말뚝주변 마찰력과 한계상대변위

On the Critical Relative Displacement between Pile Shaft and Surrounding Soil

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Abstract

Model pile pull-out tests have been executed to investigate the characteristics of the critical relative displacement at which the critical pile skin resistance is mobilized. Test result shows that the critical relative displacement is neither constant nor pile size dependent, but it is the most closely related with the magnitude of the critical skin resistance. The empirical relationship between the two quantities has been established.

Behavior of centrifuge physical models of skin-resistance-related problems has been investigated on a quantitative basis by a computational method. A pile downdrag problem has been employed as an example of the skin-resistance-related problems. A simple transfer function type method has been developed for the analysis of the downdrag. It is concluded from the analysis that centrifuge physical modeling of skin-resistance-related problems may lead to an erroneous result on an unconservative side, as may have been expected due to the violation of the similarity rule by the quantity of the critical relative displacement.

요약

말뚝의 한계 주변 마찰력을 발생시킬때의 말뚝과 주변 흙 사이의 상대변위 즉, 한계 상대 변위의 특성을 조사하기 위하여 모형 말뚝 인발시험을 하였다. 시험 결과에 의하면, 한계 상대 변위는 일정한 값을 갖지도 않고, 말뚝 크기에 비례하지도 않으나 한계 주변 마찰력의 크기와 밀접한 관계가 있음을 알았다. 본 논문에서는 모형실험 결과로부터 상대 변위와 한계 주변 마찰력 사이의 관계식을 제시하였다.

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주변 마찰력과 관련된 문제 해결을 위하여 원심력 시험기를 이용한 모형실험을 시행하는 경우, 실험결과의 신빙성을 정량적으로 분석하기 위하여 계산적 방법을 이용하였다. 그리고 주변 마찰력에 관련된 문제의 한 예로 부주변 마찰력 문제를 분석하였으며 부마찰력 계산을 위하여 간단한 전이 함수방법을 개발하였다.

분석결과로부터 얻은 결론은 주변마찰력 관련 문제에 대한 원심력 모형실험은 완전하지 않은 쪽의 오차를 초래할 수 있는데, 그 이유는 한계 상대 변위량이 상사법칙을 만족시키지 않기 때문인 것으로 생각된다.

Introduction

Foundation piles have gained their use more and more as buildings and engineering structures have become taller and larger in their sizes, and as the site conditions have become more and more complicated to meet all kinds of construction purposes. In spite of such a vast usage of pile foundations, it is true that the basic mechanism of pile behavior is still not completely understood. A load transfer problem between a pile shaft and surrounding soil belongs to this category.

In this study, research interest is focused, first, on the characteristics of the critical relative displacement at which the full skin frictional force bwteeen the pile shaft and surrounding soil is mobilized, and secondly, on the behavior of centrifuge physical models of a skin resistance related problem.

Due to the lack of the complete understanding of its behavioral mechanism and many uncertainties in analysis of pile foundation, it has been customary to perform full-scale pile tests at the site of important projects to verify actual pile response to various situations. In recent years, however, a quantitative modeling technique for geotechnical structures named as the centrifugal modeling method has become popular as a substitute measure for the full-scale tests because of its distinctive merits over prototype tests such as cost efficiency, quickness and ease of problem handling, still producing comparable results on a quantitative basis.

Modern researches on pile behavior have established(or, at least, declared so) that full mobilization of shaft resistance requires a relative displacement between the pile shaft and surrounding soil of 0.25 in. (0.635 cm) to 0.40 in. (1.016 cm), regardless of pile size and length(5, 6). However, if such declaration concerning the critical relative displacement precisely holds, since the similarity rules cannot be kept under such circumstances, centrifuge physical modeling effort, for skin-resistance-related pile problems on a quantitative basis, has to be discouraged or, at least, has to find the proper way of interpretation of test data in prior to the execution of modeling experiment.

To this end, model pile pulling-out tests have been performed, varying pile diameter, embedment depth, and pile surface condition, to verify the relations of the magnitude of critical relative displacement and pile shaft resistance with pile diameters and embedments. It is also attempted, by numerical analysis, to compare the predictions of physical modeling with prototype quantities. For this comparison, the pile downdrag problem is employed as an example of the skin-resistance-related problems, and a simple transfer function type method has been developed to analyse the downdrag problem.

