Jour. of the KGS Vol 16, No. 3, June 2000, pp 157~162

Interaction Factors of One-Row Pile Groups Subjected to Lateral Soil Movements

측방 유동을 받는 일렬 군말뚝의 상호 작용 계수

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요 지

측방유동을 받는 일렬 군말뚝의 그룹효과를 파악하기 위해 3차원 유한요소해석을 수행하였다. 국내의 대표적인 화강풍화토 지반에 선단지지된 말뚝을 대상으로 측방으로 지반변위 발생시 말뚝 두부조건과 중심간격(2.5D, 5.0D, 7.0D, 단독말뚝)및 말뚝주면의 접촉효과를 고려한 군말뚝의 상호작용계수를 산정하였다. 본 연구 결과, 단독말뚝과 비교하여 군말뚝의 간격이 좁아짐에 따라 상호작용계수는 현저하게 감소하였으며 말뚝 두부조건이 회전구속, 힌지, 자유단의 순으로 감소정도가 크게 나타났다. 이는 실내모형실험을 통해 산정된 상호작용계수와도 비교적 잘 일치함을 보였다.

Abstract

The behavior of one-row pile groups was investigated based on three dimensional finite element analysis. The emphasis was on quantifing interaction factor of pile groups subjected to lateral soil movement. A nonlinear three-dimensional analysis of pile groups in deposited weathered soil was performed for different cap rigidity and spacing-to-diameter ratios varying from 2.5 to 7.0 for identifying group effect in a group. Based on limited parametric studies, it is found that the interaction factor of the pile in a group was reduced as pile spacing decreases in the order of unrotated, hinged and free cap rigidity and prediction by present analysis is in good agreement with general trend observed by experimental results.

Keywords : Pile groups, Interaction factor, Lateral soil movement, Group effect, Cap rigidity, Pile spacing

1. Introduction

In many cases, piles are designed to sustain lateral soil movements when they are used to stabilize unstable slopes or potential landslides, bridge abutments adjacent to approach embankments and excavation. The lateral loads resulting from the soil movements induce deflections and bending moments in the pile, which may lead to their structural damage or failure. Much work has been done on the behavior of lateral soil movement on pile(Brown, Clark and Reese, 1988; Chen and Poulos, 1994). However most of the previous studies have not led to firm conclusions for practical use to cover a wider spectrum of the general problem. Moreover, issues such as the group effect on the lateral pile response, and the estimation of lateral soil movements, are not well understood.

The behavior of a pile in a group is primarily influenced by the presence of and loadings on neighboring piles when piles are closely spaced. To obtain detailed information on the behavior, it is necessary to simulate the three-dimensional geometry and nonlinear behavior of the soil. A non-linear analysis is more appropriate and realistic than a linear one to consider the important mechanism of pile-soil interaction.

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Over 90% of the pile foundations constructed in Korea are embedded in weathered rocks through the uppermost weathered soil deposit. The weathered soils, which occupy two-thirds of the total land area of thr Korean peninsula are generally the results of the physical weathering of granite-gneiss of varying thicknesses ranging up to 40 meters.

The overall objective of the present study is to investigate the pile-soil-pile interaction factor by using a three dimensional finite element analysis(Brown and Shie, 1990). As a result, interaction factors of pile groups installed in weathered soils will be proposed to design one row pile groups subjected to lateral soil movements based on the pile spacing and cap rigidity.

2. Soil Constitutive Model

The extended Drucker-Prager model is used in this study. Indeed this model uses effective strength parameters (c, ϕ) for nonlinear analysis. This model also allows limitation of the shear stress that can be developed at the pile-soil interface. According to the original Drucker-Prager model(Drucker and Prager, 1952), the incremental plastic strain vector has a negative volumetric component, implying a volume increase or dilation during shear and at failure. This is in conflict with the known behavior of normally consolidated clays and sands which have a tendency to compress or decrease in volume during shear. This discrepancy may be due to the fact that the normality rule may not be valid(Desai and Siriwardane, 1984).

