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Foundation Design Practice for Highrise Buildings in Korea

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Abstract

It is common for tall buildings in Korea to have a ground response that is highly sensitive to the behavior of the structure. Therefore, the geology of the ground needs to be carefully assessed and considered in the design process to accurately predict the performance of the foundation system. This paper sets out a systematic design approach and ground investigation methodology for the soil conditions frequently encountered in Korea. Various foundation design methods are introduced along with several case studies conducted in Korea.

Keywords: High rise building foundation, Ground in Korea, Design procedure, Ground investigation, Design methods

1. Introduction

The design of the foundation and building structures are highly correlated, especially in the case of high-rise buildings wherein the behavior of the foundation is largely influenced by the performance of the building. This paper presents a systematic design procedure as a guideline to the interactive work conducted by geotechnical designers and structural designers during the designing of high-rise buildings. Rock formations are generally used as the founding ground in Korea. In this paper, the rock classification locally adopted in Korea is introduced and compared to the internationally recognized classification system to help understand the local terminology. Ground investigation methods and interpretation of data for each classified rock type are presented. Three types of foundations, namely, raft, pile, and piled raft foundations, and the corresponding design sequences are introduced. The design methods for the evaluation of the soil-bearing capacity and vertical settlement – from simple methods to rigorous numerical analysis – are introduced.

2. General Comprehension of Foundation Design for Tall Buildings

2.1. General overview

Collaboration between the geotechnical designer and structural designer is important in the foundation design of tall buildings to ensure that the foundation design is appropriately captured in the structural design and that a wide range of loading conditions are adequately assessed in the geotechnical design (Kim et al., 2008).

In particular, understanding of soil-structure interactions is essential for a performance-based tall building design.

Three main design components are considered in the building foundation design. (Fig 2.1).

The three major considerations for foundation design are as follows:

- Geotechnical capacity
- Structural capacity
- Vertical and lateral movement (serviceability)

The geotechnical capacity and the structural capacity can be evaluated by either of the following approaches:

- Factor of Safety Design
- Limit State Design

The vertical and lateral movement should be less than the allowable deformation for the structural and adequate building performance.

2.2. Design process

In general practice, the foundation design follows the procedures illustrated below (Fig. 2.2).

However, generally, it is preferred to conduct foundation

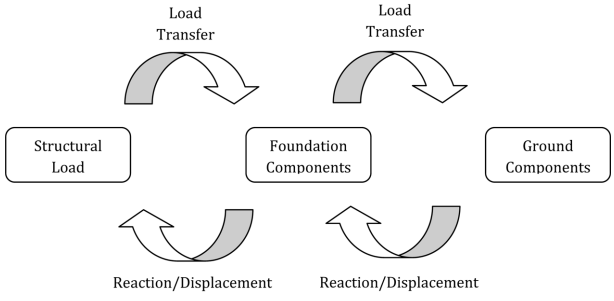


Figure 2.1. Design Components in Foundation Design.

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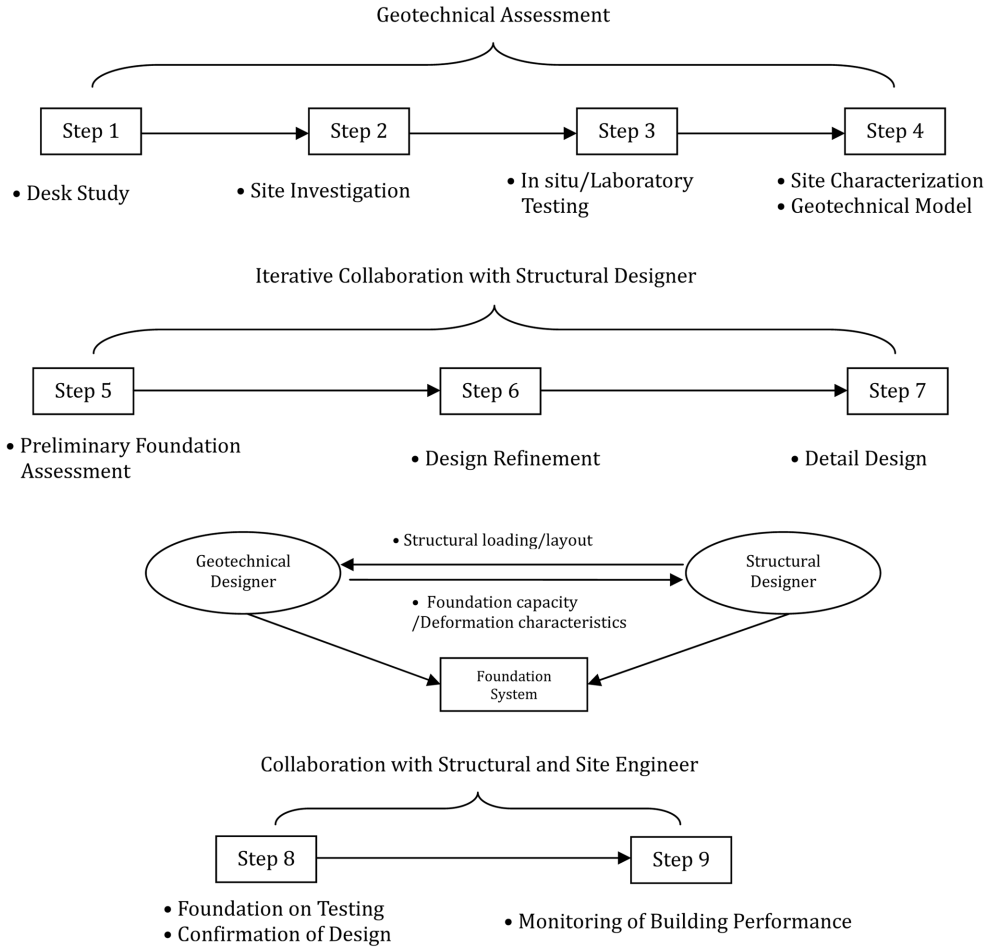


Figure 2.2. Steps in Foundation Design.

testing in Step 8 in the earlier design stage to reduce the geotechnical uncertainties. This helps improve the efficiency of design and constructability, especially in the case of pile foundations.

3. Typical Founding Ground Conditions in Korea

3.1. Founding rock characteristics

The ground in Korea generally comprises transported soil laid over residual soil layer underlain by weathered rock. The degree of weathering in rock profiles varies, with the weathering typically decreasing with depth.

In the design of tall buildings in Korea, the soil layer is seldom used as a founding ground for shallow foundations or as the end-bearing toe for pile foundations.

The typical rock characteristics in Korea are as follows:

- The rock strength generally decreases closer to the surface owing to the influence of weathering
- The rock mass frequently contains joints and sheared and crushed zones resulting from various geological activities, including the movement of faults. This results in local weakening and an increase in weathered zones within the rock profile.

In Korea, the types of rock are classified from various geotechnical aspects by several public organizations.

The rock classification summarized in Table 3.1 is primarily based on rock core strength.

Local classification and terminology of rocks used in Korea are different from those typically used in international practice. Therefore, it is recommended to use internationally recognized rock classification along with the local standards when conducting foundation design in Korea. Table 3.2 lists the rock classification by ISRM (1981) based on the uniaxial rock strength.

Alternatively, the rock sample can be described based on the degree of weathering, and this is often useful in practice owing to the simplicity of this classification. The classification for weathered rock (ISRM, 1981) is rearranged in Table 3.3.

3.2. Geotechnical parameters of rock for foundation design

In the design of tall buildings in Korea, the rock forma-

Table 3.1. Rock Classification in Korea (summarized)

Rock description	Korea Engineering & Consulting Association (2004)	Seoul City (2006)
Weathered Rock	$q_u < 50$ kg/cm^2 SPT(N) 50/10	$q_u < 100$ kg/cm^2 SPT(N) 50/10
Soft Rock	< 300	100-500
Moderate Rock	300-800	500-1000
Hard Rock	800-1500	1000-1500
Extremely Hard Rock	1500-1800	> 1500

Note: q_u, Unconfined Compressive Strength
SPT(N), Number of Standard Penetrometer Test

Table 3.2 Classification of Rock Material Strengths (summarized), ISRM (1981)

Grade	Description	Compressive Strength
R6	Extremely Strong Rock	> 250 MPa
R5	Very Strong Rock	100-250
R4	Strong Rock	50-100
R3	Medium Weak Rock	25-50
R2	Weak Rock	5-25
R1	Very Weak Rock	1-5
R0	Extremely Weak Rock	0.25-1

Table 3.3. Classifications for Weathered Rock (summarized), ISRM (1981)

Item	Fresh (F)	Slightly Weathered (SW)	Moderately Weathered (MW)	Highly Weathered (HW)	Completely Weathered (CW)
Color	No visible sign	Slight sign	Discoloration throughout	Discoloration	Discoloration
Decomposition	No visible sign	No visible sign	Weaker minerals decomposed	Most minerals somewhat decomposed	Decomposed to soil, but fabric/structure preserved
Field features	Ringing sound under hammer impact	Similar to [F]	Cores cannot be broken by hand or scraped by knife	Cores can be broken by hand with effort or can be shaved by knife	Specimen can be easily crumbled or penetrated

tion is generally considered to bear most of the vertical building load. However, the ground condition, including the soil within the critical pile length of typically 10D, plays an important role in offering the lateral resistance. Additionally, in a piled raft system, the soil layer below the raft bears a portion of the vertical load and the overturning moment from the building; hence, the ground condition should be carefully considered to ensure reliable performance-based design.

In this section, the focus is on weathered rock and soft rock from among the four rock types – weathered rock (WR), soft rock (SR), moderate rock (MR), and hard rock (HR) – in a typical Korean rock classification. This is because the engineering features of these materials are weaker and more sensitive to building behavior than in the case of strong rock formations such as MR and HR.

