stiffness and dynamic stiffness fitted the data well with a linear graph that has a R^2 (coefficient of determination) of 0.80. as shown in Figure 3. The correlation is listed as Eq.5.

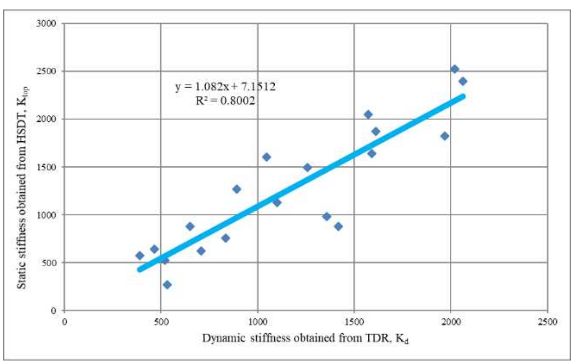

Figure 3. Static stiffness vs. dynamic stiffness of pile top

Finally, two equations are proposed for the two settlement criterions for linear and non-linear regions of the load-displacement curves. According to the ICTAD guide lines maximum allowable settlement under allowable working load is 12mm (ICTAD/DEV/16, 1997). If a pile can carry maximum structural capacity without exceeding the guidelines set by ICTAD or allowable settlement limit, the allowable carrying capacity is taken as the allowable structural capacity. If it exceeds the guidelines, corresponding load relevant to maximum allowable settlement limit is taken as the maximum allowable carrying capacity. For a conservative approach it is assumed that the maximum allowable carrying capacity occurs in the non-linear region of the load-displacement curve as a conservative approach.

3 PROPOSED SETTLEMENT CRITERIA FOR PILES

Pile vertical settlement under a load consists of two parts namely, pile shaft elastic shortening and pile toe movement. If there are no frictional resistance acts along the pile shaft, the elastic shortening of pile shaft is equal to the PL/AE. However, due to skin friction of surrounding mediums, the magnitude of elastic shortening is less than PL/AE. However even though some settlement criterion suggests the magnitude of elastic shortening is equal to PL/AE with the assumption that there is no skin friction acting along the shaft, in this study the elastic shortening is taken as αPL/AE, where α is an integer less than unity. Investigation studies of elastic shortening for the linear and non-linear regions of typical load-settlement curves are listed in the section 3.1 and 3.2 respectively. Typical load-settlement curve with two regions is shown in Figure 4. The pile toe settlement can be determined from the pile toe load and the pile toe stiffness. Refer section 2.3 for the calculations of pile toe load and pile toe stiffness. The proposed settlement criterion can be expressed mathematically as shown in Eq. 1.

$$\Delta = \alpha \frac{P_{top} L}{AE} + \frac{P_{toe}}{K_{toe}} \tag{1}$$

Where,
$\Delta = $ *Settlement under the pile top load*
$\alpha = $ *Constant*
$P_{top} = $ *pile top load*
$P_{toe} = $ *pile toe load*

$K_{toe} = \text{pile toe stiffness}$
$L = \text{Pile length}$
$A = \text{Cross section area of pile}$
$E = \text{Elastic modulus of concrete}$

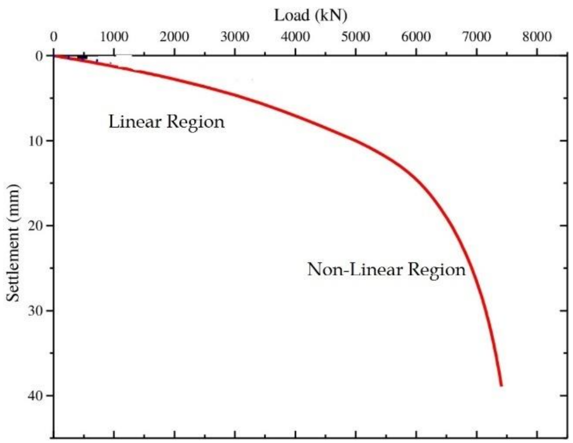

Figure 4. Load-Settlement Curve

3.1 Study of shaft shortening under the pile top load in the linear region of load-displacement curve

The shaft shortening is determined by subtracting the pile toe displacement from the pile top displacement. The shaft shortening and the the value of PL/AE, was plotted against each other as shown in Figure 5. A linear graph with a R^2 (coefficient of determination) of 0.98 fits the data well as seen in Figure 5. It is interesting to see that socket length, pile length, pile diameter and the location have a little influence on the fraction of elastic shortening of a pile, probably due to fixity provided at the bedrock level. According to the graph, actual elastic shorting is equal to the 0.62PL/AE.

