Application of Statistical Method on Field Pile Loading Test Result Analysis

Dohyun Kim
Assistant Professor, Dept. of Civil and Environmental Engineering, Hanbat National University, Daejeon, Republic of Korea

ABSTRACT: In this study, analysis on a real-scale field pile loading test results on a 20 fully instrumented steel pipe test piles was carried out, to propose a modified empirical formula for estimating the bearing capacity of the steel pipe prebored and precast pile. The formula is proposed based on a statistical approach of the data points from the field pile loading test, in order to ensure safe engineering practice while finding a reliable formula. The statistical analysis of the data points indicated that the existing formula has been underestimated the bearing capacity of the steel pipe prebored and precast pile. The proposed formula estimates 20% higher pile tip bearing capacity and the shaft resistance compared to the existing formula.

1. INTRODUCTION

Among various piling methods, the study on prebored and precast pile (here and after PPP) has not been thoroughly established. Due to this, the estimation of the bearing capacity of PPP relies only on empirical methods based on the limited consideration of the soil. Furthermore, the empirical formula itself is not entirely based on real-scale field test of the actual prebored and precast pile due to insufficient field test data, but rather derived from other test results which was carried out for bored piles or drilled shafts (cast-in-place piles). For this reason, the conventional empirical formula for PPP carries numerous assumptions, reported to be conservative, and cannot provide a safe and effective solution in actual engineering practice (Park, 2004).

Compared to conventional piling methods, PPP is effective in reducing public nuisance during pile installation, such as noise and vibration. Since PPP uses manufactured piles and placed in the borehole rather than driven in, the structural integrity of the pile can be secured. For this reason, PPP is widely used in construction projects in major metropolitan areas in East Asian countries. In addition, PPP is more cost-effective compared to drilled shafts (cast-in-place pile) in the range of small to medium diameter piles which are frequently used in urban construction projects and provides higher lateral resistance (Ng et al. 2001; Kim et al. 2018; Jeong and Kim. 2018).

PPP has a unique pile tip condition and shaft behavior characteristics. However, compared to conventional driven piles and drilled shafts, studies on clarifying the bearing mechanism of PPP, let alone sufficient real-scale field tests including pile loading tests on PPP have not been properly conducted. Studies related to PPP have been limited to empirical methods derived from insufficient field data, which lead to formulas roughly predicting the ultimate bearing capacity based on the limited SPT-N value of the ground. Moreover, existing studies are based on an application of the studies done for conventional piling methods and not based on studies or tests done exclusively for PPP (Kim et al. 2020).

In this study, a new and accurate formula for estimating tip bearing capacity and shaft resistance of steel pipe PPP will be proposed. The tip bearing capacity and the shaft resistance behavior will be thoroughly observed by analyzing 20 cases of pile tip data, and over 200 data points along the shaft. The data points are obtained from 20 cases of real-scale pile loading tests carried out on fully instrumented test pile installed in granite-gneiss weathered rocks, which occupies two-thirds of the Korean peninsula, and found in the broad northeastern region of North America (Canadian Shield) (Kim et al. 2020). Moreover, to consider the actual capacity of the ground, expanded SPT N-value will also be presented and applied in deriving the proposed formula.

2. STEEL PIPE PREBORED AND PRECAST PILE

PPP is installed based on a unique and complex installation process. As shown in Figure 1, PPP is installed by boring a hole in the ground while the soil is being removed. The precast PHC or steel pipe pile is placed in the borehole right after the 1st cement paste injection. The cement paste injection is filled up to a certain height to improve bearing capacity. The installation is finished by casting additional cement paste around the placed pile up to the top of the borehole (2nd cement paste injection). Light hammering on the head (two to three blows) of the PPP is conducted to secure the integrity of the pile tip bearing condition. Due to this installation process, the behavior and the characteristics of the PPP is distinguished from the conventional driven piles and drilled shafts − such as plugging effect on the pile tip and the infiltration of the cement paste to surrounding grounds.

