Chapter 1 Introduction

1.1 Background

Piles, as deep foundations, have been used to transfer axial loads occurred by upper structures. Especially, use of drilled shafts is the more increasing heavy structures, such as long span bridges and super high rising buildings, are more constructed. Also, the drilled shaft could apply to subsurface mixed with boulder stone and gravel, or very dense residual soil and weathered rock where driven pile could not construct. (KGS, 2002)

Researches for axial bearing capacity of drilled shafts have been carried out since 1970s. Most of the researches were based on results which had been derived from foreign load tests, and suggested equations for bearing capacity of the drilled shaft about soil and rock mass layers respectively. Many researchers have recognized problems for evaluating bearing capacity of drilled shaft constructed in transition layer between soil and rock mass, and have researched about methods for evaluating the bearing capacity since mid-1990s. As a result of the research, the transition layer have been defined as Intermediate Geo-Materials (IGMs), and evaluating method for drilled shafts constructed in IGMs were suggested.

In Korea, although the use of drilled shafts are increasing, researches about the bearing capacity are incomplete. Most of specifications in Korea, introduce the foreign methods for evaluating bearing capacities of drilled shafts, which have not been considered features of Korean subsurface. Especially,

although weathered zone, which is very similar to IGMs, consisted of residual soil and weathered rock are widely and thickly distributed in Korea, most of the specifications suggested that subsurface was just distinguished soil and rock mass, and evaluating methods for bearing capacities of drilled shafts suggested only about the soil and rock mass. After mid-2000s, researchers in Korea have recognized importance of evaluating bearing capacities of the weathered zone and have researched the bearing capacity by using IGMs theory suggested in FHWA (1999). However, a systematic criterion and method for evaluating a bearing capacity of the drilled shaft constructed in weathered zone has not been yet established.

1.2 Scope of the Research

This research deals with the design and analysis of drilled shafts socketed in weathered zone, in other words, residual soil and weathered rock. This study primarily focuses on problems suggested on several specifications in Korea, and improved methods for evaluating bearing capacity of drilled shafts, which are based on quantitative parameters on subsurface investigation reports.

The scope of these investigations were limited to the followings:

1. The behaviors of weathered rock socketed cast-in-place concrete

pile subjected to axial resistance only, the displacement is not considered; i.e., the skin friction and end bearing capacity.

2. Design parameters are only quantitative properties of weathered zone; i.e., q_u (unconfined uniaxial strength), RQD, socket depth.

3. Pile-load tests used to verify the improved method are performed in Korea.

1.3 Objectives

The objectives of this research are as follows:

1. To analyze drawbacks of several specifications in Korea for drilled shafts socketed weathered zone, which is specifically about determining design parameters and evaluating bearing capacity.

2. To suggest improved method for determining design parameters and evaluating bearing capacity, which are based on quantitative parameters on subsurface investigation reports, and to verify the improved method.

1.4 Dissertation Organization

Chapter 2 reviews literatures that are concerned with determining weathered zone and procedures for evaluating the bearing capacity of drilled shafts in weathered zone. Also, this chapter presents problems of the specifications.

Chapter 3 provides the improved method for evaluating the bearing capacity of drilled shafts in weathered rock, which includes determining boundaries of weathered zone and modifying the procedure for evaluating the bearing capacity. This chapter also provides the results of verification.

Chapter 4 provides summary and conclusions.

Chapter 2 Literature Review

2.1 Determination of Weathered Zone

2.1.1 Current State in Korea

Since the subsurface shows the characteristics that are existed bedrock at a relatively shallow depth, drilled shaft tips are generally positioned to weathered rock or soft rock in order to ensure the safety of the drilled shafts. Figure 2.1 is statistical data showing the bearing stratum of drilled shafts in Korea. Weathered zone composed of residual soil and weathered rock serves as the main bearing strata where side friction resistance and end bearing resistance are mobilized, therefore determining the depth and the thickness of the weathered zone is very important.

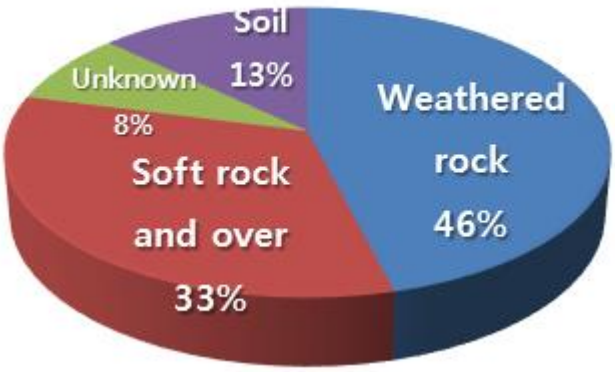

Figure 2.1 Bearing stratum of drilled shafts in Korea (KICT, 2008)

The designers generally determine the thickness of the weathered zone, by using the depth of each stratum recorded in subsurface investigation report. The recorded depth of weathered zone is determined by engineers who synthetically review the qualitative information that are judged by boring technician such as slime, boring rate, boring sound etc. and the quantitative information that are measured values such as SPT N-value, unconfined uniaxial strength (q_u), elastic wave velocity (V_p), total core recovery (TCR), rock quality designation (RQD) etc.

Several institutions have suggested the qualitative and quantitative characteristics of weathered zone in order to assist the engineers in classifying the weathered zone. The classification criteria of weathered zone are as follows, which are suggested by major institutions in Korea.

Table 2.1 Classification criteria (KEC, 2009)

Subsurface layer	Qualitative characteristic	Quantitative criteria
Residual soil	Rock material is severely weathered, and a clods state which is very easy to break	N < 50blows/15cm V_p (km/s) ≤ 3.0 q_u ≤ 25MPa
Weathered rock	Because of physical and chemical effects, fracture zones and joints are randomly developed.	N ≥ 50blows/15cm 3.0 < V_p (km/s) < 3.5 TCR ≤ 40% RQD ≤ 20% q_u ≤ 60MPa

Table 2.2 Classification criteria (KRNA, 2011)

Subsurface layer	Qualitative characteristic	Quantitative criteria
Residual soil	-	-
Weathered rock	Rock material exists in state of flinders, but cylindrical core sample could not obtained. Even a weak blow, rock could be broken.	$N \geq 50$blows/15cm V_p (km/s) < 3.5 TCR : No suggestion RQD : No suggestion $q_u \leq 5$MPa

*** -** : No suggestion

Table 2.3 Classification criteria (KGS, 2009)

Subsurface layer	Qualitative characteristic	Quantitative criteria
Residual soil	-	-
Weathered rock	Rock material is corroded and has 1~10cm length of cracks. Pickax could be used to excavate the rock in some cases, but gunpowder is usually applied to excavation.	N : No suggestion 0.7 < V_p (km/s) < 1.2 TCR $\leq$ 20% RQD : No suggestion 30MPa $\leq q_u \leq$ 5MPa

*** -** : No suggestion

Table 2.4 Classification criteria (Seoul, 2006)

Subsurface layer	Qualitative characteristic	Quantitative criteria
Residual soil	Since rock-forming minerals are completely weathered, coherence as a rock is extinct. Most joints are filled with secondary minerals such as clay and remain just traces. Shear strength is noticeably decreased on saturated state.	$N < 50$blows/10cm V_p (km/s) < 1.2
Weathered rock	Because of highly weathering, rock material is discolored and most joints are filled with secondary minerals or opened. Even a weak blow, rock could be broken and scratched with knife. Spacing of joints is narrow or very narrow. Rock flinders are only recovered, on boing.	$N \geq 50$blows/10cm $1.0 < V_p$ (km/s) < 2.5 $10\% \leq TCR \leq 30\%$ $RQD \leq 10\%$ $q_u < 10$MPa

2.1.2 Analysis of Drawbacks

Several drawbacks for determining the depth and thickness of the weathered zone are analyzed as a result of review of the current states, which is based on boring logs and the classification criteria. The analyzed drawbacks are as follow.

(A) Subjective Determination of Weathered Zone.

Determining the depth and thickness of weathered zone is very important process for evaluating the socket depth and bearing capacity of the drilled shafts. Therefore, specific and consistent method for determining the weathered zone is required. However, due to causes as follows, the depth and thickness could also be determined differently by the subjective judgment of the author of boring log, in spite of the same target site.

(1) Different criteria of weathered zone

Quantitative classification criteria which are suggested by several major institutions in Korea are arranged in Table 2.5. As showed in Table 2.5, types of geotechnical properties used to classify the weathered zone are not unified. Furthermore, in spite of the same type of geotechnical properties, the suggested values of the criteria are not equal. In other words, a standardized classification criterion is not yet established in Korea, the depth and thickness

of weathered zone could be variable, according to the criterion selected by boring log author. The subjective classification is usually more used than the quantitative criteria in the actual work, which is variable according to experience of investigator. In case of an important subsurface investigation, experienced experts comprehensively analyze the depth and thickness of weathered zone by using both the qualitative and quantitative information, but objective and consistent results are rarely suggested.

(2) Discontinuous estimations of N-value

Although each institutions suggest the qualitative classification criteria of residual soil and weathered rock, the boundary between the residual soil and the weathered rock could be rarely determined with just observation because the residual soil and the weathered rock are transition state. So, the boundary is generally determined by using SPT N-value which is almost performed on boring investigation.

Standard Penetration Test (SPT) is performed at every 1m or 1.5m of depth spacing, measured N-values are recorded with an each test depth on boring log, as shown in Figure 2.2. Each N-values are representative value in each sections, and have a characteristic of discontinuity.

Table 2.5 Quantitative classification criteria suggested by each institution

Institution	Classification criteria							
	Residual soil			Weathered rock				
	N[1)] (blows/cm)	V_p (km/s)	q_u (MPa)	N (blows/cm)	V_p (km/s)	q_u (MPa)	TCR (%)	RQD (%)
KEC	≤ 50/15	≤ 3.0	≤ 25	≥50/15	3.0~3.5	≤ 60	≤ 40	≤ 20
KRNA	≤ 50/15	-[2)]		≥50/15	≤ 3.5	≤ 5	-	
Seoul	≤ 50/10	≤ 1.2	-	≥50/10	1.0~2.5	≤ 10	10~30	≤ 10
KGS	-			-	0.7~1.2	30~70	≤ 20	-

1) N value means the number of blows required to achieve a penetration of 30cm, but N value expressed 50/penetration depth for more than 50 blows in Korea. The less penetration depth value means the more hardened subsurface.

2) No suggestion.

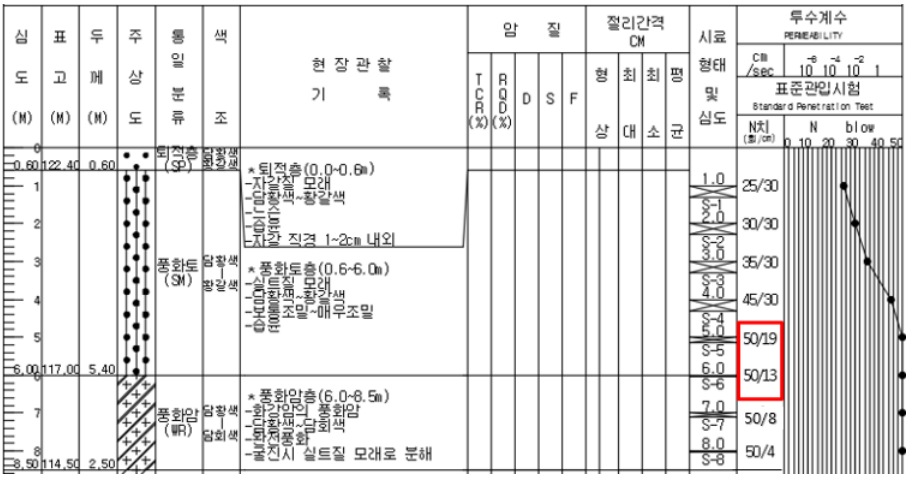

Figure 2.2 An example of boring log

Because of the discontinuity of N-value, the author of boring log could not determine the boundary between residual soil and weathered rock. For example, N-values are measured 50blows/19cm at 5m depth and 50blows/13cm at 6m depth respectively in Figure 2.2, but the criteria of the both layers, 50blows/15cm is not measured actually. In that case, the author could determine the boundary as ① the depth of 6m for conservative estimation or ② the middle depth of 5.5m or ③ the predicted depth where N-value could be measured the criteria, 50blows/15cm by linear interpolating N-values. Generally, the conservative method is used.

Likewise, it is hard for the author to apply the quantitative criteria of N-value (50blows/15cm or 50blows/10cm), when the exact criterion of N-value which is suggested by each institutions are not

measured. Therefore, the boundary of the both layers could be determined subjectively.

(B) The Absence of Criteria for Hardened Residual Soil

As a result of reviewing the N-value of residual soil in Korea, the residual soil has wide distribution of N-values from very small value to more than 50, which means that the residual soil could be divided from soft one to hard one. The N-values represent the degree of density, as shows in Table 2.6. In case of the hardened residual soil, all measured N-values are more than 50, which means that very dense condition.

