Settlement Predictions for Pile Foundations

말뚝기초의 침하예측

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요 지

한계상태설계법상에서 말뚝기초는 상부구조물의 안정성과 공용성에 근거하여 지지력과 침하량을 만족하도록 설계되어야한다. 본 논문에서는 공용성을 검토하기위해 과압밀된 점토지반에 위치한 9본의 군말뚝과 2본의 단말뚝에 대하여 실무에서 많이 이용되고 있는 예측식과 현장에서 수행한 피조콘 자료를 이용하여 침하량을 산정했다.

지반에 따라 공간적으로 변화하는 탄성계수가 침하예측에 미치는 영향을 간략히 조사했다. 예측방법으로는 단순탄성론적 예측식인 베직법(Vesic's method)과 풀러스법(Poulos's method)을 이용했으며 산정된 침하량을 실지 말뚝재하실험을 통해서 계측한 것과 비교한 결과 대부분 과다하게 예측했고 지반의 탄성계수를 직선적으로 가정한 결과는 침하계산에 큰 영향은 미치지 않는 것으로 판단되었다.

Abstract

Piling engineers in limit state design should consider both capacity of a pile and settlements of pile for stability of a structure. This paper analyzes the prediction of the settlements of single piles and nine-group piles installed at an overconsolidated clay site by common prediction methods and cone penetrometer test data obtained closely at pile locations. The effects of Young's modulus, which varies spatially in soil profile, on estimating the settlements of piles have been investigated briefly.

The predicted settlements for single piles and nine-pile group by using simple linear elastic methods, Vesic's method and Poulos's method, overestimated overally the measured values, and the assumption of Youngs modulus, which are to be varied linearly through the soil layers, did not significantly affect the settlement predictions.

Keywords : Pile, Settlement, CPT, Pile group, LRFD, Overconsolidated clay

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1. Introduction

Current procedures for the design of structures are based on the consideration of acceptable performance for both ultimate limit state(ULS) and serviceability limit state(SLS). The SLS considers deformations that affect the function and/or appearance of the structure and, in some instances, the comfort of the persons who occupy the structure. Maximum tolerable deformations for serviceability limits may depend on the deformations, displacements and damage, including cracking, that affect the use or appearance of the structure, or cause damage to finishes or to non-structural elements. The establishment of tolerable deformations for the serviceability limit states is more subjective than that for the ultimate limit state(Wahls, 1994). The tolerable movement of buildings and other structures has been a continuing concern to geotechnical and structural engineers, and the concepts for modeling load transfer and settlement for single piles and pile groups are complex.

One of the important questions in the geotechnical design of foundations is to decide how much total and differential settlement can be allowed. The total and differential settlement must be restricted in order not to break connecting parts in the structure and not to cause structural damage affecting the serviceability.

Eurocode 7, Geotechnical Design(1993), provides a guideline as shown in Table 1 for foundation settlement requirements.

Table 1. Guidelines for tolerable foundation settlement (Eurocode 7, 1993)

Settlement Components	Values
a. Total settlement	
Isolated foundation	25mm
Raft foundation	50mm
b. Differential settlement between adjacent columns	
Open frames	20mm
Frames with flexible cladding or finishes	10mm
Frame with rigid cladding or finishes	5mm
c. Relative rotation	5mm
d. Tilt	determined in design of superstructure

For predictions of pile settlement, the key geotechncial parameter required is the stiffness of the soil. If an analysis based on elastic continuum theory is used, then the soil stiffness can be expressed in terms of a Young's modulus(E_s) or shear modulus(G_s)(Poulos, 1994). The distribution and magnitude of these moduli are not constant along the pile shaft, but depend on many factors such as soil type, stress history, installation method, and stress level imposed by the pile. These effects also lead to uncertainties in the predictions. The most satisfactory procedure for assessing the soil modulus is to carry out pile load tests on prototype piles and backfigure the modulus from the observed load-settlement response(Poulos, 1994).

The majority of settlement of a single pile in clay occurs as an immediate settlement, about 80% of the final settlement(Poulos, 1988a), and is in general not considered as a major problem in foundation engineering. However, when piles are placed in groups(usually in matrix arrangements), the settlement of large pile group foundations can become important and should be checked because pile group settlement can differ significantly from that inferred from the settlement of a single pile. The main purposes of this paper are to predict the settlements of driven piles and to investigate the effects of Young's modulus in settlement predictions at an overconsolidated clay site.