2.1. Tip bearing capacity

PPPs are usually socketed in weathered rocks or harder rocks, to secure a stable pile tip condition. However, very few field tests have been carried out to investigate the failure behavior of pile tip

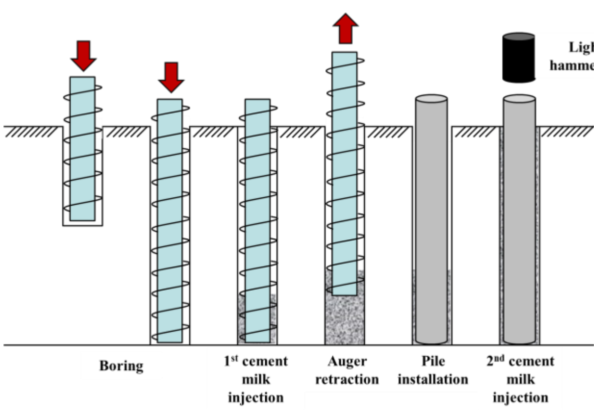

Figure 1: Prebored and precast pile installation process.

bearing in rocks. In study conducted by Johnston and Choi (1985) investigated the process of failure of a model pile socketed in simulated rock. The progressive failure modes suggested by Johnston and Choi is shown in Figure 2. They suggested that failure occurs by initial radial cracking and progresses to a fan shaped wedge. This failure process can be corresponding to a certain point in the load-settlement curve of the pile. The four points are identified as: (A) at the end of the elastic deformation; (B) right before major yielding; (C) right after major yielding; and (D) failure state, in the figure.

Different from a conventional steel pipe pile, the tip of the steel pipe PPP is commonly clogged by the 1st injected cement paste. Moreover, the cement paste injected along the shaft forms a tube-like layer bonded to the steel pipe pile, which has an effect of increasing the pile tip area. Commonly, the ultimate unit tip bearing capacity of the PPP installed in sandy soil is estimated by using the following Eq. (1) (Korean Geotechnical Society, 2009):

$$q_t = 200N_t \text{ (kN/m}^2\text{)} \quad (1)$$

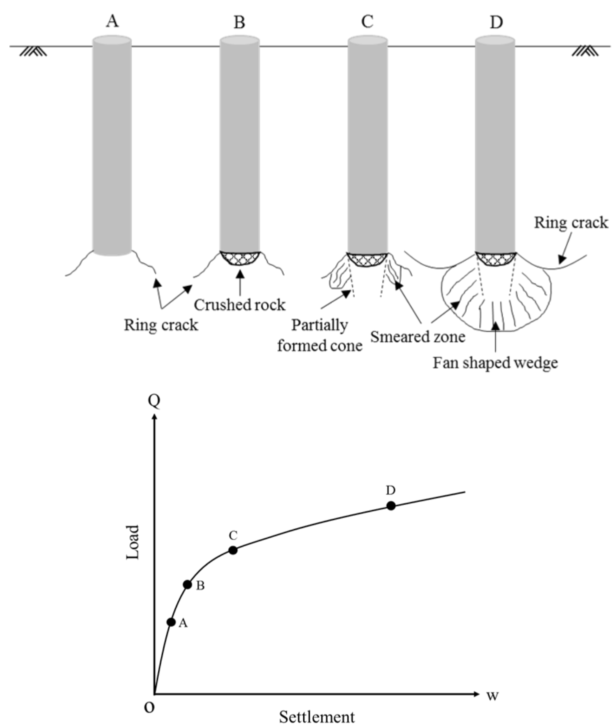

Figure 2: Four points in corresponding to the progressive failure modes (Johnston and Choi, 1985).

where, N_t is the SPT N-value at pile tip of the pile obtained from the standard penetration test ($N_t \leq 60$). Based on the Japanese foundation design standards, the average SPT N-value near the pile tip is applied in estimating the tip resistance (Architectural Institute of Japan, 2004). The formula is as follows in Eq. (2):

$$q_t = 200\overline{N}\ (\mathrm{kN/m^2}) \tag{2}$$

where, $\overline{N}$ is the average SPT N-value from 1D (diameter of pile) above the pile tip to 4D below the pile tip ($N_t \leq 60$). This formula using the average SPT N-value near the pile tip is generally assumed to be more accurate than the formulas using the SPT N-value of the pile tip. The maximum ultimate unit pile tip resistance is limited to 12,000kN/m^2.