Test Program and procedure

The test program consists of 99 model pile pulling-out tests in two different types of soils which are clay and sand. 27 tests are conducted in clayey soil, and the rest conducted in sandy soil. Identical test has been repeated four times with a few exceptions. Piles are made of aluminum whose diameter varies to be 0.156 in. (0.396 cm), 0.219 in. (0.556 cm), 0.281 in. (0.714 cm), or 0.625 in. (1.588 cm). Pile surfaces are either smooth or rough. The rough pile surfaces has been created by scrubbing the pile with a coarse grained, 50-D sand paper. Embedment depths are varied to be 10 in. (25.4 cm) or 7 in. (17.78 cm).

Sandy soils ($G_s=2.66$, $e_{max}=0.94$, $e_{min}=0.61$, $C_u=1.72$) are prepared in two different relative densities which are 96 % and 50 %. The soil densities are controlled using a raining device by adjusting a soil falling height and a width of long rectangular opening of the device. A pile is located in the middle of the empty container and supported to stand in that position by thin wires before the raining of sand onto it.

Clay soils are made by mixing Georgia Kaolin with Banny silt. The proportion of kaolin to silt is determined to be 2 : 8, which will produce such clay that consolidates in a reasonable period of time maintaining the general properties of clay. The engineering properties of this soil are shown in Table 1. The dry kaolin-silt mixture is thoroughly mixed with 50 % of water, by weight, to become a slurry state. This slurry mixture cannot hold a pile in its natural condition, thus, the mixture needs to be strengthened to have sufficient strength to hold the pile. This is achieved by consolidating the slurry inside a centrifuge by spinning the slurry mixture at 20g level for 60 minutes prior to the installation of a pile. After the installation of the pile, the pile-soil system is spinned, again, inside the centrifuge to create highter pile-soil adhesion. The g level employed at this stage is 60 with the spinning time of 80 minutes.

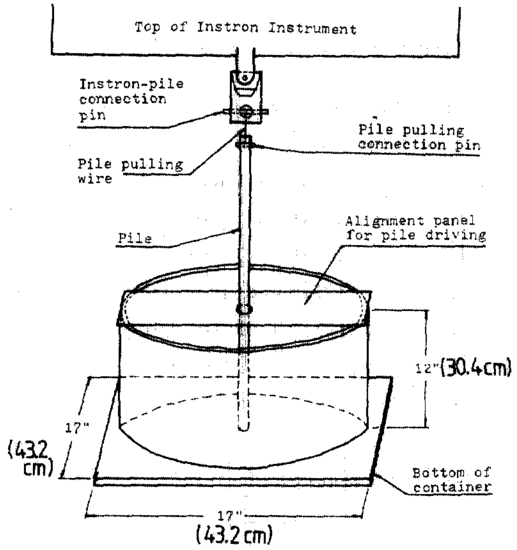

Fig. 1. Schematic Drawing of Pile Pull-out Test Arrangement.

Pile pulling-out tests are executed in an Instron loading machine. A schematic drawing of the test arrangement is shown in Fig. 1. All the piles are pulled out at the displacement rate of 0.02 in. (0.051 cm) per minute. The pulling resistance and the pulling distance (i. e., the relative displacement between pile and soil) are automatically recorded on the built-in chart of the machine.

Table 1. Engineering Properties of Laboratory Clay

Liquid Limit	LL=64.9
Plastic Limit	PL=18.6
Plasticity Index	PI = 6.3
Specific Gravity	Gs = 2.70
Undrained Shear Strength(at the stage of testing)	Cu = 0.79(psi) (0.049kg/cm^2)

Test Results and Discussions

As the main purpose of this study lies in the verification of the characteristics of the critical relative displacement, the pile diameter, the critical relative displacement and the critical pile shaft resistance are tabulated in Table 2. As observed from the table, no positive relationship can be found between the critical relative displacement and the pile diameter. The critical relative displacement (CRD), Which is determined to be the displacement at the point of maximum curvature in the pull-out resistance versus relative displacement curve, ranges from 0.002 in. (0.005cm) to 0.048 in. (0.122cm) for sand, and from 0.005 in. (0.013cm) to 0.016 in. (0.041 cm) for clay. The critical relative displacement appears to be neither constant nor pile size dependent. The wider range of the CRD in sand seems to be attributed to the wider range of the critical resistance, which idea can be verified by plotting the CRD against the resistance. These are plotted in Fig. 2. A linear regression analysis has been performed on the data shown in Fig. 2, from which the equation of the regression line is obtained as

$$Y(psi) = 23.85982X - 0.00236 \qquad (1)$$

$$= 24X(in),$$

or,

$$Y(kg/cm^2) = 0.67X(cm)$$

To correlation coefficient of the regression line is 0.88.