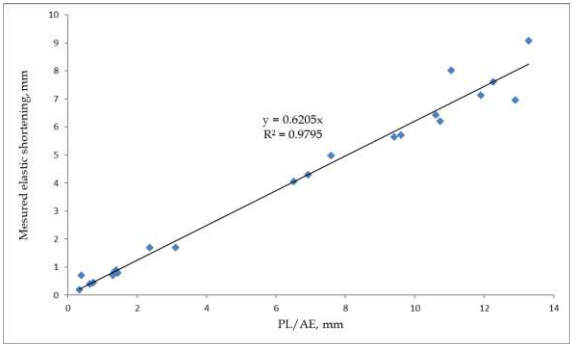

Figure 5. Measured shortening vs. interpreted shortening, PL/AE of linear region

3.2 Study of shaft shortening under the pile top Load in the non-linear region of load-displacement curve

According to the HSDT results, the maximum mobilized loads have always reached to the non-linear region of the load-displacement curves. But actual case is quite different. Under the small settlements pile could not usually reach to the non-linear region. Therefore, this is a limitation in the HSDT. Thus, it is convenient to use piles with larger settlements to determine the elastic shortening in the non-linear regions. However, some of the piles had clearly developed a non-linear region, a second region. Using data of those piles, a graph was plotted between measured shortening and value of PL/AE as shown in Figure 6. A linear graph with a R^2 (coefficient of determination) of 0.92 fits the data. According to the graph, actual shaft shortening occurs in this region is equal to the 0.64 PL/AE.

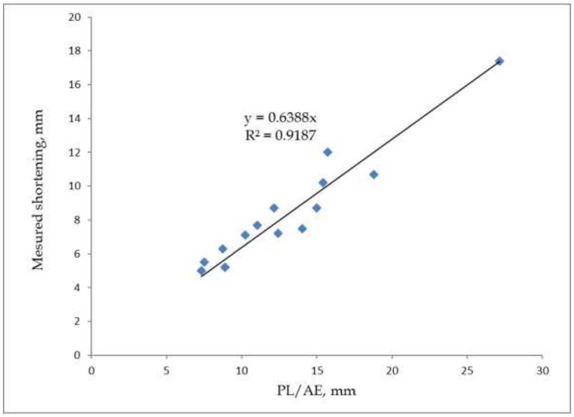

Figure 6. Measured shortening vs. interpreted shortening, PL/AE of non- linear region

3.3 Study of pile toe movement under a Load

Studies of toe movement in the linear region and non-linear region of load-displacement curve were used to formulate a pile toe movement criterion. It is observed that pile toe settlement under a load is very small. For defect-free, rock-socketed, end bearing, bored piles, toe movements have a little influence towards the total pile settlement. Generally, large portion of the total settlement is caused due the shaft shortening of piles.

The pile toe movement mainly depends on the pile toe load and the stiffness of the bedrock. Therefore, it is important to determine the pile toe load and the pile toe stiffness. As observed in the test results , in the linear region of the load-displacement graph, the average pile toe load is equal to the 0.47 times the pile top load. And for the non-linear region it is found as 0.56 times the pile top load. However, it is advisable that pile top and toe load relationship shall be made based on the actual field-testing results of a particular site. Authors suggested values are moderately conservative values for the derivations.

Therefore, two settlement criterions can be expressed using Eq. 2 and Eq. 3.

For linear region of load-displacement graph,

$$\Delta_{linear} = 0.62\frac{P_{top}L}{AE} + 0.47\frac{P_{top}}{K_{toe}} \tag{2}$$

For non-linear region of load-displacement graph,

$$\Delta_{non-linear} = 0.64\frac{P_{top}L}{AE} + 0.56\frac{P_{top}}{K_{toe}} \tag{3}$$

In order to determine the pile toe stiffness, a relationship can be made for the initial linear region of the load-displacement graph as shown in Eq. 4.

$$P_{top} = K \times \Delta_{linear} \tag{4}$$

$$K = static\ stiffness\ of\ the\ pile$$

Static stiffness can be calculated using Eq. 5 (Kodithuwakku et al., 2018).

$$K = 1.082 \times \mathrm{K_d} + 7.151 \tag{5}$$

$$\mathrm{K_d} = dynamic\ stiffness\ of\ pile$$

From the Eq. 2 and 4, following equation can be rewritten,

$$P_{top} = K \times \left[0.62\frac{P_{top}L}{AE} + 0.47\frac{P_{top}}{K_{toe}}\right] \tag{6}$$

Solving this equation yields a new equation for the K_{toe} as shown in equation 7.

$$K_{toe} = \frac{0.47}{\left[\frac{1}{K} - 0.62\frac{L}{AE}\right]} \tag{7}$$

In order to satisfy the above equation,

$$\left[\frac{1}{K} - 0.62\frac{L}{AE}\right] > 0 \quad \text{i.e,} \quad K < \frac{AE}{0.62L}$$

Therefore, the static stiffness calculated from Eq. 5 should be limited to the value of AE/0.62L. However, when the K value reaches to the AE/0.62L, Eq. 7 reaches to infinity.