Table 2.6 Relative density of granular soil (Terzaghi and Peck, 1948)

N	Description	Relative density (%)
0 ~ 4	Very loose	0 ~ 20
4 ~ 10	Loose	20 ~ 40
10 ~ 30	Medium	40 ~ 60
30 ~ 50	Dense	60 ~ 80
≥ 50	Very dense	80 ~ 100

Especially, FHWA (1999) that is a manual for design and construction of drilled shafts defined very dense and hardened soil which is in transitional state between soil and rock mass as Intermediate Geo-Materials (IGMs), and suggested methods evaluating the resistance of IGMs, which are considered the high strength characteristics of IGMs. However, since specific classifications criteria of the hardened residual soil that has more than N-

value of 50 are not yet established in Korea so, the residual soil have been generally classified without considering the degree of hardness. Therefore, the high resistance of the hardened residual soil is not recognized. When drilled shafts socketed in rock mass are designed, the bearing capacities are excessively underestimated by ignoring the side resistance of the hardened residual soil. Henceforward, the classification criteria of the hardened residual soil should be established for more efficient design.

2.2 Application of the Bearing Capacity Equations

2.2.1 Current State in Korea

Most of the drilled shafts are socketed in rock mass which is weathered rock or has higher strength than the weathered rock for safety in Korea. When axial load is applied on pile head, the resistance is generally mobilized larger at the rock mass layer than at soil layer so, the axial load are mostly resisted at rock mass layer. In addition, the displacement which is mobilized at soil layer when axial load applied is required more largely than the displacement of at the rock mass layer for mobilization of maximum resistance. Therefore, the bearing capacity is generally evaluated using only the resistance of the rock mass layer without considering the resistance of soil layer.

The weathered rock is classified as rock mass according to the classification criteria in Korea, showed in Table 2.5. So, the bearing capacity equations which is suggested for the rock mass should be

applied in order to evaluate end bearing resistance and side friction resistance of weathered rock. The equations suggested for rock mass are presented by using unconfined uniaxial strength of intact rock and discontinuity characteristics such as RQD, joint spacing, weathering degree as a design parameters. Since weathered rock has generally a lot of joints, so TCR of the rock core sample is usually low. So, the unconfined uniaxial strength might not be obtained. Using a triple core barrel is recommended to evaluate the bearing capacity exactly. However, because of the cost of boring investigation, in many case a SPT test is just performed, and the bearing capacity is evaluated by using measured N-value.

The bearing capacity equations used for rock mass are as follows.

(A) Bearing Capacity Equation Using the Unconfined Uniaxial Strength

(1) Carter and Kulhawy (1988)

■ Side Friction Resistance

Carter and Kulhawy (1988) suggested the equation for evaluating unit side friction resistance as Equation (2.2.1), which is based on Rowe and Armitage's (1984) research.

$$f_{ult} = 0.63 p_a \alpha_E \left(\frac{q_u}{p_a} \right)^{0.5} \approx 0.20 \alpha_E q_u^{0.5} \ \ (MPa) \qquad (2.2.1)$$

Where,

P_a is atmospheric pressure (≒0.101MPa);

q_u is unconfined uniaxial strength of intact rock;

If the intact rock is stronger than concrete, q_u of concrete is used.

α_E (=E_m/E_i, E_m : deformation modulus of rock mass, E_i : deformation modulus of intact rock) is the reduction factor for reflecting the characteristic of discontinuity of rock mass, which is suggested according to RQD and joint conditions as showed in Table 2.7. However, the reduction factors are not suggested in the range of RQD lower than 20%, and recommended to evaluate the unit side friction resistance by performing the pile load test.

Table 2.7 Reduction factor (Carter and Kulhawy, 1988)

RQD (%)	$\alpha_E = E_m/E_i$	
	Closed Joints	Open Joints
100	1.00	0.60
70	0.70	0.10
50	0.15	0.10
20	0.05	0.05

■ End Bearing Resistance

Carter and Kulhawy (1988) suggested a lower bound solution for unit end bearing resistance for a drilled shaft bearing on randomly jointed rock as Equation (2.2.2). In addition, this method is based partially on the work of Hoek (1983). In this method the rock has mass properties *s* and *m*, which are roughly equivalent to c (cohesion) and ϕ (internal friction angle) for a soil, respectively. The assumption is made that the joints are drained but the rock between the joints is undrained and the shear stresses in the rock mass are nonlinearly dependent on the normal stresses at failure. The joints are not necessarily oriented preferentially. The joints may be closed or open and even filled with weathered geomaterial.

$$q_{ult} = [s^{0.5} + (m \cdot s^{0.5} + s)^{0.5}]q_u \quad (MPa) \qquad (2.2.2)$$

Where, values of the parameters and the jointed rock mass are evaluated from Table 2.8.

Table 2.8 Values of *s* and *m* with rock type (Hoek, 1983)

Rock quality	Constants	Rock type A = Carbonate rocks with well developed crystal cleavage— *dolomite, limestone, and marble* B = Lithified argrillaceous rocks—*mudstone, siltstone, shale, and slate (normal to cleavage)* C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage—*sandstone and quartzite* D = Fine grained polymineralic igneous crystalline rocks— *andesite, dolerite, diabase, and rhyolite* E = Coarse-grained polymineralic igneous and metamorphic crystalline rocks—*amphibolite, gabbro, gneiss, granite, norite, quartz-diorite*				
		A	B	C	D	E
INTACT ROCK SAMPLES Laboratory size specimens free from discontinuities. CSIR rating: *RMR* = 100	m s	7.00 1.00	10.00 1.00	15.00 1.00	17.00 1.00	25.00 1.00
VERY GOOD QUALITY ROCK MASS Tightly interlocking undisturbed rock with unweathered joints at 3–10 ft. CSIR rating: *RMR* = 85	m s	2.40 0.082	3.43 0.082	5.14 0.082	5.82 0.082	8.567 0.082
GOOD QUALITY ROCK MASS Fresh to slightly weathered rock, slightly disturbed with joints at 3–10 ft. CSIR rating: *RMR* = 65	m s	0.575 0.00293	0.821 0.00293	1.231 0.00293	1.395 0.00293	2.052 0.00293
FAIR QUALITY ROCK MASS Several sets of moderately weathered joints spaced at 1–3 ft. CSIR rating: *RMR* = 44	m s	0.128 0.00009	0.183 0.00009	0.275 0.00009	0.311 0.00009	0.458 0.00009
POOR QUALITY ROCK MASS Numerous weathered joints at 2 to 12 in; some gouge. Clean compacted waste rock. CSIR rating: *RMR* = 23	m s	0.029 3×10^{-6}	0.041 3×10^{-6}	0.061 3×10^{-6}	0.069 3×10^{-6}	0.102 3×10^{-6}
VERY POOR QUALITY ROCK MASS Numerous heavily weathered joints spaced < 2 in with gouge. Waste rock with fines. CSIR rating: *RMR* = 3	m s	0.007 1×10^{-7}	0.010 1×10^{-7}	0.015 1×10^{-7}	0.017 1×10^{-7}	0.025 1×10^{-7}

(2) FHWA (Federal Highway Administration, 1999)

■ Side Friction Resistance

FHWA (1999) suggested the equation for evaluating unit side friction resistance as Equation (2.2.3), which is based on Horvath and Kenney's (1979) research.

$$f_{ult} = 0.65\alpha p_a \left(\frac{q_u}{p_a}\right)^{0.5} \approx 0.20\alpha q_u^{0.5}\ MPa \qquad (2.2.3)$$

Where,

P_a is atmospheric pressure (≒0.101MPa);

q_u is unconfined uniaxial strength of intact rock;

If the intact rock is stronger than concrete, q_u of concrete is used.

α is the reduction factor suggested by O'Neill (1999) for reflecting the characteristic of discontinuity of rock mass as showed in Table 2.9. FHWA (1999) suggested that α could be determined after select the α_E suggested in Table 2.7. The reduction factor, α is only suggested in the range of RQD larger than 20%, and recommended to evaluate the unit side friction resistance by performing the pile load test.

Table 2.9 Reduction factor (O'Neill et al. 1999)

E_m/E_i	α
1.00	1.0
0.5	0.8
0.3	0.7
0.1	0.55
0.05	0.45

■ End Bearing Resistance

FHWA (1999) proposed that the unit end bearing resistance could be evaluated using the Equation (2.2.4) for cases where the rock mass is sedimentary jointed and where the joints are primarily horizontal, which was suggested by Canadian Foundation Engineering Manual (CGS, 1985), based on the work of Ladanyi et al. (1974). The depth factor (D') and the bearing capacity factor (K_{sp}) are used to consider the socket depth and the effects of discontinuity of rock mass respectively.

$$q_{ult} = 3K_{sp}q_u D' \ (MPa) \tag{2.2.4}$$

Where,

q_u is unconfined uniaxial strength of intact rock;

If the intact rock is stronger than concrete, q_u of concrete is used.

K_{sp} is an empirical factor and can be obtained from Table 2.10.

D' is depth factor and can be calculated from Equation (2.2.5)

$$D' = 1 + 0.4\frac{L_s}{D_s} \leq 3 \qquad (2.2.5)$$

Table 1.10 Bearing capacity factor (FHWA, 1999)

Joint spacing (m)	K_{sp}
0.3 ~ 1	0.1
1 ~ 3	0.25
≥ 3	0.4

(3) CFEM (Canadian Foundation Engineering Manual, 2006)

CFEM suggested that the unit side friction resistance could be evaluated by using the same equation except for the reduction factor, α_E, which promised by Carter and Kulhawy (1988) i.e. The Equation (2.2.1). The evaluating methods for unit end bearing resistance were proposed using the FHWA (1999) methods, which are suggested the Equation (2.2.4), (2.2.5) and the Table 2.10.

(4) AASHTO (American Association of State Highway and Transportation Official, 2010)

AASHTO proposed that the unit side friction and the unit end bearing resistance could be evaluated by using FHWA (1999) method and Carter and Kulhawy (1988) method, respectively.

(B) Bearing Capacity Equation Using the SPT N-value

(1) The Equations for General soil layer

The core sample of weathered rock might not be obtained in performing boing investigation. In that case, the unconfined uniaxial strength of the weathered rock is rarely measured. So, the bearing capacity is evaluated using SPT N-values.

The weathered rock has been assumed as very hardened soil layer in Korea, and the bearing capacity has been evaluated in accordance with the bearing capacity equation which suggested by JSCE (2002). JSCE (2002) proposed that the unit side friction resistance of a drilled shaft in soil layer could be evaluated by using the Equation (2.2.6). Since the condition of soil layer is affected by construction of a drilled shaft, JSCE (2002) suggested that applying 3MPa to the unit end bearing resistance regardless of the strength of the soil layer.

$$f_{ult}(MPa) = 0.005N_{60}\ (\leq 0.2MPa) \tag{2.2.6}$$

(2) The Equations for Cohesionless IGM

■ Side Friction Resistance

FHWA (1999) suggested that a cohesionless IGM is a sand-like or gravel-like material (transported or residual) that exhibits SPT N-Value is larger than 50blows/0.3cm, and the unit side friction resistance in such soils could be evaluated by using the Equation (2.2.7). O'Neill et al. (1996) recommend the following procedure using the SPT N-value, based on the work of Mayne and Harris (1993). This method has been used and verified by load testing of full-scale drilled shafts.

$$f_{ult}(MPa) = \sigma_v' K_0 \tan\varphi' \tag{2.2.7}$$

Where,

$$K_0 = (1-\sin\phi')OCR^{\sin\phi}, \quad OCR = \frac{\sigma_p'}{\sigma_v'}, \quad \sigma_p' = 0.2 p_a N_{60}$$

$$\phi' = \tan^{-1}\left\{\left[\frac{N_{60}}{12.3+20.3\left(\frac{\sigma_v'}{p_a}\right)}\right]^{0.34}\right\}$$

σ_v' is a vertical effective stress at average depth of the stratum, K_0 is coefficient of earth pressure at rest, calculated using OCR and internal

friction angle. It is recommended by O'Neill that N_{60} not be taken to be more than 100 with this method, regardless of the actual value of N_{60} measured.

- End Bearing Resistance

FHWA (1999) suggested that could be evaluated by using the Equation (2.2.8), which was based on the work of Mayne and Harris (1993)

$$q_{ult} = 0.59\left[N_{60}(p_a/\sigma_p')\right]^{0.8}\sigma_v' \tag{2.2.8}$$

Where N_{60} is the SPT blow count in blows/0.3m immediately below the base of the shaft. It is recommended by O'Neill that N_{60} not be taken to be more than 100 with this method, regardless of the actual value of N_{60} measured.

2.2.2 Analysis of Drawbacks

Several drawbacks of the method for evaluating the bearing capacity of the weathered zone are analyzed as a result of review of the current states. The analyzed drawbacks are as follow.

(A) The Ignorance of the Hardened Residual Soil Resistance

Since the strength and stiffness of the rock mass are much larger than those of the soil, the resistance is highly mobilized in rock mass. Therefore, an axial load applied to a pile head is mostly resisted by the side friction resistance and end bearing resistance mobilized in socketed area. The side friction resistance which is mobilized at the soil, including the residual soil has generally not been considered.

However, the hardened residual soil, as the layer which exhibits SPT N-Value is larger than 50blows/0.3cm, is very dense and hard. So, the strength and stiffness of the hardened residual soil are larger than that of the general soil. Recognizing the properties of the hardened residual soil as mentioned, FHWA (1999) suggested that a cohesionless IGM is a sand-like or gravel-like material (transported or residual) which exhibits SPT N-Value is larger than 50blows/0.3cm. Recently, the researches of evaluation the bearing capacity of the weathered zone have been performed, in Korea.

