현장타설말뚝의 전단강도 조정계수 결정법
Determination of Shear Strength Modification Factors in Drilled Shaft

김명학[1], Myung-Hak Kim and Michael W. O'Neill[2]

[1] 인제대학교 토목공학과 조교수, Assistant Professor, Dept. of Civil Engineering, Inje University
[2] 휴스턴대학교 토목공학과 교수, Professor, Dept. of Civil and Envir. Engineering, Univ. of Houston

SYNOPSIS : 팽창토에 설치된 직경 305 mm 현장타설말뚝의 18개월간에 걸친 거동을 관찰하였다. 계절적 함수량 변화에 따른 말뚝주변 흙의 부피 변화가 발생시킨 말뚝의 인발력을 측정하였고, 측정한 인발력에서 말뚝 단위 표면적당의 전단 응력을 계산하였다. 본 실험 말뚝에서는 최대 전단 응력은 54 kPa 이 계산되었다.

An experimental study is described in which a 305-mm-diameter instrumented drilled shaft was installed in a moderately expansive clay soil during the dry season and monitored over a period of about 18 months. The purpose of the study was to investigate the effects of seasonal moisture changes in the soil on the shear stresses imposed on the sides of the drilled shaft and movements of the shaft head. The soil in the vicinity of the test shaft was instrumented to measure suction and ground surface movement, and the relation between suction, total stress and shear strength of the soil at the test site was determined through laboratory triaxial compression testing. Daily rainfall and temperatures were also monitored at the test site, the National Geotechnical Experimentation Site at the University of Houston, where control on surface grading and vegetation existed. Over the course of the study induced unit side shear values of up to 54 kPa were measured in the test shaft. A simple computational model was developed that related observed suction changes to unit side shear induced by the expansion of the soil through the use of the laboratory suction-total stress-shear strength relation.

주요어 (Key words) : Drilled Shaft, Expansive Clay, Shear Strength

1. Introduction

Drilled shaft foundations are frequently used to bypass expansive surficial soils that otherwise possess strength and compressibility properties adequate to support shallow foundations (O'Neill and Poormoayed, 1980). However, expansive clays near the surface can exert large shearing stresses on the perimeters of drilled shafts that can produce excessive upward movement and even tensile failures when the shafts support light structures.

In designing a drilled shaft to penetrate through expansive soils, it is incumbent upon the designer to predict the maximum magnitude of upward-directed shearing stresses that will be exerted on the drilled shaft by the expanding soil. These efforts are necessary in order to determine the appropriate anchorage details in the soil or rock located below the expansive soil and to choose a reinforcing steel schedule that will not result in tensile failure of the drilled shaft. Traditionally, these shearing stresses have been estimated for design purposes by empirical means. For example, O'Neill (1988) recommended that the maximum upward-directed shearing stress imposed on the perimeters of drilled shafts due to expansive soils, f_{max}, be estimated as αs_u, where s_u is the undrained shear strength of the soil at the time of sampling and α is a shear strength reduction factor. Based on limited data, α was estimated to be 0.6 in stiff clays.

However, this simplistic method suffers from the fact that undrained shearing probably does not occur in the process and that the reference value of s_u is not necessarily the value of s_u at the time of sampling. In order to improve on the prediction of uplift stresses imposed on drilled shafts, an experimental study of the behavior of a drilled shaft was performed in which the shear strength of the soil in which it was embedded was correlated to total stress and matric suction over a long period of time.

2. Shear Strength of Unsaturated Soils

An expression for the shear strength of an unsaturated soil, in terms of the stress state variables, Eq. (1), was proposed by Fredlund et al. (1978).

$$s = c' + (\sigma_n - u_a) \tan \phi' + (u_a - u_w) \tan \phi^b, \tag{1}$$

where s = shear strength; c' = effective cohesion intercept; σ_n = total normal stress on the shearing surface; ϕ' = angle of internal friction with respect to $(\sigma_n - u_a)$; ϕ^b = angle indicating the rate of increase in shear strength with $(u_a - u_w)$; u_w = pore water pressure (always negative), and u_a = pore air pressure (usually atmospheric). The term $(\sigma - u_a)$ is the net normal stress, and the term $(u_a - u_w)$ is termed the matric suction. It is significant to note that the total stress changes and matric suction changes do not produce equivalent shear strength and volume changes in an unsaturated soil.

