COMPRESSIVE LOAD CAPACITY OF BORED PILES BASED ON FIELD TESTS COMPARED TO THEORETICAL AND EMPIRICAL METHODS

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ABSTRACT

Static axial compressive load tests on pile foundations provide valuable information on the load-settlement behavior that can be utilized to determine the ultimate load capacities required for design purposes. This paper presents the field load test data of four individual 600 mm diameter bored piles from four different construction sites in Bangladesh. The ultimate compressive load capacities of the piles determined from the load tests were compared with the results calculated using different theoretical and empirical methods available in the BNBC 2020 and AASHTO LRFD Bridge Design Specifications 2017. Critical analysis of different methods was performed to reveal whether the theoretical and empirical calculations agree with the field test results. The findings reveal that the AASHTO 2017 method and static bearing equations of BNBC 2020 closely anticipate the ultimate capacities of the piles. On the other hand, the SPT-based calculations from BNBC 2020 yield approximately 1/3 of the field capacity for the same soil profile. The load-settlement behavior derived from AASHTO 2017 was also compared with the actual field load-settlement curves. It was observed that the lower range of normalized load transfer curves approximately predicts the field load-settlement behavior. The outcome of this study is expected to facilitate the professional personnel in enhancing the decision-making process for practical design purposes. Moreover, the analyses will assist future research works on further improvement in the available theoretical and empirical equations of determining the static axial compressive load capacity of bored piles.

Keywords: *bored pile; compression capacity; load test; load-settlement behavior*

1. INTRODUCTION

Piles are structural components that transfer loads from the superstructure to the deeper earth layers. When a shallow foundation is not possible to employ, the pile is preferred as a foundation element. Soft soil is very common in Bangladesh; thus, the pile foundation becomes the best possible solution. Even for the most experienced geotechnical engineer, precisely estimating pile capacity is a challenging task. There are many conventional techniques for determining pile capacity, but choosing one requires knowledge of the soil properties as well as the restrictions or applicability of each technique over a geographical boundary. Traditionally, pile capacity has been determined using a borelog from a subsoil report [1], and then confirmed by a static load test. Accordingly, driven pile and bored pile static load testing is very time-consuming, expensive, and requires ongoing process monitoring. It is generally difficult to guarantee the likelihood of accuracy and precision. Furthermore, the test has various flaws, such as frictional defects preventing the load from being transferred to the pile. Furthermore, a manual data gathering technique raises the possibility of human error. In these situations, foundation engineers needed a viable substitute for cross-checking or static load testing. Several studies have been done comparing different codes (Chinese code, Egyptian code, DIN 4014) with the AASHTO code [2, 3]. Different empirical correlations based on AASHTO 2002, AASHTO LRFD 2012, AASHTO LRFD 2017, IS 2911, and BNBC 2020 were used to conduct a comparative analysis for axial pile load carrying capability [4]. This was the motivation for the current study. This study compared the field results to theoretical and empirical methods. The study focuses only on the capacity of a single pile under compressive loading. Of course, seldom single piles are used; however, the capacity of group piles entirely depends on the capacity of a single pile within a group [1]. It should be noted that the pile group capacity is not the intention of this study. To conduct this study, excel sheets were prepared to calculate ultimate capacity using BNBC 2020 and AASHTO 2017. Later, the capacities from both methods were compared to field capacity. Finally, field load settlement curves were prepared using AASHTO 2017 and compared to field load-settlement curves.