2. Descriptions of the Site and CPT Tests

The National Geotechnical Experimentation Site-University of Houston(NGES-UH) is

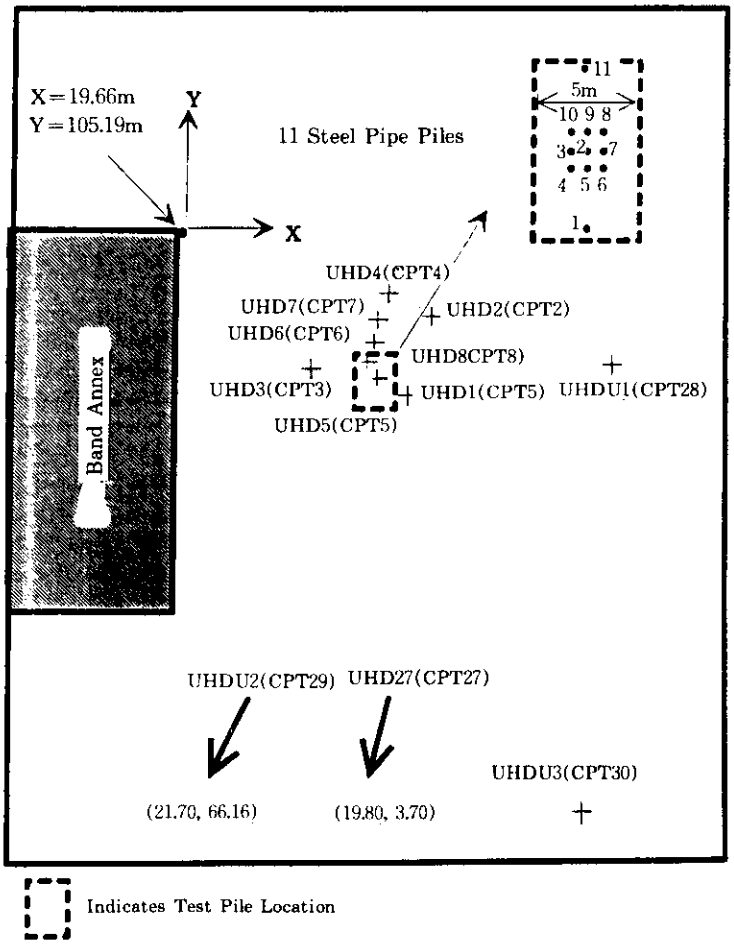

Fig 1. Detailed plan of location A at the NGES-UH

located on the University of Houston campus, in Houston, Texas. It consists of a series of Pleistocene pro-delta clays that have been preconsolidated by desiccation. The Location A within the site, which is the area of this study, is shown in Fig. 1.

Detailed geological descriptions of the site, geotechnical properties of the soil and information from previous tests are presented by Mahar and O'Neill(1983), Dunnavant(1986), and O'Neill and Yoon(1995). A generalized soil profile developed from sample boring and cone penetration test results are shown in Fig 2. The depth of phreatic ground water is about 2 m, with minor seasonal fluctuations. The Beaumont formation, from the ground surface to about 8 m depth, is a clay that is highly overconsolidated due to desiccation, with OCR values decreasing with depth. The soil is highly plastic, with a network of variable, closely spaced, discontinuous fissures and slickensides, which are inherent planes of weakness.

The soil profile has been subdivided into several subunits or layers, based on visual description and index properties, as shown in Fig. 2. The Beaumont formation(Layers 1-3) consists of stiff to hard gray and tan clay with tan sandy clay(CH), and the Lissie formation(Layers 4-8) consists of very stiff to hard gray sandy clay(CL), interbedded with clay, sand and silt layers.