3.2.1. Weathered rock

WR is usually found below the residual soil layer; it is frequently located close to the foundation level and is generally quite thick. Therefore, a thorough understanding of the geotechnical characteristics within the site is important during the design. This rock has a wide range of rock strength and deformation characteristics because by definition in the Korean rock classification, it lies in between soil and soft rock layers, and the degree of weathering is widely described as completely to highly weathered, and sometimes even occasionally to moderately weathered.

The weaker side of WR is tested during borehole drilling during a standard penetrometer test (SPT) as done in the case of a soil layer, but with a tiny amount of samples obtained using a split spoon. The stronger side of WR is too hard for the SPT test, and the material is generally too hard to be cored because of the disintegrated characteristics resulting from its high degree of fractures or weathering.

Because it is difficult to determine the geotechnical characteristics of WR owing to the lack of WR samples and its wide variation in strength, in many situations, the strength and deformation values are considered very conservatively based on the conservatively assumed SPT(N) for WR, as say N=100 or so.

However, in order to realize efficient and accurate foundation design for tall buildings, it is necessary to determine the well-defined geotechnical characteristics of WR.

3.2.2. Soft rock

Similar to WR, SR too is frequently located around the founding level. It is mainly described as a moderately weathered rock frequently containing some reddish iron oxide that stains joint faces. Core recovery of this material is generally poor owing to its fractured character and sometimes local degraded rock conditions.

The evaluation of the geotechnical parameters of SR is rather difficult because of its poorly retrieved samples. Hence, various field tests have to be conducted to precisely determine the engineering parameters.

3.2.3. Moderate and hard rock

MR and HR are generally considered as solid engineering materials with sufficient strength and practically negligible deformation characteristics.

Table 3.4 Test methods for rock characteristics and design application

	Test Method	Result and Usage	Remark
Laboratory	Rock core strength test	Direct result: Rock core strength, q_u, Rock core elasticity, E Correlated data: Rock mass strength, Deformation Design usage:: Settlement, Bearing capacity	• Not applicable for WR • Usually interpreted with RQD or seismic velocity
In-situ Borehole Test	SPT	Direct result: SPT(N) Correlated data: ϕ, E Design usage: Settlement, Bearing capacity	• Applicable only for the weaker side of WR
	Pressuremeter Test	Direct result: P_l, E Design usage P_l: Bearing capacity E: Settlement	• Efficient test methods for weak rock with rare cores
	Seismic velocity Test	Direct result: V_S Correlated data: G_0, E_0, G, E Design usage: Settlement, Ground classification	• Meaningful for its continuous data • G_0 and E_0 corrected to G and E, respectively, for determining adequate strain range of founding ground
	Borehole camera	Direct result: View for in situ joint information Design usage: Refine information for rock mass structure	• Meaningful for its continuous visual data • Useful to detect features of local defects
Field Test	Rock Face Mapping	Direct result: Rock mass information (strength, joints, weathering…) Rock classification (e.g., GSI, RMR) Correlated data: E Design usage: Subdivision of rock mass feature for settlement, Bearing capacity	• Investigation of exposed rock faces, especially in raft design • Actual rock conditions superseding the initial assumption
	Large Plate Loading Test	Direct result: Pressure vs. Settlement with time relation Correlated data: E, Rock strength, Creep data Design usage: Settlement(short/long term) Bearing capacity Dynamic behavior	• Efficient method to check complex material as a whole such as a fractured zone or fault • Size of plate to be decided in consideration of target ground size
	Pile Load Test	Direct result: Load vs. Settlement with time relation Correlated data: f_s, f_b, E Creep data Design usage: Pile settlement Pile capacity Dynamic behavior	• Most effective way to check pile behavior and capacity, including pile construction effect • Pile group effect should be considered especially in the settlement analysis of weak rock
Monitoring during building construction/usage	Building Monitoring	Direct result: Building Pressure vs. Settlement Correlated date: E, creep Design usage: Update of design parameters Valuable design information for similar ground conditions and construction method	• Valuable actual data • Anticipation of foundation settlement with building construction progress

The knowledge of the depth and variation of their presence below the foundation is important because they affect the overall ground deformation and building settlement.

The following investigation approaches (Table 3.4) have been carefully adopted in various projects in Korea to evaluate the geotechnical parameters of rocks.

3.2.4. Legend

SPT(N): Number of standard penetrometer test

P_l: limit pressure, : internal friction angle

V_S: shear wave velocity, $G_0(E_0)$: dynamic shear(elastic) modulus

G(E): shear(elastic) modulus, $f_s(f_b)$: pile friction(base) capacity

4. Raft Foundation

4.1. Design sequence for raft foundation

In Korea, the raft foundation system is generally planned for tall buildings when the founding ground comprises WR or other solid ground types. However, such foundations exhibit flexible characteristics to some extent even under stiff ground conditions owing to the relatively low thickness-to-width ratio unlike in the case of massive footings such as in a bridge foundation. These flexible features become more evident with uneven loading or under softer and varying ground conditions. The combination of such factors along with the differential settlement of soil becomes a crucial component in building foundation design and needs to be controlled sufficiently through collaboration between the geotechnical and structure engineers. A typical design sequence for a raft foundation in consideration of diverse factors is shown in Fig. 4.1; this sequence can be altered and adjusted on a case-by-case basis subject to the project conditions.

4.2. Raft design

4.2.1. Bearing capacity

In raft foundation design, in general, the bearing capacity of ground is not a major factor.

However, as in the case of WR, the bearing capacity needs to be carefully checked owing to the wide strength range and difficulties in obtaining core samples.

When the rock core can be recovered and tested for strength, and when the rock type is often SR or better according to Korean Rock Classification, the bearing capacity can be evaluated based on the following methods (Table 4.1).

Evaluation of the bearing capacity of WR requires a different approach because rock core samples are seldom obtained through borehole investigation owing to the brittle nature of WR. The following methods (Table 4.2) can be applied with reasonable accuracy for WR or better rock conditions.

4.2.2. Settlement

When the settlement of a raft foundation is analyzed, it is important initially to realize the major influencing factors and then properly apply them to the analysis.

The theory of elasticity is a useful approach to initially understand the behavior of foundations and to approximate the settlement evaluation based on simplified ground and

Table 4.1. Evaluation methods of bearing capacity based on compressive strength of rock

Methods	Formula
1. Canadian Foundation Engineering Manual (2006)	$q_{all} = K_{sp} \times UCS$ (1) q_{all}: allowable rock bearing capacity with F.O.S = 3 K_{sp}: bearing capacity coefficient UCS: unconfined compressive strength
2. Foundation on Rock by Duncan C. Wyllie (1999)	$q_{all} = C_{f1} S^{1/2} UCS[1+(ms^{-0.5}+1)^{0.5}]/F.O.S$ (2) C_{f1}: shape factor, 1.25 for squares s, m: empirical constants based on rock mass quality, RMR and rock types F.O.S: 3 for dead load + live load, 2 for temporary load condition such as wind and earthquake (US department of the Navy, 1982)
3. Ralph Peck (1974)	$q_{all}(MPa) = 1+\left(\frac{ROD}{16}\right) \Big/ \left[1-\left(\frac{ROD}{130}\right)\right] < UCS$ (3)

4. Goodman Kulhawy (1987)	Rock Mass Properties			
	RQD	UCS of Rock Mass	Cohesion (C)	Friction Angle (ϕ)
	0-70	0.33UCS	0.1UCS	30°
	70-100	0.33-0.8UCS	0.1UCS	30°-60°
	Note: Bearing capacity can be evaluated from the above rock mass properties by using the bearing formula.			