2.METHODOLOGY

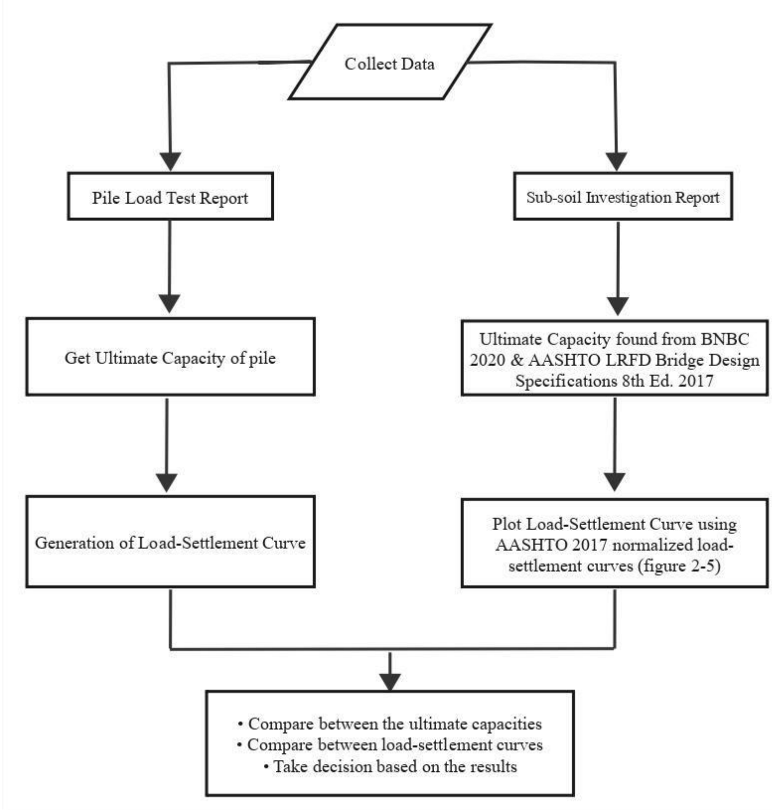


Figure 1:Methodology of the Study

First the ultimate capacity from the static load test is calculated according to ASTM D1143/D1143M-07 (2013). Later it is compared to BNBC 2020 (SPT-based and static bearing equations) and AASHTO 2017 ultimate capacity.
Ultimate pile capacity is denoted as, Q_{ult} (Fry et al., 2019)

$$Q_{ult} = Q_s + Q_b \quad (1)$$

Here, Q_s= skin friction, Q_b= end bearing.

$$Q_s = f_s A_s \quad (2)$$

Here, f_s= Skin frictional resistance on the unit surface area of the pile A_s= skin friction area.

$$Q_b = f_b A_b \quad (3)$$

Here, f_b= End bearing resistance on unit tip area of the pile, A_b= cross-sectional area of pile tip.
For a layered soil system containing n number of layers, end bearing resistance can be calculated considering soil properties of the layer at which the pile rests and the skin friction resistance considers all the penetrating layers calculated as,

$$Q_s = \sum \Delta z_i \times n\ _{i}{=}1\ (Perimeter)_i \times (f_s)_i \quad (4)$$

Here, Δz_i represents the thickness & $(f_s)_i$ represents the unit skin friction of any ith layer. $(\text{Perimeter})_i$ represents the perimeter of the pile in that layer.

Pile ultimate capacity according to BNBC 2020 (SPT based) (BNBC_2020-Vol-2.Pdf, 2020):
Cohesionless soil: The following relations may be used for the ultimate capacity of concrete bored pilesin cohesionless soil and non-plastic silt.

Skin friction for sand:

$$f_s = 1.0\ N_{60}\ (in\ kPa) \leq 60\ kPa \quad (5)$$

Skin friction for non-plastic silt:

$$f_s = 0.9\ N_{60}\ (in\ kPa) \leq 60\ kPa \quad (6)$$

End bearing for sand:

$$f_b = 15\ N_{60}\ (L/D)\ (in\ kPa) \leq 1500 N_{60}\ \&\ \leq 4000\ kPa \quad (7)$$

End bearing for non-plastic silt:

$$f_b = 10\ N_{60}\ (L/D)\ (in\ kPa) \leq 100 N_{60}\ \&\ \leq 4000\ kPa \quad (8)$$

Cohesive soil: The following relations may be used for ultimate capacity of concrete bored piles in clay soil and plastic silt.

For skin friction the relationship is,

$$f_s = 1.2\ N_{60}\ (in\ kPa) \leq 70\ kPa \quad (9)$$

For end bearing the relationship is,

$$f_b = 25\ N_{60}\ (in\ kPa) \leq 4000\ kPa \quad (10)$$

Where N_{60} is the average N-value over the pile shaft length and N_{60} is the N-value in the vicinity of the pile tip.

