C.R. Coll. Int. Renforcement des Sols. Paris 1979.

GOUGHNOUR R.R. et BAYUK A.A.

Vibroflotation Foundation Company, U.S.A.

A field study of long term settlements of loads supported by stone columns in soft ground

Etude in-situ des tassements à long terme de pieux ballastes dans des sols mous

On a conduit un essai de charge verticale à long terme sur un ensemble de poteaux de pierre pour vérifier les bases de calcul d'un project utilisant des poteaux de pierre pour élargir un remblai existant, ainsi que plusieurs rampes sur la route I-64 a Hampton en Virginie. L'essai a eu lier dans un terrain marécageaux peu profond. Le profil du sol obtenu par sondages aux abords immédiats, indiquait une couche supérieure de limon argileux très tendre à tendre de 12 à 15 pieds (environ 3,7 a 4,6 mètres) d'épaisseur reposant sur du sable limoneux meuble à semi compact.

L'essai était concu pour reproduire les conditions existantes lorsque un remblai de 20 pieds (6 mètres) est construit sur un ensemble de poteaux de pierre. On avait installé des appareils de mesure et de réglage à plusieurs endroits pour mesurer la pression interstitielle, le tassement et las pression totale. On s'est particulièrement intéressé à l'étude du rapport temps/tassement et à sa corrélation avec l'analyse théorique intitulée "Analyse de l'interaction entre poteau de pierre et sol portant sous charge verticale" qui est présentée au cours d'une autre séance de ce congrès.

Introduction

The proposed expansion of the interchange ramps connecting Interstate Route 664 with Interstate Route 64 at Hampton, Virginia involved numerous high embankments and bridge structures over marshlands. A problem encountered by the Virginia and Federal Highway Engineers was: How to construct embankments, up to 35 ft (10.7 m) high on a thick deposit of highly compressible, weak, sensitive silts and clays in a relatively short period of time, at a reasonable cost, without causing major embankment failures and without producing post construction settlements of intolerable magnitudes? Very strict environmental constraints complicated the problem even further. Several alternatives were considered including excavation and replacement of the soft soils and the use of piled bridge structures in lieu of embankments. The most attractive alternate appeared to be stabilization of the in situ soils by the installation of stone columns.

The Vibroflotation Foundation Company installed a group of stone columns through 12 to 15 ft (3.7 to 4.6 m) of the very soft to soft silts and clays, terminating them at an average depth of 21 ft (6.4 m) within loose to medium compact sands. A long term, vertical load test was performed to simulate the embankment conditions and to verify the design principles used to establish the stone column spacing and patterns for the proposed project.

Presented in this paper is a description of the field installation, instrumentation and performance of a stone column group under a vertical area loading. The observed field test results are compared to results predicted by calculations using a theoretical analysis (Goughnour and Bayuk, 1979). Correlations to various design parameters are discussed in order to develop design procedures for future stone column installations.

General Site Conditions and Soil Properties

The field survey of the proposed expansion of the interchange ramps connecting I-664 with I-64 at Hampton, Virginia indicated that the major portion of the construction would be located within a shallow swamp with brush reaching eight feet (2.4 m) in height. The ground surface elevation in the swamp was approximately El. +2 ft (+0.6 m) mean sea level. The groundwater table was influenced by the tide and appeared to vary between El. +1 ft (+0.3 m) and El. -1 ft (-0.3 m).

The general soils profile revealed 12 to 15 ft (3.7 to 4.6 m) of very soft to soft silts and clays overlying loose to medium compact silty sands. Logs showing soil classification and static cone resistances of borings taken in the immediate vicinity of the load test area are presented in Fig. 1 and indicate the high compressibility of the upper silts and clays.