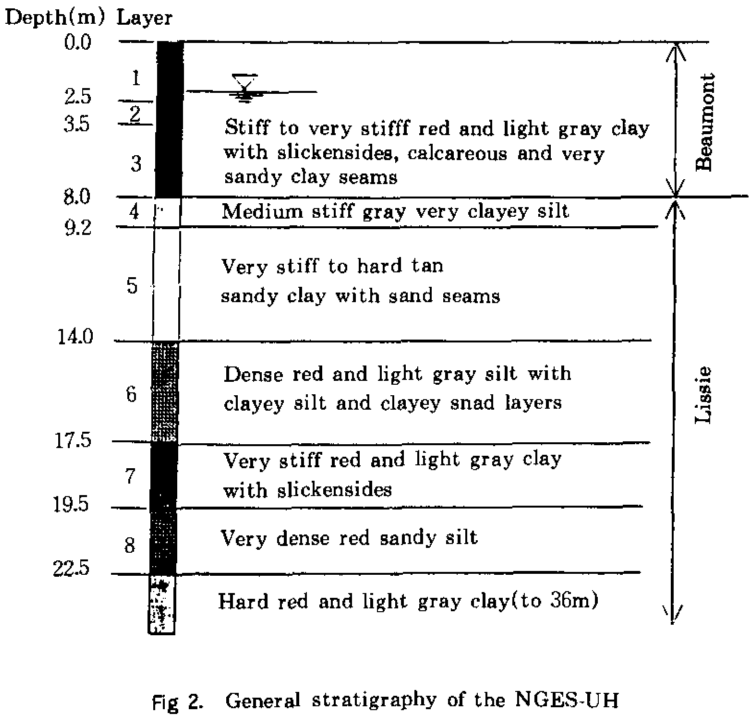

Fig 2. General stratigraphy of the NGES-UH

Twenty-eight electric Fugro-type cone penetrometer(10cm^2 bearing area, 150cm^2 sleeve area) soundings with a capacity of 20 tons of thrust from a truck were made near each pile location shown in Fig. 1. The cone is hydraulically pushed into the ground at a constant speed of 20mm/sec. All cone tests were performed according to American Standard Testing Material D 3441-86, which is an internationally recommended standard method. For each of the cone soundings, continuous recordings of the cone sleeve friction, f_s, and cone tip resistance, q_c, were taken and averaged for computational purposes, with an averaging vertical distance of 0.15 m for each layer in the soil profile. A composite profile of CPT tip resistance is shown with a pile profile in Fig. 3. These will be used to determine Young's modulus of soil layers of interest later.

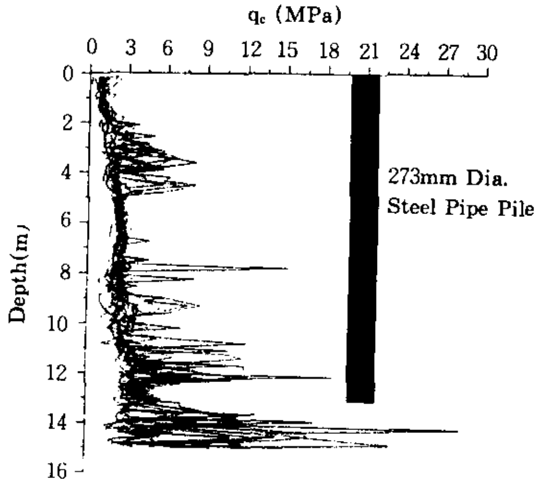

Fig 3. Composite CPT q_c profile with pile profile

3. Single Piles and Nine-Pile Group

Eleven identical steel pipe piles in Fig. 4, which consist of two isolated single piles and a nine-pile group, were installed at the NGES-UH, which had an outside diameter of 273 mm, a wall thickness of 9.27mm. These piles were all driven closed-ended with a Raymond No.1 hammer over a period of three days to a penetration of 13.1m below the ground surface. The test piles were loaded axially to plunging conditions, which define the failure load(resistance) used here. Load tests occurred approximately four weeks after installation, after excess pore water pressures produced by driving had dissipated. Nine of the piles in Fig. 4 (No. 2 through No. 10) were loaded simultaneously, but there was no physical evidence of these piles mutually affecting the capacity of neighboring piles. Two reference piles in Fig. 1(No. 1 and No. 11) were loaded independently. These piles had mean resistances almost identical to those of the nine piles loaded simultaneously. Table 2 shows the results of pile loading test. O'Neill et al.(1981) describes the installation and load testing to plunging failure of these piles. The steel closure plates, which were 25 mm thick were all welded to the toes of the piles and ground flush with the perimeters of the piles so as to have a minimal effect on shaft resistance. Each of the piles was instrumented to measure applied load and resulting settlement and to delineate toe and shaft resistance during static load testing.

Analysis of the test data indicated that all piles achieved failure completely independently; that is, there was no measurable group effect on pile resistance during the first load tests, which are the very tests considered here. The absence of group effect on capacity is a major assumption in the analysis that follows. The piles were loaded in increments of increasing constant load until they failed by plunging over a period of about 8 hours. Each load increment was held for approximately one hour. Loads were not cycled. The measured settlements with ultimate resistance obtained from pile loading test will be discussed later.

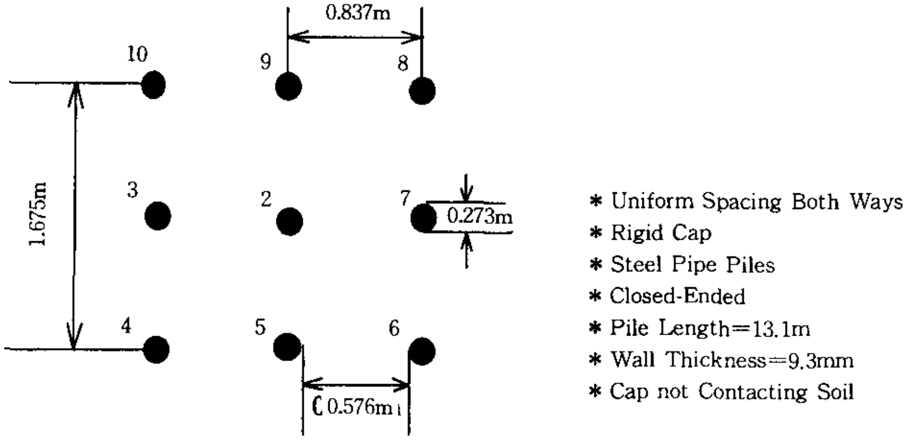

Fig 4. Nine-Pile Group, Locations and Dimension

Table 2. Guidelines for tolerable foundation settlement (Eurocode 7, 1993)

