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Comparison of Performance between Regular Drilled Shaft and Isolation Tube Drilled Shafts

일반 현장 타설 말뚝과 분리형 현장 타설 말뚝의 거동 비교

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요 지

과압밀 점토층에 설치된 4개의 직경 305 mm현장 타설 말뚝 (1개의 일반 현장 타설 말뚝과 불포화 점토층에서 발생하는 주면 마찰력을 경감시키기 위해 분리형 튜브를 사용한 3개의 분리형 현장 타설 말뚝)의 거동을 분석 비교하기 위한 실험적 연구를 수행하였다. 흡수력 측정을 하기위해 말뚝 주변에 지반흡수력 측정 장치를 설치하였고 지표면 표고 변화도 측정하였다.

지표면 표고는 최고 40 mm까지 팽창하였고, 일반 말뚝은 5 mm, 분리형 말뚝은 1.5 mm 미만의 말뚝 두부 수직인발변위가 관측되었다. 분리형 말뚝에서 관측된 주면마찰력은 일반형 말뚝에 비해 미소하여 분리형 튜브를 사용한 분리형 현장타설 말뚝이 주위 지반의 팽창 수축에 의해 말뚝에 발생하는 주면 마찰력을 경감시키는데 매우 효과적이었다.

Abstract

An experimental study that included detailed observation of four 305-mm-diameter drilled shafts, one reference shaft of standard design and three test shafts with isolation tubes to mitigate skin friction in the vadose zone of a clay soil profile, is described. The shafts were loaded only by naturally expanding and contraction soil over a period of 17 months. The soil at the test site was instrumented to track suction and elevation changes.

Maximum ground surface movements exceeding 40 mm were observed. Heave movements of less than 1.5 mm were observed in the test shafts with isolation tubes, while movements of 5 mm were observed in the reference shaft. Unit side shear loads in the shafts protected by the isolation tubes were minimal compared to those measured in the reference shaft. This indicates that the isolation tubes were very effective.

Keywords : Expansive soil, Drilled shaft, Isolation tube, Suction, Swell, Clay

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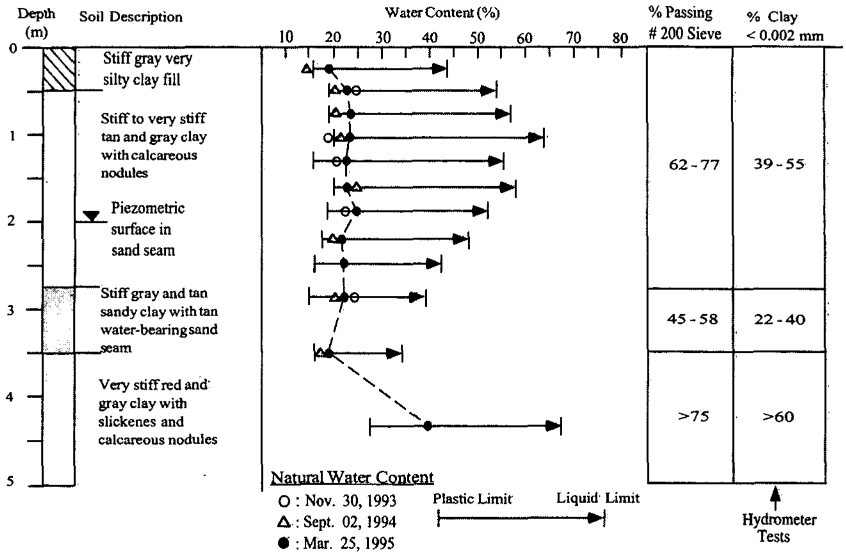