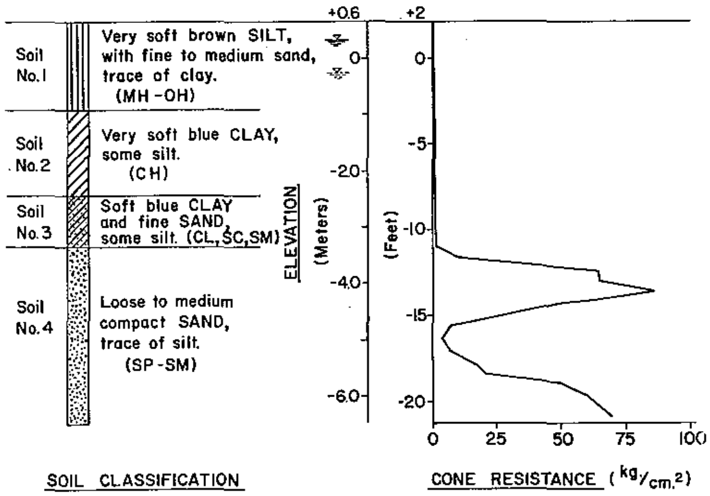

Fig. 1 - Soils profile at test site.

Consolidation tests, performed on undisturbed samples of these very soft soils, showed compression indices, C_c, ranging up to 1.04 with a void ratio, e_o, of 2.693. The coefficients of consolidation for vertical drainage, c_v, were determined to be as low as 0.025 ft^2/day ($2.69 \times 10^{-8} m^2/s$) for loadings comparable to the maximum proposed embankment heights.

In situ vane shear tests were performed to estimate the undrained shear strength, s_u, of the very soft soil deposits. The strength profile, as determined by the field vanes, is shown in Fig. 2. The shear strength profile shows a lower limit for s_u of 140 psf (6.71 kN/m^2) near the surface with an increase in strength with depth. In situ vane shear tests located nearest to the load test area are plotted on the strength profile. It should be noted that these values are below the 400 psf (20 kN/m^2) criteria at which Thorburn (1976) suggested caution be used in the use of stone columns.

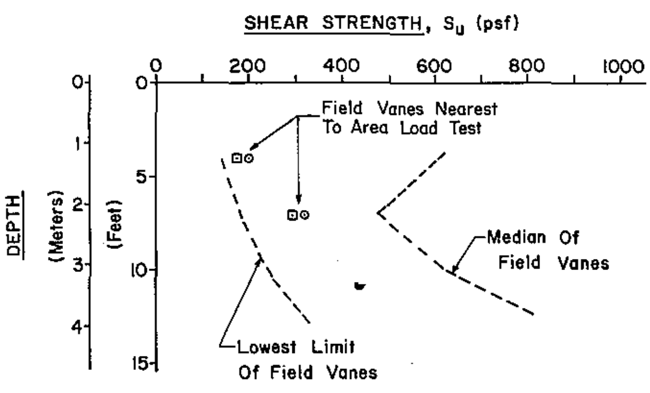

Fig. 2 - Field vane test results.

Effective stress strength paramaters were established from consolidated undrained triaxial tests with pore pressure measurements (CU tests). The results of these tests indicated that the very soft clays have effective stress strength parameters of c = 50 psf (2.40 kN/m^2) and $\phi = 26^0$.

Atterberg limits of the upper fine-grained soils and subsequent liquidity index determinations suggested a sensitivity classification in the very sensitive range (4 to 8). Experience with stone columns in sensitive soils, up to now, has been limited to clays having sensitivities not exceeding five (Baumann and Bauer, 1974). Peak and remolded undrained strengths were obtained by field vane tests and sensitivities determined. The sensitivity profile for the upper soft soils is shown in Fig. 3. These tests showed a median sensitivity of approximately two, ranging in sensitivity from slight to medium.

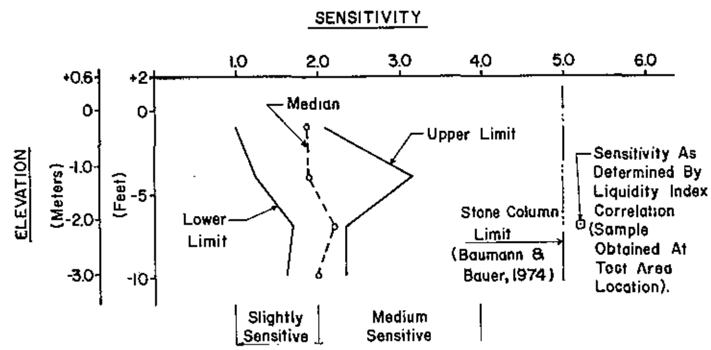

Fig. 3 - Field vane sensitivity results.