4 ESTIMATION OF ALLOWABLE CARRYING CAPACITY

The equations derived in the previous chapters can be successfully used to determine the vertical allowable carrying capacity of a pile. Following steps shall be performed to determine the capacity.

Step 1: The pile head should be trimmed and brushed to prepare a smooth surface for the accelerometer installation and for the hammer hit. Then instrumented LSPT should be performed accurately. After the testing, data should be analysed and interpreted. The dynamic stiffness (K_d) should be determined through the TDR method (Frequency analysis of velocity and force).

Step 2: The static stiffness (K) of the pile should be determined using the dynamic stiffness (K_d) with the Eq. 5.

Step 3: The pile toe stiffness should be determined through the Eq. 7.

Step 4: Determine the maximum allowable structural capacity from Eq 8.

$$Structural\ capacity = 0.25 \times f_{cu} \times A \qquad (8)$$
$$f_{cu} = concrete\ strength$$
$$A = cross\ sectional\ area\ of\ the\ pile$$

Step 5: Apply maximum structural capacity in to the non-linear settlement equation (Eq. 3) and determine the corresponding settlement. Check whether this settlement exceeds the allowable settlement limit (12mm in ICTAD guildines) or not. If the settlement is under the allowable settlement limit, the maximum allowable capacity is equal to the maximum allowable structural capacity. If the settlement exceeds the allowable settlement limit, corresponding P_{top} value for the allowable settlement limit is taken as the maximum allowable carrying capacity.

In order to verify the accuracy of suggested methodology, the mobilized pile top loads from the HDST results are compared with the corresponding calculated pile top loads under the same mobilized pile top settlements as shown in Figure 7. The results are listed in Table 1. As observe in the graph, predicted pile loads and actual mobilized loads show a strong relationship with a R^2 (coefficient of determination) of 0.92. Therefore, it verifies that proposed settlement criterions deliver accurate results.

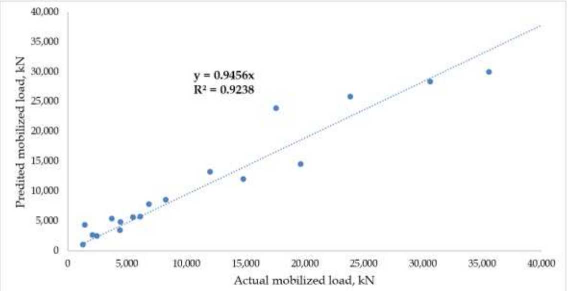

Figure 7. Predicted pile top load vs actual mobilized pile load

5 CONCLUSION

It was shown that the instrumented low strain pile integrity test can be successfully used to determine the allowable carrying capacity of a rock-socketed, end-bearing, bored pile. For the defective piles, it is advisible to use advanced method such as HSDT to determine the allowable carrying capacity. The reliability of the results depends both on accurate and clear data and user's experience with the TDR method. In some cases, it may become almost impossible to compute all the parameters as mentioned in this paper. Therefore, advanced methods are needed to determine the allowable carrying capacity of piles.

Table 1: Comparison of mobilized load with load calculated from proposed method under the same mobilized pile top settlement.

Pile No	$\delta_{mobilize}$ d/mm	$P_{mobilize}$ kN	$P_{calculated}$ kN
P1	8.6	3,741	-
P2	8.6	1,451	4,347
P3	2.3	1,267	1,035
P4	2.6	2,074	2,687
P5	14.7	6,122	5,705
P6	10.6	3,707	5,420
P7	5.5	8,262	8,527
P8	6.4	6,833	7,806
P9	14.1	19,658	14,544
P10	18.0	14,833	12,003
P11	2.7	5,525	5,676
P12	2.9	4,434	4,834
P13	3.8	4,395	3,433
P14	4.0	2,442	2,522
P15	19.0	35,588	29,974
P16	18.9	41,793	-
P17	13.6	30,629	28,350
P18	21.4	17,597	23,909

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