Fig. 1. Test site stratigraphy

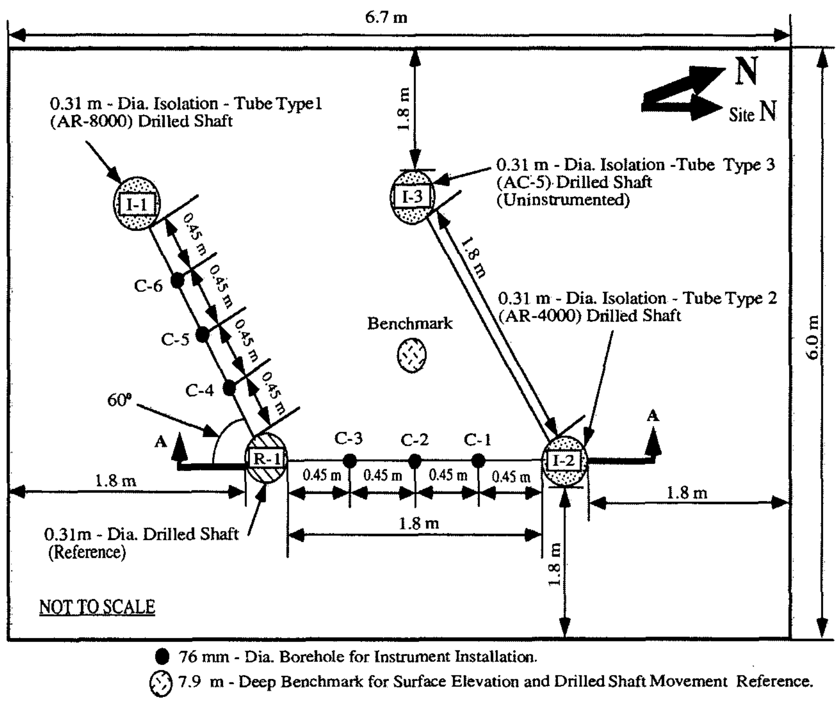

Fig. 2. Plan view of test site

1. Introduction

Soils with a potential for shrinking and swelling are found throughout the world (Johnson and Snethen, 1978). This phenomenon is driven by the soil moisture retention potential, termed "soil suction." Soil suction is a negative pore pressure that is directly related to effective stress in the soil framework. Since suction is a component of effective stress, it is an accurate indicator of the volume and strength states of specific soils (Fredlund, 1983; Johnson and Snethen, 1978). Therefore, soil suction change is an appropriate indicator of volume change and shear strength in expansive soils.

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.

This paper focuses on the performance of foundations in expansive soils, particularly the use of isolation tubes installed during construction to mitigate the effect of swelling soils on movements and tensile stresses in drilled shafts. A description of the mechanisms for producing such movements and stresses in a drilled shaft, in particular the standard, reference shaft in this study, was given by Kim and O'Neill (1998).

2. Description of Experimentation Site

The study was performed in the field at the National Geotechnical Experimentation Site at the University of Houston, Texas, USA (NGES-UH). Test site stratigraphy is shown in Fig. 1. The site is flat and vegetated with short, native grass.

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 gray and tan clay, which classified as CL to 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(CL). The last stratum extended to a depth of at least 7 m. Minor amount of free ground water was encountered at a depth of 2.1 m during test shaft excavation.

3. Laboratory Tests

A laboratory study was performed to relate total suction (Ψ) in the soil in the upper 2.0 m (above piezometric surface) to mean normal total stress (σ_c) and vertical strain (ε_v), considering the soil to be initially uncracked and to be constrained from straining laterally (Kim, 1996). σ_c was defined as σ_v (vertical total stress) times $[(1+2K_o)/3]$, in which K_o , the coefficient of earth pressure at rest, was defined from profiles determined in earlier studies at the test site (O'Neill and Yoon, 1995). Space limitations do not permit the presentation of details of the laboratory study, but the mathematical relation derived from the laboratory experiments, performed by using 72-mm-diameter samples and modified triaxial cells, for decreasing suction (swelling soil), can be written as shown in Equation 1 (Kim, 1996).