To verify the validity of the above equation, it is intended to compare the CRD calculated by the equation with a field data reported in a literature(Whitaker and Cooke, 1966,⁽⁷⁾). Pile no. 16 in the reference paper is picked up for the comparison as it is the only pile whose uplift resistance-displacement curve is presented in the paper. The relevant data are shown in Table 3 and in Fig. 3. A

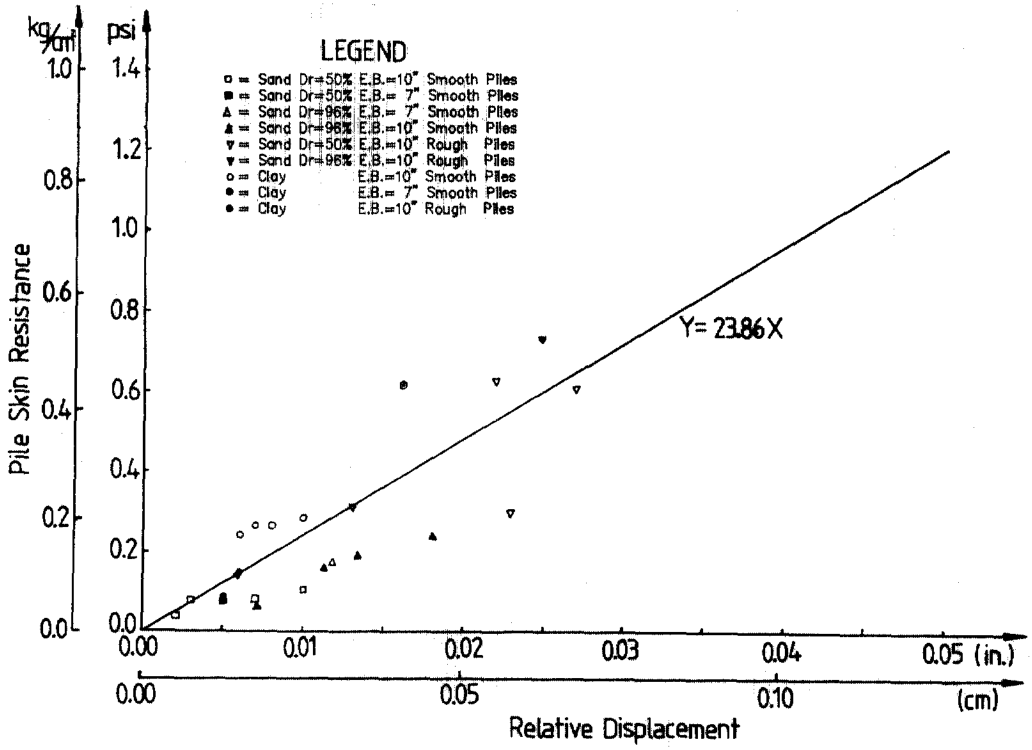

Fig. 3. Critical Relative Displacement vs Critical Pile Skin Resistance.

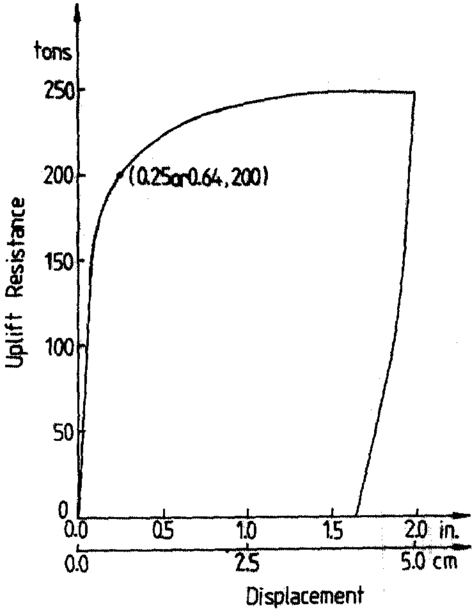

Fig. 3. Pile No. 16 Uplift Resistance vs Displacement.(From Whitaker and Cooke, 1966)

critical point in the resistance-displacement curve of pile no. 16 is set to have 0.25 in. (0.635 cm) displacement and 200 ton uplift resistance as indicated in Fig. 3. For the critical uplift resistance, the critical relative displacement can be calculated as

$$X(in)=\frac{1}{24}\left(\frac{200(ton)}{250(ton)}\right)\left(0.569\left(\frac{ton}{ft^2}\right)\right)\left(13.89\left(\frac{lb\cdot ft^2}{ton\cdot in^2}\right)\right) \quad (2)$$
$$=0.26(in.),$$

or,

$$X(cm)=0.660(cm).$$

This computed value, 0.26 in (0.66cm) agrees very well with the field data, 0.25 in (0.635 cm). It also belongs to the range of the CRD established by some colleagues in recent years. In fact, as the average unit skin resistance of friction piles being used in practice may