Pile No.	x,y Coordinates(m)	Measured Resistances(kN)			Single or Group
		R_s	R_b	R_u	
1	30.4, 94.2	630	138	768	Single
2	30.4, 97.8	559	155	714	Group
3	29.6, 97.8	519	110	629	Group
4	29.6, 97.0	515	133	648	Group
5	30.4, 97.0	515	120	635	Group
6	31.3, 97.0	542	102	644	Group
7	31.3, 97.8	506	75	581	Group
8	31.3, 98.7	466	138	604	Group
9	30.4, 98.7	519	204	723	Group
10	29.6, 98.7	564	142	706	Group
11	30.4, 101.5	497	182	679	Single

Piles in cohesive soils, such as those at the NGES-UH are normally considered "friction piles" or "floating piles", since most of the resistance is developed in shaft resistance, or "skin friction", rather than toe resistance. For example, the measured ultimate pile resistance(R_u) of eleven driven piles in NGES-UH consisted of about 80 % shaft resistance(R_s) and 20% toe resistance(R_b). In friction piles the toe resistance develops very slowly compared to the shaft resistance and is seldom fully mobilized until the settlement reaches 10-20 % of the toe diameter. By contrast, the shaft resistance is fully mobilized when the settlement reaches only about 0.5 % of the shaft diameter and then remains more or less constant with increasing settlement(Hansbo, 1994). At the serviceability limit state, when the shaft resistance is not yet fully mobilized, the settlement of friction piles will mainly be governed by the elastic modulus of the soil, regardless of the Young's modulus(E_s) or the shear modulus(G_s), which is the primary soil parameter to be evaluated in the design models.

4. Settlement of a Single Pile in Cohesive Soils

4.1 Vesic's Semi-Empirical Method

For design purposes, the settlement of a pile can be broken into the following three components(Vesic, 1977),

$$S^1 = S_s + S_p + S_{ps}\ , \tag{1}$$

where S^1 = total pile settlement for a single pile,

S_s = settlement due to axial deformation of pile shaft,

S_p = settlement of pile toe caused by load transferred at the toe, and

S_{ps} = settlement of pile toe caused by load transferred along the pile shaft.

These three components can be determined separately from the following equations and then added together,

$$S_s = \frac{Q_{ta} + \alpha_s Q_{sa}}{A_p E_p} L, \tag{2}$$

where Q_{ta} = actual toe load transferred to the pile under working conditions,

Q_{sa} = actual shaft resistance load transferred by the pile under working conditions,

L = pile length,

A_p = pile cross-sectional area,

E_p = modulus of elasticity of the pile and

α_s = a number that depends on distribution of shaft resistance along the pile.

Vesic(1977) recommended α_s=0.5 for uniform shaft resistance distribution along the pile shaft, and α_s=0.67 for triangular(zero at pile head and maximum at pile base) shaft resistance distribution. However, the real shape of shaft resistance distribution can only be obtained by load test. Sharma and Joshi(1988) indicated that the total settlement estimated on the basis of unifor or triangular distribution is not sensitive to α_s. Therefore, for practical pur poses, both values of α_s will provide reasonable settlement estimates. The α_s $=0.5$ will be used here.

The following relationships have been established on the basis of theoretical analysis and empirical correlation between soil properties and ultimate toe resistance(q_t) for a number of construction sites as reported by Vesic(1977),

$$S_p = \frac{C_p Q_{ta}}{D q_t}, \tag{3}$$

$$S_{ps} = \frac{C_s Q_{sa}}{L q_t}, \tag{4}$$

where C_p = empirical coefficient(Table 3),

$$C_s = 0.93 + 0.16\sqrt{\frac{L}{D}}\, C_p,$$

Q_{ta} = actual toe load transferred to the pile under working conditions,

Q_{sa} = actual shaft resistance load transferred by the pile under working conditions,

q_t = ultimate toe resistance(force/area),

D = pile diameter, and

L = embedded pile length.

In these estimates, it has been assumed that the bearing stratum under the pile toe extends at least ten pile diameters below the toe. Also, the soil below the pile toe is of equal stiffness to or higher one than that at the elevation of the toe.