$$\varepsilon_v(\%) = [9.14 - 0.12\sigma_c(\text{kPa})][1.28 - \log\Psi\ (\text{bars})],\ \sigma_c < 76\ \text{kPa}\ ;\ \Psi > 3\ \text{bars}, \tag{1}$$

in which 1 bar = 100 kPa. Equation 1 was used to predict the movement of the soil surface over the testing period of 17 months from measured changes in suction (Ψ) at the site. Using the maximum suction profiles measured in the soil at the test site over the 17 month test period, shown later in Fig. 5, a ground heave of 68 mm at the test site was predicted by using Equation 1. This is well in excess of the 5 to 10 mm of relative movement between soil and shaft commonly believed to be necessary to develop the full shearing strength of the soil at the soil-shaft interface. The corresponding maximum measured value, however, was only 41 mm, which was still more than sufficient to develop large values of side shear. This discrepancy suggests that a factor to account for internal lateral swelling into secondary features such as open joints and slickensides is required to multiply the value of ε_v from Equation 1. For this site, that value empirically is 41/68 = 0.60. The 41 mm of heave occurred entirely in the soil above the piezometric surface (depth of 2 m).

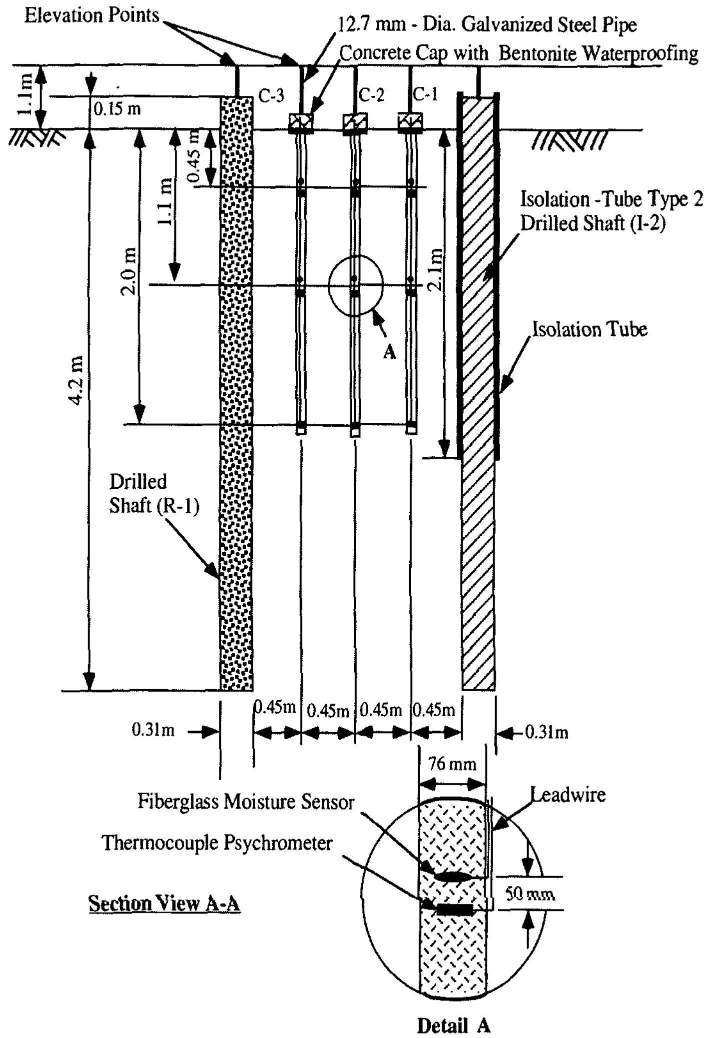

Fig. 3. Section view A-A (Fig. 2) showing dimensions of the drilled shafts and locations of ground instruments

4. Field Experiment

4.1 Test Site Layout and Installation of Instruments

Three drilled shafts with isolation tubes and one reference drilled shaft, constructed in the usual way without isolation tubes, casing or drilling fluid, all 0.305 m in diameter and 4.2 m deep, were installed at the test site. The depth of 4.2 m was selected so that all test shafts would be adequately anchored in moisture-stable soil below 2m during the experiment. Figures 2 and 3 show the test site layout, indicating locations of instruments and drilled shafts, and a section view showing dimensions of the drilled shafts and the locations of the ground instruments. Throughout the experiment, which lasted for 17 months after the test shafts were installed, no load was applied to the shafts except through swelling and shrinking of the subsurface soils brought about by entirely natural rainfall and thermal changes in the surface soils.