2.2. Shaft resistance capacity

Compared to conventional piling methods, the shaft resistance makes up for a larger portion in the total bearing capacity in PPP. The behavior of PPP along the shaft is like drilled shafts, where relative displacement (elastic slip) between the pile-soil interface is observed due to loading, which mobilizes shear stress along the shaft (Broms 1979; O'Neill et al. 1996). This is reported by numerous studies that the ultimate shaft resistance for bored piles is mobilized after relative displacements of the shaft with respect to the surrounding soil, as well as the tip resistance (Reese and O'Neill 1988). Shear load transfer (t-z) characteristics along the shaft of the drilled shaft socketed in rocks were investigated, and modified load-transfer curves were proposed, which increased the accuracy of the numerical predictions (Seol et al. 2009; Kim et al. 2020). Johnston and Lam (1989) made detailed investigations, as shown in Figure 3, of the pile-rock interface to clarify the shear load transfer behavior for drilled shafts. Despite the existence of the precast pile within the cement layer, the interface between the cement (in case of drilled shaft, the pile) and the rock can be considered in having an identical condition. For the cement (pile)-rock interface, shearing results in dilation as one roughness overrides another. If the surrounding rock mass is not deformed sufficiently, an inevitable increase in normal stress occurs during shearing. This so called constant normal stiffness, which can be determined by a theoretical approach based on an expanding infinite cylindrical cavity in an elastic half-space. In case of PPP, constant normal stiffness criteria can be also caused by the circulation loss (infiltration) of the injected cement paste, which is common during installation. The constant normal stiffness boundary condition produces an increase in stress normal to the interface and a corresponding increase in the frictional resistance between the cement (pile) and the rock. Even though the PPP shows the similar shaft behavior due to circulation loss of injected cement paste, the study on the matter is not yet properly carried out. For this reason, the constant normal stiffness boundary condition can provide an indirect view on determining the shaft resistance behavior of the PPP and should be considered in estimating the shaft resistance.

In estimating or predicting the shaft resistance of PPP, empirical correlations between the ultimate unit shaft resistance of PPP and the SPT N-value are widely used. The empirical formula based on the Korean foundation design standard (2009) is shown in Eq. (3):

$$f_{max} = 2.5N_s\ (\mathrm{kN/m^2}) \tag{3}$$

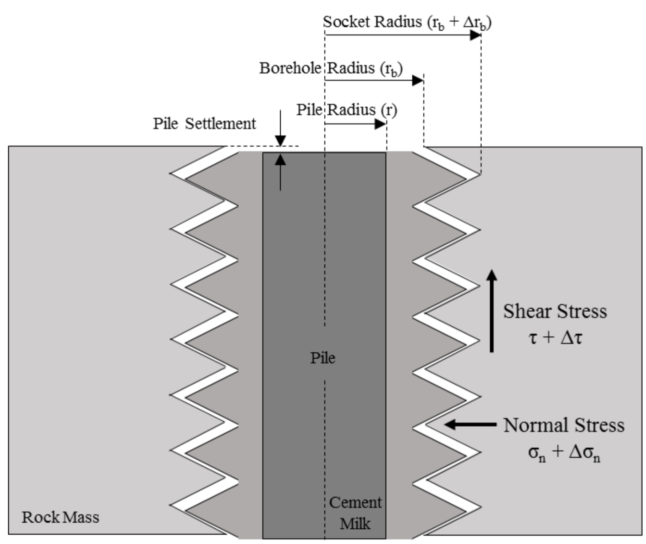

Figure 3: Schematic of idealize rock-socketed pile (Johnston and Lam, 1989)

where, N_s is the SPT N-value along the shaft of the pile obtained from the standard penetration test ($N_s \leq 50$). The most used formula for estimating the ultimate unit shaft resistance of PPP in Japan is identical to the Korean foundation design standard.