Table 3. Typical values of coefficient C_p(after Vesic, 1977)

Soil type	Driven Piles	Bored Piles
Sand(dense to loose)	0.02-0.04	0.09-0.18
Clay(stiff to soft)	0.02-0.03	0.03-0.06
Silts(dense to loose)	0.03-0.05	0.09-0.12

4.2 Poulos's Linear Elastic Method

Poulos(1988a) suggested that the pile-head settlement of a single elastic pile, S^1, of length L and diameter D in an elastic soil, under the action of a vertical load Q, may be evaluated by

$$S^1 = \frac{Q}{E_{sL} D} I_p, \tag{5}$$

where E_{SL} = Young's modulus of the soil at the pile toe, and

I_p = the settlement influence factor.

Values of I_p for typical friction piles derived from boundary element analysis, are shown in Fig. 5, using the nonhomogeneity factor, $\eta = E_{SO}/E_{SL}$, in which E_{SO} is the Young's modulus of soil at the surface. The relative pile stiffness, K_b, is defined as

$$K_b = \frac{E_P R_A}{E_{SL}}, \tag{6}$$

where E_p = Young's modulus of the pile material,

R_A = area ratio of the pile(= $A_{CROSS SECTIONAL}/A_{TOTAL}$), and

E_{SL} = Young's modulus of the soil at the level of the pile toe(soil not affected by high strains caused by loading the pile).

The elastic modulus, E_{SO} and E_{SL} can either be undrained values(for modeling short-term loading) or drained values(for modeling deformations and elastic consolidation produced by long-term loading). Note that the soil Poisson ratio ν_s is generally unimportant for settlement calculations. For short-term loading in saturated clay, ν_s, can be taken to be 0.45 to 0.5, and a value of 0.3 can be used for long-term(consolidation) settlements in clays(Poulos, 1988a). In this study, only short-term loading settlements results from elastic deformation of cohesice soils will be considered because piles were installed at the highly overconsolidated clay site, where consolidation settlement can not be expected.

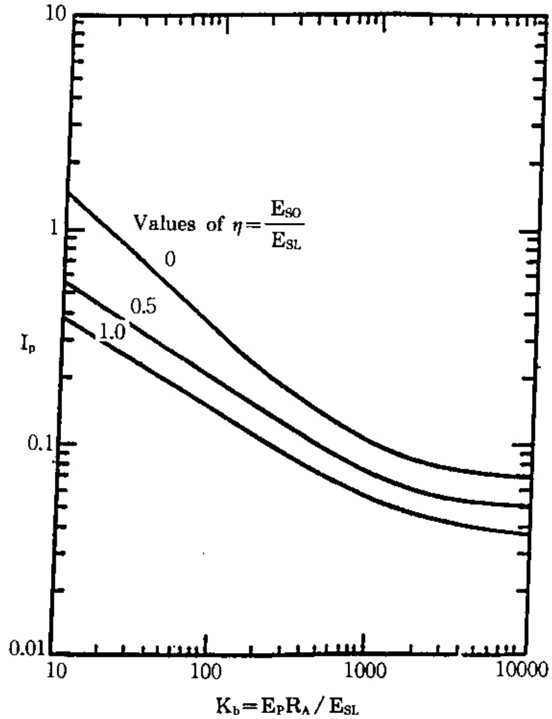

Fig 5. Values of influence factor, I_p, for typical friction pilesl : $\frac{L}{D} = 50$, without underreams (after Poulos, 1988a)

5. Settlement Prediction for a Pile Group in Cohesive Soils

Many methods are available for predicting pile group settlements with varying approximations but it is known that the use of more sophisticated analytical methods in routine design may not be worthwhile because of the complexity of soil characteristics and effects of pile installation(Hansbo, 1994 ; Van Impe, 1991). Moreover, the deviations between the settlements obtained from simplified and more sophisticated approaches, with the same input parameters, are generally negligible. designers prefer to use the simplified methods. In this paper, two commonly used prediction methods are utilized to compute pile group settlements.

5.1 Vesic's Empirical Method

For design purposes, the simplest method is that of Vesic(1977),

$$S_g = S^l \sqrt{\frac{B}{D}}, \tag{7}$$

where S_g = group settlement at load per pile equal to that of the single pile,

S^l = settlement of a single pile estimated or determined from load test,

B = width of pile group(smaller dimension) and

D = individual pile diameter.

In the absence of field test data on pile groups, equation(7) is generally recommended in engineering practice(e. g., Canadian Foundation Engineering Manual(CFEM), 1992).