Two types of instruments were installed to measure suction. One was a device distributed by Soiltest, Inc., that consisted of fiberglass fibers whose collective resistivity changed as their moisture content changed. The second was a thermocouple psychrometer with a 400 mesh stainless steel protective tip manufactured by the J. R. D. Merrill Company. All of these devices were calibrated in the laboratory prior to installing them in the ground (Kim, 1996).

The test site was instrumented with six columns of suction sensors. The fiberglass sensors were placed in and near the zone of saturation because the psychrometers do not function well at the saturation moisture content. Three instrument columns were installed between the reference drilled shaft, R-1, and isolation tube shaft, I-1, and the remaining three columns were placed between R-1 and isolation tube shaft I-2. Each column had three suction measuring points, 0.45 m, 1.1 m, and 2.0 m below grade, the upper two levels of which were psychrometers.

4.2 Isolation Tubes

Isolation tubes were installed to a depth of 2.1 m to bypass the potentially expansive surface soil unit. An isolation tube consisted of a pair of concentric pressed fiber tubes in which the annular space between the tubes (6.3 mm thick) was filled with asphalt to minimize the transmission of shear stress from the soil to the drilled shaft. The outside diameters of the isolation tubes were 0.337 m (the nominal as-drilled diameters of the boreholes to a depth of 2.1 m). Asphalts with three different viscosities were provided in the isolation tubes. Isolation Tube 1 (I-1) contained an AR-8000 grade asphalt, which was the most viscous of the three. Isolation tube 2 (I-2) contained AR-4000 grade asphalt, which was of intermediate viscosity, while Isolation Tube 3 (I-3) contained AC-5 grade asphalt, which was least viscous. The isolation tube for I-3 was difficult to handle in hot weather because the asphalt tended to run. The other two tubes posed no difficulty in handling as long as they were kept out of the direct sunlight.

Figure 4 shows a schematic elevation of an isolation-tube drilled shaft. No continuous longitudinal bars were used in any of the shafts. Tension links (electronic load cells), connected to the longitudinal steel, were installed through the "no-tension material" zones to measure uplift forces in R-1, I-1 and I-2 (Kim and O'Neill, 1998). I-3 was uninstrumented.

In order to ensure that the isolation tubes remained in intimate contact with the expansive soil during the course of the experiment, any small spaces remaining between the exterior of the tube and the soil after insertion of the tube into the borehole for the drilled shaft was filled with fluid cement- water-bentonite grout.

4.3 Elevation Measurements

A benchmark was established in the middle of research site. The benchmark was set at a depth of 7.9 m, which is well below the potential expansive zone and which can be considered stable. It served as the reference for the ground surface and top-of-shaft elevation measurements that were acquired through the 17-month study. The benchmark consisted of a 19-mm-diameter galvanized steel pipe supported on a 70-mm diameter circular plate, the bottom of which, in turn, was firmly seated on the bottom of a 102-mm diameter augured borehole. The pipe was placed inside a 102-mm O.D. PVC pipe that had been driven approximately 25 mm into the soil at the bottom of the borehole to seat it from outside moisture intrusion. The benchmark pipe was supported laterally within the PVC pipe by loose-fitting rings that maintained the pipe in an essentially vertical alignment. The PVC pipe was sealed internally with a heavy bentonite slurry to ensure that no moisture from the surface could reach the soil at the bottom of the bentonite slurry.