However, the empirical formulas stated above are reported in numerous reports and literatures as conservative and tends to underestimate the pile tip capacity of the PPP. It shows low accuracy due to insufficient field data used in the deriving of the formula and lacks consideration of the characteristics of the PPP (Kim et al., 2020).

3. EVALUATION OF BEARING CAPACITY OF STEEL PIPE PREBORED AND PRECAST PILE

In this paper, real-scale field tests were carried out in order to investigate the unique behavior of steel pipe PPP and propose a new empirical formula.

3.1. Test preparation and procedures

20 cases of real-scale field tests were conducted on a fully instrumented test piles in four test sites. After a series of site investigations and laboratory tests, the test site was found to have a layer of land fill, sedimentary layer, weathered soil and weathered rock layers. The mechanical properties of the soil and rock of the test site are shown in Table 1.

The vibrating wire type strain gauges (*GEOKON*) measuring the settlement and the load-transfer (t-z) curve of the test pile was installed along the shaft of the test piles. The gauges were placed on two sides of the pile for each depth to prevent the loss of records due to gauge failure. Table 2 summarizes the installation conditions of the 20 test piles. The test piles were installed at a various pile tip condition, with different assigned SPT N-value, to observe the effect of different pile tip condition.

Table 1: Properties of test site and cement paste.

	γ (kN/m^3)	E (MPa)	v	c (kPa)	φ (˚)
Pile	75	200,000	0.2	-	-
C.P[1)]	20	2,700	0.2	-	-
Fill	17	10	0.3	0	29
S.L[2)]	19	20	0.3	3	28
W.S[3)]	20	13	0.3	33	39
W.R[4)]	21.5	185	0.3	35	32

[1)]Cement paste, [2)]Sedimentary layer
[3)]Weathered soil, [4)]Weathered rock

The static pile loading test was carried out 30 days after the pile installation, to secure maximum curing of the injected cement paste. The procedure of the loading test is a combination of the slow maintained standard loading test following the instructions stated be the American Society of Testing and Materials (ASTM) D1143 (2009). The loading process was carried up to the ultimate state of the pile. The settlement of the pile head due to axial loading is recorded based on the measurement through the LVDT (Linear Variable Differential Transformer) installed on the test pile. The ultimate stage of the pile capacity is assumed to be reached when a sudden settlement (i.e., failure) occurred during the loading process. After the ultimate state, the test is completed.

Table 2: Test pile installation conditions.

No.	Diameter (m)	Length (m)	SPT N-value
A-1	0.457	12.0	50/6
A-2		14.0	50/3
B-1		12.5	50/10
B-2		11.5	50/15
B-3		11.0	50/13
B-4		10.4	50/17
C-1	0.508	11.9	50/5
C-2		13.4	50/2
C-3		7.3	50/20
C-4		8.0	50/15
D-1		9.7	50/20
D-2		10.8	50/9
D-3		9.8	50/5
D-4		13.4	50/2
D-5		10.0	50/14
D-6		9.7	50/10
D-7		10.2	50/11
D-8		11.0	50/8
D-9		8.8	50/13
D-10		9.0	50/15

Figure 4 shows the instruments used in the field loading tests including a load cell, hydraulic jack, pump, LVDTs, earth anchors for reaction and beams (support and reaction). Eight anchor (reaction) piles were constructed by boring 150mm borehole with 10 12.7mm steel bars installed. The borehole is grouted with cement of 50% water-cement ratio, to reach capacity of 1,000kN per anchor.