3. Description of Site

The experimentation site was the National Geotechnical Experimentation Site (NGES-UH) at the University of Houston, Texas. Test site stratigraphy is shown in Fig. 1. The site is flat and vegetated with short, native grass. The soil "loading" was exclusively by natural rainfall and atmospheric thermal effects. There were no surface membranes, and neither the drainage pattern nor the vegetation was varied during the experiment.

Three separate soil strata of concern to this research were identified on the site. The top 2.7 m consisted of stiff to very stiff tan and gray clay, which classified as CH in the Unified Soil Classification System. The next 0.8 m consisted of a stiff gray and tan very sandy clay with waterbearing sand seams (CL), and the underlying stratum was a stiff to hard red and light gray clay (CH). The last stratum extended to a depth of at least 7 m. Minor amounts of free ground water were encountered at a depth of 2.8 m during shaft excavation.

4. Field Test

Details of field test set-up are given by Kim and O'Neill (1998) and Kim (1996).

4.1 Uplift Forces and Unit Side Resistance

Figure 2 shows the tensile forces in the test shaft as functions of time and depth in the shaft. Positive values indicate tension forces in the shafts. The maximum tensile force in the shaft was 55 kN and occurred at a depth of 2.0 m in March and April, 1995, about six months after shaft construction.

As with the changes in soil surface movements, the loads within the shaft responded rapidly to changes in rainfall. For example, the tensile load at a depth of 2.0 m increased from about 2 kN at the time of the heavy rains in October, 1994, to about 32 kN one week later.

Unit side shear resistances as functions of time at different depths in the shaft are shown in Fig. 3. Positive values indicate an upward-directed side resistance. After the heavy rain in October, 1994, side resistances were always positive. During the wet and cool season (November, 1994 to March, 1995), unit side resistance was highest in the depth range of 0.0 to 0.45 m, where it reached a value of 54 kPa.

5. Side Shear Modification Factor

The upward-directed shearing stress imposed on the drilled shaft for a given suction state is not necessarily identical to the shear strength of the soil indicated from the laboratory tests for that suction state, for several reasons. First, the structure of the soil directly against the face of the drilled shaft is modified by drilling the borehole for the shaft, and perhaps strengthened slightly by migration of lime or cement from the concrete. Second, the isotropic stress conditions assumed in the laboratory tests may be in error for shallow depth ranges. Third, the suction in the soil immediately adjacent to the drilled shaft may not be equal to the suction at the points in the soil mass at which the suction sensors were placed (0.45 m from the shaft).

From a practical perspective it is important to determine the relation of the developed values of imposed side shearing stresses, f, to the suction-dependent shear strength of the soil. The procedure for accomplishing this objective is described in this section.

Weekly averaged suction values in the two suction columns were determined at each depth and denoted $\tau_{0.45\ m}$ and $\tau_{1.1\ m}$. τ in the depth range between 0 and 0.45 m, denoted as $\tau_{0.23\ m}$, was assumed to be equal to the average of the two weekly averages measured at the depth of 0.45 m ($\tau_{0.45\ m}$), while τ in the depth range between 0.45 m and 1.1 m ($\tau_{0.78\ m}$) was assumed to be the average value of the two weekly averages of τ measured at depths of 0.45 m ($\tau_{0.45m}$) and 1.1 m ($\tau_{1.1\ m}$). The depth of the zone of seasonal moisture change (active zone) was established at 1.68 m based on the measurements of liquidity index over a long period of time, reported by O'Neill and Poormoayed (1980). τ at the depth between 1.1 m and the active zone depth (1.68 m) was assumed as the average of the two weekly averaged values at 1.1 m ($\tau_{1.1\ m}$) and 3.0 bars, which is the equilibrium suction value (τ_e) at the NGES-UH (Kim, 1996). Figure 4 indicates the approximate extreme ranges of wet and dry suction for the site (boundary suction profile). Wet boundary suctions decrease from 3.0 bars above 1.68 m and dry boundary suctions increase from 3.0 bars above 1.68 m.