Pile ultimate capacity according to BNBC 2020 (Static Bearing Equations) (BNBC_2020-Vol-2.Pdf, 2020):

α Method: The α-method is based on total stress analysis (TSA). It is normally used to estimate the short-term load capacity of piles embedded in fine-grained soils. In this method, a coefficient α is used to relate the un-drained shear strength c_u or s_u to the adhesive stress (f_s) along the pile shaft. Hence,

$$Q_s = f_s A_s = \alpha c_u A_s \tag{11}$$

From BNBC 2020,
c_u= average of $12.5\ N_{60}$ and $10\ N_{60}$ (in kN/m^2)
$\alpha = 1.0$ for clays with $c_u \le 25\ kN/m^2$
$\alpha = 0.5$ for clays with $c_u \ge 70\ kN/m^2$
$\alpha = 1 - (c_u\text{-}25)/70)$ for clays with $25\ kN/m^2 < c_u < 70\ kN/m^2$
The end bearing in such a case is found by,

$$Q_b = f_b A_b = (c_u)_b N_c A_b \tag{12}$$

N_c is a bearing capacity factor and for deep foundation, the value is usually 9. c_u is the undrained shear strength of soil at the base of the pile. The general equation for N_c is, however, as follows,

$$N_c = 6 + [1 + 0.2\ (L\ /D_b)] \le 9 \tag{13}$$

D_b represents the diameter of the pile at the base and L is the total length of the pile. The skin friction value, f_b should not exceed 4.0 MPa.
β Method: Piles in cohesionless soil shall be designed by effective stress methods of analysis for drained conditions. For piles in cohesionless soil, the ultimate side resistance may be estimated using the following formula:

$$Q_s = (2/3) f_s A_s = \beta\ \sigma'_z\ A_s \tag{14}$$

Where σ'_z is the effective vertical stress under consideration. The values of β are as follows:
$\beta=0.10$ for $\varphi= 33^o$
$\beta=0.20$ for $\varphi= 35^o$
$\beta=0.35$ for $\varphi= 37^o$
The following equation, as used for cohesive soil, may be used to compute the ultimate end-bearing capacity of piles in sandy soil in which, the maximum effective stress, σ'_z allowed for the computation is 240 kPa.

$$Q_b = (1/3)\ f_b A_b = (\sigma'_z)_b \times (N_q) \tag{15}$$

$N_q= 8$ to 12 for loose sand, loose sand $\le N=10$
$N_q = 12$ to 40 for medium sand, medium sand $10 \le N \le 30$
$N_q = 40$ for dense sand, dense sand $N \ge 30$
Critical Depth: The vertical effective stress (σ'_z) increases with depth. Hence the skin friction should increase with depth indefinitely. In reality, skin friction does not increase indefinitely. It is believed that skin friction would become a constant at a certain depth. This depth is named critical depth. Following approximations may be used for the critical depth in relation to the diameter of the pile, D
$D_c= 10D$ for loose sand
$D_c=15D$ for medium sand
$D_c= 20D$ for dense sand.

Pile Ultimate Capacity According to AASHTO LRFD Bridge Design Specifications 8th Ed. 2017 (AASHTO LRFD, 2017):
α Method: The α method is based on total stress. The adhesion factor is an empirical factor used to correlate the results of full-scale load tests with the material property or characteristics of the cohesive

soil. It is related to S_u and it is derived from the results of full-scale pile and drilled shaft load tests.