3.2. Supplementary site investigation

After the completion of the pile loading test, the actual pile tip and shaft condition was thoroughly investigated to consider the actual pile tip and shaft condition in deriving the new empirical formula.

The pile tip condition was investigated after the field loading test. The SPT N-value of the pile tip was measured at the bearing layer by drilling a hole at the center of the test pile. The pile tip bearing layer was reached after drilling through the injected cement paste, cured in the hollow center portion of the test pile. The results of the investigation on the actual pile tip are summarized in Table 3. By comparing the results, it was found that in some cases the SPT N-value differed significantly between the initial installation condition and after the loading tests, due to disturbance or occurrence of slime during the installation process. In site A, A-2 showed a slight decrease in pile tip condition compared to the targeted (assigned) SPT-N value at installation process. Test pile 4 in site C showed significant decrease in SPT-N value between the targeted N-value and the actual value. The SPT-N value of

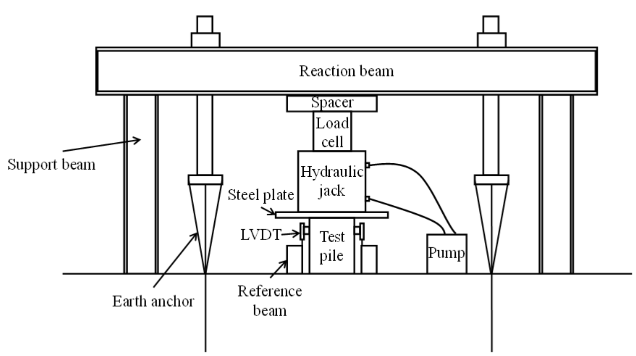

Figure 4: Schematic of the loading system for field loading test

Table 3: Actual SPT N-value by SPT N-value.

No.	Assigned SPT N-value	Actual SPT N-value	Remarks
A-1	50/6	50/5	Normal
A-2	50/3	50/10	Disturbed
B-1	50/10	50/20	Disturbed
B-2	50/15	50/25	Disturbed
B-3	50/13	7/30	Slime
B-4	50/17	50/17	Normal
C-1	50/5	50/4	Normal
C-2	50/2	50/2	Normal
C-3	50/20	50/17	Normal
C-4	50/15	3/30	Slime
D-1	50/20	50/21	Normal
D-2	50/9	50/9	Normal
D-3	50/5	50/18	Disturbed
D-4	50/2	50/3	Normal
D-5	50/14	50/7	Normal
D-6	50/10	50/15	Normal
D-7	50/11	50/6	Normal
D-8	50/8	50/4	Normal
D-9	50/13	50/30	Disturbed
D-10	50/15	50/11	Normal

the pile tip was found to be extremely low (3/30), which indicated the existence of slime layer. The cause of the slime was caused by the rise of the groundwater, or by a layer of residual soil during the borehole excavation. To prevent this from happening, the agitation of the residual soil and the injected cement paste might be necessary. However, even though it is an abnormal circumstance, it is a possibility which can occur in the engineering practice. Thus, these data points are also considered in the formula deriving process, as a data point with extremely low SPT N-value and low tip resistance.

The condition along the shaft of the test piles were also investigated through a borehole image profile system (BIPS). The BIPS investigation was carried out by boring a hole right next to the PPP. From that borehole, the surface was taken as an image to observe the cement paste infiltration to the surrounding ground. Through BIPS investigation it was clearly observed that PPP shows two different types of shaft behavior due to the infiltration of cement paste injected during installation. This will be discussed in detail in the following section of this paper.

3.3. Test results and discussions

3.3.1. Pile tip bearing capacity

To analyze the relation between the SPT N-value and the tip bearing capacity, load-settlement curves, load-transfer curves, and load-distribution curves along the shaft obtained from the field loading test was intensively analyzed.

The SPT-N values on the pile tip and the unit tip bearing capacities (kN/m^2) obtained from 20 field test cases were used in the process. As discussed above, in some cases the pile tip conditions were uncertain and showed very low SPT-N value, and these cases were found to show very low tip bearing capacity. By taking the actual installation condition into account, the proposed formula will be able to consider the actual PPP tip characteristics as well as provide a safe estimate of the tip bearing capacity.