Table 2. Critical Relative Displacement vs Shaft Resistance

Pile Diameter in. (cm)	CRD in.(cm)	Shaft Resistance psi(Kg/cm²)	Ramarks
0.156(0.396)	0.007(0.018)	0.090(0.0063)	Sand,
0.219(0.556)	0.003(0.008)	0.080(0.0056)	Smooth Surface Pile,
0.281(0.714)	0.010(0.025)	0.100(0.0070)	Dr = 50%,
0.625(1.588)	0.002(0.005)	0.040(0.0028)	embedment = 10 in. (25.4cm)
0.281(0.714)	0.005(0.013)	0.090(0.0062)	embedment = 7 in. (17.78cm)
0.156(0.396)	0.013(0.033)	0.200(0.0141)	Sand,
0.219(0.556)	0.011(0.028)	0.150(0.0105)	Smooth Surface Pile,
0.281(0.714)	0.018(0.046)	0.230(0.0169)	Dr = 98%,
0.625(1.588)	0.007(0.018)	0.070(0.0049)	embedment = 10 in. (25.4cm)
0.281(0.714)	0.012(0.030)	0.160(0.0112)	embedment = 7 in. (17.78cm)
0.156(0.396)	0.022(0.056)	0.630(0.0443)	Sand,
0.219(0.556)	0.013(0.033)	0.310(0.0218)	Rough Surface Pile,
0.281(0.714)	0.027(0.069)	0.610(0.0429)	Dr = 50%
0.625(1.588)	0.023(0.058)	0.310(0.0218)	embedment = 10 in. (25.4cm)
0.156(0.396)	0.028(0.071)	1.140(0.0801)	Sand,
0.219(0.556)	0.025(0.064)	0.740(0.0520)	Rough Surface Pile,
0.281(0.714)	0.051(0.130)	1.400(0.0984)	Dr = 96%,
0.625(1.588)	0.048(0.122)	0.750(0.0527)	embedment = 10 in. (25.4cm)
0.156(0.396)	0.007(0.018)	0.260(0.0183)	Clay,
0.219(0.556)	0.006(0.015)	0.240(0.0169)	embedment = 10 in. (25.4cm)
0.281(0.714)	0.010(0.025)	0.280(0.0197)	Smooth Surface Pile.
0.625(1.588)	0.008(0.020)	0.260(0.0183)	
0.281(0.714)	0.016(0.041)	0.620(0.0436)	Rouh Surface Pile.
0.281(0.714)	0.006(0.015)	0.160(0.0112)	Smooth Pile, 7 in. (17.78cm)
0.281(0.714)	0.005(0.013)	0.100(0.0070)	Spinned at Lower g.
0.281(0.714)	0.006(0.015)	0.150(0.0105)	Longer Spin Time.

Table 3. Uplift Test on Anchor Pile(After Whitaker and Cook, 1966)

Pile No.	Age at Test week	Dia. in. (cm)	Length ft(m)	Weight + Suction(tons)	Total uplift resistance (ton)	Frictional resistance t/ft^2(t/m^2)
13	8	35(89)	41.0(12.5)	24.0	234	0.560(6.028)
17	4	37(94)	41.5(12.7)	27.0	305	0.692(7.449)
16	15	37(94)	40.5(12.3)	26.5	250	0.569(6.125)
10	62	37(94)	41.0(12.5)	26.8	282	0.643(6.921)

Tabel 4. Pile Surface Effect

Pile Dia. in. (cm)	Rough Pile Resistance/Smooth Pile Resitance		
	Sand		clay
	Dr=96%	Dr=50%	
0.156(0.396)	5.70	7.00	N.A.*
0.219(0.556)	4.93	3.88	N.A.
0.281(0.714)	6.09	6.10	2.21
0.625(1.588)	11.19	7.75	N.A.

*N.A. : Not Available.