$$q_s = \alpha\, S_u \tag{16}$$

Here,
$\alpha = 0.55$ for $S_u/P_a \leq 1.5$
$\alpha = 0.55 - 0.1(S_u/P_a - 1.5)$ for $1.5 \leq S_u/P_a \leq 2.5$
Here,
S_u= Undrained shear strength = average of $12.5\ N_{60}$, $10\ N_{60}$ and $0.06\ N_{60}$ Pa (in kN/m^2)
P_a= Atmospheric pressure
α= Adhesion factor
Skin fiction is not considered for the top 5 ft of the shaft and periphery of belled shaped end (AASHTO LRFD, 2017). For axially loaded shafts in cohesive soil, the nominal tip resistance, q_p, by the total stress method as provided in Brown et al. (2010) shall be taken as:

$$q_p = N_c\, S_u \leq 80\ ksf \tag{17}$$

$$N_c = 6\,[1 + 0.2\,(Z/D)] \leq 9$$

where, D= diameter of the shaft, Z= depth of the drilled shaft, S_u= undrained shear strength. The value of Su should be determined from the results of in-situ and/or laboratory testing of undisturbed samples obtained within a depth of 2.0 diameters below the tip of the shaft. If the soil within 2.00 diameters of the tip has $S_u < 0.50$ ksf, the value of N_c should be multiplied by 0.67.
β Method: The side resistance for cohesionless soil can be calculated using the β method.

$$q_s = \beta\, \sigma'_v \tag{18}$$

in which ,

$$\beta = (1 - \sin\varphi'_f)\,(\sigma'_v/\sigma'_p)^{\sin\varphi'_f} \tan\varphi'_f \tag{19}$$

here,
β= load transfer co-efficient
φ'_f = friction angle of cohesionless soil layer
σ'_v = vertical effective stress at soil layer mid-depth
σ'_p = effective vertical pre-consolidation stress
the effective friction angle can be found from the following relation:

$$\varphi'_f = 27.5 + 9.2\log[(N_1)_{60}] \tag{20}$$

$(N_1)_{60}$= SPT-N value corrected for effective overburden stressPre-consolidation stress can be calculated as follows.
For sands, $\sigma'_p/p_a = 0.47\,(N_{60})^m$ (21)

here,
m=0.6 for clean quarzitic sand m=0.8 for silty sand to sandy siltsp_a= atmospheric pressure
for gravelly soils, $\sigma'_p/p_a = 0.15(N_{60})$ (22)

the nominal tip resistance, q_p, for drilled shafts in cohesionless soil can be calculated by the method described by Brown et al. (2010):
If $N_{60} \leq 50$

$$q_p = 1.2\, N_{60} \tag{23}$$

N_{60} = average SPT blow count (only corrected for hammer efficiency)
the value of q_p from equation (23) should be limited to 60 ksf unless greater values are justified. where a shaft is tipped in a strong soil layer overlying a weaker layer, the base resistance shall bereduced if the shaft base is within 1.5D of the top of the weaker layer. A weighted average should beused that

varies linearly from the full base resistance in the overlying strong layer at a distance of1.5D above the top of the weaker layer to the base resistance in the overlying strong layer at the top of the weaker layer.

After calculating the pile load capacity for AASHTO 2017 the ultimate load was calculated using the normalized load transfer graphs for side resistance transfer and end bearing transfer. As soil is not homogeneous in real fields a weighted average was used to calculate the side resistance transfer. The used graphs are provided below:

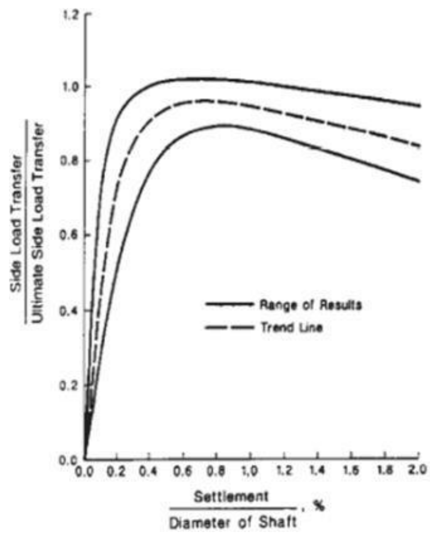


Figure 2:Normalized load transfer in side resistance vs settlement in cohesive soil