Therefore, for more reasonable and economical design, the side friction resistance of the hardened residual soil is required to be included, which can contribute to the bearing capacity

(B) The Limitation of Applying the Reduction Factor to Weathered Rock

The reduction factor have to be determined in order to use the bearing capacity equations using the unconfined uniaxial strength, such as Carter and Kulhawy (1988) method, FHWA (1999) method and AASHTO (2010) method. The reduction factor could be determined by using the Table 2.7, Table 2.9, but, the RQD of the weathered rock in Korea is less than 20%. Therefore, the side resistance of the weathered rock could not be evaluated, since the reduction factor is only suggested in the range of more than 20% of RQD. In that case, the upper value of the side friction resistance proposed at general soil layer is applied to evaluating the side friction resistance of the weathered rock. However, applying the upper value to the weathered rock which has larger strength and stiffness than those of the soil is very conservative design method. For more reasonable and economical design, the researches for more reasonable methods which evaluate the side friction resistance of weathered rock in Korea are required.

Chapter 3 Improved Methods & Verification

3.1 Determination of Weathered Zone

3.1.1 Securement of the Thickness of Weathered Zone

The thickness and depth of the weathered zone could be differently determined with designers, because of the ununified criteria and discontinuous estimations of N-value, as mentioned in Chapter 2.1. For the consistant determination of the thickness of weathered zone which is a one of the major design parameters, it is recommend that classifing the weathered zone according to the criteria suggested by KEC, as follows in Table 3.1.

Table 3.1 Classification of weathered zone in Korea

Subsurface layer	Quantitative criteria
Residual soil	· N < 50blows/15cm
Weathered rock	· N ≥ 50blows/15cm (low boundary) · RQD ≤ 20% (upper boundary)

As a result of the review of the ununified classfication criteria which is presented in the Table 2.5, the criteria suggested by KEC is recommended to unified the classification criteria, which is the conservative criterian. Subsurface parameters, such as N-values, V_p, TCR, RQD, that are used to classify the layers must present the characteristics of the strength and the discontinuity of the weathered zone, and must be easily obtained from subsurface investigation reports. Therefore, it is reasonable to use N-values and RQD, which are generally measured on perfoming the subsurface investigation and are applied to evaluating the bearing capacity of the drilled shaft.

If a N-value (50blows/15cm) which is the boundary between the residual soil and the weathered rock is not measured as mentioned in the Chaper 2.1.2-(B), it is recommended that designers should determine the boundary by interpolating the measured N-values.

The classification and determination method are standardized in this study in order to secure the thickness consistantly. With the classificaiton and determination method, bearing capacities of drilled shafts constructed in Korea are evaluated and compared with the results of pile load test to verify the improvement effect.

■ Outline of the Analysis

- Targets : Drilled shafts socketed in weathered rock
- Evaluation method of the bearing capacity

In case 0, the thickness of weathered rock that is presented in subsurface investigation report is applied as it is. The side friction resistance of residual soil is neglected, and the upper limit of the resistance suggested to general soil such as sand is applied to calculating the side friction resistance of weathered rock except for CFEM method which do not apply the reduction factor, since all of the RQD of the weathered rocks are less than 20%. The bearing capacity equations that suggested in the Chapter 2.2.1 are used to calculate the end bearing resistance of the drilled shafts.

In case 1, the standardized thickness of weathered rock that is suggested in this Chapter is applied, the equations to calculate the side friction resistance and the end bearing resistance are same to case 0.

Table 3.2 Comparison of evaluation methods (Case 0, Case 1)

Case No.	Subsurface layer	Evaluation method		Note
		Side	Base	
Case 0	Residual soil	-		Neglect the side friction resistance
	Weathered rock	The upper limit of soil bearing capacity f_{ult}=0.2MPa	The bearing capacity equation for rock mass	***Use the thickness presented in site investigation report*** *Upper limit of soil bearing capacity (JSCE, 2002) (2.2.1-(B)-(1)) * CFEM method (2.2.1-(A))
Case 1	Residual soil	-		Neglect the side friction resistance
	Weathered rock	The upper limit of soil bearing capacity f_{ult}=0.2MPa	The bearing capacity equation for rock mass	***Use the standardized thickness*** *Upper limit of soil bearing capacity (JSCE, 2002) (2.2.1-(B)-(1)) * CFEM method (2.2.1-(A))

■ Analysis Result

From the Table 3.3 the bearing capacities of the drilled shafts are largely estimated in order of CFEM (2006), FHWA (1999), AASHTO (2010) and Carter & Kulhawy (1988), regardless of the standardized method for determining the thickness of the weathered rock. The bearing capacities which are calculated using AASHTO (2010) and Carter & Kulhawy (1988) methods are same, since the bearing capacity equation of the end bearing resistacne are same and the upper limit of soil layer capacity are applied to the side friction resistance.

According to comparing the results of case 0 and case 1, the bearing capcities that are estimated in case 1 are generally increased. This result is caused by applying the standardized method of determining the boundary of weathered zone, which evaluate thickness of the weathered rock thicker than the existed method. The increased thickness of weathered rock are presented in Table 3.4

Table 3.3 Comparison of ultimate bearing capacity (Case 0, Case 1)

Pile No.	Measured capacity (kN)	Predicted Capacity (kN)					
		C&K, AASHTO		CFEM		FHWA	
		Case 0	Case 1	Case 0	Case 1	Case 0	Case 1
1	16700	2013	2076	31616	32050	23069	23132
2	13000	1825	1888	28961	29395	21529	21592
3	12000	1919	2202	30288	32243	22299	22582
4	18500	4050	4333	28076	30031	21016	21299
5	*21330	3924	4553	26306	30650	19989	20618
6	2550	1531	1506	8169	8056	2899	2874
7	2130	768	819	7061	7259	4837	4887
8	1070	761	811	4944	5143	2711	2761
9	1100	760	810	4763	4962	2530	2580
10	*3100	1531	1606	9672	10087	2865	2941
11	2950	2285	2361	9117	9375	3642	3717
Mean rate of increase, %		7.2		5.1		1.6	

* Measured capcity which is estimated using extrapolated load-diplacement curve

Table 3.4 Thickness of the weathered rock before and after modification

Pile No.	Thickness (m)		Pile No.	Thickness (m)	
	Case 0	Case 1		Case 0	Case 1
1	2.3	2.4	**7**	3	3.2
2	2	2.1	**8**	3	3.2
3	2.15	2.6	**9**	3	3.2
4	1.9	2.35	**10**	6	6.3
5	1.7	2.7	**11**	9	9.3
6	6	5.9	**Average**	3.64	3.93

Table 3.5 Comparison of Measured capacity/Predicted capacity

Pile No.	Measured capacity/Predicted capacity=K					
	C&K, AASHTO		CFEM		FHWA	
	Case 0	Case 1	Case 0	Case 1	Case 0	Case 1
1	8.29	8.04	0.528	0.521	0.724	0.722
2	7.12	6.89	0.449	0.442	0.604	0.602
3	6.25	5.45	0.396	0.372	0.538	0.531
4	4.57	4.27	0.659	0.616	0.880	0.869
5	5.44	4.69	0.811	0.696	1.07	1.03
6	1.67	1.69	0.312	0.317	0.880	0.887
7	2.77	2.60	0.302	0.293	0.440	0.436
8	1.41	1.32	0.216	0.208	0.395	0.388
9	1.45	1.36	0.231	0.222	0.435	0.426
10	2.03	1.93	0.321	0.307	1.08	1.05
11	1.29	1.25	0.324	0.315	0.810	0.794
Average	3.84	3.59	0.413	0.392	0.714	0.704

Table 3.5 presents the value of K which is defined as the ratio of the measured capacity and the predicted capacity. The measured capacities are the load in mobilizing the displacement of 24mm, which is suggested by Terzaghi in 1943. If the mobilized displacement is less than 24mm in applying the maximum load, the measured capacities are determined by using the extrapolated load-displacement curve. The value of K is larger than 1 means that the measured capacity is larger than the predicted capacity and that the bearing capacity is underestimated. In addition, the value of K converges the more closely to 1 the bearing capacity is the more accurately evaluated.

Carter & Kulhawy (1988) and AASHTO (2010) methods evaluate the value of K to be more than 1 and CFEM (2006), FHWA (1999) methods present the opposite result, regardless of the application of standardized determination method of the weathered zone. This tendency is explained with the constant that is multiplied to the unconfined uniaxial strength of the weathered rock in calculating the end bearing capacity, as described in the Chapter 2.2.1. The constants applied in Carter & Kulhawy (1988) and AASHTO (2010) methods are evaluated double to five times larger than those of CFEM (2006) and FHWA (1999) methods (Jung et al., 2007).

In this study, applying the thickness of the weathered rock as mentioned in the case 1, Carter & Kulhawy (1988) and AASHTO (2010) methods which estimate the bearing capacity conservatively evaluate the capacity more accurately, but CFEM (2006) and FHWA (1999) methods more overestimate the bearing capacity.

Since the conservative design is very important for the drilled shaft, it is reasonable to use, Carter & Kulhawy (1988) and AASHTO (2010) methods. In that case, applying the standardized classification criteria and the improved method for determing the boundary of the layers are more reasonable, since the value of K converges to 1, more closely.

3.1.2 Classification of the Residual soil with degree of hardness

Although the resistance of general soil such as sand might not be reflected in evaluating the bearing capcity of the drilled shaft socketed in weathered rock, it is reasonable to consider the resistance of the hardened residual soil that have the property of the high strength and resistance. So, a criterian which is applied to classifying the hardened residual soil. The hardened residual soil is located above the weathered rock and the properties of the strength and stiffness are higher than those of the general soil. The hardened residual soil also has the similar properties of cohesionless IGMs which are suggested by FHWA (1999). Therefore, it is suggested that the residual soil should be classified by applying the classification criteria of the cohesionless IGMs as shown in Table 3.6.

Table 3.6 Classification criteria of the residual soil

Subsurface layer	Quantitative criteria
Soft residual soil	• N < 50blows/30cm
Hard residual soil	• 50blows/30cm ≤ N < 50blows/15cm

FHWA (1999) describes that the IGMs is stronger and stiffer than the general soil, especially, the mobilized side friction resistance is as simiar as that of rock mass. So, the sid friction resistance of the hardened residual soil can clearly affect the bearing capacity of the drilled shaft. With classifying the hardened residual soil considering the side friction resistance of it, the bearing capacity of the drilled shaft is more reasonably evaluated.

3.2 Evaluating methods of the Bearing Capacity

3.2.1 Inclusion of the Bearing Capacity of the Hardened Residual Soil

The side friction resistance of the residual soil is generally neglected in evaluating the bearing capaciy of the drilled shaft socketed in rock mass , in Korea. However, the hardened residual soil, as mentioned in the Chapter 3.1.2, has the property of high strength and stiffness, and especially, the side friction resistance of the cohesionless IGMs which have similar

characteristics with the hardened soil is as high as the rock mass. Therefore, including the side friction resistance is required to evaluate the bearing capacity of drilled shaft, more reasonably.

In this Chapter, the effect of the side friction resistance of the hardened residual soil to the bearing capacity of drilled shaft is analyzed, is calculated by using the equation of cohesionless IGMs and that is included in the bearing capacity of drilled shaft.

■ Outline of the Analysis

- Targets : Drilled shafts socketed in weathered rock
- Evaluation method of the bearing capacity

In case 1, the standardized thickness of weathered rock that is suggested in the Chapter 3.1.1 is applied. The side friction resistance of residual soil is neglected, and the upper limit of the resistance suggested to general soil such as sand is applied to calculating the side friction resistance of weathered rock except for CFEM method which do not apply the reduction factor, since all of the RQD of the weathered rocks are less than 20%. The bearing capacity equations that suggested in the Chapter 2.2.1 are used to calculate the end bearing resistance of the drilled shafts.

In case 2, as mentioned above, the side friction resistance of the hardened residual soil which is calculated by using the equation of

cohesionless IGMs and that is included in the bearing capacity of drilled shaft. The equations to calculate the side friction resistance and the end bearing resistance are same to case 1.

Table 3.7 Comparison of evaluation methods (Case 1, Case 2)

Case No.	Subsurface layer	Evaluation method		Note
		Side	Base	
Case 1	**Residual soil**	-		***Neglect the side friction resistance***
	Weathered rock	The upper limit of soil bearing capacity f_{ult}=0.2MPa	The bearing capacity equation for rock mass	These are same to the resistances of case 1 calculated in the Chapter 3.1.1
Case 2	**Residual soil**	Cohesionless IGMs equation	-	***Include the side friction resistance***
	Weathered rock	The upper limit of soil bearing capacity f_{ult}=0.2MPa	The bearing capacity equation for rock mass	These are same to the resistances of case 1 calculated in the Chapter 3.1.1

■ Analysis Result

As shown in the Table 3.9, the bearing capacities of case 2 which include the side friction resistance of the hardened residual soil (Table 3.8) are larger than those of case 1. As mentioned in the Chapter 3.1.1, since the end bearing capacity which estimated by applying Carter & Kulhawy (1988) and AASHTO (2010) methods are small, the rate of increse of the bearing capacity is relatively small, as compared with the other methods.

Carter & Kulhawy (1988) and AASHTO (2010) methods which underestimate the bearing capacity evaluate the capacity more accurately, but CFEM (2006) and FHWA (1999) methods more overestimate the bearing capacity. (Table 3.10)

As mentioned in the Chapter 3.1.1., since the conservative design is very important for the drilled shaft, it is reasonable to use, Carter & Kulhawy (1988) and AASHTO (2010) methods. In that case, including the side friction resistance of the hardened residual soil is more reasonable, since the value of K converges to 1, more closely.