5.2 Poulos's Linear Elastic Method

Poulos(1980, 1988a) suggests that an evaluation of the settlement of a friction or free-standing group pile, S_g, can also be obtained by assuming linear elastic behavior for the both the soil and the piles and either full flexibility or infinite stiffness of the cap connecting the pile heads; the behavior of the real foundation being actually intermediate between these two limit situations. The effect of adjacent piles on the settlement of one pile in the group can be evaluated using a two-pile interaction factor, α, defined by

$$\alpha = \frac{\textbf{Increase in settlement due to loading adjacent pile with unit load}}{\textbf{Settlement of single pile under unit load}}. \tag{8}$$

The interaction factor α is a function of the length(penetration) of the pile, L, and of the stiffness of the pile in relation to the surrounding soil as expressed by the stiffness factor, K, as shown in Fig. 6. The factor, K is defined by

$$K = \frac{E_P}{E_S} R_A, \tag{9}$$

where the terms are as defined previously. The use of a single value of E_s for soil Young's modulus implies approximation of an uniform soil conditions.

In order to analyze the settlement behavior of a general pile group under vertical loading, superimposability of the two-pile interaction factors is assumed. Then, the settlement of a pile i, S_i, in a group of N identical piles may be expressed by

$$S_i = S^I \sum_{j=1}^{N} Q_j \alpha_{i,j} , \qquad (10)$$

where S^I = settlement of a single pile under unit load,

Q_j = load on a pile j, and

$\alpha_{i,j}$ = interaction factor for a spacing equal to that between the centers of piles i and j(Fig. 6).

Therefore, the settlement of each pile in the group may be calculated in terms of the loads Q_j in the piles.

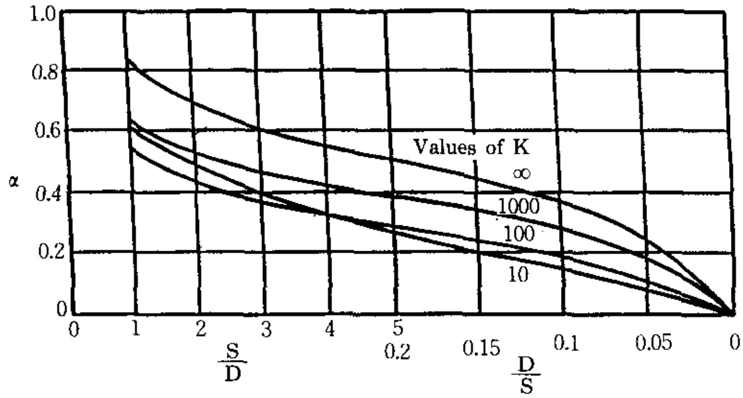

Fig 6. Settlement interaction factor according to the ratio of pile spacing to diameter for $\frac{L}{D}$ = 50 and ν_s=0.5(after Poulos and Davis, 1980)

6. Prediction of Settlements for Single Pile and Nine-Pile Group at the NGES-UH

For the purpose of settlement predictions, the prediction methods described above were used to compute settlements of a single pile and the nine-pile group, whose results can be compared to the measured values and may demonstrate the premise that settlement problems is a noncritical limit state. The objective of this analysis is to show that the settlements at a upper limit values of the service limit load are less than the allowable settlements quoted in Tables 1.

In order to compute settlements, one needs to know the external loads to be applied, which are called service loads in limit state design. AASHTO(1994) defined the service loads as "load combination relating to the normal operational use of the bridge with 90km/hr wind, and with all loads taken at their nominal values." The service loads are almost always less than the ultimate pile resistance. In working stress design, these service loads are considered as working or allowable loads, which can be determined by dividing nominal ultimate resistance by factors of safety of two to three. Meyerhof(1994) suggested that the allowable settlement of a structures can only be determined for each particular case. Therefore, the service load limit should be determined by site conditions(e.g., sand or clay) and particular pile types(driven or bored). The service loads in this study are assumed as 75 % of mean value of the measured ultimate pile resistance, which represents an extreme upper bound.

Settlement Prediction for a Single Pile at NGES-UH

The pile dimension(closed-ended steel pipe pile) are as follow:

- Pile Diameter = 0.273m(outside diameter)
- Wall Thickness = 9.3mm
- Pile Length(embedded length) = 13.1m
- Pile Cross-sectional Area(A_p) = $\sqrt{\frac{\pi}{4}}[0.273^2-0.254^2]=0.00786\mathrm{m}^2$
- Pile Toe Area(A_t) = $\sqrt{\frac{\pi}{4}}(0.273)^2 = 0.057\mathrm{m}^2$
- Pile Shaft Area(A_s) $=\pi\times0.273\times13.1 = 11.1\mathrm{m}^2$
- Young's Modulus of Pile(E_p) = 30×10^6(psi) = $6.89\times30\times10^6$(kPa)=206700MPa.