Table 5. Embedment Effect on Pile Shaft Resistance(0.281 in.(0.714cm) Smooth Surface Pile)

Soil Type	Shaft Resistance of 7 in. Pile/10 in. pile (17.78cm) (25.4cm)
96% Sand	0.70
50% Sand	0.90
Clay	0.57
Average	0.72

be assumed to have a certain renge, the declaration of a certain range of CRD like 0.25 in. - 0.40 in. (0.635 - 1.000cm) may have a practical meaning.

It may be also noticed from table 2 that the shaft resistances of 0.625 in. (1.588 cm) smooth pile in sand are much smaller than those of differnet size piles, which may be attributed to the different frictional characteristices of different types of aluminum. The 0.625 in. (1.588 cm) pile is made of 6061 T-6 aluminum, and the rest made of 2024 T-3 aluminum.

Table 4 reflects the effect of the pile surface conditions. As noticed from the table, pulling resistance ratio of rough to smooth piles in dense sands falls in between 5 to 6 except that of 0.625 in. (1.588 cm) pile which is even greater than 10. The resistance ratio for loose sands ranges from 4 to 8, while the ratio for clay is only slightly bigger than 2. It is regarded reasonable that piles in cohesive soil are less affected by the pile surface condition than those in chohesionless soil.

Effect of pile embedment depths is shown in Table 5. The ratio of pulling resistance of 7 in. (17.78 cm) embedment pile to that of 10 in. (25.4 cm) embedment pile is 0.70, 0.90, and 0.57 for 96% sand, 50% sand, and clay, respectively. The ratio is 0.72 in average. It may be said that pulling resistance is approximately proportional to the embedment depth for the piles tested in this study.

Computational Predictions on Downdrag

As discussed earlier, the critical relative displacement, at which the critical pile shaft resistance developes, appears to be dependent upon the critical stress instead of pile size, which implies the violation of the similarity rule by the critical relative displacement quantity when the skin-resistance-related problems are physically modeled at a reduced scale in a centrifuge. To verify the validity

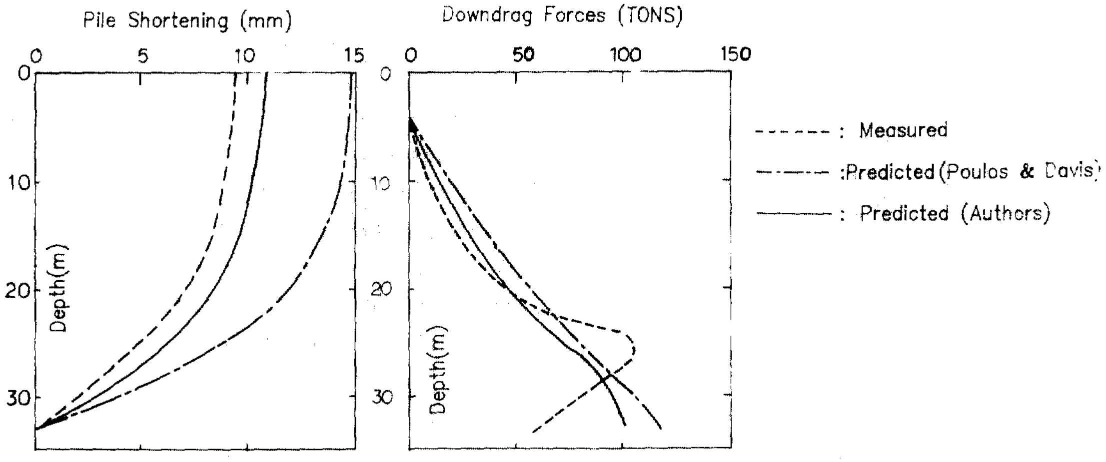

Fig. 4. Comparison with Pile Test of Bjerrum et. al. (1969) at Heroya

of the centrifuge physical modeling predictions on such problems, the pile downdrag problem is employed as an example, and a simple transfer function type method has been developed to analyze the downdrag problem.

The simple method may be summarized as follows :

1. The pile is divided into a number of small elements.

2. The critical relative displacement (CRD) is calculated for a given pile for a given pile-soil system using the empirical equation, $\tau_{av}=24$ CRD [in terms of lb-in. units], where τ_{av} represents the pile-soil shear strength at the mid-depth of pile embedment.