Lateral stresses during confined compression are of particular importance in a stone column analysis (Goughnour and Bayuk, 1979). The ratio of horizontal to vertical stress at rest, K_o, is a function of the overconsolidation ratio and the plasticity index (Lambe and Whitman, 1969, p. 300). Consolidation testing of the very soft upper soils indicated that these soils are normally consolidated and should have a K_o value of approximately 0.6.

Installation of Stone Columns

The load test area was located north of the westbound lane and near station 630+00 of I-64 in Hampton, Virginia. The brush in the test area was cut to ankle height and a three feet (0.9 m) high working platform was installed. The plan dimensions of the platform were approximately 36 ft (11 m) by 50 ft (15 m). The material used for construction of the platform was a fine to medium sand with some silt and was spread with a dozer equipped with swamp tracks.

A total of forty five stone columns were installed within a period of three and one-half days. The layout and spacing of the stone columns are shown in Fig. 4.

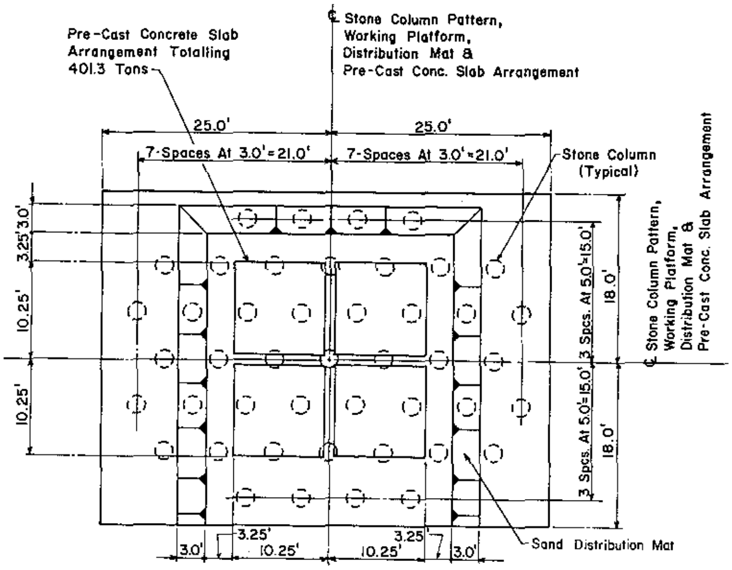

Fig. 4 - Stone column pattern and load test plan.

Equipment - The stone columns were installed using a 16 in. (406 mm) diameter vibrator, approximately 7 ft (2.1 m) long, specially connected to a 12 in (305 mm) diameter follower tube. The complete assembly, called a Vibroflot, was approximately 39 ft (11.9 m) in length, weighed approximately 4.3 tons (3.9 metric tons) and was supported from a 35-ton (31.8 metric tons) crawler crane.

The lower part of the Vibroflot housed a 100 hp electric motor which drove an eccentric weight at a speed of 1800 revolutions per minute at 60 Hz, creating vibrations in a horizontal plane. Twenty tons (18.2 metric tons) of centrifugal force were generated, producing an amplitude of 14 mm at the tip of the Vibroflot when unconstrained. A portable 440 volt, 3 phase, 60 Hz, 125 KW generator provided electric power to the Vibroflot.

The stone was brought up to the holes and dumped utilizing a rubber tired side-dump loader. The loader bucket was carefully measured to determine its level-full capacity by filling the loader bucket level-full with water and carefully bailing out the water with a six gallon (22.7 liters) bucket. The loader bucket, when level-full, contained 40 cu ft (1.1 m^3) of loose stone.

A six inch (150 mm) high capacity, high pressure water pump, drawing water from nearby Newmarket Creek provided a supply of water for stone column installation.

Stone Column Characteristics - The stone columns were formed using a special run, crushed, angular granite with representative gradation as follows:

Sieve Sizes (inches)	(mm)	Percent Passing (By weight)
2-1/2	64	100
1-1/2	38	65-79
3/4	19	6-10
1/2	13	1-5

Previously performed field shear tests on stone columns, formed with material similar to this, indicated an angle of internal friction of 38° (Engelhardt and Golding, 1976).

A total of 692 tons (624 metric tons) of rock were brought to the site, and it is estimated that approximately 615 tons (559 metric tons) were placed in the 45 stone columns. The estimated consumption for each column was determined on the basis of number of loader buckets of stone dumped into each stone column and assuming the loose density of rock as 100 pcf (1.6 tons/m^3).