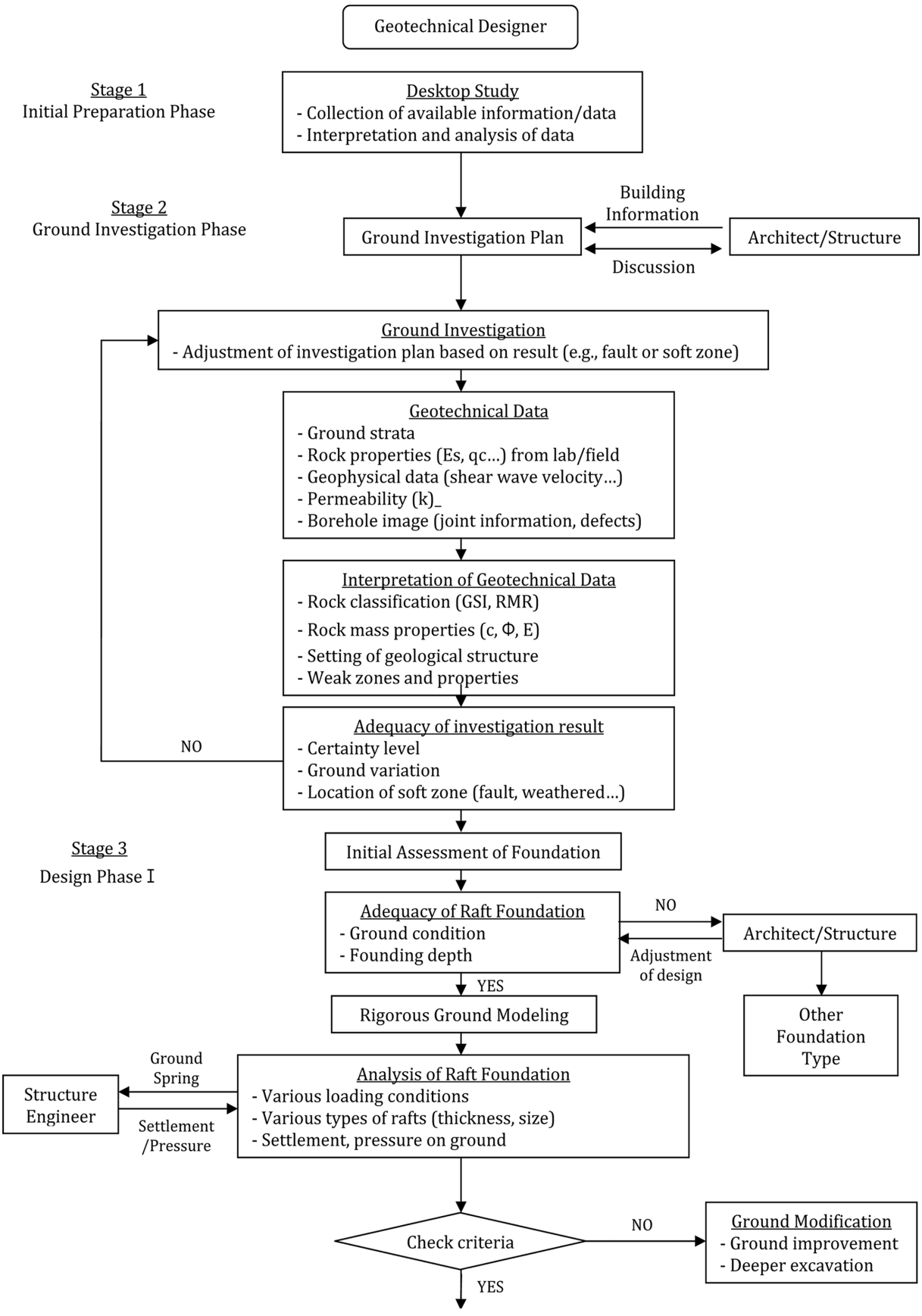


Figure 4.1. Typical Raft Foundation Design Sequence.

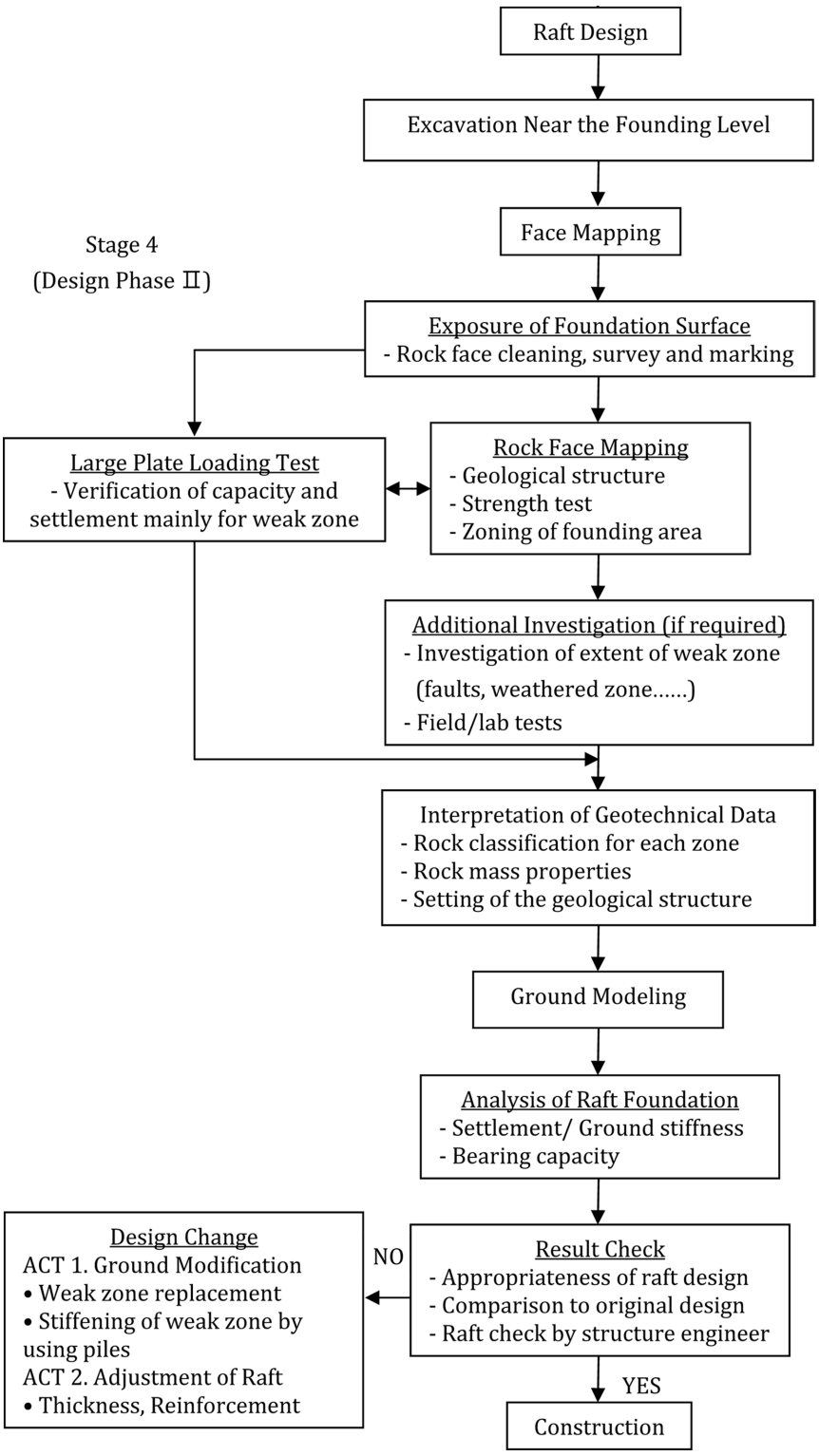


Figure 4.1. Typical Raft Foundation Design Sequence (Continued).

loading models. The analysis is then developed using rigorous numerical methods to accommodate the large complexities and variables involved depending on the ground conditions, and thus to obtain a more refined result. The procedures are illustrated below (Fig 4.2).

(1) Simple approach based on elasticity theory

The design method developed by Mayne and Poulos

Table 4.2. Evaluation Methods of Bearing Capacity for Weak Rocks with Scarce Cores

Bell Solution (1915)	$q_{all}=\frac{C_{f1}CN_c+C_{f2}(B\gamma/2)N_r+\gamma DN_q}{F.O.S}$ C_{f1}, C_{f2} = correction factor; 1.25, 0.8, respectively, for squares N_c, N_r, N_q = bearing capacity factors $N_\phi = \tan^2(45+\phi/2)$ $N_c = 2N_\phi^{1/2}(N_\phi+1)$, $N_r = 0.4N_\phi^{1/2}(N_\phi^2-1)$ $N_q = N_\phi^2$ γ: unit weight B: footing width	(4)
Pressure-meter Test Method (1957)	$q_{all}=\frac{K(P_l-P_0)+q_0}{F.O.S}$ k: bearing factor, 0.8 P_l: limit pressure P_0: at rest earth pressure at the time of the test q_0: overburden pressure after construction	(5)
Large Plate Loading Test	The test is useful and effective for the evaluation of WR. The results provide • Allowable bearing capacity (q_a) • Approximate strength parameter (C, ϕ) from back-analysis • Pressure-settlement relation and creep pressure	

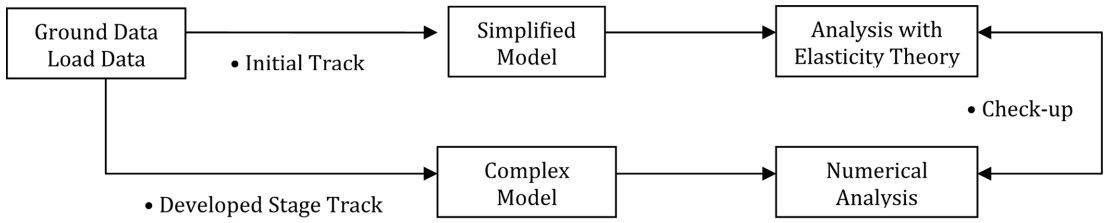


Figure 4.2. Analysis approaches for raft foundations.

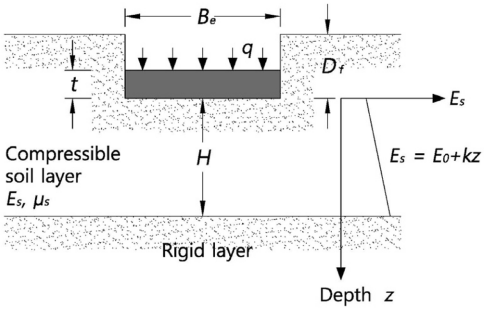


Figure 4.3. Mayne and Poulos' Procedure (1999) for Settlement Calculation.

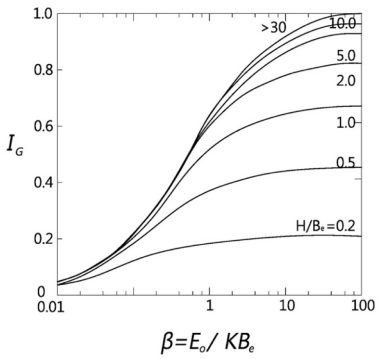


Figure 4.4. Variation in I_G with β.

(1999) has improved the previous solutions by clearly presenting the design factors in the formula, allowing easy applicability in practice.