3.3.2. Pile shaft resistance capacity

The pile shaft resistance from the field test was thoroughly analyzed based on the 211 data points obtained from the strain gauges attached along the shaft of the test piles. Based on the load-transfer curves from the field test and the BIPS investigation, it was found that PPP shows two different types of shaft behavior. The main cause of the different behavior is due to the cement paste infiltration to surrounding ground. By analyzing the field load-transfer, the representative load-transfer curves of PPP can be displayed as “elasto-perfectly plastic behavior” and “brittle behavior”. There was no cement milk infiltration observed in C-1 and 3, and the load-transfer curves showed “elasto-perfectly plastic behavior”. However, C-2 and 4 showed a “brittle behavior” load-transfer curve, which was associated with the cement milk infiltration. The shaft resistance at a designated location along the shaft is paired with the SPT N-values from the field investigation. The relationship between the SPT N-value and the shaft resistance are, similar to the tip bearing capacity case, cannot be properly considered due to the limited consideration of the ground condition of SPT N-value up to 50 blows. This may cause the inaccuracy of the proposed empirical formula.

4. PROPOSED UNIT BEARING CAPACITY FORMULA

New empirical formula estimating the unit bearing capacity of steel pipe PPP will be proposed based on the field test results and statistical processing. Moreover, to fully consider the installation condition of the steel pipe PPP, expanded SPT N-value criterion will be applied.

4.1. Expanded SPT N-value criterion

Existing empirical formula could consider limited capacity of the ground by limiting the SPT N-value of the ground to maximum of 50. This was the main cause of the conservative PPP foundation design. To fully consider the condition of the ground in which the PPP is installed and estimate realistic bearing capacity of PPP, the expanded SPT N-value will be applied in the new formula. The linear expanded SPT N-value is stated in Table 4. The expanded SPT N-value will be presented up to the upper limit of weathered rock (SPT N-value = 50/3, or 3cm penetration in 50 hammer blows) for the pile tip bearing capacity and lower limit of the weathered rock (SPT N-value = 50/10, or 10cm penetration in 50 hammer blows) for the shaft resistance, which was the main scope of this study and field test.

4.2. Proposed empirical formula

The new formula estimating the unit pile tip bearing capacity of steel pipe PPP will be proposed. The correlation between the expanded SPT N-value and the unit tip bearing capacity from the field test is displayed in Figure 5. The maximum value of the expanded SPT N-value will be set to 500 to fully cover the range of the

Table 4: Expanded SPT N-value.

N-value	1/30	30/30	50/30	50/15	50/3
Existing	1	30	50	50	50
Expanded	1	30	50	100	500

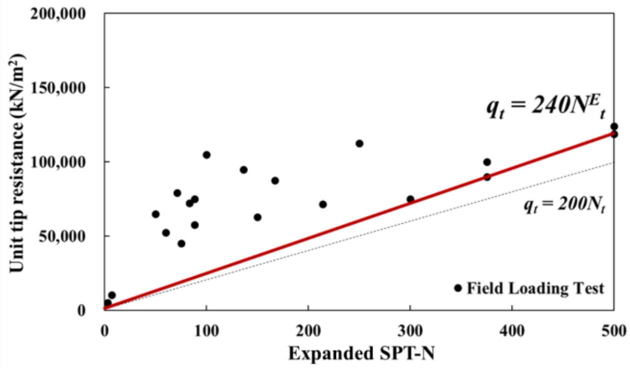

Figure 5: Correlation between tip bearing capacity and expanded SPT N-value (N^E_t)

weathered rock. Due to the limited test cases, the proposed formula for the unit tip bearing capacity is based on the most conservative trendline, while considering the plugging effect of the injected cement paste in the pile tip, which is displayed as a bold red trendline in the figure. The proposed formula is shown in Eq. (4):