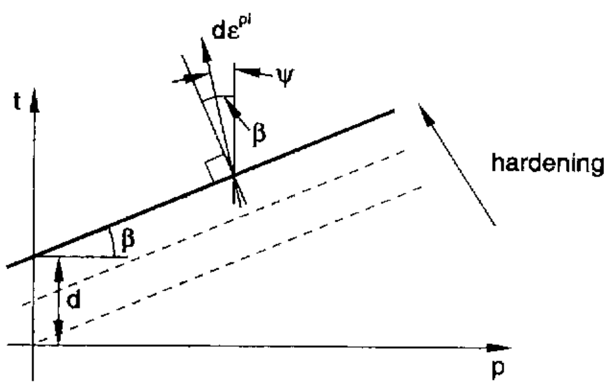


Fig. 1 Schematic diagram of the p-t plane

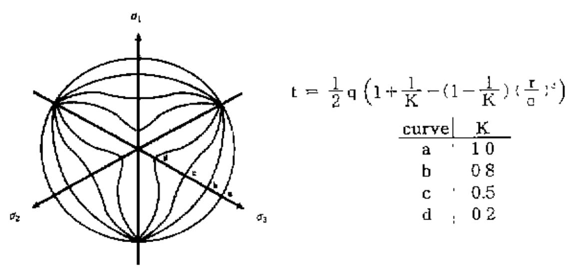


Fig. 2 Dependence of t on K

The extended Drucker-Prager model (Fig. 1) used in this study, however, allows control of the change in volume by using a non-associated flow rule as well as a yield surface in the deviatoric plane based on the third stress invariant. This model in intended to simulate material response under essentially monotonic loading. The basic characteristics of this model are: the material is initially isotropic, the yield behavior depends on the hydrostatic pressure, the material harden or soften isotropically, the inelastic behavior can be generally accompanied by some volume change. This model uses a smooth Mohr-Coulomb yield surface, inelestic flow in the deviatoric plane, and separate dilation(Ψ) and friction angle(β). There are two fundamentals in the formulation of a constitutive law based on this model.

2.1 Yield Function

Under axisymmetric triaxial loading ($\sigma_2 = \sigma_3$), the yield surface of the original Drucker-Prager model is defined as follows:

$$f = \sqrt{(J_{2D})} - \alpha J_1 - k = \left(\frac{1}{3} q^2\right)^{\frac{1}{2}} - 3 \cdot \alpha \cdot p - k = 0 \quad (1)$$

where

$$J_{2D} = \frac{1}{3}(\sigma_1 - \sigma_3)^2, \quad J_1 = \sigma_1 + 2\sigma_3,$$
$$q = \sigma_1 - \sigma_3, \quad p = \frac{J_1}{3}$$

and σ_1 and σ_3 are maximum and minimum principal stresses. The model used in this study extendeds the form of Eqn(1) to include dependence of the yield function on the third stress invariant and also permits non-associated flow. By letting $\tan\beta = 3\sqrt{3}\alpha$ and $d = \sqrt{3}k$ in Eqn(1), the form of yield function is

$$f = t - p \cdot \tan\beta - d \qquad (2)$$

where

$$t = \frac{1}{2} q\left[1 + \frac{1}{K} - \left(1 - \frac{1}{K}\right) \cdot \left(\frac{r}{q}\right)^3\right] \qquad (3)$$

and r is the third deviatoric stress invariant,

$$r = \left(\frac{9}{2} S_{ij} S_{jk} S_{ki}\right)^{\frac{1}{3}} \qquad (i, j, k = 1,2,3) \qquad (4)$$