Equation representing results of suction-confinging pressure-shear strength tests are given by

$$s(\text{kPa}) = 17.3e^{0.001775(\text{kPa})} \qquad \text{(Kim, 1996)} \qquad (2)$$

Equation (2) was then used to estimate the time- and depth-dependent shear strength in the soil surrounding the shaft based on the measured suction interpreted as described above.

The measured side shear stress distributions in the shaft in the depth range between 1.1 m and 2.0 m were transformed to a depth range corresponding to the depth of the active zone (1.1 - 1.68 m), that is, assuming that all of the imposed shearing resistance measured between 1.1 and 2.0 m actually occurred between 1.1 and 1.68 m. The resulting side shear stress distribution was divided by the distribution of shearing strength obtained from the suction values to obtain the shear strength modification factor, termed $\alpha(\tau, \sigma_c)$ as a function of time and depth, as illustrated in Fig. 10. Note that this is not the traditional α factor for drilled shafts or piles, but it is a factor that relates the unit induced side shear stress to a suction- and total stress-correlated shear strength, which varies with time and depth. The numerical calculations indicated in Fig. 4 were performed at several discrete depths, and the results are shown in Figs. 5 and 6 for two selected depths.

5.1 Discussion

From Figs. 5 and 6, the pattern of $\alpha(\tau, \sigma_c)$ from the ground surface to a depth of 1.68 m can be described. From Fig. 5 (depth range of 0.45 m - 1.1 m) $\alpha(\tau, \sigma_c)$ appears to exhibit three different patterns with time. The most critical condition for induced uplift exists when the combination of the products of $\alpha(\tau, \sigma_c)$ and s within all of the depth zones considered is a maximum. Both $\alpha(\tau, \sigma_c)$ and s are dependent upon suction and therefore upon time. This phenomenon is examined for the study described here in terms of time phases associated with surface moisture, first in the upper depth range (Fig. 11), then in the lower.

1. During the Phase 1 in the upper zones (above 1.1 m), (path 1 in Fig. 7) in which $\alpha(\tau, \sigma_c)$ peaks at 1.37, the appropriate soil suction value is about 2 bars at the time $\alpha(\tau, \sigma_c)$ maximizes, and the mean confining pressure σ_c is the mean total vertical pressure at a depth of 0.45 m, or 12.3 kPa, for the upper segment (0 to 0.45 m) or the value at a depth of 1.1 m, or 24.5 kPa, for the second segment (0.45 to 1.1 m). Figure 4 shows that there is little dependence of s on σ_c for the study site, so that in a practical sense, s varies only with τ. Thus, a unique relation, Eq. (2), is assumed to exist between s and τ in all depth intervals. The high value for $\alpha(\tau, \sigma_c)$ of 1.37 at this shallow depth may a result of anisotropic total stresses in the soil not having been replicated in the laboratory tests on which Eq. (2) is based.

2. During the early part of the Phase 2 (path 2 in Fig. 13), $\alpha(\tau, \sigma_c) = 0.95$ and $\tau = 6$ bars. During the latter part of Phase 2 (path 2) $\alpha(\tau, \sigma_c)$ is around 0.4, which corresponds to a τ value of 11 bars.

3. During Phase 3 (path 3) $\alpha(\tau, \sigma_c)$ varies from about 0.3 to -0.1, which corresponds to a suction value of about 9 bars. Note that the low values for $\alpha(\tau, \sigma_c)$ in Phases 2 and 3 indicate that once the soil shrinks away from the shaft, the suction measurements made in the soil mass are not relevant to the shear strength at the soil-shaft interface.