$$q_t = 240N^E_t \text{ (kN/m}^2\text{)} \tag{4}$$

where, N^E_t is the expanded SPT N-value at pile tip from the standard penetration test ($N^E_t \leq 500$). The maximum unit tip bearing capacity for the steel pipe PPP will be 120,000kN/m^2. Compared to the existing empirical formula the new proposed formula gives 20% higher tip bearing capacity in case of ground condition below the SPT N-value of 50, even with the consideration of the characteristics of the PPP, such as weak tip bearing conditions and the plugging effect. Although the existing formula gives identical tip bearing capacity for piles installed in ground with higher SPT N-value of 50, new formula can estimate the pile tip bearing capacity based on realistic consideration of the ground condition for PPP which is commonly installed in conditions equal to or higher than weathered rocks.

The correlation between the expanded SPT N-value and the unit shaft resistance measured during the field test is displayed in Figure 6. In case of the shaft resistance formula, the maximum value of the expanded SPT N-value is set to 150, the lower boundary of the weathered rock. Different from the pile tip bearing capacity, sufficient number of data, over 200 data points

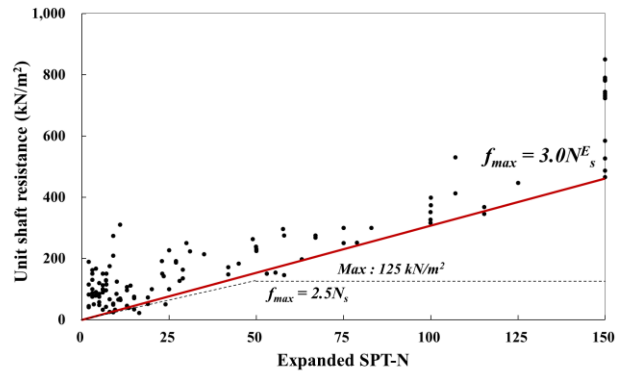

Figure 6: Correlation between tip bearing capacity and expanded SPT N-value (N^E_s)

which exceeds the threshold for a statistically large number of 30, were obtained. Therefore, the new formula estimating the shaft resistance was deducted by applying statistical processing. Assuming that the data points follow the gaussian contribution, and to deal with the uncertainties of the field test results and safe design of the PPP foundation, the “two-sigma rule” was applied in deriving the new formula. The “two-sigma rule”, which is often considered as the lowest cutoff value in hypothesis testing and is frequently used to establish confidence intervals. By this, the lower 97.5% data point was applied in proposing a new formula as in Figure 7 and Eq. (5):

$$f_{max} = 3.0N^E_s \text{ (kN/m}^2\text{)} \tag{5}$$

where, N^E_s is the expanded SPT N-value along the pile shaft obtained from the standard penetration test ($N^E_s \leq 150$). The maximum unit shaft resistance for the steel pipe PPP will be 450kN/m^2. Compared to the existing empirical formula, the new proposed formula for the shaft resistance also gives 20% higher results in case of ground condition

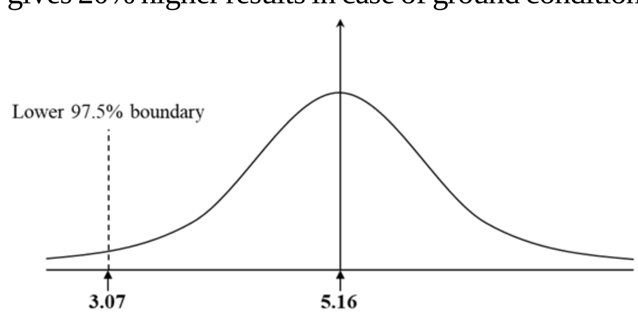

Figure 7: Statistical deduction of the new empirical formula for shaft resistance based on “two-sigma rule”

condition below the SPT N-value of 50. The existing formula gives maximum unit shaft resistance of 125kN/m^2, which is severely conservative compared to the field test results. Proposed shaft resistance formula can estimate the effective pile shaft resistance based on realistic consideration of the ground condition.