The settlement below the center of the foundation can be estimated by the formula below:

$$S=\frac{qB_eI_GI_RI_E}{E_0}(1-\mu_s^2) \tag{6}$$

q = magnitude of the uniformly distributed load

B_e = equivalent diameter of a rectangular foundation or the diameter of a circular foundation

$$B_e=\sqrt{\frac{4BL}{\pi}} \tag{7}$$

I_G = influencing factor for the variation in E_S

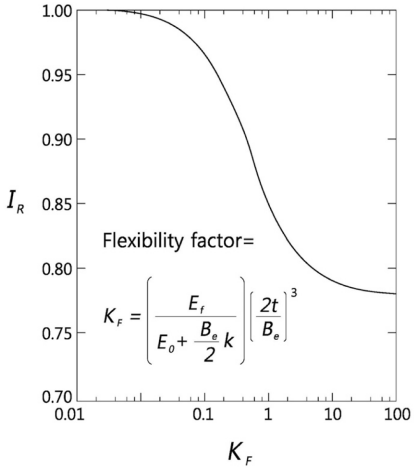


Figure 4.5. Variation in I_R with K_F

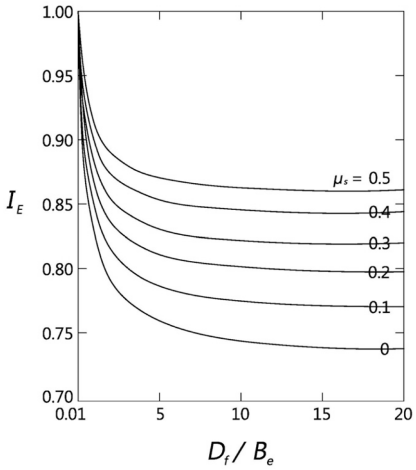


Figure 4.6. Variation in I_E with μ_s and D_f/B_e.

I_R = foundation rigidity correction factor

$$I_R = \frac{\pi}{4} + \frac{1}{4.6 + 10\left(\frac{E_f}{E_0 + (B_e/2)k}\right)\left(\frac{2t}{B_e}\right)^3} \quad (8)$$

where E_f = elastic modulus of the raft
t = raft thickness
I_E = embedded correction factor

$$I_E = 1 - \frac{1}{3.5 exp(1.22\mu_s - 0.4)\left(\frac{B_e}{D_f} + 1.6\right)} \quad (9)$$

In the design of raft foundation, differential settlement is a major factor that affects the building structure.

The normalized differential settlement was defined by Horikoshi and Randolph (1997) as shown in the figure below:

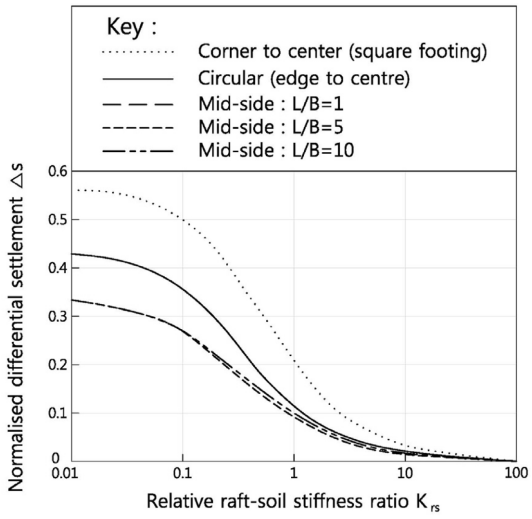


Figure 4.7. Normalized differential settlement as a function of raft stiffness ratio.

$$\Delta S = (S_A - S_B)/S_{av} \quad (10)$$

S_A = settlement at the center
S_B = settlement at the middle or corner
S_{av} = average settlement

$$K_{rs} = 5.57(E_f/E_{S_{av}})[(1-v_s^2)/(1-v_f^2)](B/L)^{0.5}(t_f/L)^3 \quad (11)$$

In practical design, the differential settlement ($S_A - S_B$) can be approximated based on the assumption that S_{av} is the settlement of a rigid footing of $I_R = 0.78$ and $S_{av} = (S_A + S_B)/2$; thus, ΔS can be estimated using the actual rigidity.

The angular distortion of $(S_A - S_B)/L$ (distance AB) can be checked for the permissible criteria.

The other difficulty in practice is how to obtain the representative elastic modulus especially in a multi-layered ground of different stiffness, including weak layers for which it is often a challenge to determine simple trend of the elastic modulus.

In such situations, it is recommended to set a layered model and evaluate a representative value from the model.

The approach of averaging the arithmetic mean elastic modulus tends to underestimate the high local settlement, especially in cases where there exists large deformable ground. It is therefore appropriate to adopt a harmonic mean approach with weight factors since the settlement is proportional to the inverse of the ground elastic modulus, with the stress decreasing with depth.

The mean value can be obtained by the formula below for the case of equal Poisson ratios, as reported by Wardle and Fraser (1975).

$$\frac{1}{E} = \frac{\Sigma_{i=1}^{n} \frac{1}{E_i} \Delta I^i}{\Delta I^{total}} \quad (12)$$

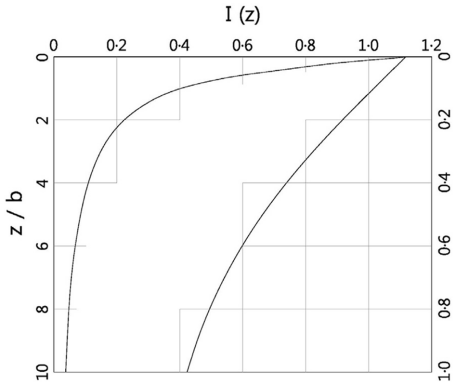


Figure 4.8. Vertical settlement influencing factor.

(2) Rigorous approach based on numerical modeling

Although the elasticity theory approach is relatively simple and quick to use, it has limitations in accurately representing the actual load owing to the complexity of building structures and the variety of ground conditions. In such cases, numerical modeling is particularly useful.

The great advantage of a numerical approach is that it not only allows the analysis of the possible complex conditions, but also helps better understand the foundation behavior and sensitivity under various design factors.

The implication and application of the design factors used in numerical approach are illustrated below.

$$\frac{q \cdot B_e \cdot I_G \cdot I_R \cdot I_E \cdot (1-\mu_s^2)}{E_0} = S \tag{13}$$

i) Loading Term, q

Method	Application Range
Elasticity Formula	- Mainly used for uniformly distributed vertical loading conditions
Numerical Model	- Used for various loading conditions • column load, wall load, distributed load • vertical/horizontal/rotational loading

ii) Structural Terms (B_e, I_R)

Method	Application Range
Elasticity Formula	- Raft shape: circular/rectangular plan shape, constant raft thickness (t)
Numerical Model	- Raft shape: all shapes, varying raft thickness - Inclusion of diverse building structures such as core wall - Advantages: • Differential settlement can be evaluated in detail • Efficient raft shape can be designed • Stiffening effect of the core wall • Accurate collaboration with structural engineers

iii) Geotechnical Terms (E, I_G, I_E)

Method	Application Range
Elasticity Formula	- Elasticity modulus (E): idealized constant or linearly changing values - Rigid layer depth (I_G): constant depth below the raft - Embedment depth (I_E): constant depth all around the raft
Numerical Model	- Elasticity modulus (E): • varying ground conditions including local variations • non-linear ground deformation characteristics - Rigid layer depth/embedment depth: accommodation of diverse changing conditions of depth Note: Diverse cases of ground and their significance are shown in Fig 4.9.

In a rigorous assessment of foundation settlement, it is useful to consider the rock mass properties based on the following assumptions.

- The deformation modulus for loading-reloading or cyclic loading conditions is necessary to reflect the

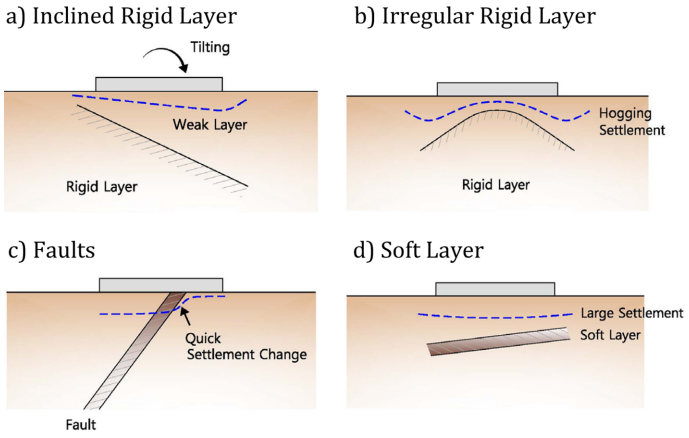


Figure 4.9. Examples of settlement trends affected by rock variation and defects.

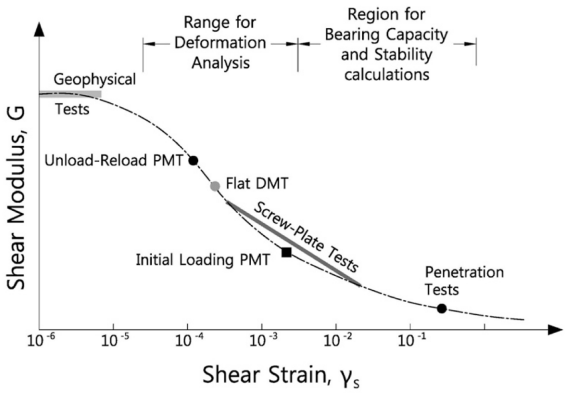


Figure 4.10. Conceptual variation in shear modulus with strain level under static monotonic loading and its relevance to in situ tests.

actual site condition or loading condition.

- The deformation modulus varies according to the range of strain and should be adjusted in consideration of the test method and the expected stress or strain level in ground.

The shear modulus (G) and shear strain (γ_s) in relation to test method are shown in Fig 4.10 (Wyllie, 1999).

iv) Settlement Term (S)

Method	Application Range
Elasticity Formula	- Typically the maximum settlement at center and the symmetrical settlements at both edges of the raft are obtained.
Numerical Model	- Settlement at the raft and beyond. - Superposed settlement by interaction between buildings. - Differential settlement between a tower and surrounding low-rises and also within the tower. - Overturning settlement due to earthquake or wind. - Assessing the influence of building settlement on its surrounding structures.