S_{ij} are the deviatoric shear stresses. The measure of deviatoric stress, t, allows matching of different stress values in tension and compression in the deviatoric plane thus providing flexibility in the yield surface depending on the third stress invariant and in the material parameter of K (Fig. 2) and a smooth approximation to the Mohr-Coulomb yield surface. The value of K is chosen to match the Mohr-Coulomb values on both the tensile and compressive meridians. The values of β and d be expressed in terms of angle of internal friction, ϕ, and cohesion, c, by using conventional triaxial compression tests. In this case the values of $\tan\beta$ and d are:

$$\tan\beta = \frac{6\sin\phi}{3 - \sin\phi}, \quad d = c \cdot \left(\frac{6\cos\phi}{3 - \sin\phi}\right) \qquad (5)$$

2.2 Flow Rule

A non-associated flow rule, where the direction of the inelastic deformation vector is normal to the plastic potential is assumed, so that

$$d\varepsilon^{pl} = \frac{d\,\overline{\varepsilon}^{pl}}{\left(1 - \frac{1}{3}\tan\Psi\right)} \cdot \frac{\partial g}{\partial \sigma} \qquad (6)$$

where $d\,\overline{\varepsilon}^{pl}$ is the plastic strain increment vector assumed to be equal to the uniaxial compressive plastic strain, $d\,\overline{\varepsilon}^{pl}$, and **g** is the flow potential, chosen in this model as

$$g = t - p \cdot \tan\Psi \qquad (7)$$

where Ψ is the dilation angle in the p-t plane. A geometric interpretation of Ψ is shown in the p-t diagram of Fig. 1. This flow rule definition precludes dilation angles Ψ >71.5 degree ($\tan\Psi$>3) since it is unlikely it will be the case for real materials. To avoid volume dilation in soft clay during shear, this model uses a non-associated flow rule by taking into account diffrent values of dilation (Ψ) and friction angle(β).

3. Finite Element Model

The response of groups is analyzed by using a three-dimensional nonlinear finite element approach. The finite element mesh for a typical case is shown in Fig. 3. The mesh consists of three-dimension 8-noded solid elements and is assumed to be resting on a rigid layer, and the vertical boundaries at the left- and right-hand sides are assumed to be on rollers to allow movement of soil layers. The interface elements were two-dimensional elements comprising two surfaces compatible with the adjacent solid elements; the two surfaces coincide initially. The interface elements can transfer only shear forces across their surfaces when a compressive normal pressure acts on them.

The pile elements are assumed to remain elastic at all times, while the surrounding soil is idealized as an elasto-plastic material. This model was selected from among the soil models in the library of ABAQUS (version 5.8 from Hibbit, Karlsson and Sorensen, Inc.), the commercial finite element package used for this work. For certain group configurations, the use of symmetry reduced the size of the mesh. The actual size of the mesh is related to the pile length; the lower rigid boundary has been placed at a depth equal to pile point and the side boundary is extended laterally to r_m=2.5L(1- υ) (Randolph & Wroth, 1978).