Table 3.8 Side friction resistance of the hardened residual soil

Pile No.	Side resistance (kN)	Pile No.	Side resistance (kN)
1	46	**7**	19
2	93	**8**	19
3	115	**9**	19
4	139	**10**	15
5	115	**11**	15
6	26	**Average**	57

Table 3.9 Comparison of ultimate bearing capacity (Case 1, Case 2)

Pile No.	Measured capacity (kN)	Predicted Capacity (kN)					
		C&K, AASHTO		CFEM		FHWA	
		Case 1	Case 2	Case 1	Case 2	Case 1	Case 2
1	16700	2076	2123	32050	32096	23132	23179
2	13000	1888	1981	29395	29488	21592	21685
3	12000	2202	2317	32243	32358	22582	22698
4	18500	4333	4472	30031	30170	21299	21438
5	*21330	4553	4668	30650	30765	20618	20733
6	2550	1506	1533	8056	8082	2874	2900
7	2130	819	837	7259	7278	4887	4906
8	1070	811	830	5143	5162	2761	2780
9	1100	810	829	4962	4981	2580	2599
10	*3100	1606	1621	10087	10102	2941	2956
11	2950	2361	2376	9375	9390	3717	3732
Mean rate of increase, %		2.7		0.3		0.5	

* Measured capcity which is estimated using extrapolated load-diplacement curve

Table 3.10 Comparison of Measured capacity/Predicted capacity

Pile No.	Measured capacity/Predicted capacity=K					
	C&K, AASHTO		CFEM		FHWA	
	Case 1	Case 2	Case 1	Case 2	Case 1	Case 2
1	8.04	7.87	0.521	0.520	0.722	0.720
2	6.89	6.56	0.442	0.441	0.602	0.599
3	5.45	5.18	0.372	0.371	0.531	0.529
4	4.27	4.14	0.616	0.613	0.869	0.863
5	4.69	4.57	0.696	0.693	1.03	1.03
6	1.69	1.66	0.317	0.316	0.887	0.879
7	2.60	2.54	0.293	0.293	0.436	0.434
8	1.32	1.29	0.208	0.207	0.388	0.385
9	1.36	1.33	0.222	0.221	0.426	0.423
10	1.93	1.91	0.307	0.307	1.05	1.05
11	1.25	1.24	0.315	0.314	0.794	0.790
Average	3.59	3.48	0.392	0.391	0.704	0.700

3.2.2 Determination of the Side Resistance Reduction Factor of Weathered Rock in Korea

The weathered rock is defined as the rock mass stratum which exhibits RQD is less than 20% according to the classifiction criteria of KEC that is selected in this study. However, the bearing capacity evaluation equations using the uniaxial unconfined strength such as Carter & Kulhawy (1988), FHWA (1999), AASHTO (2010) methods did not suggest the reduction factor of the side friction resistance in the range of RQD < 20%, so it is hard to evaluate the side friction resistance of the drilled shafts, reasonably. In that case, the upper limit of the side friction resistance suggested to general soil has been applied in Korea, which is very conservative method.

In this study, for more reasonable evaluation of the side friction resistance of the weathered rock, the reduction factor which was suggested in the ranage of RQD > 20% was extrapolated to 0% of RQD, and it was applied (Case 4). For verifying the reasonability of that method, the bearing capaciy which was calculated by using that method were compared with the case using the upper limit of the side friction resistance of the general soil (Case 2) and the side resistance of the cohesionless IGMs (Case 3), respectively.

■ Outline of the Analysis

- Targets : Drilled shafts socketed in weathered rock
- Evaluation method of the bearing capacity

In case 2, the standardized thickness of weathered rock that is suggested in the Chapter 3.1.1 is applied. The side friction resistance of residual soil is included, which is calculated by using the cohesionless IGMs equation. The upper limit of the resistance suggested to general soil such as sand is applied to calculating the side friction resistance of weathered rock except for CFEM method which do not apply the reduction factor, since all of the RQD of the weathered rocks are less than 20%. The bearing capacity equations that suggested in the Chapter 2.2.1 are used to calculate the end bearing resistance of the drilled shafts.

In case 3, the side friction resistance of the hardened residual soil which is calculated by using the equation of cohesionless IGMs and that is included in the bearing capacity of drilled shaft. The equations to calculate the side friction resistance and the end bearing resistance are same to case 2.

In case 4, , as mentioned above, the extrapolated reduction facor is applied to calculation of the side friction resistance of the drilled shaft. The side friction resistance of the hardened residual soil and the weathered rock are same to case 2.

Table 3.11 Comparison of evaluation methods (Case 2, Case 3, Case 4)

Case No.	Subsurface layer	Evaluation method		Note
		Side	Base	
Case 2	**Residual soil**	Cohesionless IGMs equation	-	Include the side friction resistance
	Weathered rock	The upper limit of soil bearing capacity f_{ult}=0.2MPa	The bearing capacity equation for rock mass	These are same to the resistances of case 1 calculated in the Chapter 3.1.1
Case 3	**Residual soil**	Cohesionless IGMs equation	-	***Cohesionless IGMs equation was applied to the weathered rock which is RQD <20%***
	Weathered rock		The bearing capacity equation for rock mass	
Case 4	**Residual soil**	Cohesionless IGMs equation	-	Same to Case 2
	Weathered rock	The bearing capacity equation for rock mass		***Extrapolated reduction factor was applied to the weathered rock which is RQD <20%***

■ Analysis Result

As shown in the Table 3.12, the side friction resistances of case 3 are larger than those of case 2. Therefore, the bearing capacities of the drilled shafts are increased in case 3, in which the side friction resistances of the weathered rock are calculated by using the cohesionless IGMs equation (Table 3.13). However, since CFEM (2006) method did not suggest the reduction factor, the calculated bearing capaicity in case 2 and case 3 are same.

In case of applying Carter & Kulhawy (1988) and AASHTO (2010) methods which underestimate the bearing capacity, the K values are increased in case 4. This is the result from the increased side friction resistances. But, because the bearing capacites were under estimated on a few piles (No. 8, No. 9), the additional verifications are required, and the resonable factor of safety or the resistance factor are considered (Table 3.14)

Table 3.12 Comparison of Side friction resistance of the weathered rock

Pile No.	Side resistance (kN)		Pile No.	Side resistance (kN)	
	Case 2	Case 3		Case 2	Case 3
1	1508	2455	**7**	804	1163
2	1319	2147	**8**	804	1175
3	1634	2652	**9**	804	1167
4	1477	2401	**10**	1583	2312
5	1696	2751	**11**	2337	3420
6	1483	2158	**Average**	53	

Table 3.13 Comparison of ultimate bearing capacity (Case 2, Case 3)

Pile No.	Measured capacity (kN)	Predicted Capacity (kN)					
		C&K, AASHTO		CFEM		FHWA	
		Case 2	Case 3	Case 2	Case 3	Case 2	Case 3
1	16700	2076	2123	32096		23179	24126
2	13000	1888	1981	29488		21685	22513
3	12000	2202	2317	32358		22698	23716
4	18500	4333	4472	30170		21438	22362
5	*21330	4553	4668	30765		20733	21788
6	2550	1506	1533	8082		2900	3576
7	2130	819	837	7278		4906	5264
8	1070	811	830	5162		2780	3150
9	1100	810	829	4981		2599	2961
10	*3100	1606	1621	10102		2956	3684
11	2950	2361	2376	9390		3732	4815
Mean rate of increase, %		39.5		0.00		11.0	

* Measured capcity which is estimated using extrapolated load-diplacement curve

Table 3.14 Comparison of Measured capacity/Predicted capacity

Pile No.	Measured capacity/Predicted capacity=K				
	C&K, AASHTO		CFEM	FHWA	
	Case 2	Case 3	Case 2, Case 3	Case 2	Case 3
1	7.87	5.44	0.520	0.720	0.692
2	6.56	4.63	0.441	0.599	0.577
3	5.18	3.60	0.371	0.529	0.506
4	4.14	3.43	0.613	0.863	0.827
5	4.57	3.73	0.693	1.03	0.98
6	1.66	1.15	0.316	0.879	0.713
7	2.54	1.78	0.293	0.434	0.405
8	1.29	**0.89**	0.207	0.385	0.340
9	1.33	**0.92**	0.221	0.423	0.371
10	1.91	1.32	0.307	1.05	0.84
11	1.24	0.85	0.314	0.790	0.613
Average	3.48	2.52	0.391	0.700	0.624

Applying the cohesionless IGMs equation to evaluation of the side friction resistance of weathered rock is more resonable than using the upper limit of the side friction resistance of the general soils. So, the bearing capacity in case 3 are tried to compare with those of in case 4 where the extrapolated reduction factor was applied.

As shown in the Table 3.15, the bearing capacities in case 4 are generally larger than those of case 3. However, the beraring capacities which were calculated by using Carter & Kulhawy (1988) method are decreased,

since the side friction resistance of the weathered rock were very underestimated, which used the extrapolated reduction factor.

As presented in Table 3.16, the K values of the bearing capacity estimated by using Carter & Kulhawy (1988) method are highly increased, because of the highly underestimated side friction resistance. On the other hand, when applying AASHTO (2010), FHWA (1999) method to calculating the side friction resistace of the weathered rock, the average K values in this case is less than that of case 3, because of the side resistance are generally increased.

Since several bearing capacities which were evaluated by using AASHTO (2010) method are underestimated, additional verifications should be required. However, because all of the K values are larger than 0.9, if the reasonable factor of safety or resistance factor are applied, the reasonable design of the drilled shafts could be performed. Consequently, applying AASHTO (2010) method to design of drilled shafts is the most reasonable, which means that the bearing capacity are generally underestimated and the K values converges to 1, more closely.

Table 3.15 Comparison of ultimate bearing capacity (Case 3, Case 4)

Pile No.	Measured capacity (kN)	Predicted Capacity (kN)						
		C&K		AASHTO		CFEM	FHWA	
		Case 3	Case 4	Case 3	Case 4	Case 3, Case 4	Case 3	Case 4
1	16700	3070	792	3070	4337	32096	24126	25393
2	13000	2808	816	2808	3918	29488	22513	23622
3	12000	3336	876	3336	4716	32358	23716	25096
4	18500	5396	3883	5396	8569	30170	22362	25535
5	*21330	5723	4144	5723	9622	30765	21788	25687
6	2550	2208	163	2208	2429	8082	3576	3797
7	2130	1196	87	1196	1167	7278	5264	5236
8	1070	1200	80	1200	1163	5162	3150	3113
9	1100	1192	79	1192	1163	4981	2961	2932
10	*3100	2349	186	2349	3155	10102	3684	4489
11	2950	3458	174	3458	2893	9390	4815	4249
Mean rate of increase, %		-74.1		25.9		0.00	5.55	

* Measured capcity which is estimated using extrapolated load-diplacement curve

Table 3.16 Comparison of Measured capacity/Predicted capacity

Pile No.	Measured capacity/Predicted capacity=K						
	C&K		,AASHTO		CFEM	FHWA	
	Case 3	Case 4	Case 3	Case 4	Case 3, Case 4	Case 3	Case 4
1	5.44	21.09	5.44	3.85	0.520	0.692	0.658
2	4.63	15.93	4.63	3.32	0.441	0.577	0.550
3	3.60	13.70	3.60	2.54	0.371	0.506	0.478
4	3.43	4.76	3.43	2.16	0.613	0.827	0.724
5	3.73	5.15	3.73	2.22	0.693	0.98	0.83
6	1.15	15.64	1.15	1.05	0.316	0.713	0.672
7	1.78	24.49	1.78	1.83	0.293	0.405	0.407
8	0.89	13.37	0.89	**0.92**	0.207	0.340	0.344
9	0.92	13.89	0.92	**0.95**	0.221	0.371	0.375
10	1.32	16.63	1.32	**0.98**	0.307	0.84	0.69
11	0.85	16.93	0.85	1.02	0.314	0.613	0.694
Average	2.52	14.69	2.52	1.89	0.391	0.624	0.584

3.2.3 Evaluation of Bearing Capacity when There is No Unconfiend Uniaxial Strength

The bearing capacity have been generally evaluated by using the upper limit of the resistance of general soil in Korea, when the unconfined uniaxial strength of the weathered rock could not be obtained, which is very conservative way supposing the rock mass to soil.

Therefore, for more reanoable and economical design, the rock mass properties such as the strength and the stiffness which are more bigger than those of the general soil are need to be considered. In this study, when there is no the unconfined uniaxial strength of the weathered rock, the weathered rock were supposed to be the hardened residual soil suggested in the Chapter 3.2.1 and the bearing capacity of the weathered rock were calculated by using the cohesionless IGMs equation in order to evaluate the bearing capacity of drilled shaft more reasonably.

■ Outline of the Analysis

- Targets : Drilled shafts socketed in weathered rock
- Evaluation method of the bearing capacity

In case 5, the standardized thickness of weathered rock that is suggested in the Chapter 3.1.1 is applied. The side friction resistance of residual soil is included, which is calculated by using the cohesionless

IGMs equation. The upper limit of the resistance suggested to general soil such as sand is applied to calculating the side friction resistance of weathered rock.

In case 6, as mentioned above, the side resistance and the end bearing resistance of the weathered rock were estimated by using the cohesionless IGMs equation. The side resistance of the hardened residual soil are same to those of in case 5.