4.3. Rock face mapping

The characteristics of founding ground in the design of foundations are typically interpreted and modeled from results by various site investigation and test methods. However, it is often noted that after exposure of the rock face, there are some or significant discrepancies between the interpreted and actual founding grounds.

This becomes more evident under the complex rock conditions that are usually influenced by diverse geological activities such as faulting, folding, intrusion, and chronologic weathering – the typical geological conditions in Korea.

In fact, determining composite, heterogeneous, and discontinuous rock mass features through detailed interpretation of complex rock is very difficult; often, this involves relying on geological assumptions and predicting possible scenarios.

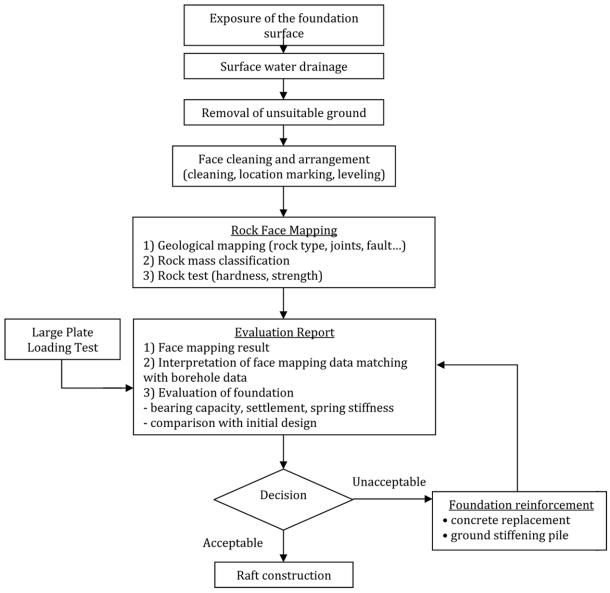


Figure 4.11. Process of rock face mapping and evaluation.



Figure 4.12. A photograph showing the ground exposed after final excavation (Bird's Eye View).

Such limitations can be effectively overcome by incorporating face mapping in the design process, in particular, for rock mass founding ground. Face mapping can be considered an extended measure to produce a reliable and accurate design for raft foundations.

Rock face mapping results should be matched with and interpreted using the initial investigation data to establish a rock mass model for the ground below the foundation. In case of any ambiguity or contradiction within the interpretation, further geotechnical investigation may be required to clarify the knowledge of ground conditions further.

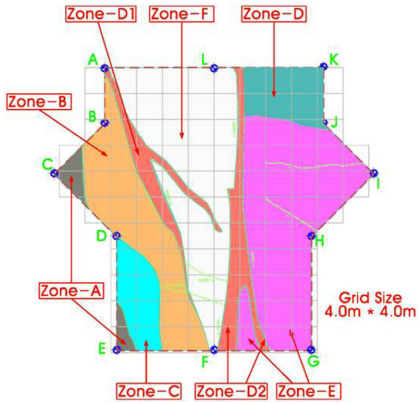


Figure 4.13. Zoning by face mapping (Plan).

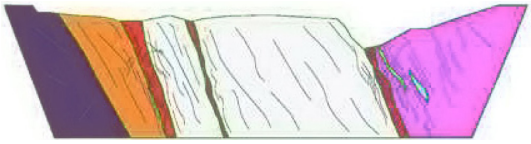


Figure 4.14. Interpreted ground (Section).

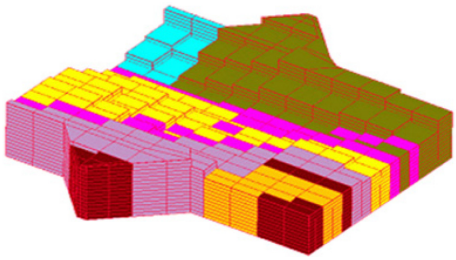


Figure 4.15. Ground modeling.

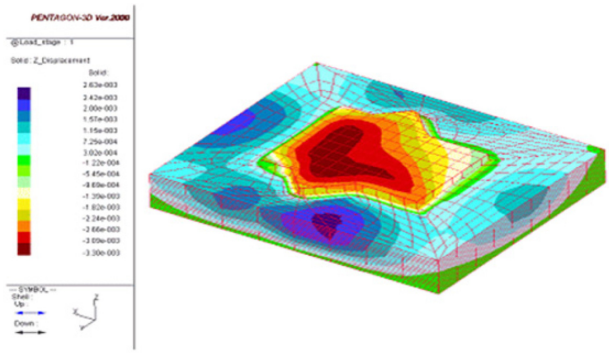


Figure 4.16. Settlement analysis.

The process of face mapping and examples of its applications are shown in Figs. 4.11 to 4.17 (Arup, 2007).

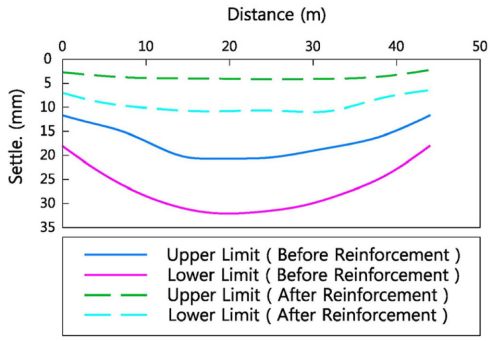


Figure 4.17. Comparison of settlement before and after weak ground reinforcement by concrete replacement.

5. Pile Foundation

5.1. Design sequence for pile foundations

The pile foundation system is generally planned when the ground does not meet the design-bearing capacity or tolerable settlement criteria. In the construction of tall buildings in Korea, large-diameter drilled piles are usually used and installed into the rock. From the viewpoint of pile behavior, the grouping effect of piles for vertical, lateral, and momental loading components should be thoroughly analyzed and considered. The analysis results for the behavior of each pile group are then expressed in terms of pile stiffness and are provided to the structural engineers for the building design. The overall procedure of the pile design is shown in Fig 5.1.

5.2. Pile design

5.2.1. Bearing capacity

In Korea, in the design of pile foundations for tall buildings, it is common practice to install the pile into the rock layer. This allows the vertical load to be supported by both the shaft friction and the end bearing. The vertical bearing capacity of the pile can be primarily estimated by the socket rock quality. However the actual pile capacity is influenced by other construction factors such as the piling method and practice, time, quality of piling, and hydraulic conditions for bored pile construction; it is difficult to assess these factors during the design stage. Therefore, it is essential to conduct a pile test during pile design to consider these effects and to reflect them in the final assessment of pile capacity. The bi-directional pile load test conducted by installing a hydraulic jack in the pile is frequently applied in Korea because of the simplicity and reliability of the construction and test. The evaluating methods of pile capacity based on the ground investigation data are summarized below in Table 5.1 and Table 5.2 for shaft friction capacity and end bearing capacity, respectively. With regard the weathered rock, the application of the formula is difficult because of the lack of rock core samples. Alternatively, SPT(N) or pressuremeter results