The most critical multiplicative combination of $\alpha(\tau, \sigma_c)$ and s above 1.1 m (highest numerical value) needs to be used determine the maximum value of fmax in that depth range. In this case, it is $\alpha(\tau, \sigma_c) = 0.95$ and s based on τ of 6 bars (early part of Phase 2). The most critical condition for the depth range between 1.1 and 1.68 m (Fig. 7) occurred in May of 1995, about 8 months after installation, near the end of Phase 2, when $\alpha(\tau, \sigma_c) = 1.1$ and τ was approximately 7 bars.

6. Conclusions

A study to investigate the underlying mechanisms of the effects of seasonal movements of expansive soils on a drilled shaft foundation at a flat site with no surface cover except natural grasses was performed.

The following are the significant conclusions of the study.

1. The critical shear strength modification factors $\alpha(\tau, \sigma_c)$ for the drilled shaft tested, to a depth of 1.1 m, can be categorized in three phases, described below. Only the first value would be critical for design.
   - $\alpha(\tau, \sigma_c)$ = 0.95 when the soil is at the end of the initial rapid wetting phase or beginning of the following drying phase and suction is less than 6 bars or less in the drying phase,
   - $\alpha(\tau, \sigma_c)$ = 0.4 when suction is greater than 6 bars in the slow drying phase, and
   - $\alpha(\tau, \sigma_c)$ = 0.2 when the soil is in a second slow wetting phase.
2. The critical value of $\alpha(\tau, \sigma_c)$ was 1.1 for z > 1.1 m but above the base of the zone of seasonal moisture change for the drilled shaft tested.

REFERENCES

1. Fredlund, D.G. (1983). "Prediction of Ground Movements in Swelling Clays," Presented at the 31st Annual Soil Mechanics & Foundation Engineering Conference, ASCE, Invited Lecture, Minneapolis, MN.
2. Fredlund, D.G., Morgenstern, N.R., and Widger, R.A. (1978). "The Shear Strength of Unsaturated Soils, " Canadian Geotechnical Journal, Vol. 15, pp. 313-321.
3. Johnson, L.D., and Snethen, D.R. (1978). "Prediction of Potential Heave of Swelling Soils," Geotechnical Testing Journal, ASTM, Vol. 1, No. 3, pp. 117-124.
4. Kim, M. H. (1996). "Effect of an Expansive Clay on the Behavior of Drilled Shaft," Ph. D. Dissertation, University of Houston, Houston TX.
5. Kim, M. H. and O'Neill M.W. (1998). " Side Shear Induced in Drilled Shaft by Suction Change," Journal of Geotechnical and Geoecvironmental Engineering, Vol. 124, No. 8, August, pp. 771-780.
6. O'Neill, M.W. (1988). "Adaptive Model for Drilled Shafts in Expansive Clay," ASCE Geotechnical Special Publication, No. 16, pp. 1-20.
7. O'Neill, M.W., and Poormoayed, N. (1980). "Methodology for Foundations on Expansive Clays," Journal of the Geotechnical Engineering Division, ASCE, Vol. 106, No. GT 12, December, pp. 1465-1488.