The test site had a total of six ground surface elevations and four drilled shaft elevation points. The elevation points for the ground surface elevation were constructed with 1-m high × 12.7-mm diameter galvanized steel pipes cast in concrete blocks 300 mm × 300 mm × 150 mm high to hold the pipe in position. The elevation points for the drilled shafts, installed on the tops of the shafts during casting, were 1-m high × 12.7-mm diameter galvanized steel pipes. Elevation measurements were made at all the elevation points with respect to the benchmark by using a manometer formed from 12.7-mm I.D. clear plastic tubing. Elevations were read to the nearest 0.5 mm. The difference in thermal elongation and contraction of the benchmark and elevation points was also considered because the exposed length of each pipe in each was different. The

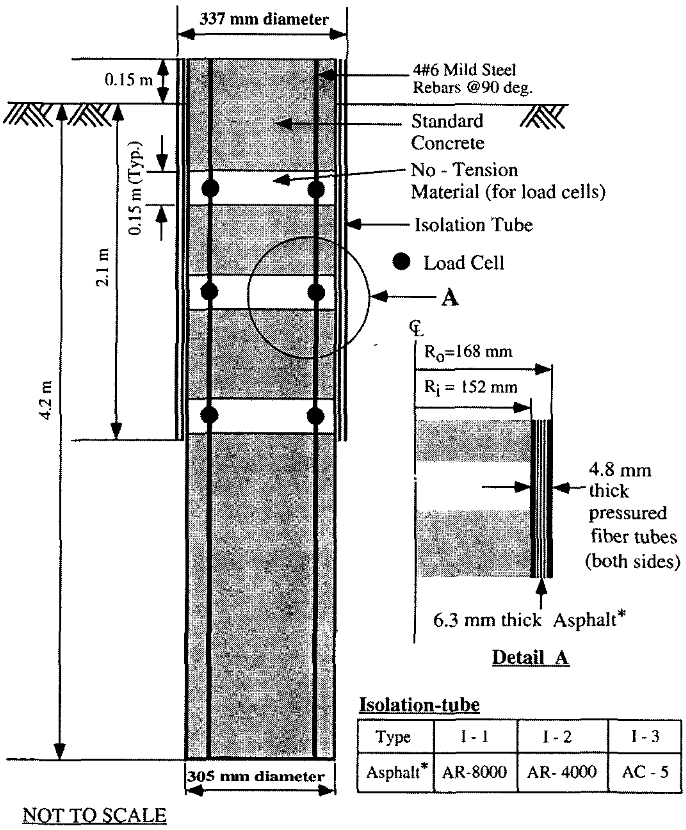

Fig. 4. Elevation view of instrumented test shaft

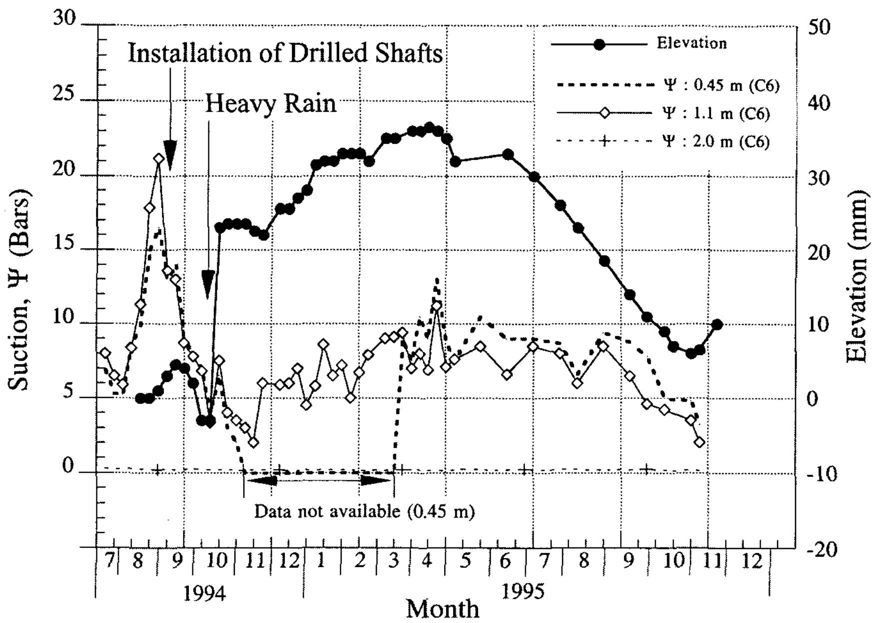

Fig. 5. Suction, Ψ, and surface elevation vs. time at location C6 (1 Bar = 100 kPa)

difference in elongation was well within the precision of the manometer, which was 0.5 mm.