5. CONCLUSIONS

In this study, new empirical formula for estimating the tip bearing capacity and the shaft resistance of the steel pipe PPP was proposed. The formula was derived by analyzing 20 cases of real-scale pile loading tests and considering the distinct characteristic of the PPP. Based on the tests conducted in this paper and deriving process the following conclusions can be drawn:

(1) It was clear that the pile tip bearing capacity / shaft resistance and the SPT N-value was in direct proportional relation. To fully consider the ground condition the SPT N-value was expanded and applied, up to 500, which can distinctively consider the full range from soil to weathered rock.

(2) The pile tip bearing capacity formula was proposed based on the lowest trendline of the field test results. Even though the lowest trendline was applied, compared to the conventional formula, it still estimates higher tip bearing capacity by 20%.

(3) The shaft resistance formula was proposed through statistical processing of 211 data points. To deal with the uncertainties of the results and secure the safety of the PPP design, low 97.5% trendline was applied in proposing the new formula for estimating the shaft resistance. The new formula estimating the shaft resistance also provides higher shaft resistance by 20% compared to the conventional formula.

6. ACKNOWLEDGEMENT

This study was funded by the National Research Foundation (NRF) of Korea (RS-2022-00166673).

7. REFERENCES

AIJ (2004), Recommendations for design of building foundations, Architectural Institute of Japan, Tokyo.

American Society for Testing and Materials D1143-87. 2009. Standard Test Method for Piles Under Static Axial Compressive Load.

Broms, B. (1979), "Negative skin friction", *Proceedings of the 6th Asian Regional Conference of Soil Mechanics and Foundation Engineering*, Singapore, Vol. 2, 41–75.

Jeong, S. and Kim, D. 2018. "Estimation of the load sharing ratio of pre-installed columns in top-down buildings on Korean rock", *KSCE Journal of Civil Engineering*, 22(12), 4852-4861.

Johnston, I., and Choi, I. (1985), "Failure mechanism of foundations in soft rock", *Proceeding of 1st International Conference on Soil Mechanics and Foundation Engineering*, San Francisco, CA, No. 3, 1397-1400.

Johnston, I., and Lam, T. (1989), "Shear behavior of regular triangular concrete/rock joints-analysis", *ASCE Journal of Geotechnical Engineering*, Vol. 115(5), 711-727.

Kim, D., Jeong, S., Jung, G. and Park, J. 2018. "Load-sharing ratio of prebored and precast pile in top-down method construction process", *Structural Design of Tall and Special Buildings*, 27(10), 1-14.

Kim, D., Jeong, S., and Park, J. 2020. "Analysis on shaft resistance of the steel pipe prebored and precast piles based on field load-transfer curves and finite element method", *Soils and Foundations*, 60(2), 478-495.

Korean Geotechnical Society. 2009. Standards for Foundation Design of Structures. CIR.

Ng, C., Zhang, L. and Nip, D. 2001. "Response of Laterally Loaded Large-Diameter Bored Pile Groups", *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, 127(8), 658-669.

O'Neill, M., Townsend, F., Hassan, K., Buller, A., Chan, P. 1996. "Load transfer for drilled shafts in intermediate geomaterials", Federal Highways Administration.

Park, J. 2004. "Strength and friction behavior of cement paste poured in the bored pile", *Journal of the Korean Geoenvironmental Society*, 5(3), 31-39.

Reese, L. and O'Neill, M. 1988. "Drilled shafts: Construction procedures and design methods", Publication No. FHWA-HI-88-042, Federal Highway Administration, Washington, D.C.

Seol, H., Jeong, S. and Cho, S. 2009. "Analytical method for load-transfer characteristics of rock-socketed drilled shafts", *ASCE's Journal of Geotechnical and Geoenvironmental Engineering*, 135(6), 778-789.