It is found that this size was sufficient for single pile

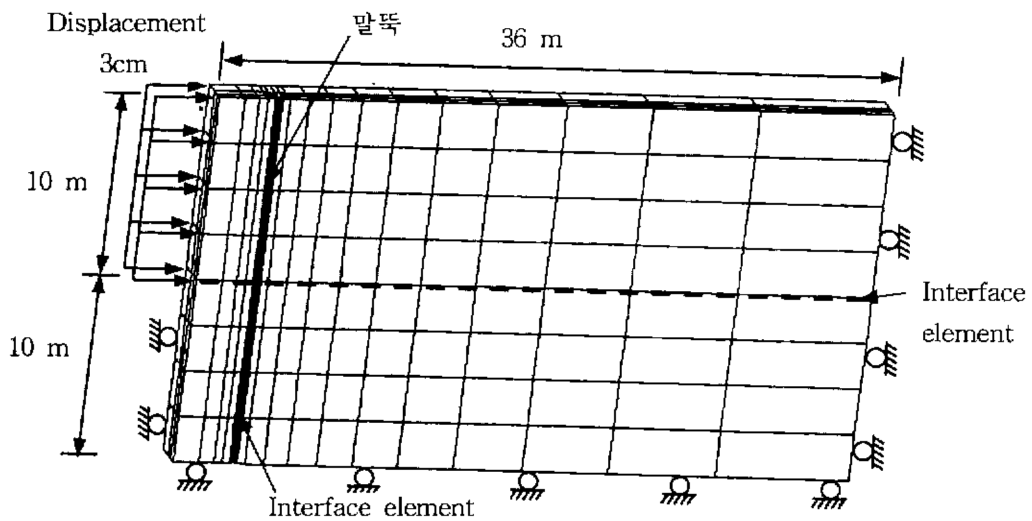


Fig. 3 Three-dimensional finite element mesh

analysis, but a mesh for pile groups is extended laterally to rm from group perimeter for estimating accurately the lateral interaction between neighboring piles. The meshes with various degrees of refinement, especially in the region surrounding the piles and near the pile head and tip where singularities occur, were tested until satisfactory convergence of the soil stresses around the pile was reached.

4. Parametric Study

The behavior of a pile in a group is influenced by the presence of and loadings on neighboring piles when piles are closely spaced. This is referred to as group effect. A major parameter influencing the group effect is the spacing between piles and pile head condition, so the cases of a single pile and of pile groups are analyzed.

To understand the true behavior, yielding of the soil at the pile-soil interface was considered by taking into account the effective strength parameters of a deposited weathered soil: the effective cohesion, c' and the effective friction angle, ϕ'.

The elastoplastic analyses were run to take into account the local yielding at the pile-soil interface and used an iterative and incremental analysis. For an elastoplastic state, the material stiffness was continually changed and thus the iterative process was repeated until the changes in material stiffness between successive iterations were negligible. The incremental procedure was composed of dividing the external load into many small and equal increments which are applied incrementally.

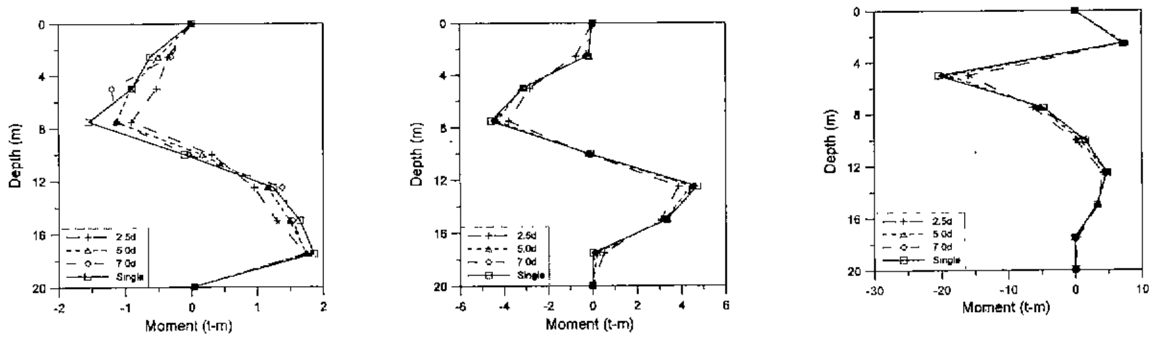
The cases of numerical analyses on pile groups were performed for different cap rigidity and spacing between piles (Table 1). The cases of a single pile and of pile groups are analyzed. The material properties used in this study was chosen to represent a typical weathered soil and are shown in Table 2.

Table 1. Numerical analysis for pile groups

	Cases
Pile Spacing (D=diameter)	2.5D
	5.0D
	7.0D
	Single
Pile head conditions	Free
	Unrotated
	Hinged

5. Group Effect By Interaction Factors

The information obtained by the three-dimensional parametric analysis can play a significant role in determining the group effect between piles. In order to investigate the effect of pile-soil interaction on the pile behavior, the response of each individual pile within a group will be compared with that of a single pile.