Knowing the depth penetrated by the Vibroflot and using an estimated compacted density of the rock in each stone column, the average diameter was calculated for each column. Tabulated results are shown in Table 1.

	Averages Per Stone Column
1. Depth:	
Feet	21.1
(Meters)	(6.4)
2. Stone Consumption:	
Tons/Foot	0.65
(Metric Tons/Meter)	(1.94)
3. Installation Time:	
Minutes	32.1
4. Stone Col. Diam.:*	
Feet	3.6
(Meters)	(1.1)

* Based on a compacted stone column density of 125 pcf (2.0 tons/m^3).

Table 1 - Tabulated Results of Stone Column Installation.

Load Test Arrangement and Instrumentation

After installation of the stone columns, a dozer stripped the working platform down to approximately El. +4.5 ft (+1.37 m) mean sea level.

Instrumentation, which included pneumatic piezometers and load cells, was installed as shown in Fig. 5. The piezometers were Sinco Model No. 51481 (with well screen) and were placed approximately three feet (0.9 meters) radially from the center of the stone column, which was located at the center of the load test area. The load cells were Sinco Model No. 51482 and were placed at approximately El. +4.0 (+1.22 m).

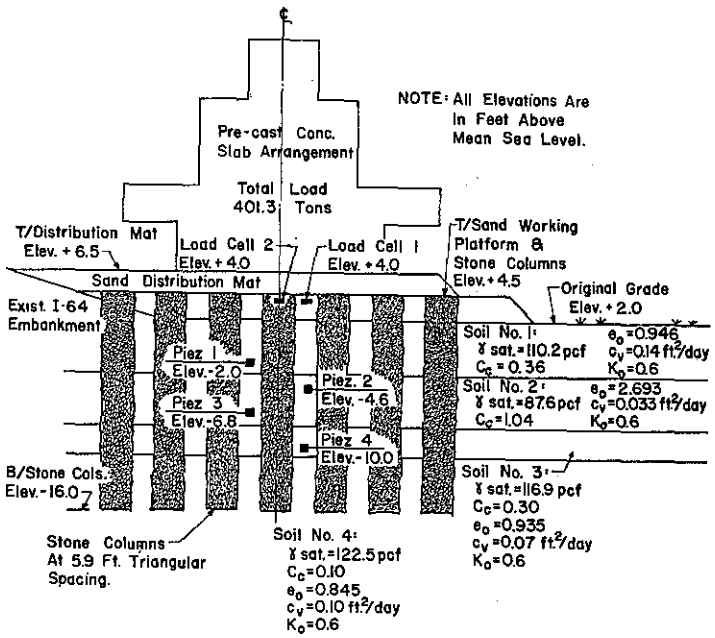

Fig. 5 - Load test arrangement and instrumentation.

One of the load cells, Cell 2, was located at the center of the central stone column and the other, Cell 1, was located in the working platform directly between the central stone column and the most adjacent stone columns.

A pneumatic settlement device, Petur Model No. SP-105, was installed at the center stone column and at El. 14.0 ft (1.22 m). The settlement device performed very well for the first 200 hours of the load test, but failed to function after this initial period, perhaps due to freezing. Subsequent settlement measurements were made with a surveyors level located approximately 30 ft (9.1 m) from the center of the load test area. Readings were taken at the center of the load test area and at each of its four corners.

After installation of the instrumentation, a sand distribution mat was placed and compacted in approximately one foot (0.3 meter) layers up to El. +6.5 ft (+2.0 m). The approximate dimensions of the sand distribution mat are shown in Fig. 4. After completion of the distribution mat, the pneumatic settlement device indicated 0.4 in. (10 mm) of settlement.

Four precast concrete slabs, measuring 10 ft x 10 ft x 1.33 ft (3.0 m x 3.0 m x 0.4 m) each, were placed with a six inch (150 mm) gap separating one slab from the other as shown in Fig. 4. The slabs were placed in this manner to make the subsequent loading as flexible as possible. Three additional layers of similar concrete slabs were positioned over these base slabs. However, lack of additional pre-cast concrete slabs of the same size prevented a continuation of this arrangement. The available slab sizes dictated that the upper layers of weights span across these bottom layers so that, in reality, the load was only semi-flexible.