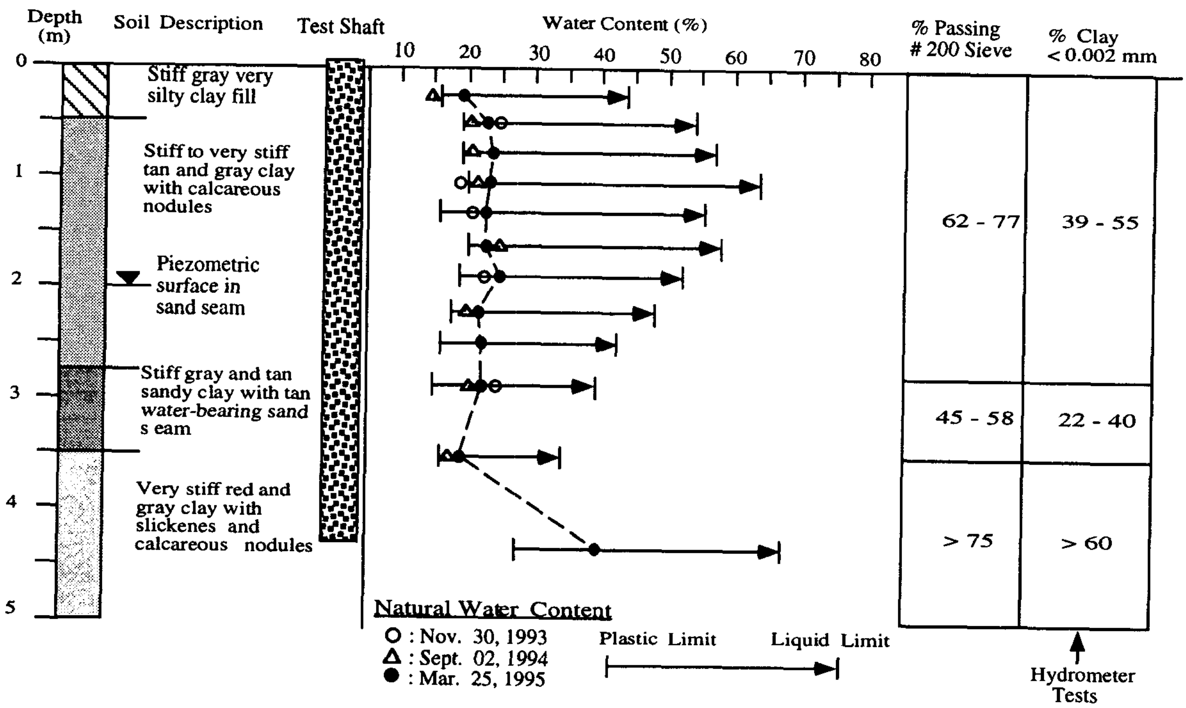

Fig. 1. Test Site Stratigraphy

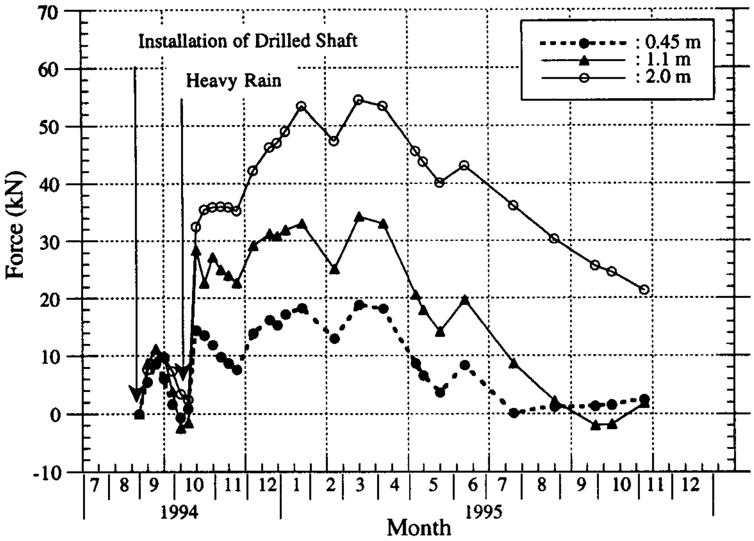

Fig. 2. Uplift Force vs. Time

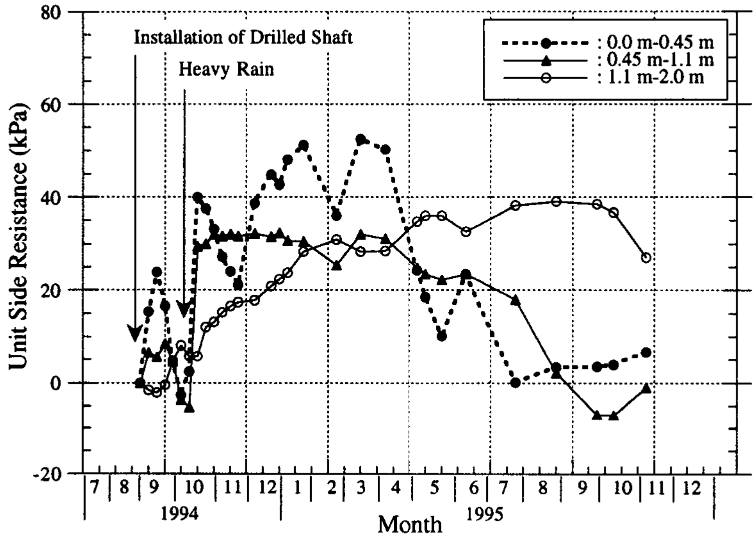

Fig.3. Unit Side Resistance vs. Time