(a) Free head (b) Unrotated head (c) Hinge head

Fig. 4 Bending moment vs. depth

Group effect may be assessed in terms of loadings or bending moments and deflections for laterally loaded piles. For the present study, the group effect was primarily assessed based on the predicted bending moments.

In using pile bending moment for evaluating the group effect on the lateral pile response, an interaction factor is introduced which compares the maximum bending moment of a pile from a pile group analysis with that of the single pile analysis at the same amount of the free-field soil movement, and is expressed as follows;

$$\mu = \frac{M_{G(max)}}{M_{S(max)}} \tag{8}$$

where

μ = group interaction factor

$M_{G(max)}$ = maximum bending moment of the pile in a group

$M_{S(max)}$ = maximum bending moment of the single pile

Fig. 4 shows the bending moment distribution for a free head (Fig. 4-a), an unrotated head (Fig. 4-b) and for a hinge head (Fig. 4-c). This figure shows that there is a group effect and therefore a reduction in maximum bending moment for the three spacings studied and for different cap rigidities. The reason for this reduction is that as the pile spacing become smaller, the lateral soil movement between the piles is prevented more and more by the piles. In addition, it can be seen that the maximum bending moment is increased in order of free, unrotated and hinged head condition. This is because the soil movement pushes the piles laterally while the hinge head resists more lateral movement than free head.

Based on the non-linear analysis, the interaction factor defined as Eqn(8) can be summurized in Table 3. The interaction factor presented in the Table is ratio of the maximum bending moment on a pile in the group to the maximum bending moment on a single pile. Here, experiment means the testing result performed by the authors. For more detailed information of the experimental results, readers are referred to Jang et al. (1999). Based on the results, the prediction by FEM analysis is in fairly good agreement with the measurements by the authors.

6. Conclusion

The main objective of the analysis described herein was to investigate the interaction factor of pile groups subjected to lateral soil movement based on a three dimensional finite element model. A limited parametric study of the response of one row pile groups was performed to examine the interaction factor for different cap rigidity and spacing-to-diameter ratios varying from 2.5 to 7.0. The major parameters highly influencing the interaction factors are the group spacing. The interaction factors of the pile in a group was reduced as pile spacing decreases in order of unrotated, hinged and free cap rigidity and prediction by present analysis is in good agreement with general trend observed by experimental results.

Table 2. Material properties for pile groups

Soil	Unit-weight (kN/m^3)			17.66
	Plastic	Drucker-Prager	β ($^\circ$)	50
			Dilation Angle ($^\circ$)	0
		Drucker-Prager Hardening	Yeild Stress (Pa)	33983
			Plastic Strain	0
	Elastic		Elastic Modulus (Pa)	1.0×10^7
			Poisson's Ratio	0.3
Pile	Unit-weight (kN/m^3)			77
	Plastic		Elastic Modulus (Pa)	2.1×10^{11}
			Poisson's Ratio	0.25
	Length (m)			20
	Diameter (m)			0.4
	Thickness (mm)			9
	Head Condition			Free Unrotated Hinged
	Tip Condition			Horizontal Free Vertical Rigid
Interface	Soil-Soil (tan ϕ)			0.577
	Pile-Soil (tan δ)			0.364

Table 3. Interaction factor (μ)

Pile head condition		Free		Unrotated	Hinged	
		FEM	Experiment	FEM	FEM	Experiment
Pile spacing	2.5D	0.59	0.58	0.83	0.78	0.80
	5.0D	0.73	0.76	0.94	0.92	0.89
	7.0D	0.78	0.98	0.97	0.97	0.99
Single		1.00	1.00	1.00	1.00	1.00

Acknowledgements

This study was supported by the Ministry of construction and Transportation (Project No. 970009-2). This support is gratefully acknowledged.

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