4.4 Suction and Surface Elevation Changes

The total suction history of the soil at the site is summarized in Fig. 5 (for Location C6, Fig. 2), which also shows the corresponding surface elevation changes at Location C6. Note that both the sensors and test shafts were installed at the end of a hot summer period when soil suction was high. Approximately six weeks after construction a very heavy and prolonged rain (approximately 500 mm in 4 days) occurred, causing an immediate heave at the soil surface, as can be seen in Fig. 5.

The pattern of indicated soil suction and surface elevation response at C6 is typical of the response elsewhere on the site. At first, indicated soil suction increased, then decreased, in the six weeks between the time the ground instruments were installed and the installation of the drilled shafts. Indicated ground elevation also increased slightly during this time. The rapid indicated increase in suction during August, 1994, before the drilled shafts were installed, may have been the result of air paths between the sensors and the atmosphere that had not yet been sealed by the compacted clay backfill that was placed around the lead wires. When the drilled shafts were installed on September 2, 1994, the indicated suction dropped sharply, possibly because of the availability of excess water to the soil from the concrete in the drilled shafts not required for hydration of the cement. At the same time the ground surface elevation decreased slightly. This indicated surface elevation decrease may not have accurately reflected real ground movements because ongoing construction activity on the site (demobilization of construction equipment) may have influenced the ground movement readings. Six weeks after the drilled shafts were installed, the heavy rain occurred, which produced a near-immediate ground surface heave of about 25 mm. Soon, but not immediately, after the prolonged heavy rain, the suction at 0.45 m and 1.1 m dropped sharply. These indicated suction decreases lagged the ground heave by several days, presumably because by the time of the heavy rains the psychrometers were well insulated from the ground surface and moisture infiltration because the clay soil had by then sealed well around the lead wires. Furthermore, the sensors had been installed by removing cylindrical plugs of soil with a thin-walled sampling tube and embedding the sensors within the centers of "clods" in the plugs bounded by slickensides and fissures, then pushing the instrumented soil plug back into the borehole from which it was recovered. Since the suction sensors were relatively far away from open moisture paths, they did mot "feel" the effects of the deluge of surface water as rapidly as the soil adjacent to slickensides and fissures, especially the soil very near the surface, which swelled almost immediately. Within about one month after the heavy rains, however, the sensors at the depth of 0.45 m apparently became saturated and failed to give meaningful values of suction for a period of about four months.

Beginning in late November of 1994 (early winter period), the ground surface slowly began to heave even more, while the suction at a depth of 1.1 m slowly increased. Then, beginning in April, 1995 (spring), the ground surface slowly began to recede, while the suction at both the 0.45 m and 1.1 m depths remained approximately constant. This behavior continued throughout the summer of 1995. No unusual meteorological events occurred on the site after October, 1994.

One would expect ground surface heave to be accompanied temporally by decreasing suction. It appears that after November, 1994, the ground surface movements lagged the suction changes at depths of 0.45 and 1.1 m by several months. For example, the suction increases that occurred in November, 1994, through January, 1995(cool, dry period), were not manifested in ground surface recession until April, 1995, and suction decreases that began to occur in late August of 1995 were not manifested in surface heave until early November of that year. The reasons for this lag are not clear, but they appear to be related with apparent differences that existed in moisture paths to the sensors compared to the moisture paths to the soil that was undergoing significant volume change near the ground surface. Nonetheless, the gross changes in soil suction over the long course of the experiment were good predictors of ground heave via Equation 1 with the empirical correction for the effects of internal swelling into joints and slickensides described earlier.

Elevation changes measured around the test site at the six surface elevation points are shown in Fig. 6 for the period from July, 1994, through November, 1995. Although the site is visually uniform, considerable variability in surface movement occurred around the site during this period. The surface elevation at the time of ground instrument installation was considered to be the reference elevation. The absolute elevation of the site is about +12.0 m MSL.