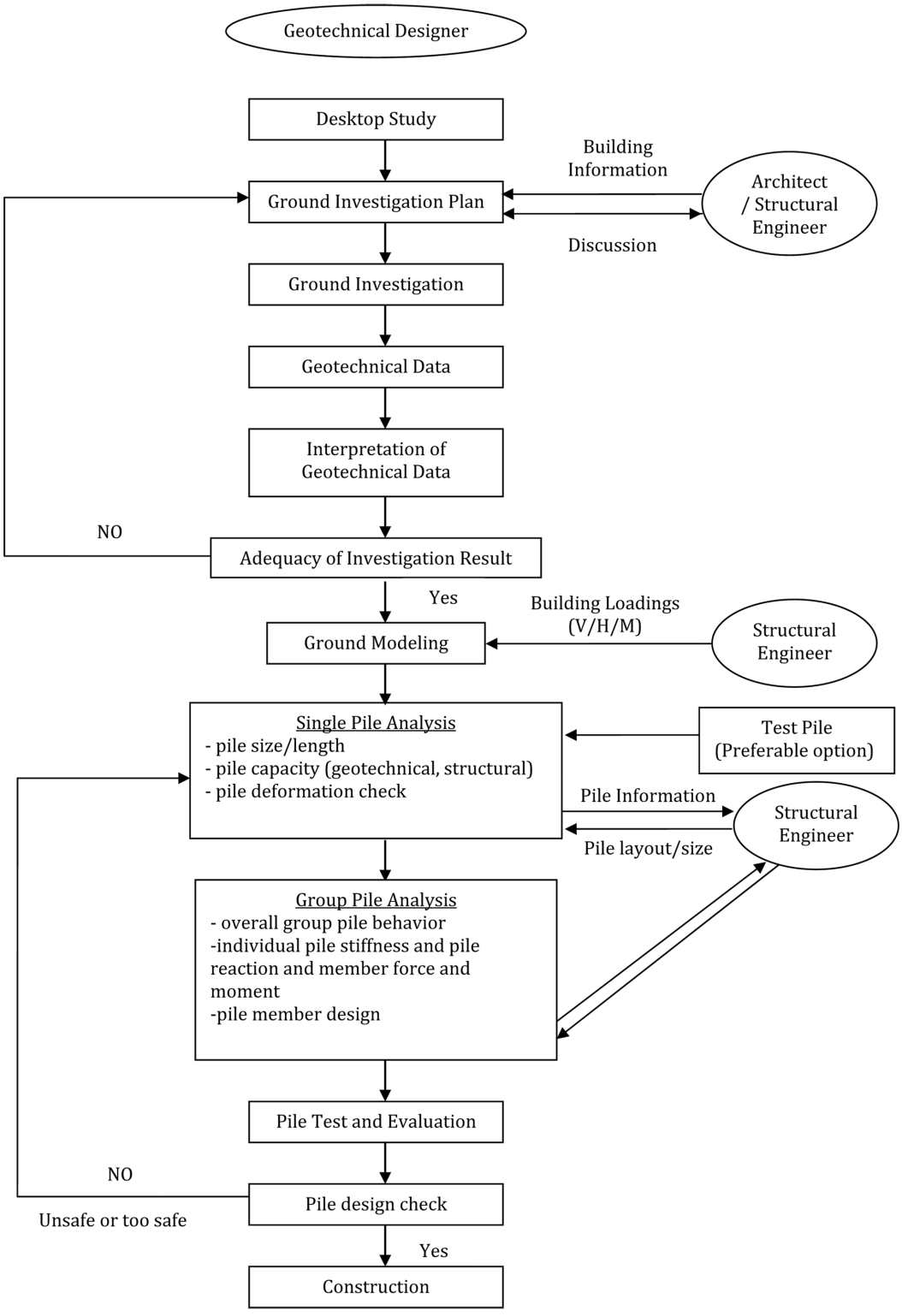


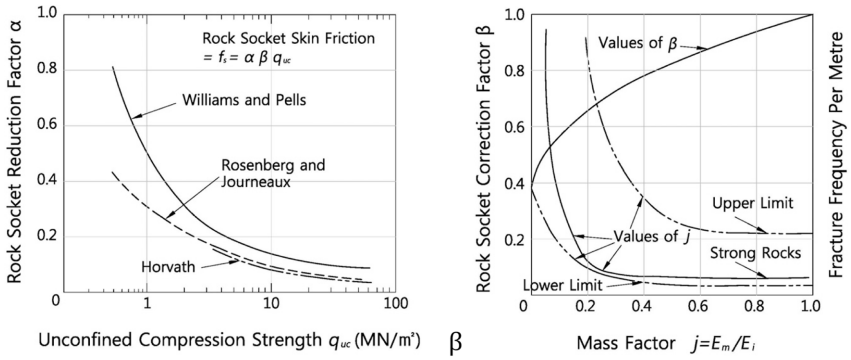
Figure 5.1. Pile design procedure.

can be used to obtain the rock core strength indirectly.

5.2.2. Settlement

The settlement of a single pile socketed into a rock formation is practically negligible if pile construction is properly executed. However, the settlement of a group pile is important, particularly, in the case of high-rise tower foundations and can be estimated considering the following

Table 5.1. Typical correlation between q_{uc} and shaft friction

Evaluation Method	Detail		
	α	β	From
$f_{sa} = \alpha(q_{uc})^{\beta}/SF$	0.2-0.5	0.5	Horvath and Kenny (1979)
	0.22	0.6	Meigh and Wolski (1979)
	0.45	0.5	Rowe and Armitage (1987)
	0.2	0.5	Carter and Kullaway (1988)
$f_{sa} = \alpha\beta(q_{uc})/SF$	 Tomlinson (1987)		

Note: q_{uc}, Uniaxial compressive strength of rock cores
f_{sa},Allowable shaft friction

Table 5.2. Typical formula for allowable end bearing (f_{ba})

Evaluation Method	Detail
$f_{ba} = \alpha q_{uc} / SF$	α = 2.7 Rowe and Armitage (1987) α = 1-4.5 Findlay et al. (1997)
$f_{ba} = N_{ms} q_{uc} / SF$ (ASSHTO, 1989)	N_{ms} is the variable values correlated to rock classification rates from RMR or Q or RQD and rock type
$f_{ba} = \alpha(q_{uc})^{\beta}/SF$	α = 4.83, β = 0.51 Zhang and Einstein (1998)
$f_{ba} = q_{uc} K_{sp} D$ (Canadian Foundation Engineering Manual, 1985)	$K_{sp} = (S+S/B)/[10(1+300g/s)^{0.5}]$ (14) $D = 1+0.4(L/B) \leq 3.4$ (15) K_{sp}: empirical factor, D: depth factor s: discontinuity space, g: aperture of discontinuities B: shaft width or diameter, L: shaft length
$f_{ba} = f(RQD)$	$f_{ba}(MPa) = 1+\dfrac{\left(\dfrac{ROD}{16}\right)}{1-\left(\dfrac{ROD}{130}\right)} < q_{uc}$ Peck(1974) (16)
$f_{ba} = f(P_l)$ (Canadian Foundation Engineering Manual, 1985) Pressuremeter test method	$f_{ba} = [K_b(P_l - P_0)+\sigma_0]/SF$ (17) P_l: limit pressure, P_0: at rest stress σ_0: overburden pressure, K_b: empirical depth coefficient

two major factors:

- Pile-to-pile influence through ground medium as the superposed pile settlement with the numerically estimated pile-to-pile influencing factors.
- Complex ground conditions considered as done in the case of raft foundation design and includes the variation in rock strength and local defects such as faults and cavities.

Hence, in the rigorous design approach, the detailed overall ground model should be set up accurately to determine the foundation performance, which could not be determined by the simple pile group design approach. With regard the horizontal movement, the conditions of the soil close to the founding level and the inertia of the pile structure are more influential factors than the rock conditions at greater depths.

Vertical group pile analysis may be performed mainly by three methods with different levels of accuracy as below.

(1) Settlement ratio method

This method can easily estimate group pile settlement based on single pile settlement. The solution, however, does not reflect or consider the actual ground variation below. This method is more suitable for the initial settlement assessment of a pile group. The formula is shown below:

$$S_G = R \cdot S_S \tag{18}$$

where S_G = pile group settlement
S_S = single pile settlement at the average load
R = settlement ratio

$$R = n^e \text{ (Fleming et al., 1992)} \tag{19}$$

n: pile numbers
e: Efficiency exponent e = typically 0.3-0.5

For use as reference, the efficiency exponent *e* can be estimated in detail based on the method by Randolph (1994). In this method, the ground pile stiffness ratio, pile spacing ratio, ground homogeneity ratio, and Poisson ratio are used. The influence of ground conditions below the pile toe, such as the rigid layer depth or ratio of ground stiffness of shaft and toe can be considered, as reported by Poulos and Davis (1980).

(2) Equivalent raft and equivalent pier method

§ Equivalent raft method

Tomlinson (1987) suggested that a pile group can be replaced by a raft foundation acting at a representative depth below the founding level with some effective embedment depth. The configuration of the equivalent raft for the pile group embedded into the rock formation is shown in Fig. 5 for cases of shallow or deep sockets in rock. The settlement analysis of the equivalent raft can be evaluated by the elastic method through an approach similar to that adopted for shallow foundation analysis with the effective embedment depth of two thirds of the rock embedment (L) or by the numerical method with the equivalent raft set in the ground model.

The pile group settlement W is then represented as follows:

$$W = W_{raft} + \Delta W \tag{20}$$

where W_{raft} = settlement of the equivalent raft
ΔW = elastic compression of the piles above

Note that this approach yields a conservative outcome, especially for a deeply socketed pile, because a significant part of the magnitude of the vertical load is eliminated by the action of the circumferential shaft friction of the embedded portion, and this phenomenon is not accounted for in the equivalent raft method.

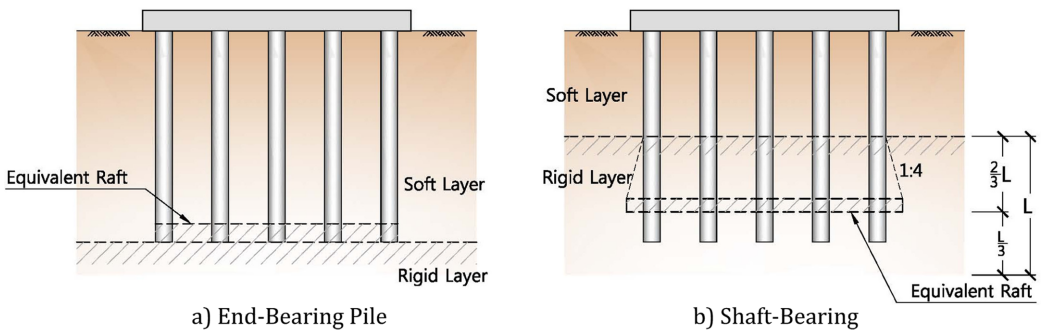
§ Equivalent pier method

In the equivalent pier method, the pile group can be replaced by an equivalent single pier with the composite stiffness of the ground and pile based on the same concept as that used for reinforced concrete columns. The size and stiffness of the equivalent pier are as follows:

$$d_{eq} = \left[\frac{4}{\pi} A_g\right]^{0.5} = 1.13(A_g)^{0.5} \tag{21}$$

d_{eq}: diameter of equivalent pier
A_g: pile group area
$E_{eq} = E_s + (E_p - E_s)\, A_p/A_g$
E_{eq}: equivalent Young's modulus
E_s: average Young's modulus of soil
E_p: Young's modulus of pile
A_p, A_g: area of piles and soil block, respectively, encompassed by the pile group

The elastic solutions for the settlement of a single pile as presented by Randolph (1994) can be applied with the equivalent pier for the given ground conditions. Alternatively, the equivalent pier can be set in the numerically modeled ground to incorporate the complex ground condition into the analysis. The analytical advantage of the equivalent pier method over the equivalent raft method is that the perimetrical resistance of a pile group is taken into consideration. This result provides more accurate and realistic settlement for long friction piles, in particular.



a) End-Bearing Pile b) Shaft-Bearing

Figure 5.2. Equivalent raft method.