Table 3.17 Comparison of evaluation methods (Case 5, Case 6)

Case No.	Subsurface layer	Evaluation method		Note
		Side	Base	
Case 5	Residual soil	Cohesionless IGMs equation	-	Include the side friction resistance
	Weathered rock	***The upper limit of soil bearing capacity*** f_{ult}***=0.2MPa***	***The upper limit of soil bearing capacity*** q_{baset}***=3MPa***	These are same to the resistances of case 1 calculated in the Chapter 3.1.1
Case 6	Residual soil	Cohesionless IGMs equation	-	Same to Case 5
	Weathered rock	***Cohesionless IGMs equation***		FHWA (1999)

■ Analysis Result

As shown in the Table 3.15, the average bearing capacity of case 6 are increased about 124% compared to that of case 5, and the bearing capacities are estimated in the case 6 are closer to the measured capacity than those of case 5. However, applying the cohesionless IGMs equation, the bearing capacities are overestimated on six drilled shafts. Especially, for several drilled shafts (pile No.6, No.8, No.9, No.11), the bearing capacities are excessively overestimated so, additional verifications should be reqired. Therefore, when the unconfined uniaxial strength of the weathered rock could be obtained, applying the equation that were suggested to the cohesionless IGMs are not reasonable but using the upper limit of the bearing capacity that were suggested to the general soil are reasonable for the purpose of the conservative design. However, although the upper limit of the bearing capacity is applied, K value is less than 1 in some cases, which means unsafe design. So, the reasonable factor of safety or the resistance factor are need to be applied.

Table 3.18 Comparison of ultimate bearing capacity and K value (Case 5, Case 6)

Pile No.	Measured capacity (kN)	Predicted Capacity (kN)	
		Case 5	Case 6
1	16700	3911	10740
2	13000	3768	10470
3	12000	4105	10969
4	18500	3972	10756
5	*21330	4168	11050
6	2550	1886	3280
7	2130	1200	2260
8	1070	1200	2291
9	1100	1200	2271
10	*3100	1975	3428
11	2950	2729	4545
Mean rate of increase, %		122	

Pile No.	Measured capacity/Predicted capacity=K	
	Case 5	Case 6
1	4.27	1.55
2	3.45	1.24
3	2.92	1.09
4	4.66	1.72
5	5.12	1.93
6	1.35	**0.78**
7	1.78	**0.94**
8	0.89	**0.47**
9	0.92	**0.48**
10	1.57	**0.90**
11	1.08	**0.65**
Average	2.55	1.07

* Measured capcity which is estimated using extrapolated load-diplacement curve

Chapter 4 Conclusions

In this study, the existed classification methods of the weathered zone and the evaluating methods for the drilled shafts socketed in the weathered zone in Korea were reviewed in order to estimate the bearing capacities of the drilled shafts more reasonably, which were based on the quantitative information exhibited in subsurface investigation report. And then the improved methods for the more reasonable evaluation of the bearing capacity were suggested and verified by using the pile load test results.

At first, the classification criteria were suggested, which could be applied to determination of the socket depth that was very important factor for evaluating the bearing capacity of the drilled shafts. The upper boundary and low boundary of the weathered rock were defined as N value (50blows/15cm) and RQD (20%), respectively. In addition, in case of determining the boundary between the residual soil and the weathered rock, using the interpolated N value was recommended.

As based on the criteria and determination method for the layer boundaries which were mentioned above, the thickness of the hardened residual soil and the weathered rock were determined, and then the bearing capacity of the drilled shaft was estimated. For calculating the resistance of the hardened residual soil which was generally not considered, the evaluating equation for the cohesionless IGMs which was suggested by FHWA (1999) was applied. And also the side friction resistance of weathered rock which

were in the range of RQD <20% were evaluated by using the extrapolated reduction factor. In addition, the upper limit of bearing capacity of the general soil and the evaluating equation for the cohesionless IGMs were applied to evaluating the bearing capacity of the weathered rock whose unconfined uniaxial strength could not be obtained.

As a result of evaluating the bearing capacity of the drilled shafts, if the unconfined uniaxial strength of the weathered rock could be obtained, applying AASHTO (2010) method to estimating the bearing capacity with considering the resistance of the hardened residual soil, modifying the thickness of the layer and applying the extrapolated reduction factor was the most reasonable. However, since the several drilled shafts were overestimated, the reasonable factor of safety or the resistance factor were needed to apply. If the unconfined uniaxial strength of the weathered rock could not be obtained, using the upper limit of the resistance of the general soil was more reasonable for the purpose of a conservative design. Also, in that case, since the several drilled shafts were overestimated, the reasonable factor of safety or the resistance factor must be applied.

Bibliography

서울특별시 (2006), 지반조사편람

일본토목학회 (2002), 도로교 시방서

이정훈, 김원철, 서덕동, 남현우, (2006), “IGM에 근입된 현장타설말뚝의 설계사례 연구”, 한국지반환경공학회 논문집, 제7권, 제1호, pp. 19-28.

장학성 (2009), “대구경 현장타설말뚝의 설계”, 유신기술회보 제 16호, pp. 194-206.

정성준, 김성렬, 소진만, 김명모 (2007), “풍화암에 근입된 현장타설말뚝 지지력 공식의 신뢰성 분석”, 한국지반공학회 논문집 제23권, 제2호 2월, pp. 85-92.

한국도로공사 (2009), 도로설계요령

한국지반공학회 (2009), 구조물 기초 설계기준 해설

한국철도시설공단 (2011), 철도건설공사 전문시방서

AASHTO. (2010), *AASHTO LRFD Bridge Construction Specifications. (3rd Ed.)*, American Association of State Highway and Transportation Officials Inc., Washington, DC.

Carter, J. P., and Kulhawy, F. H. (1988), *Analysis and Design of Drilled Shaft Foundations Socketed into Rock. Report EL-5918, Electric Power Research Institute, Palo Alto, CA.*

CGS (2006), *Canadian Foundation Engineering Manual*. (4th Ed.), Canadian Geotechincal Society.

Hoek, E., (1983), "*Strength of Jointed Rock Masses", Geotechnique, Vol. 33, No. 3, September, pp. 187 – 223.*

Ladanyi, B., DuFour, R., LaRocque, G. S., Samson, L., and Scott, J. S. (1974), "Report Of the Subcommittee on Foundations and Near-Surface Structures", Canadian Advisory Committee on Rock Mechanics, Canadian Geotechnical Society, pp.55

Mayne, P. W., and Harris, D. E. (1993), "Axial Load-Displacement Behavior of Drilled Shaft Foundations in Piedmont Residuum", FHWA Reference No. 41-3—2175, Georgia Tech Research Corporation, Geotechnical Engineering Division, Georgia Institute of Technology, School of Civil Engineering, Atlanta, February.

O'Neill, M. W., Townsend, F. C., Hassan, K. H., Buller, A., Chan, P. S. (1996), "Load Transfer for Drilled Shafts in Intermediate Geomaterials", Publication No. FHWA-RD-95-171, Federal Highway Administration, Office of Engineering and Highway Operations R & D, McLean, VA, November, 184 pp.

O'Neil, M. W., and Reese, L. C. (1999). Drilled shafts : *Construction Procedures and Design Methods. Report prepared for U. S. Department of Transportation and Federal Highway Administration, Report No.* FHWA-IF-99-025

Terzaghi, K. and Peck, R. B. (1948), *Soil Mechanics in Engineering Practice, John Wiley and Sons, Inc., New York, NY*

초 록

풍화토와 풍화암으로 구성되는 풍화대는 현장타설말뚝의 지지력이 발현되는 주요 지지층으로서 정량적인 지반정보에 기반하여 정확하고 합리적인 지지력 산정이 중요하다.

하지만, 풍화대는 토사와 기반암의 전이상태인 지층으로 그 경계를 명확히 구분할 수 없고, 풍화암의 암 코어 확보가 어렵기 때문에 지지력 산정에 필요한 말뚝의 암반 근입깊이와 강도정수를 객관적이고 합리적으로 평가할 수 없다. 또한, 풍화암의 지지력 산정 시 설계자에 따라 토사에 제안된 지지력의 상한값 또는 국외에서 제안된 암반 지지력 식을 혼용하고 있어 합리적인 지지력 산정방법에 대한 연구가 필요하다.

본 연구에서는 지반 조사보고서의 정량적인 정보에 기반하여 객관적으로 설계정수를 결정하고, 기존 지지력 산정식의 문제점을 개선하여 합리적으로 지지력을 산정할 수 있는 표준화된 방법을 제시하고자 하였다.

먼저, 국내 현황과 문제점을 분석하기 위해 주요기관에서 제시하고 있는 정량적인 풍화대 분류기준과 일반적으로 적용하는 지지력 산정방법을 검토하였다.

다음으로, 풍화대의 지지력을 보다 합리적으로 평가하기 위해, 분석된 문제점에 대한 개선 방안을 제안하였다. 개선된 지지력 산정 방법의 효과를 검증하기 위해 국내 풍화암에 근입된 현장타설말뚝 11본에 대하여 개선된 방법으로 지지력을 산정하였으며, 이를 재하시험 결과와 비교 평가하였다.

재하시험 결과와 비교 평가한 결과, 개선된 방안을 적용하였을 때,

일축압축강도 정보가 존재하는 풍화암의 경우, AASHTO (2010) 방법을 적용하는 것이 지지력을 안전측으로 평가하며 재하시험으로 측정된 지지력과 가장 유사하게 지지력을 산정하는 것으로 평가되었다. 일축압축강도 정보가 존재하지 않는 풍화암의 경우, 안전측 설계를 위해 토사 지지력의 상한값을 적용하는 것이 적절한 것으로 평가되었다.

본 연구를 통해, 풍화대 분류 및 지층 경계 결정과정에서부터 현장타설말뚝의 연직 극한지지력을 지반조사보고서의 정량적인 정보에 기반하여 합리적으로 산정할 수 있는 방법 및 과정을 제안함으로써, 향후 설계단계에서 숙련되지 않은 설계자들도 보다 객관적이고 합리적으로 풍화대에 근입된 현장타설말뚝의 지지력을 산정할 수 있을 것으로 기대된다.

주요어 : 풍화대, 현장타설말뚝, 극한지지력, 체계화

학 번 : 2013-20934

Appendices

Table A.1 Side friction resistance and end bearing resistance in case 0

Pile No.	Ultimate bearing capacity (kN)					
	C&K, AASHTO		CFEM		FHWA	
	Side	Base	Side	Base	Side	Base
1	1445	568	9991	21624	1445	21624
2	1257	568	8688	20273	1257	20273
3	1351	568	9340	20948	1351	20948
4	1194	2856	8254	19822	1194	19822
5	1068	2856	7385	18921	1068	18921
6	1508	23	6777	1391	1508	1391
7	754	14	2978	4083	754	4083
8	754	7	2988	1957	754	1957
9	754	6	2988	1776	754	1776
10	1508	23	8314	1357	1508	1357
11	2262	23	7737	1380	2262	1380

Table A.2 Side friction resistance and end bearing resistance in case 1

Pile No.	Ultimate bearing capacity (kN)					
	C&K, AASHTO		CFEM		FHWA	
	Side	Base	Side	Base	Side	Base
1	1508	568	10426	21624	1508	21624
2	1319	568	9122	20273	1319	20273
3	1634	568	11295	20948	1634	20948
4	1477	2856	10208	19822	1477	19822
5	1696	2856	11729	18921	1696	18921
6	1483	23	6664	1391	1483	1391
7	804	14	3177	4083	804	4083
8	804	7	3187	1957	804	1957
9	804	6	3187	1776	804	1776
10	1583	23	8730	1357	1583	1357
11	2337	23	7995	1380	2337	1380

Table A.3 Side friction resistance and end bearing resistance in case 2

Pile No.	Ultimate bearing capacity (kN)					
	C&K, AASHTO		CFEM		FHWA	
	Side	Base	Side	Base	Side	Base
1	1554	568	10472	21624	1554	21624
2	1412	568	9215	20273	1412	20273
3	1749	568	11410	20948	1749	20948
4	1616	2856	10347	19822	1616	19822
5	1812	2856	11844	18921	1812	18921
6	1509	23	6691	1391	1509	1391
7	823	14	3195	4083	823	4083
8	823	7	3206	1957	823	1957
9	823	6	3205	1776	823	1776
10	1598	23	8745	1357	1598	1357
11	2352	23	8010	1380	2352	1380

Table A.4 Side friction resistance and end bearing resistance in case 3

Pile No.	Ultimate bearing capacity (kN)					
	C&K, AASHTO		CFEM		FHWA	
	Side	Base	Side	Base	Side	Base
1	2502	568	10472	21624	2502	21624
2	2240	568	9215	20273	2240	20273
3	2768	568	11410	20948	2768	20948
4	2539	2856	10347	19822	2539	19822
5	2866	2856	11844	18921	2866	18921
6	2184	23	6691	1391	2184	1391
7	1182	14	3195	4083	1182	4083
8	1194	7	3206	1957	1194	1957
9	1186	6	3205	1776	1186	1776
10	2327	23	8745	1357	2327	1357
11	3435	23	8010	1380	3435	1380