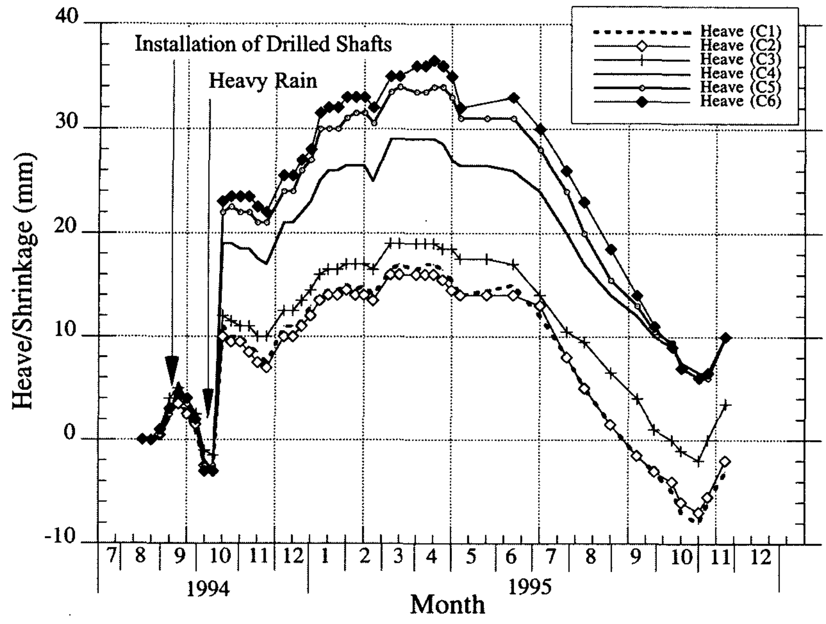

Fig. 6. Ground surface elevation change

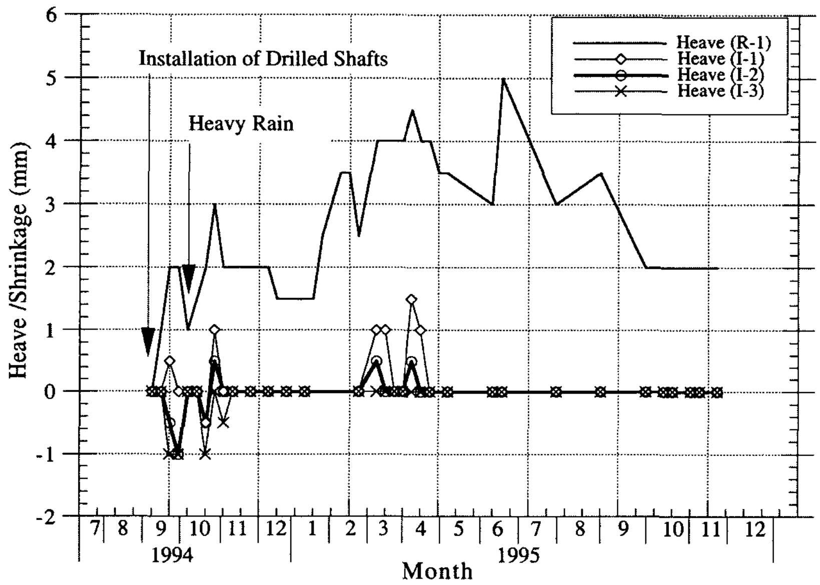

Fig. 7. Elevation changes at the heads of reference and isolation-tube drilled shafts

4.5 Shaft-Head Elevation Changes

The test shafts were installed on September 2, 1994. The elevation changes at the heads of the reference shaft and those of the isolation-tube shafts are shown in Fig. 7. The maximum upward vertical movement change of the reference drilled shaft was recorded at 5.0 mm, due mostly to tensile strains in the shaft, while isolation tube shafts moved no more than 1.5 mm (I-1). The shaft with the most viscous asphalt (I-1) exhibited up to 1.5 mm movement, while those with lower viscosity asphalt exhibited only 0.5 mm or less of upward movement.