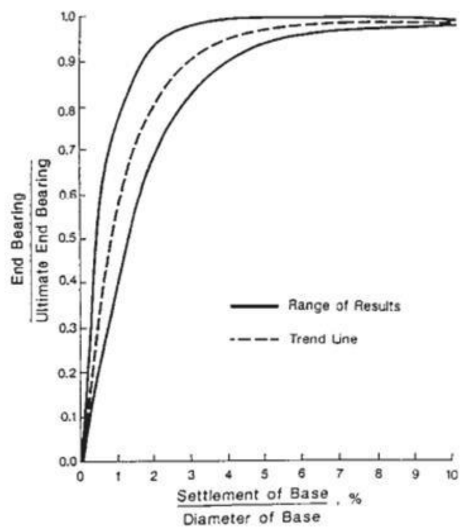


Figure 3:Normalized load transfer in end bearing vs settlement in cohesive soil

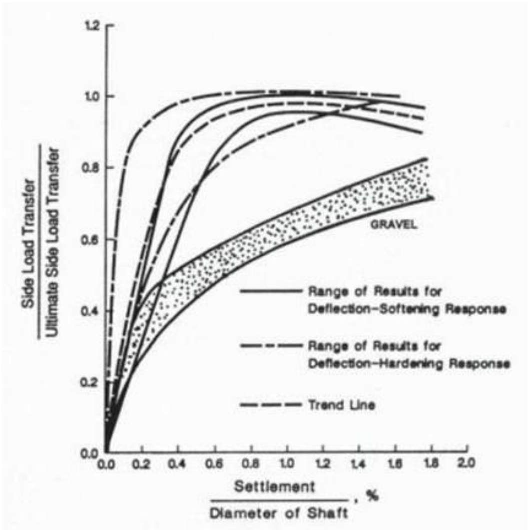


Figure 4: Normalized load transfer in side resistance vs settlement in cohesionless soil

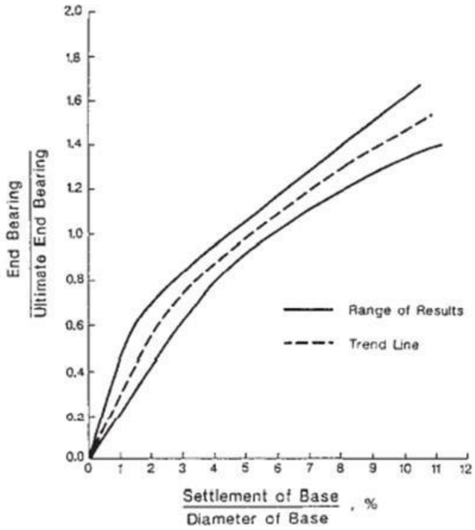
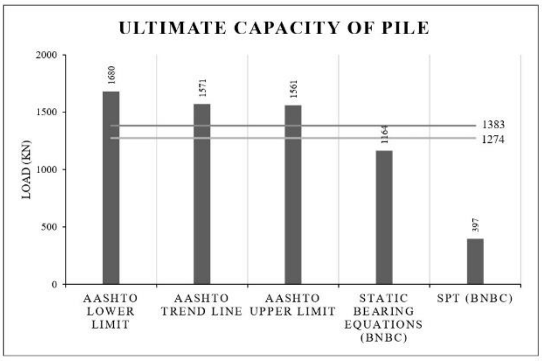


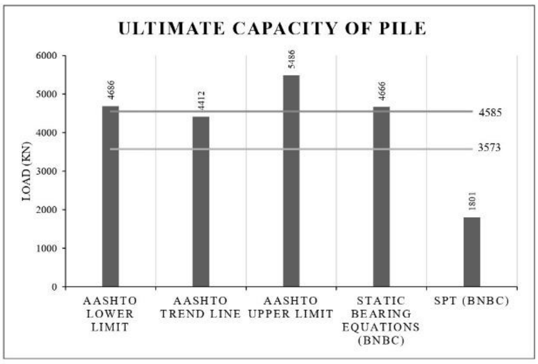
Figure 5: Normalized load transfer in end bearing vs settlement in cohesionless soil