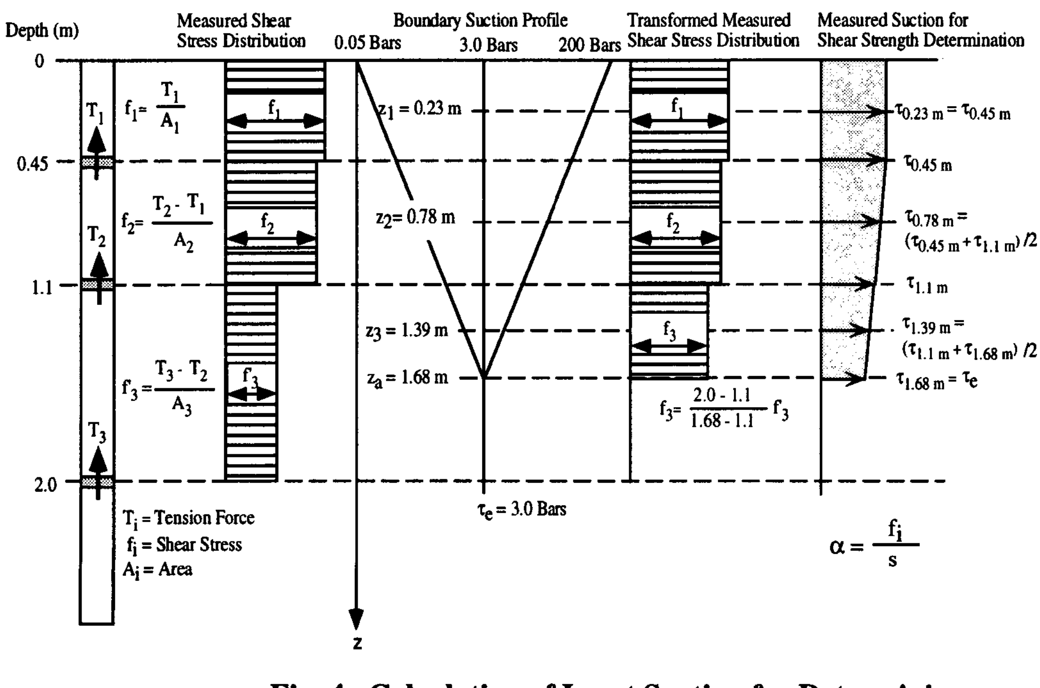

Fig. 4. Calculation of Input Suction for Determining α

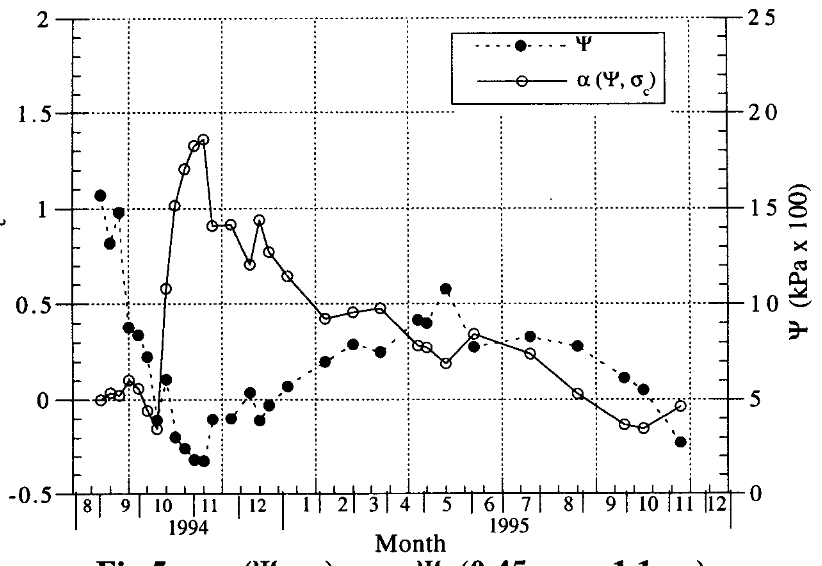

Fig.5. α (Ψ, σ_c) vs. Ψ (0.45 m - 1.1 m)