3. The soil movement at each element is compared with the CRD to determine slip elements.

4. The downdrags for the slip elements are set to be equal to the pile-soil shear strengthes for the corresponding elements, and for the non-slip elements, the downdrags are calculated by

$$P_i=\tau_{av}\times\frac{\rho_i}{CRD} \qquad (3)$$

where ρ_i : soil movement(3) at the ith element

5. The magnitude of pile shortening due to the downdrag force is calculated at each element.

6. The net soil movement at each element is obtained by subtracting the pile shortening from the original soil movement.

7. Steps from 2 to 6 are repeated using the net soil movement until identical downdrag forces are obtained. Normally, 3 iterations are enough.

For the purpose of compariosn of the pile behavior predicted by the simple method with an observation of pile behavior in the presence of downdrag, a documented case of field measurement (Bjerrum et. al., 1969,(1)) has been considered.

Test of Bjerrum et. al. --- Heroya Site, Pile A

$L=30m$ (7.5m of fill included),
$d=0.3m$ (7mm wall),
Unit wt. of fill$=19.6KN/m^2$,
Submerged unit wt. of clay$=9.81KN/m^2$
$(K_s \tan\phi_a')_{fill}=0.20$, $(K_s \tan\phi_a)_{clay}=0.25$,
$C_a'=0.0$, $E=2.06\times10^5$ MN/m^2
$\rho_{top}=20cm$(settlement of soil at the surface)

Comparisons between measured and predicted downdrag force distribution and pile shortening reveal good agreements as shown in Fig. 4. There is also good agreement between the present solution and that of

Table 6. Numerical Estimation of Centrifuge Model Behavior on Pile Downdrag

	Maximum Downdrag(ton)	Pile Top Shortening(cm)
Prototype	102.2	1.07
Model(Scale Factor of Geometry = 100)	11.1	0.0016

Poulos and Davis (1980,(4)) as shown in the same figure, which was obtained by an elastic solid method (2, 3) using a compatibility requirement between the displacements of the pile and the adjacent soil for each divided element of the pile. Using the simple method developed in this study, the behavior of a centrifuge model pile of a reduced scale by a factor of 100 simulating the pile A tested by Bjerrum et. al. has been predicted and compared with those of prototype in Table 6. As noticed from the table, the model pile, in this particular case, underestimates the maximum downdrag force by as much as 90%. The amount of the discrepancy may differ case by case depending upon the characteristics of surrounding soil movement distribution along the pile depth and the magnitude of the modeling scale factor, but as may have been expected due to the violation of the similarity rule by the quantity of the CRD, it is obvious that the centrifuge physical modeling of the skin-resistanc-related problem will always produce the under estimated results. This statement does also apply for the elastic solid method whose result should be unaffected, by its nature, by reducing the geometry of a given pile-soil system.

Conclusion

From the test results and the numerical analysis performed in this study, conclusions are drawn as the followings :

1. The critical relative displacement(X) is the most closely related with the critical pile skin resistance(Y), whose relationship has been determined experimentally as Y(psi) = 24 X (in.) ; $Y(kg/cm^2) = 0.68\ X\ (cm)$.
2. Pile surface condition can affect the pile shaft resistance(at least, for the model piles tested here)by more than a factor of 10 in the extreme case.
3. Centrifuge model tests on skin-resistance-related problems may lead to an erroneous result on an unconservative side.

Acknowledgement

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Bibliography

1. Bjerrum, L., Johannesson, H.J. & Eide, O. (1969). "Reduction of Skin Friction on Steel Piles to Rock." *Proc. of 7th ICSMFE,* vol. 2 : 27~34.
2. D'Appolonia, E. & Romualdi, J.P. (1963). "Load Transfer in End-bearing Steel H-Piles." *J. of SMFE, ASCE,* Vol. 97, SM1 : 77~93.
3. Poulos, H.G. & Mattes, N.S. (1969). "The Analysis of Downdrag in End-Bearing Piles Due to Negative Friction." *Proc. of 7th ICSMFE,* Vol. 2 : 204~209.
4. Populs, H.G. & Davis, E.H. (1980). *Pile Foundation Analysis and Design.* John Wiley and Sons.
5. Vesic, A. S.(1969). "*Load Transfer, Lateral Loads, and Group Action of Deep Foundation.*' Performance of Deep Foundations, ASTM, STP No. 444 : pp. 5~14.
6. Vesic, A.S. (1977). *Design of Pile Foundation,* National Cooperative Highway Research Program Synthesis 42 : pp. 9.
7. Whitaker, T. & Cooke, R.W. (1966). "An Investigation of the Shaft and Base Resistance of Large Bored Piles in London Clay." *Proc. of Symp. on Large Bored Piles : 7~49.*

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