Table A.5 Side friction resistance and end bearing resistance in case 4

Pile No.	Ultimate bearing capacity (kN)							
	C&K		AASHTO		CFEM		FHWA	
	Side	Base	Side	Base	Side	Base	Side	Base
1	224	568	3768	568	10472	21624	1554	21624
2	248	568	3350	568	9215	20273	1412	20273
3	308	568	4148	568	11410	20948	1749	20948
4	1027	2856	5713	2856	10347	19822	1616	19822
5	1288	2856	6766	2856	11844	18921	1812	18921
6	140	23	2406	23	6691	1391	1509	1391
7	73	14	1153	14	3195	4083	823	4083
8	73	7	1157	7	3206	1957	823	1957
9	73	6	1156	6	3205	1776	823	1776
10	164	23	3132	23	8745	1357	1598	1357
11	151	23	2869	23	8010	1380	2352	1380

Table A.6 Side friction resistance and end bearing resistance in case 5 and case 6

Pile No.	Ultimate bearing capacity (kN)			
	Case 5		Case 6	
	Side	Base	Side	Base
1	1554	3864	2502	10693
2	1412	3676	2240	10377
3	1749	3990	2768	10854
4	1616	3833	2539	10617
5	1812	4053	2866	10935
6	1509	1860	2184	3253
7	823	1181	1182	2241
8	823	1181	1194	2272
9	823	1181	1186	2252
10	1598	1960	2327	3413
11	2352	2714	3435	4529

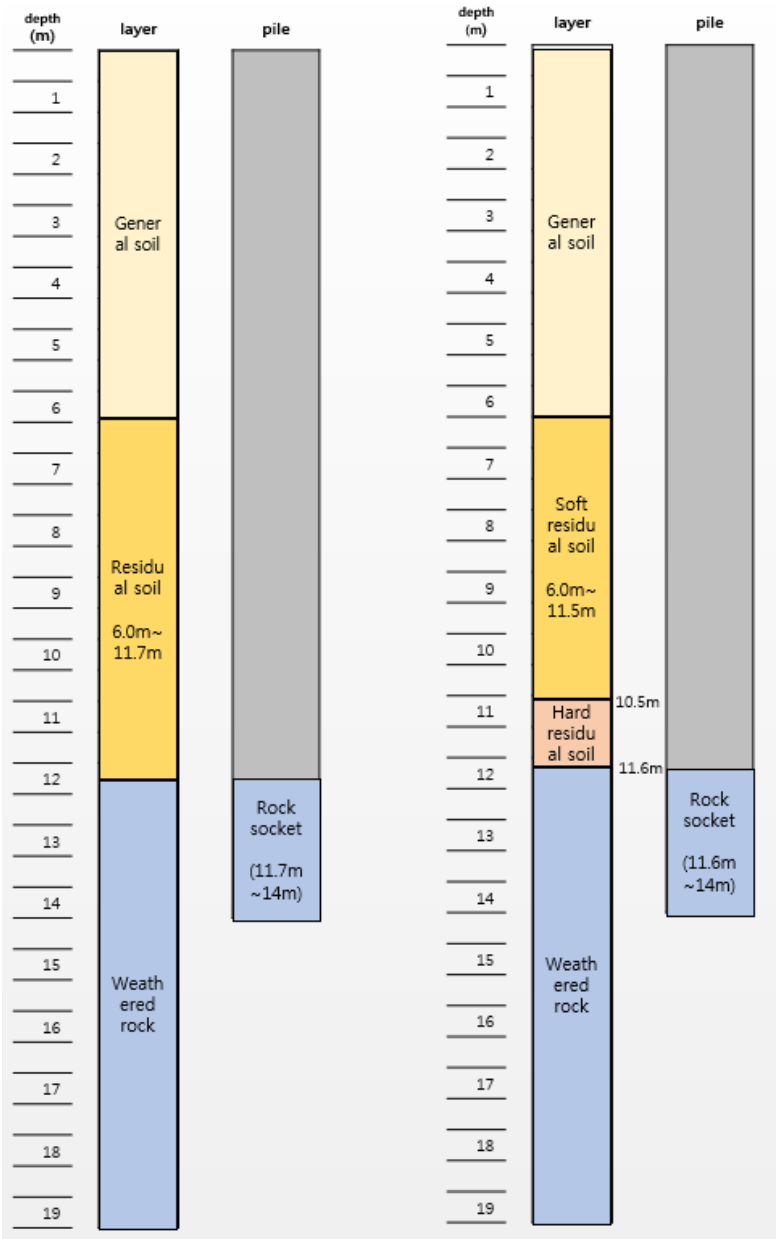

Figure A.1 Section view of the subsurface layer for pile No. 1 before and after modification

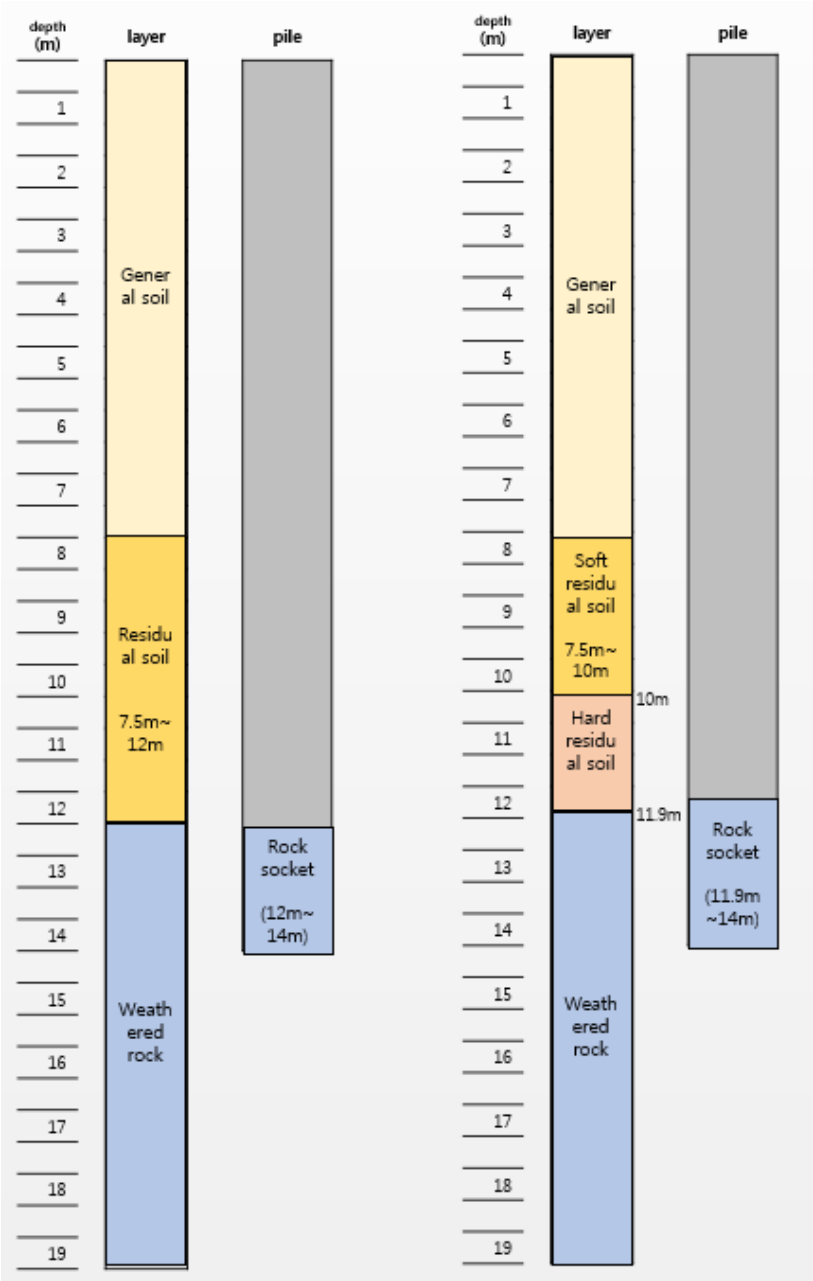

Figure A.2 Section view of the subsurface layer for pile No. 2 before and after Modification

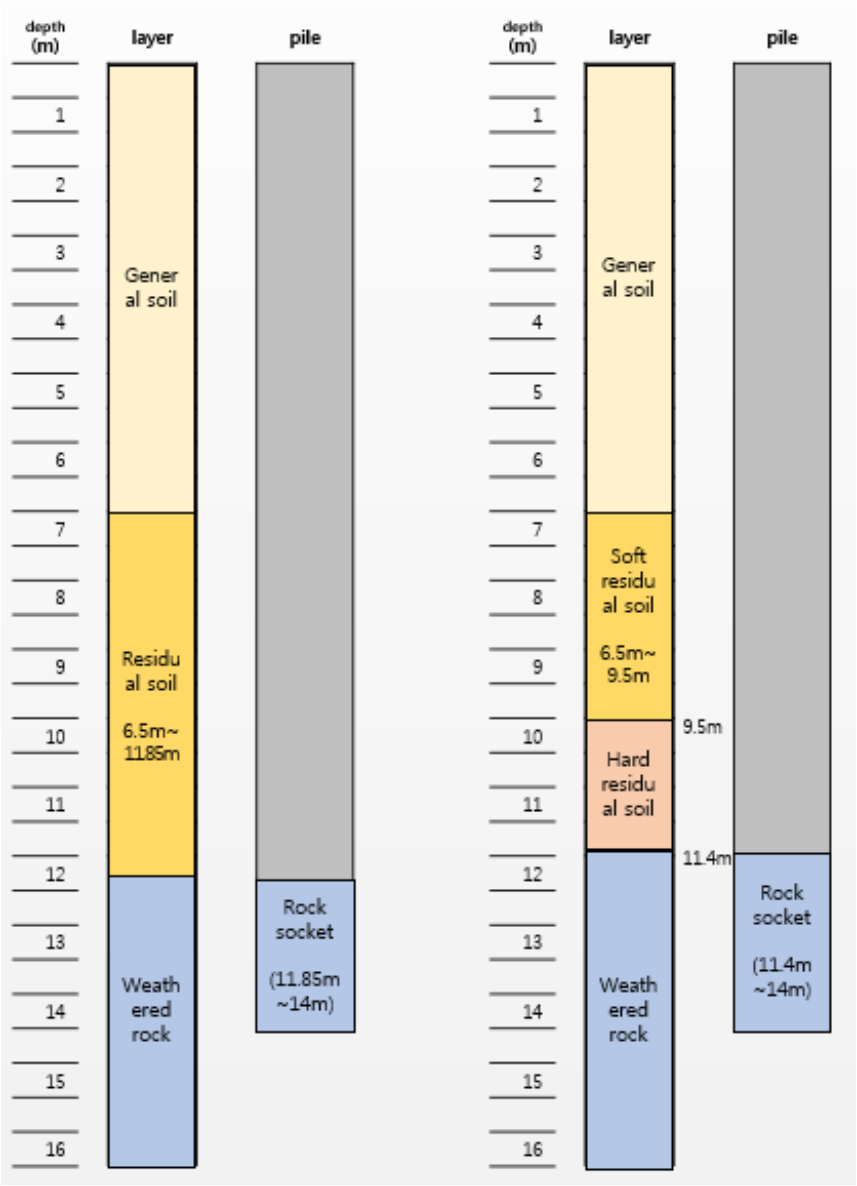

Figure A.3 Section view of the subsurface layer for pile No. 3 before and after Modification

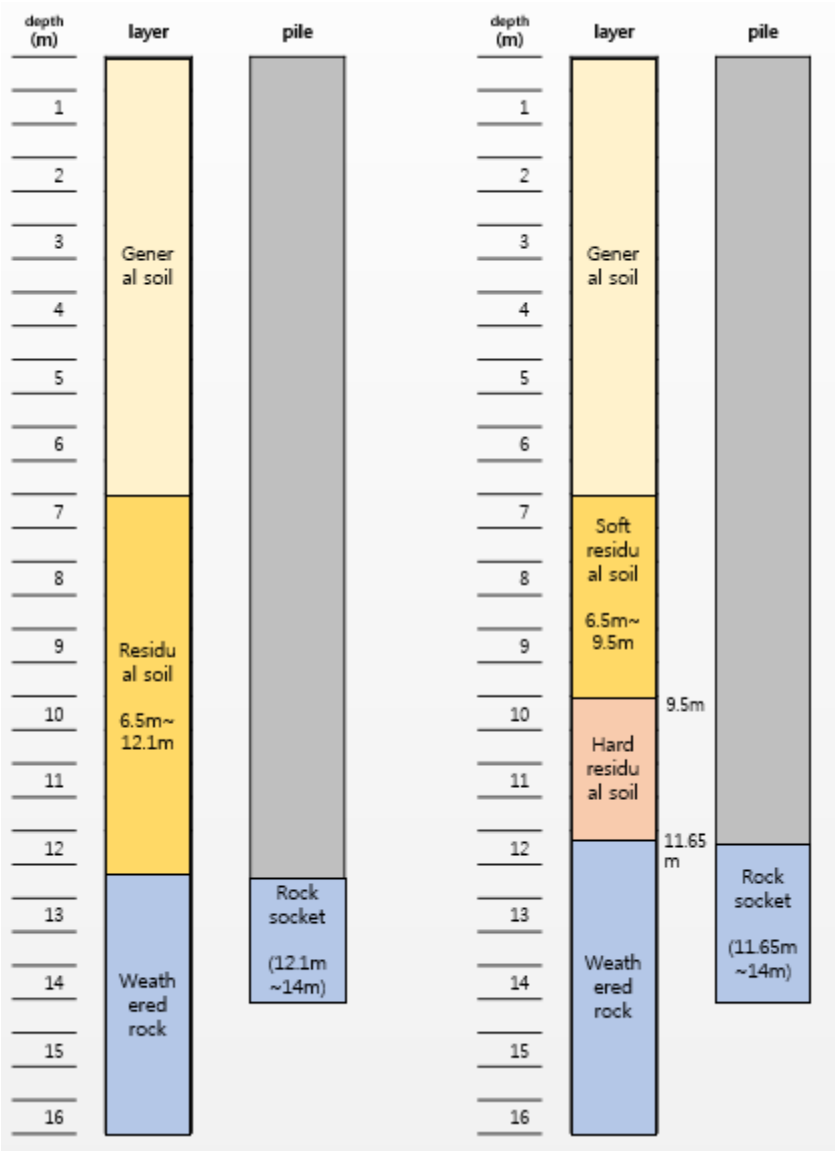

Figure A.4 Section view of the subsurface layer for pile No. 4 before and after Modification