6.1 Vesic's Semi-Empirical Method

From(1), we get $S^1 = S_S+S_P+S_{PS}$, assuming service loads, which are assumed as 75% of ultimate mean pile resistance(666 kN), are actual loads, then

$$S_s=\frac{Q_{pa}+\alpha_s Q_{fs}}{A_p E_p}L=0.75\frac{136+0.5\times530}{0.00786\times206700000}13.1=2.43\mathrm{mm},$$

$$S_p=\frac{C_p Q_{ba}}{Dq_b}=0.75\frac{0.02\times136}{0.273\times\frac{136}{0.057}}=3.10\mathrm{mm},$$

$$S_{ps}=\frac{C_s Q_{sa}}{Lq_b}=0.75\frac{(0.93+0.16\sqrt{\frac{13.1}{0.273}})\times0.02\times521}{13.1\times\frac{136}{0.057}}$$

$$=0.75\frac{2.03\times0.02\times521}{29417}=0.54\mathrm{mm}.$$

Therefore, the total settlement of a single pile is

$S_l = 2.43 + 3.10 + 0.54 \simeq 6.1$ mm.

6.2 Poulos's Linear Elastic Method

From(5), the settlement of a single pile is computed by $S^l = \frac{Q}{E_{SL}D} I_\rho$. The Young's modulus of a soil at the pile toe, E_{SL}, can be determined by CPT q_c data shown in Fig. 3, based up on the recommendation by Poulos(1988b), as

$E_{SL} = 25q_c = 25 \times 4.0\text{MPa} = 100\text{MPa}$,

where $q_c = 4.0$MPa in the mean value of q_c in Layer 5 from the 12 deeply penetrating CPTs in Location A.

The factor I_ρ from Fig. 5 is 0.14 for $K_b = 270$ and $\eta = 0.63$. Then, S^l is

$$S^l = \frac{Q}{E_{SL}D} I_\rho = 0.75 \frac{666}{100000 \times 0.273} 0.14$$
$$= 0.0026\text{m} = 2.6\text{mm}.$$

Settlement Prediction for Nine-Pile Group

As shown in Fig. 4, the piles in the nine-pile group had the same attributes as the single piles. The group was laid out in a three-by-three grid with spacing of three diameters center-to-center. The piles were connected with a reinforced concrete pile cap that can be assumed to be rigid. Note again that the 3×3 group of steel pipe piles has O.D.=0.273m and wall thickness of 9.3mm,. spacing=3.1 diameters, pile penetration depth=13.1m, and L/D=13.1/0.27=48.5.

6.3 Vesic's Method

From(8), we get $S_g = S^l \sqrt{\frac{B}{D}}$

$$= 5.99 \sqrt{\frac{1.675}{0.273}} = 14.8\text{mm}.$$

6.4 Poulos' Linear Elastic Method

Poulos(1988a) suggests if the pile cap in the group is relatively rigid, equal settlement of all piles in the group can be assumed, and it may also be assumed that the summation of applied load, Q_i, to each pile equals the load applied to the group, Q_g. Thus, the settlement of all piles in a group of N identical piles may be equated which, together with the static equilibrium equation, gives N+1 equations which may be solved to give the distribution of the N unknown loads in the group and the settlement of the group of piles. These procedures have been applied to compute the settlement of a group of nine piles of the same length as those at the NGES-UH in a 3×3×3 diameter spacing matrix, with a rigid cap. For the computation of the settlements, the Young's modulus of soil(E_s) is obtained from CPT q_c data shown in Fig. 3, which is the mean value along the top 13.1 m of the soil profile. Poulos(1988b) suggested that the values of Young's modulus of clay near the piles in a group could be taken as $E_s=25\ q_c$. Between the piles and far from the group, the E_s will be higher. However, if $E_s=25\ q_c$ is used with the Poulos's solution, an upper bound for the settlement should be realized. Therefore, the calculations are carried out with that relationship. If the q_c profile shown in Fig. 3, which represents a typical profile at the NGES-UH, is utilized to compute E_s, then E_s will be $25\times2.25=56$MPa for the Beaumont formation and $25\times4.15=104$MPa for the Lissie formation. Thus,$(E_{AVG})_{Soil}\approx(56\text{MPa}+104\text{MPa})/2$ or 80 MPa for an assumed homogeneous(uniform) profile was used to compute settlements in this study.

The interaction factors α can be obtained from several sources. Poulos and Davis(1980) plotted relationships between interaction factor and ratio of spacing/diameter of piles as shown in Fig. 6. These factors are based on uniform soil modulus in a horizontal plane. The predicted settlement from this method is 3.9 mm for a load of nine times the failure load of a single pile times 0.75, which is still much less than the allowable settlement(25mm).

7. Comparison of Predicted Settlements with the Measured Values

Mean values of the measured settlements from the full-scale pile load tests for 75% of ultimate load for the singles pile and for the nine-pile group(O'Neill et al., 1981) are given in Table 4. The simple method of Vesic's overestimated the measured settlements for both the single pile and pile group. Poulos's linear elastic method also overestimated the measured values, but by a less amount. Both methods, however, yielded values for the single pile and group that were much less than the tolerable settlement(25mm) by assuming that the service loads are 75% of the measured ultimate loads(failure loads for a single pile).