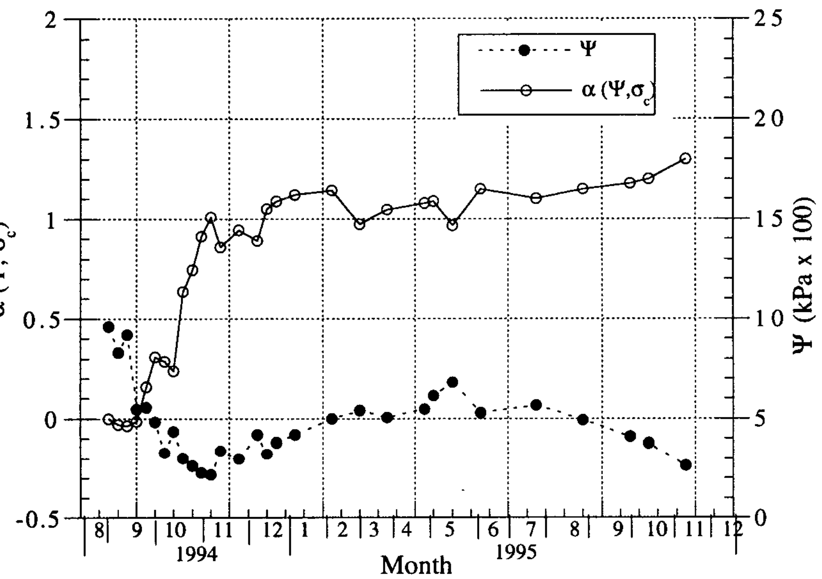

Fig. 6. α (Ψ, σ_c) vs. Ψ (1.1 m - 1.68 m)

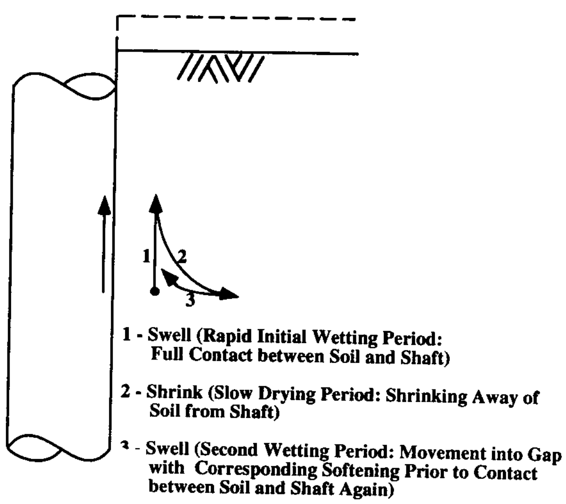

Fig. 7. Possible Path of Soil Particle Adjacent to Drilled Shaft During Swelling and Shrinkage