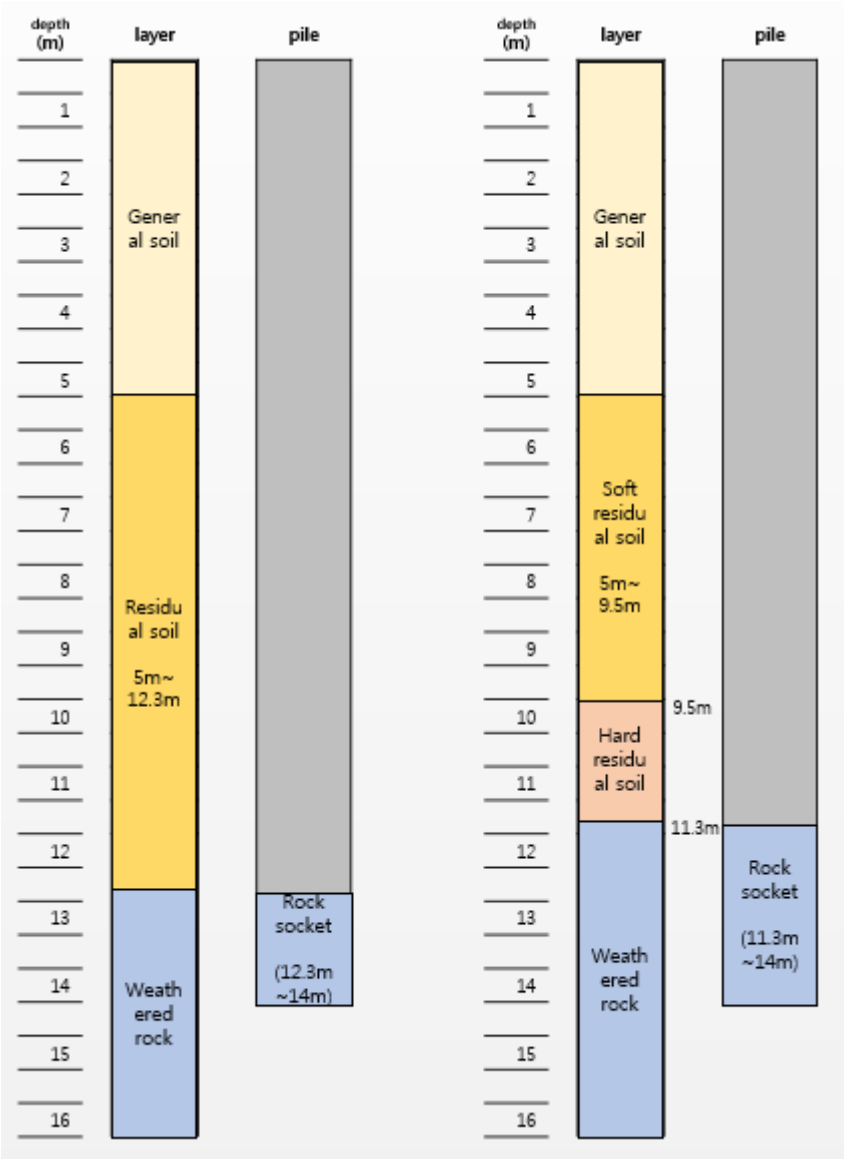

Figure A.5 Section view of the subsurface layer for pile No. 5 before and after Modification

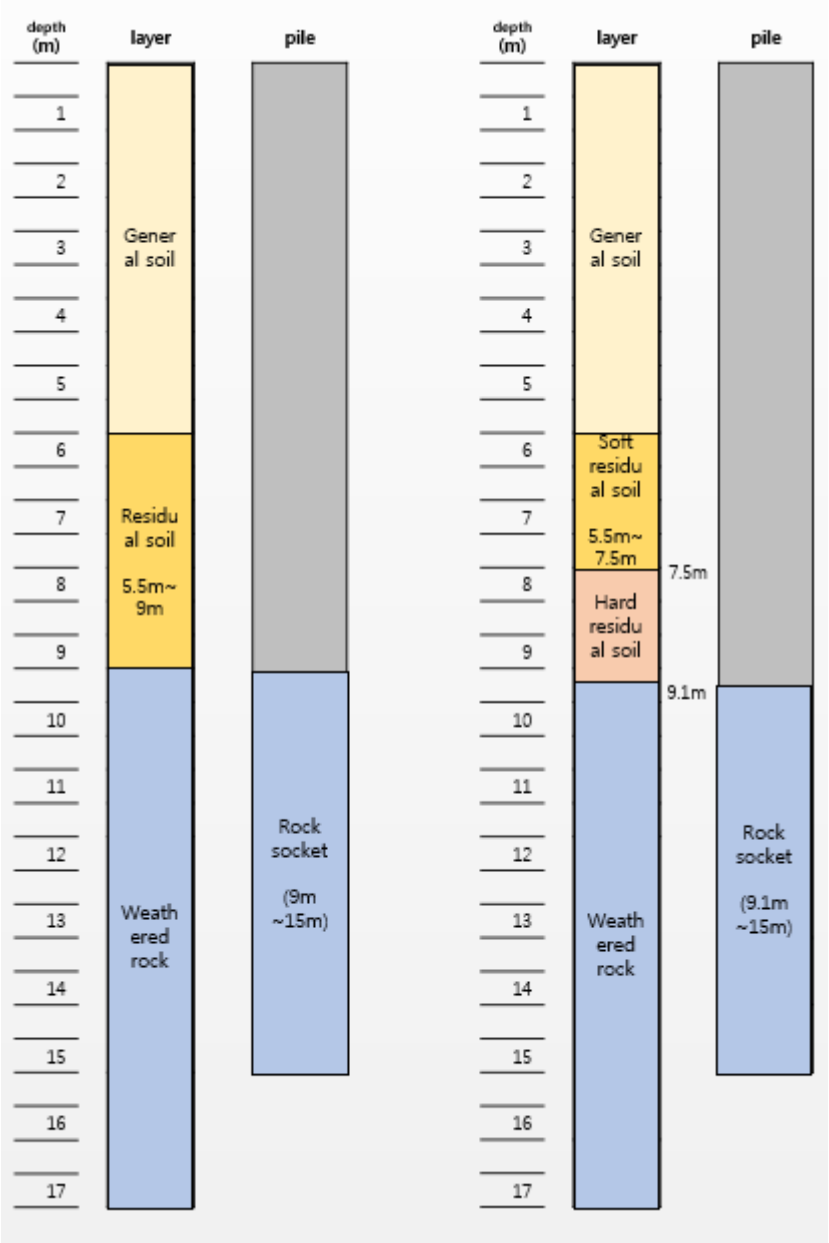

Figure A.6 Section view of the subsurface layer for pile No. 6 before and after Modification

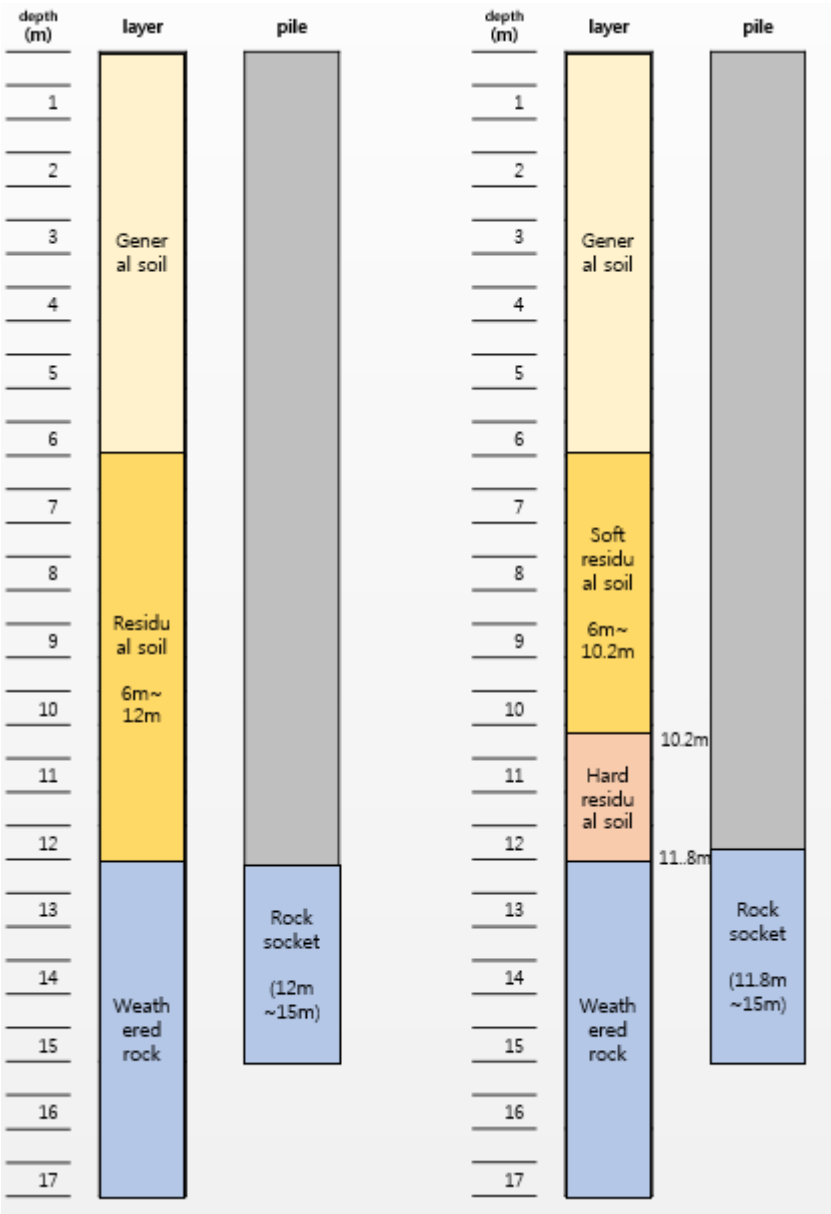

Figure A.7 Section view of the subsurface layer for pile No. 7 before and after Modification

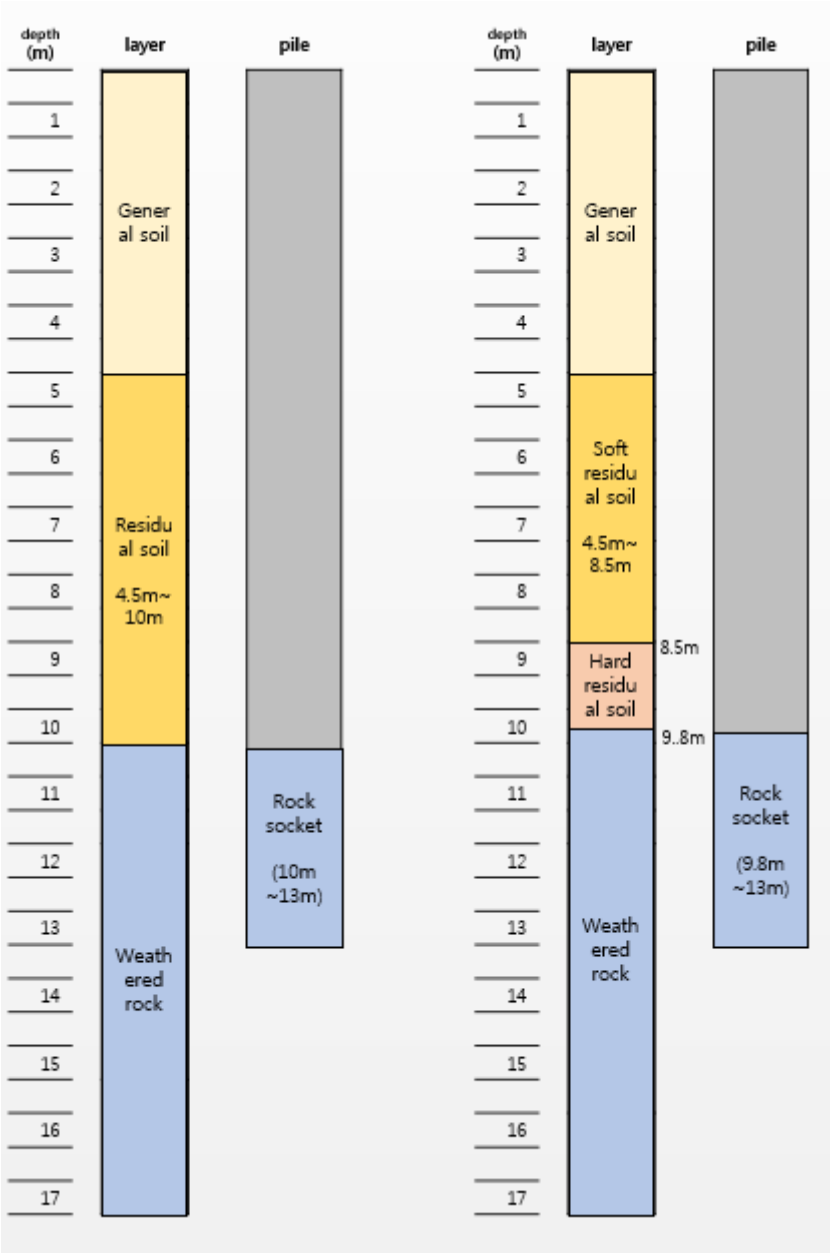

Figure A.8 Section view of the subsurface layer for pile No. 8 before and after Modification