Upward movement of the outer isolation tube components relative to the inner tube components was clearly visible

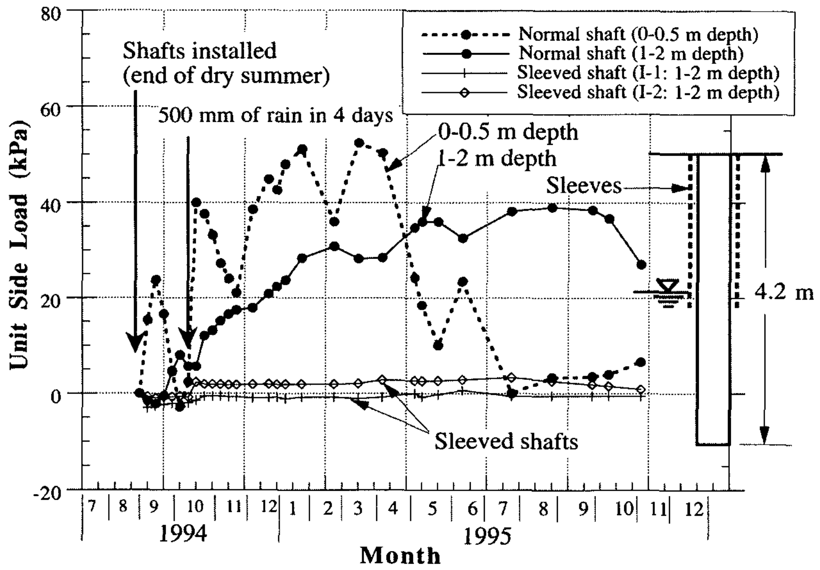

Fig. 8. Unit Side shear load acting upon reference and isolation-tube shafts vs. time

following heavy rains of October 15 - 18, 1994, immediately after which a surface heavy of up to 23 mm was recorded on the site (Fig. 6). When the ground surface elevation receded during the following spring and summer, the outer tube components remained extended by about 20 mm. That is, the shrinking soils, which shrank both vertically and laterally, apparently lost contact with the outer tubes, leaving them in an extended position.

4.6 Induced Uplift Stresses

Concurrent with the upward movement of the near-surface soil, shear stresses were induced along the sides of the sections of the test shafts within the zone of suction change, since the test shafts were anchored below the unstable zone (Kim and O'Neill, 1998). These induced shear stresses were manifested as tensile forces in the test shafts. The time histories of the side shearing stresses (unit side loads) derived from the measured tensile forces in R-1, I-1 and I-2, are shown in Fig. 8. For reference, the undrained shear strength of the soil in the upper 2 m is about 120 kPa in the dry summer season. It is evident that the largest stresses (up to approximately 47 percent of the undrained shear strength) consistently occurred in the reference shaft (R-1), and the shear stress changes occurred more rapidly in the upper 0.5 m than in the lower part of the unstable zone. The shaft with the isolation tube with the asphalt of highest viscosity (I-1) exhibited essentially zero side shear load. Very small side shear loads were observed in I-2 over the 17 months of the experiment.

5. Conclusions

Simple isolation tubes made from concentric pressed fiber tubes with annular spaces filled with asphalt and inserted into drilled shaft boreholes during construction to a depth slightly in excess of the depth of seasonal moisture change were very effective in mitigating the transfer of shear stresses from an expansive clay soil to the perimeters of the drilled shafts. Maximum unit uplift shear loads in shafts constructed with isolation tubes ranged from nearly zero to approximately 9% of the maximum unit uplift shear load in a conventional shaft after a period of heavy rainfall. The cost of making and installing the tubes would likely be offset by the saving in the cost of steel and the depth of anchorage required for conventional drilled shafts.

The soil at the test site exhibited rapid changes in vertical movement to a depth of over 1.0 m with the application of saturating moisture (heavy rain) following a dry period. Maximum ground movements exceeding 40 mm were observed during the experiment. Movements of 1.5 mm or less were observed in the test shafts with isolation tubes, while upward movements of 5 mm were observed in the reference shaft constructed in the usual manner.

Acknowledgements

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