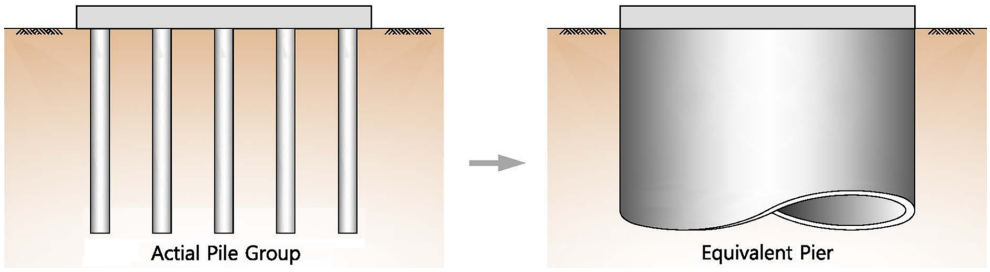


Figure 5.3. Equivalent pier method.

(3) Rigorous approach with numerical methods

More complex analyses using finite element or finite difference analysis are used to evaluate the detail settlement analysis and pile reactions for each individual pile for the ground and structural models to obtain values close to the actual design conditions.

Ideally the numerical model used for the pile design should capture the following conditions:

- Variations in ground conditions across the foundation
- Variations in pile type/length within the foundation
- The pile group interaction effects (the settlement of a pile group is always greater than the settlement of the corresponding single pile because of the overlapping of the individual zones of influence of the piles in the group)
- The interactions between piles, raft, and ground
- Non-linear pile behavior
- Variations in soil stiffness at different locations below the foundation, depending on the operating strain levels
- Structural members such as rafts and core walls
- Various loading conditions such as vertical and lateral loads and rotational moment
- Interface with the surrounding structures or facilities

5.3. Pile spring stiffness

In the building structural design, the reaction characteristics of piles are usually delivered to the structural designer in the form of spring stiffness by the geotechnical engineer owing to its ease of application in the structural model. The vertical pile spring stiffness can be obtained according to the simple Hooke's law f = kx, which is used to determine the pile reaction and pile settlement when the pile reaction is within the allowable pile capacity. As mentioned in the previous chapter, pile settlement depends on factors such as the pile group size, pile space, and pile location in a group. Therefore, the pile spring stiffness should be determined after considering these factors. The vertical spring stiffness derived from various group pile analysis methods is as below.

5.3.1. Spring stiffness from the settlement ratio method

$$k_{av}=f_{av}/S_G \tag{22}$$

k_{av}: average pile spring in a pile group.

f_{av}: average pile load on a single pile.

The spring stiffness determined by this method is used to estimate the average pile spring stiffness. This contradicts the actual behavior of a group pile where the spring stiffness of the inner piles is typically less than that of the outer piles. Hence, the settlement at the center of the raft is underestimated, while the settlements at the perimeter are overestimated; this results in a smaller differential settlement of the raft as compared to its actual behavior.

5.3.2. Spring stiffness from the equivalent raft method

In the equivalent raft method, the raft deflection can be estimated to enable the spring stiffness to be uniquely obtained according to the location in a pile group (Fig. 5.4). Structural designers adopt this approach as it yields more accurate pile information.

5.3.3. Spring from the equivalent pier method

The settlement estimated by the equivalent pier method takes the soil-resisting effect of both the raft and piles into consideration because the soil-pile composite member model of the equivalent pier is assumed in a manner similar to that in the case of piled raft foundation. This approach lets the designer evaluate the pure pile stiffness by averaging the pile spring stiffness as shown below:

$$k_{av}=f_{av}/S_{av}\cdot\frac{E_pA_p}{E_SA_S+E_pA_p} \tag{23}$$

If the equivalent pier is set in a numerical model as an equivalent mass property instead of as a member element, the deflection of the raft can be determined suitably to accurately estimate the individual pile springs.

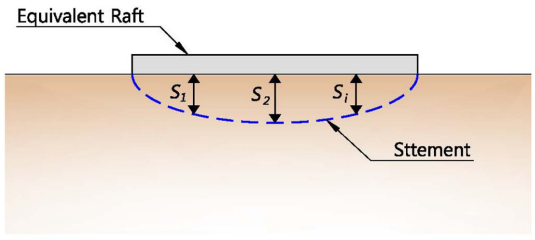


Fig 5.4 Settlement of the equivalent raft.

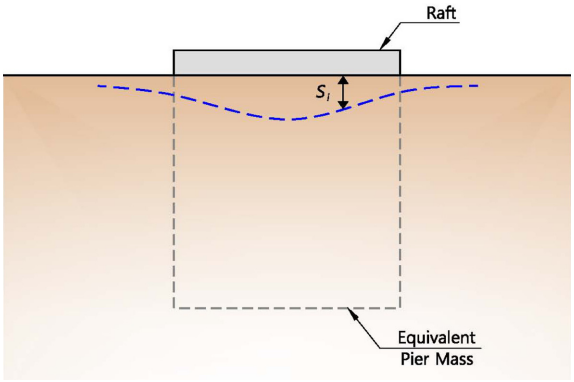


Figure 5.5. Settlement of the equivalent pier mass.

$$k_i = f_{av}/S_i \cdot \frac{E_p A_p}{E_S A_S + E_p A_p} \quad (24)$$

5.3.4. Spring from numerical method

The numerical method enables estimation of individual pile settlement and reaction forces with refined ground properties and under diverse loading conditions. This improves the accuracy of the calculated spring stiffness by allowing the pile grouping effect and soil-structure interactions to be reflected in the design. The settlement and reaction value of each pile is then used to derive the individual pile spring stiffness as illustrated in Fig. 5.6. This approach also enables the evaluation of lateral movement and reaction to determine the lateral pile spring stiffness.

6. Piled Raft Foundation

6.1. Design concept

The piled raft foundation is a composite system where the piles and raft share the load from the super-structure. This system can be generalized into two major types, namely, the raft-enhanced pile group and the pile-enhanced raft. According to the major supporting system, as illustrated in Fig. 6.1, sometimes the two systems may support the load equally.

The proportion of load sharing and settlement of the piled raft system should be evaluated based on the interaction between the piles and raft, as shown in Fig 6.2.

6.2. Piled raft design

6.2.1. Bearing capacity

The vertical ultimate bearing capacity (Q_{PR}) can be taken as the lesser of

(i) the capacity of the block containing the piles and raft, in addition to a portion of the raft beyond the pile-group perimeter

(ii) the factored ultimate capacity of the raft in addition to that of the piles as indicated in the equation below.

For the rock socket pile, the block failure can be ignored. Only condition (ii) needs to be checked using the following equation proposed by Mandolini et al. (2005):

$$Q_{PR} = lesser[\alpha_r Q_R + \alpha_p Q_p,\ f_{PR}(Q_R + Q_p)] \quad (25)$$

α_r: reduction factor for the raft, see Fig. 6.3

α_p: reduction factor for the pile, 1.0

f_{PR}: 0.8

Q_p, Q_R: ultimate bearing capacity of the pile and raft, respectively.

6.2.2. Settlement

§ Interaction factor method

The settlement of the piled raft foundation can be evaluated simply by the interaction factor method reported by Poulos and Davis (1980) and Randolph (1994). The process of analysis is shown below.

Evaluation of Individual Raft Stiffness (k_R)
Evaluation of Pile Group Stiffness (k_p)

↓

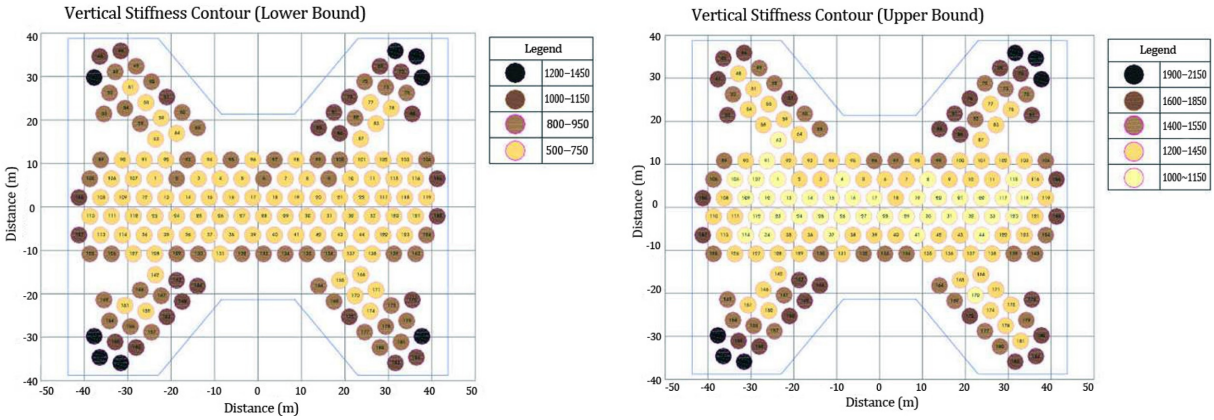


Figure 5.6. Individual Pile Spring Stiffness (151 Towers).