Table 4. Summary of settlements for a single and group pile computed by two methods for service load Equal to 75% of ultimate resistance

Pile Classification	Single Pile		Nine−Pile Group	
Prediction Method	Predicted	Measured	Predicted	Measured
Vesic's Method	6.1mm	2.2mm	14.8mm	3.3mm
Poulos's Method	2.6mm	2.2mm	6.7mm	3.3mm

Poulos(1989) also investigated settlement in this particular group by computing the settlements of single piles and group piles installed at the NGES-UH site by using several versions of the boundary element method. The results are shown in Fig. 7. Poulos applied 290kN as an applied load for the settlement computations, which was lower value than that of the author(75% of R_u=500kN).

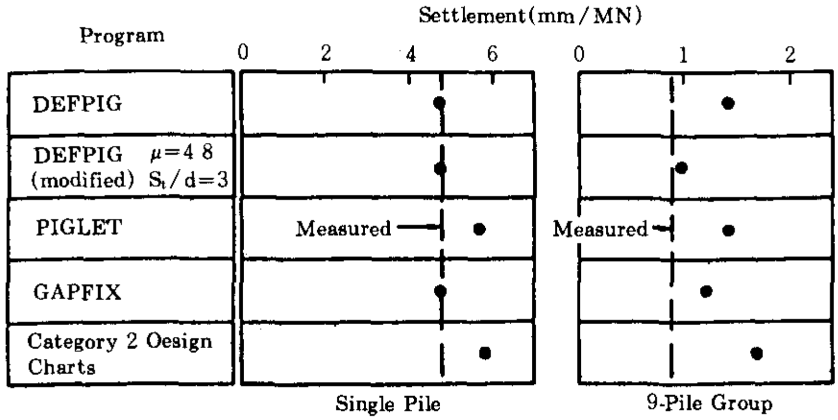

Fig 7. Predictions of Settlement of Single Pile and Nine Pile Group by Various Methods (after Poulos, 1989)

After having established the relative independence of the detailed model in making the settlement calculations, demonstrated in Fig. 7, Poulos's also investigated the effects of soil modulus selection further by using one elastic model, assuming Young's modulus of the soil to be homogeneous(constant), linearly varying(as above), consisting of two layers and layered in detail. The results of his investigation on Young's modulus effects is shown in Fig. 8, which indicates that the prediction of settlements of the single pile are relatively accurate, but prediction of the settlement of the nine-pile group pile in all cases overestimated the measured values. There is, however, only a small variation in settlement both single pile and group piles among the soil profile models for Young's modulus. It therefore appears that, in this particular case, the idealization of the soil profile is not a crucial factor in the prediction of pile group behavior.

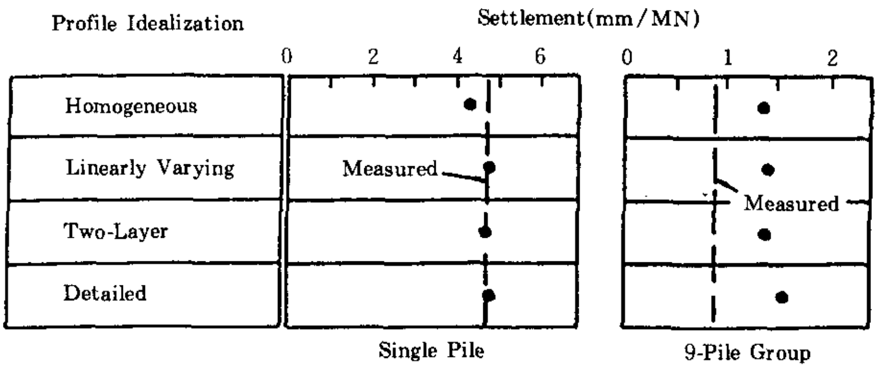

Fig 8. Effects of Young's modulus profile of the soil on the predictions of settlement of single pile and nine-pile group(after Poulos, 1989)

8. Conclusion

This paper described the serviceability limit state design problem, which mainly concerns settlements of a structure. Among various settlement prediction methods for driven piles and pile groups, commonly used simple elastic methods suggested by Vesic and Poulus empirical methods have been used to predict the settlements of a single pile and nine-pile group installed at an overconsolidated clay site. Also, the effects of Young's modulus on estimation of settlements have been investigated briefly for both single piles and group piles.

The predicted settlements by these methods overestimated overally the measured vlaues, and the assumption of Young's modulus, which assumed here to be linearly-varing through the soil deposits, has not much effects on settlement calculations. It therefore appears that the idealization of the soil profile is not a crucial factor in the prediction of pile settlement behaviors at this particular site.

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(접수일자 1997. 1. 20)