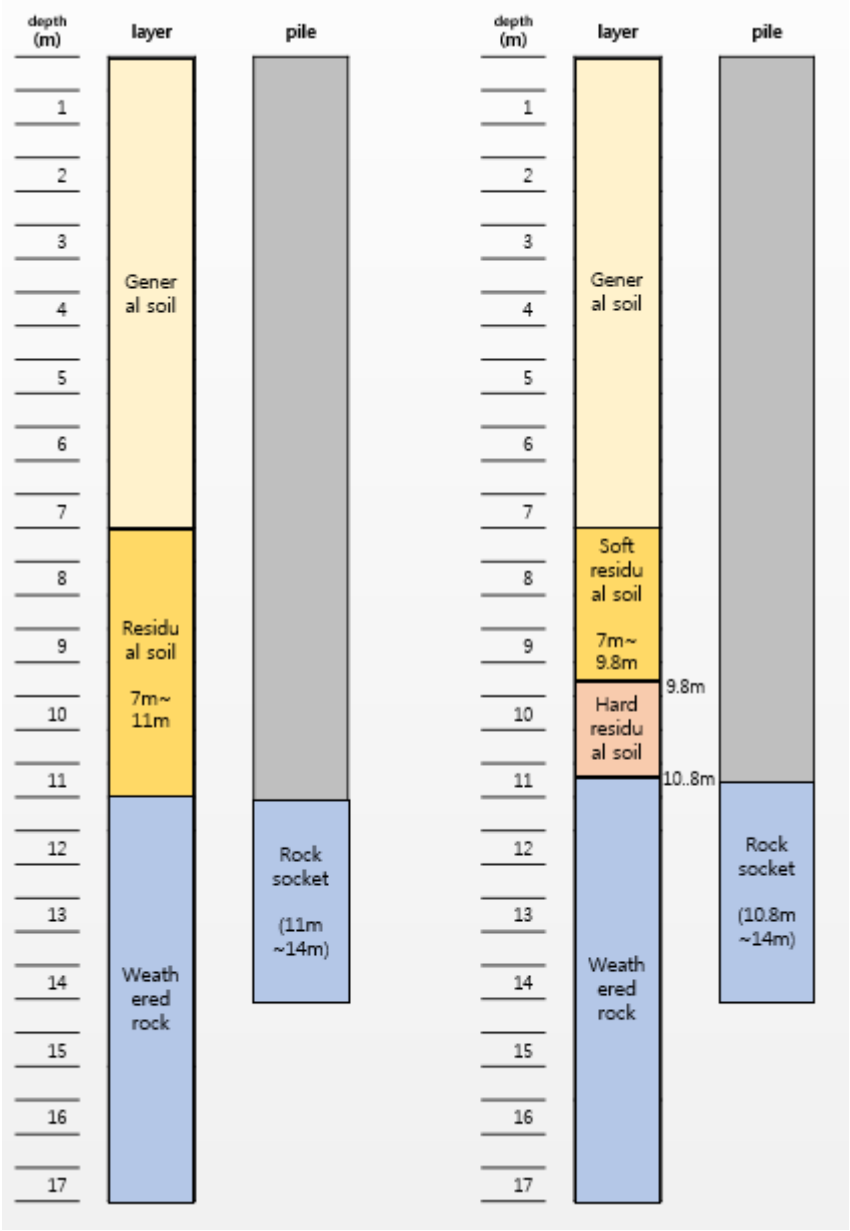

Figure A.9 Section view of the subsurface layer for pile No. 9 before and after Modification

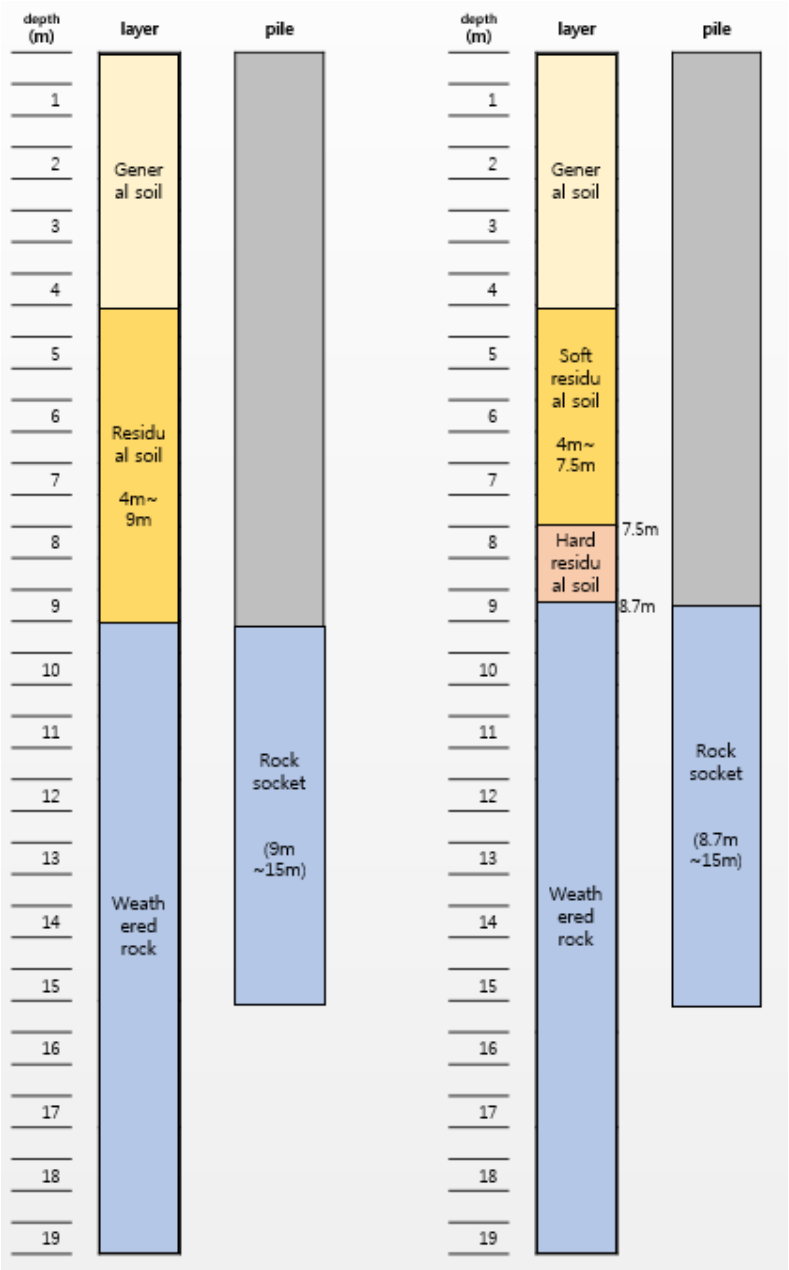

Figure A.10 Section view of the subsurface layer for pile No. 10 before and after modification

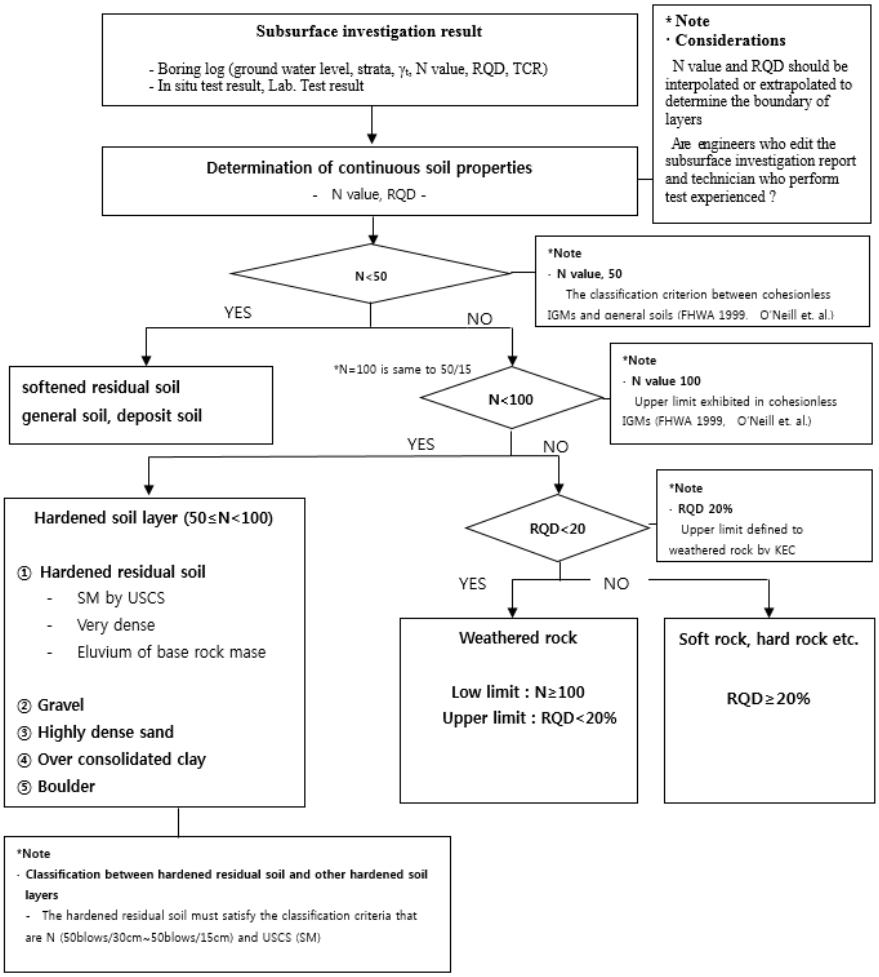

Figure A.11 Flow chart for determining the weathered zone

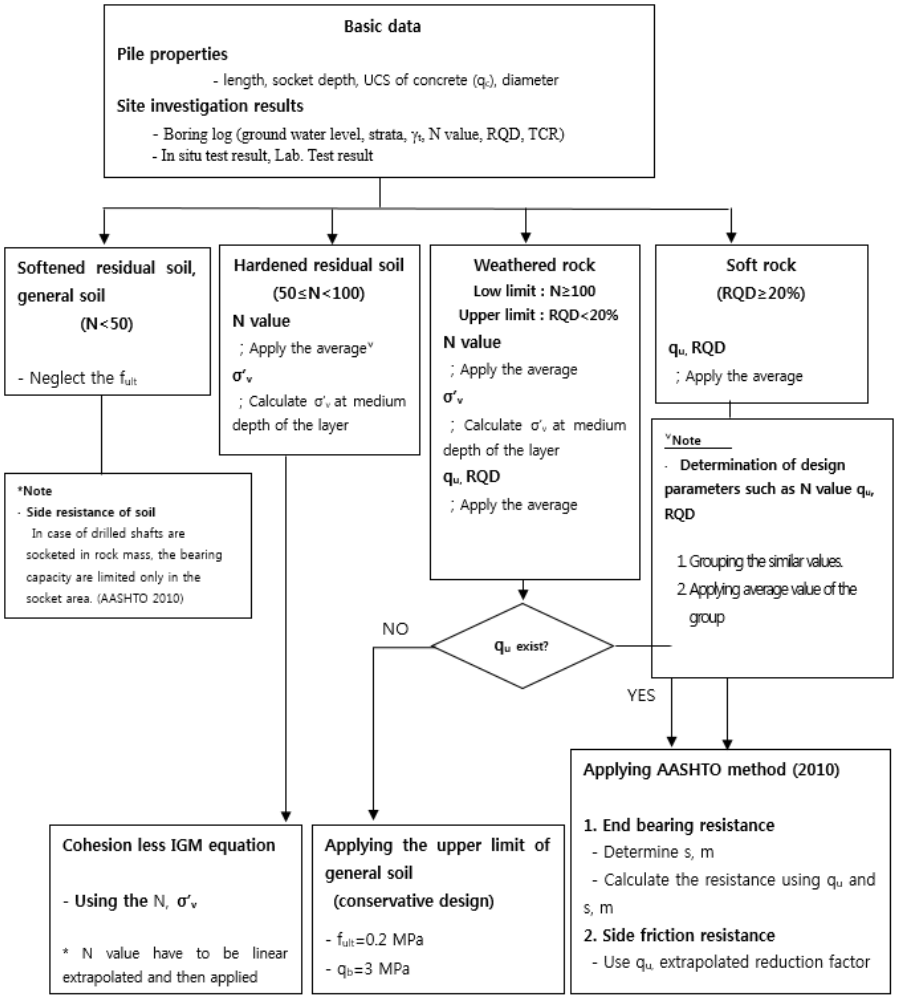

Figure A.12 Flow chart for evaluating the bearing capacity of the drilled shaft socketed in weathered zone