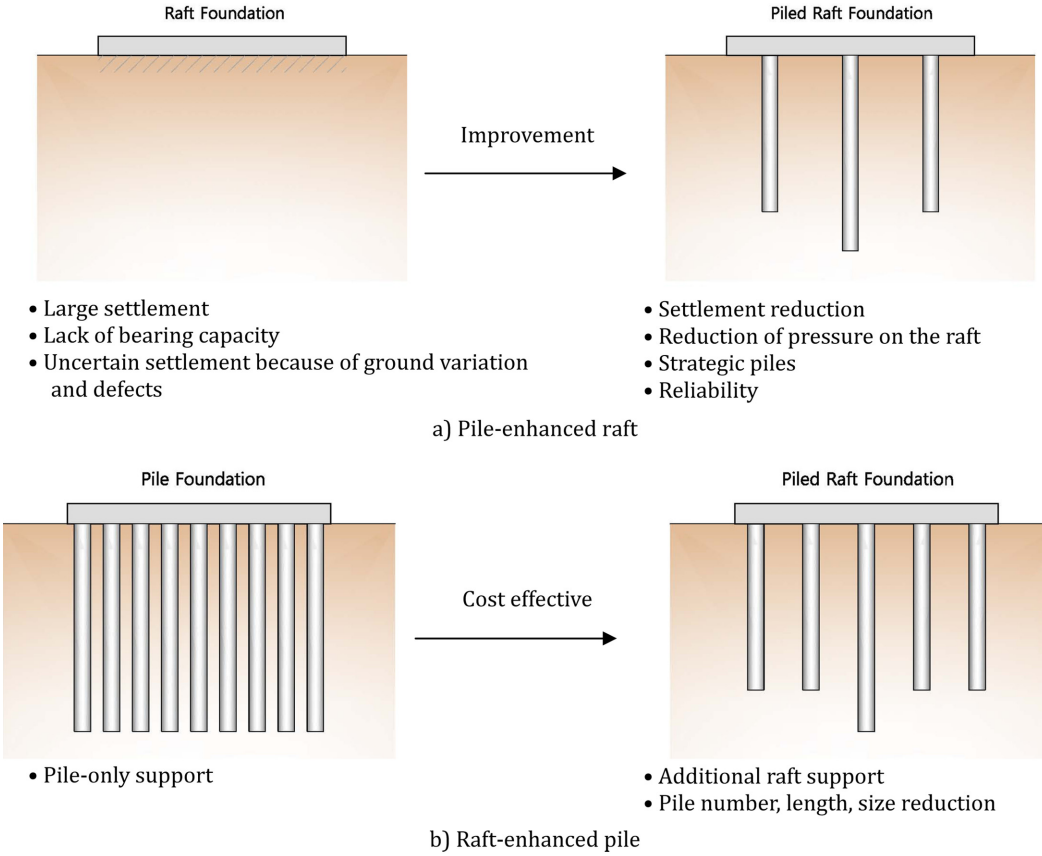


Figure 6.1. Piled raft types.

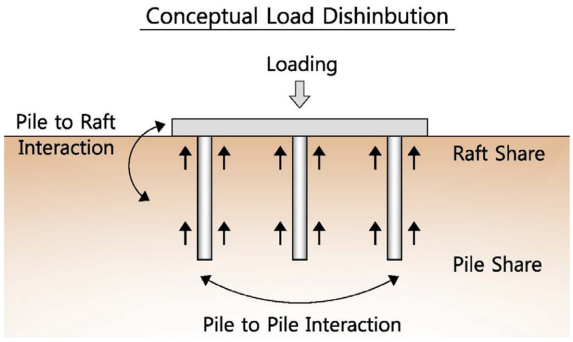


Figure 6.2. Conceptual load sharing and interaction.

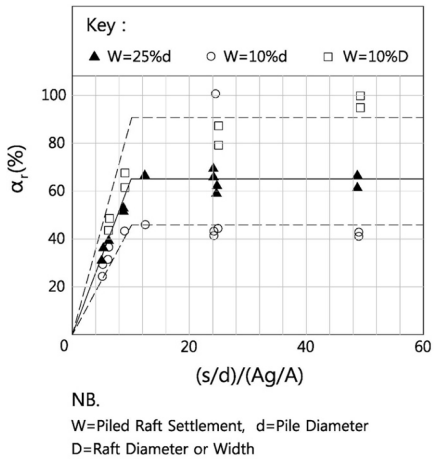


Figure 6.3. Ultimate capacity factor for the piled raft, α_r versus pile spacing and area.

Evaluation of Interaction factor (α)
Evauation of Proportion of Load by Pile (β_p)
Evauation of Proportion of Load by Raft (β_r)

↓

Evaluation of Stiffness Piled-Raft Foundation (K_F)
Evaluation of Interaction-reflected Raft Stiffness (K_{RR})
Evaluation of Pile Group Stiffness (K_{PR})

$$\alpha = \frac{0.2}{1-0.8(K_r/K_p)}\left(\frac{K_r}{K_p}\right) \tag{26}$$

$$\beta_p = 1/(1+\alpha) \tag{27}$$

$$\beta_r = \alpha/(1+\alpha) = (1-\beta_p) \quad (28)$$

$$K_F = \frac{K_p + K_c(1-2\alpha)}{1-\alpha^2 K_c/K_p} \quad (29)$$

$$K_{PR} = \beta_p K_F \quad (30)$$

$$K_{RR} = \beta_r K_F \quad (31)$$

§ Numerical Methods

Numerical models that include piles, raft, and complex ground conditions can be set up to evaluate the piled raft performance in detail. From the analysis, the load-settlement relationship of each pile within the group and the pressure-settlement variation across the raft are estimated. This approach can be used to accurately estimate the spring stiffness of the piles and raft for the benefit of the structural designer.

6.2.3. Application in Practice

§ Pile-enhanced Raft

In Korea, the basement levels of tall buildings generally comprise more than four stories, and the founding level frequently encounters the rock formation. However, owing to excessive weathering and defects such as faults, adopting a raft alone as the foundation involves uncertainties.

In such a case, pile size and locations could be strategically chosen by the numerical approach to enhance and optimize the function of the raft. Thus, such a case will be different from the case of shallow weathering depth or minor defects where it is preferable to replace the ground with engineering concrete. By adopting the pile-enhanced raft design, the designer gains the freedom to have the pile either structurally connected to or disconnected from the raft, based on the ground conditions. The ground can be reinforced, and the ground stiffness can be increased by this method. Examples of application of this method are shown in Fig. 6.4.

§ Raft-enhanced Piles

In Korea, when the raft is laid on the soil layer, and the piles are socketed into the rock, the raft-supporting function is typically ignored during the design.

However, this approach will result in highly conservative figures, especially under dense or stiff soil conditions. In the evaluation of high-rise building performance, under seismic or wind conditions, consideration of the raft resistance will help estimate the considerable overturning resistance and yield more accurate results that reflect the actual

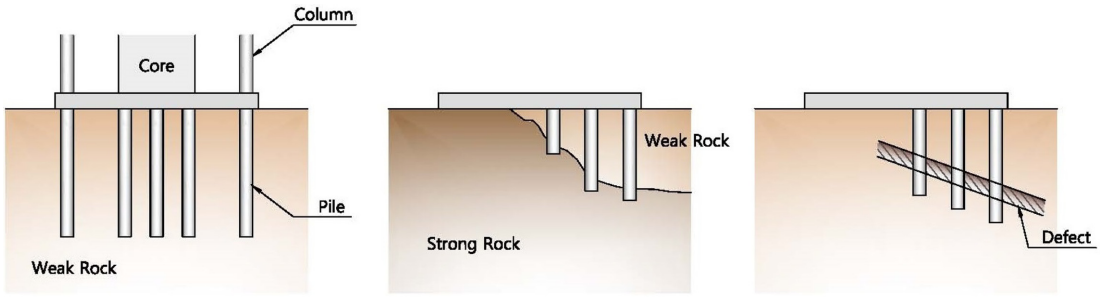


Figure 6.4. Typical cases of pile enhancement.

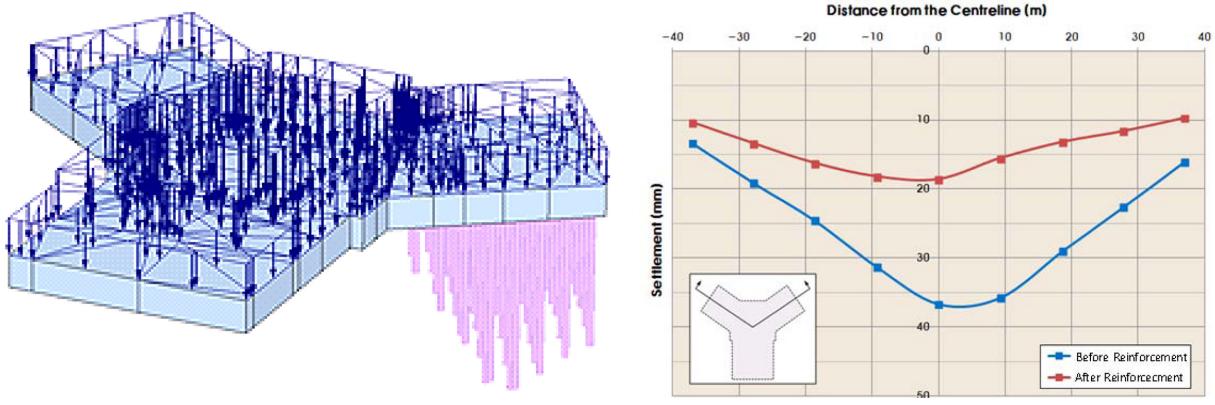


Figure 6.5. Analysis of the pile-enhanced raft.

building performance.

7. Conclusions and Recommendations

1) In this paper, the design steps for the foundation system for a high-rise building are described.
2) In Korea, the rock formation is mainly considered as the founding strata for both raft and pile foundations. The classification of rock based on the local criteria in Korea are introduced and compared to the international rock classification to help understand the local terminology.
3) Ground investigation methods for various rock types and the interpretation methods for the geotechnical parameters for foundation design are introduced.
4) The design sequences of raft, pile, and piled-raft foundations are presented.
5) The simple and rigorous design methods for evaluating the bearing capacity and settlement of rocks are introduced.
6) Case studies conducted in Korea on the design of high-rise building foundations are included for better understanding of the foundation design approach.

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