3. RESULT AND DISCUSSION

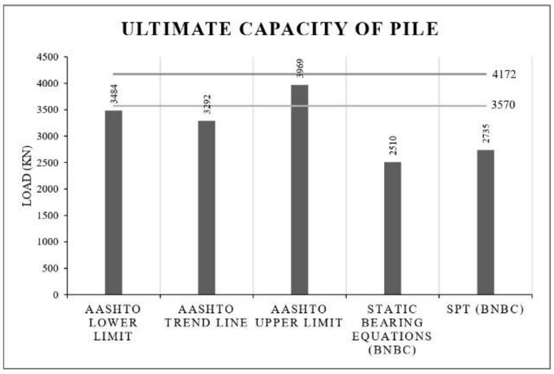
First, the graphs that show the comparison between the ultimate capacities from field tests and BNBC 2020 and AASHTO 2017 are shown. The horizontal lines show the range of results from the static field load test. The vertical columns show the ultimate capacities found in BNBC 2020 and AASHTO 2017.



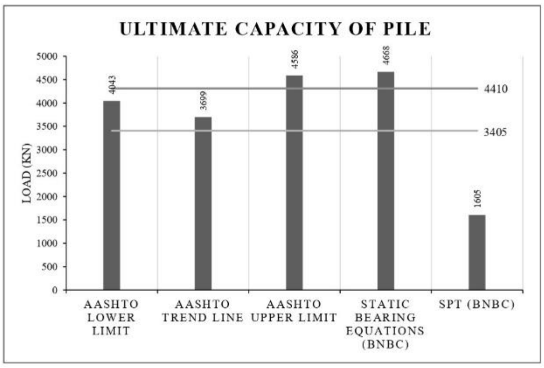
(a)



(b)



(c)



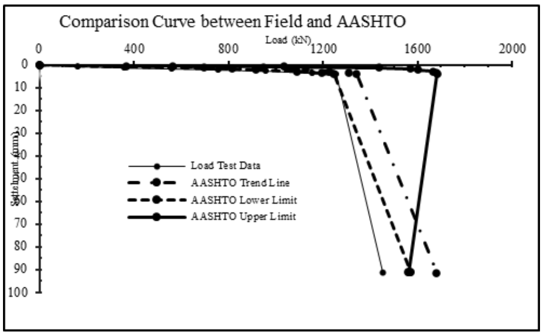
(d)

Figure 6: Comparison of ultimate capacity of piles from BNBC 2020 with AASHTO 2017 ultimate capacity of different limits for (a) cast in-situ pile-1, (b) cast in-situ pile-2, (c) cast in-situ

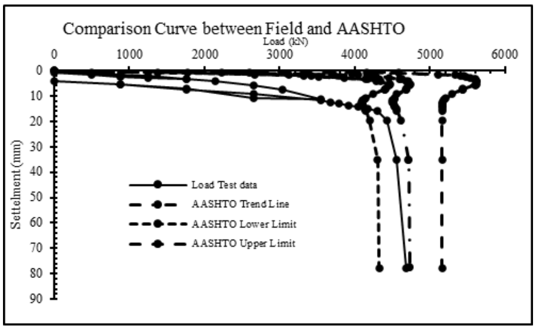
For piles 1,2 and 4, the ultimate capacity calculated from static bearing equations from BNBC 2020 and AASHTO 2017 is almost equal to the field ultimate capacity range. It is also seen that the ultimate capacity found from SPT-N based equations is almost one-third of the maximum capacity found from the static bearing equations. In the case of the 3rd cast in-situ pile, AASHTO 2017 predicts the ultimate capacity well. But BNBC 2020, in this case, predicts less than the actual value. It is also seen that the ultimate capacity from static bearing equations and SPT-based relations give almost the same value. It may happen for different soil conditions than the other three piles.

The critical depth (a certain depth after which skin friction does not increase) was considered in BNBC 2020, which led to ultimate capacity very close to the field test. No such consideration was made in AASHTO 2017. This may lead to a capacity higher than the original.

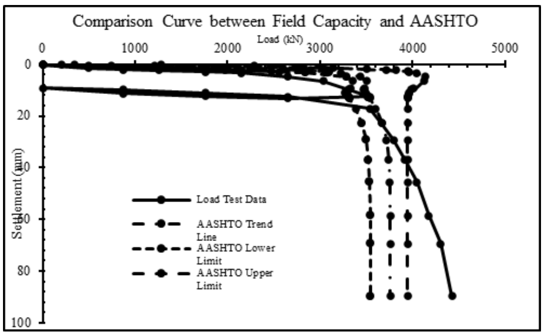
Field load-settlement graphs were generated, and it was compared to the AASHTO 2017 load-settlement graphs (using normalized load-settlement curves). Comparison graphs are provided below:



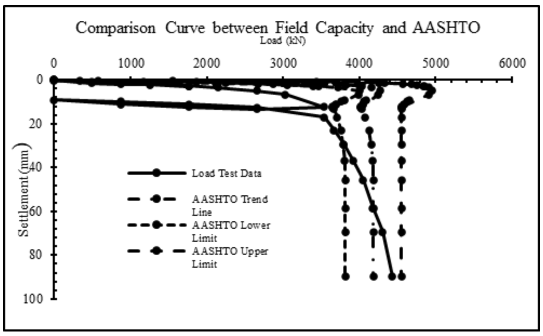
(a)



(b)



(c)



(d)

Figure 7:Comparison of the field load-settlement curve with AASHTO load settlement curves of different limits for (a) cast in-situ pile-1, (b) cast in-situ pile-2, (c) cast in-situ pile-3,(d) cast in-situ

pile-4

It showed that the lower limit of AASHTO 2017 almost merged with the field load settlement curves. Which means the AASHTO 2017 load-settlement curve for lower limit predicts the field capacity well.

4. CONCLUSIONS

The study aimed to use several methods to calculate the ultimate capacity piles from BNBC 2020 and AASHTO LRFD Bridge Design Specifications 8th Ed. 2017. The final capacity determined by these two methods were then compared to the filed capacity determined by static load tests. Finally, load-settlement curves were prepared from field load and load-settlement curves for AASHTO 2017(lower limit, trend line, and upper limit were also prepared). These load-settlement curves show how the pile behaves in that particular soil and which limit state best anticipates the pile's behavior. Pile capacity was calculated by BNBC 2020 (static bearing equations) and AASHTO LRFD Bridge Design Specifications 8th Ed. 2017. The ultimate capacity from static load test is almost equal to the ultimate capacity computed using BNBC 2020 (static bearing equations) and AASHTO 2017. The ultimate capacity using SPT-based equations (BNBC 2020) is almost one-third of the ultimate capacity from the static bearing equations (BNBC 2020). Comparing the load-settlement curves from the field and AASHTO 2017 shows that the AASHTO lowerlimit predicts the field capacity well. The normalized load-settlement curves are only available for homogeneous soil. But in real life soil is not homogeneous. Also, there are no recommendations for how to use the normalized load-settlement curves in a non-homogeneous soil profile. So, a weighted average was used to calculate the skin frictionfor different soil types. Which may lead to some error.

REFERENCES

AASHTO LRFD. (2017). Bridge Design Specifications (SI). In *8th Edition* (Issue September).

Awad, A., & Mahmoud, F. (2013). Comparative Study between the Methods Used for Estimating Ultimate Lateral Load of Piles in Sandy Soil. *5th IYGEC*.

Aziz, H. Y., Sultan, H. K., Abbas, B. J., & Tarik, R. (2021). *Comparison of pier bridge design between AASHTO and Chinese codes Comparison of pier bridge design between AASHTO and Chinese codes*. https://doi.org/10.1088/1757-899X/1090/1/012020

BNBC_2020-Vol-2.pdf. (2020).

Bowles, J. E. (1977). *FOUNDATION ANALYSIS AND DESIGN Fifth Edition*.

Fry, M. J., Cochran, J. J., Ohlmann, J. W., & Anderson, D. R. (2019). *Copyright 2019 Cengage Learning. All Rights Reserved. May not be copied, scanned, or duplicated, in whole or in part. WCN 02-200-203*

Islam, S., Hossain, S., & Fathiazar, A. (2021). *Comparative Evaluation of Load Bearing Capacity of Bored Piles Using Numerical Approach & Empirical Co-Relations. May*, 0–8.