The pre-cast concrete slab arrangement was carefully positioned to produce as uniform a load as possible and, when completed, totalled 401.3 tons (364.8 metric tons). The total load was placed within 54 hours after completion of the sand distribution mat. At this time the measured total settlement was 3.1 in. (79 mm) at the center of the load test area. The intensity of loading on the stone column group, calculated at the original ground elevation of +2.0 ft (+0.6 m), was approximately 2400 psf (115.0 kN/m^2).

The load test arrangement, including instrumentation, is shown in Fig. 5. Also presented are the consolidation parameters, prior to the stone column installation, of the various soils at the load test site. These parameters were used in developing the subsequent sections of this paper.

Theoretical Analysis

Equations have been derived theoretically (Goughnour and Bayuk, 1979) to simulate the consolidation settlement behavior of vertically loaded stone columns in soft ground. The same symbol definitions are used in this paper and, due to space limitations, will not be repeated. These equations were derived under the assumption that the behavior of a single column and its tributary soil can be represented by a unit cell similar to that in common use for sand drain computations with the equal strain assumption.

In the initial analysis, the total load, L_t, applied to this unit cell was considered to be constant with depth. If the soil layer is relatively thick with respect to the loaded surface dimensions, as is the case of this test, a decrease of L_t with depth must be considered. Since, for this test, stone columns were installed on a uniform spacing to some distance outside the limits of the loaded area, it is assumed that L_t can be modified with depth according to Boussinesq's or Westergaard's equations for vertical stress distribution beneath a loaded area.

Conventional sand drain theory has been suggested (Goughnour and Bayuk, 1979) for computations of time-settlement behavior. A summary of such equations is given by Scott, 1963, pp. 202-203.

Test Results and Comparison with Theory

In this section observed test results are reported and compared with theoretical predictions. Implications of this comparison with respect to the suitability of the theoretical model are discussed under various subheadings.

For all computations the load was considered to be flexible, and the total load on the unit cell, L_t, was modified with depth according to Westergaard's equations unless otherwise indicated.

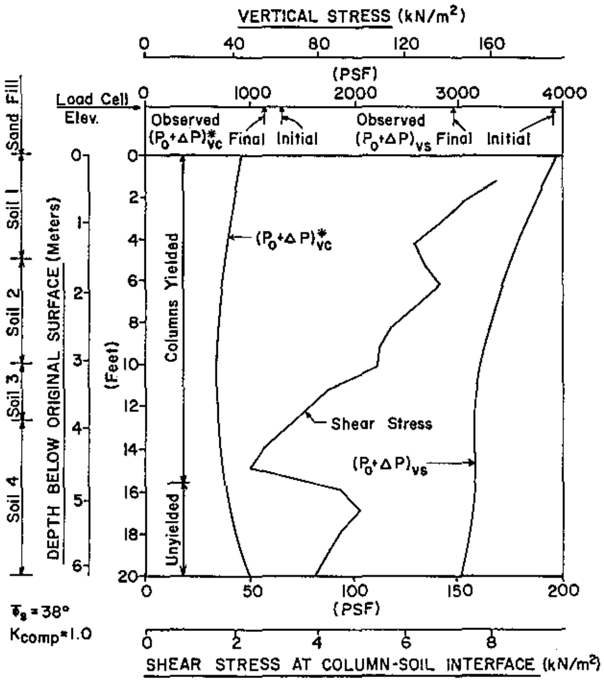

Fig. 6 - Typical computation results at center of loaded area (effective stress basis).

Fig. 6 presents typical results of calculations for conditions at the center of the loaded area. These particular results were obtained using $\bar{\phi}_s$ equal to 38° and K_{comp} equal to 1.0. A vertical increment of 1 ft (0.3 m) was used. Zero depth corresponds to the original ground surface, or El. +2 ft (+0.6 m), and the soil profile is indicated on the left side of the plot.

Under these assumed conditions, calculations indicate that the stone columns are in a yielded condition to a depth of about 15.5 ft (4.7 m). The ultimate calculated settlement is 11.5 in. (29.3 cm). Of this total settlement, approximately 61% is contributed by Soil 1, 31% by Soil 2, 6% by Soil 3 and 2% by Soil 4.

Load cell readings, also shown in Fig. 6, indicate stress adjustments beneath the loaded area occurring with time. The fact that the load was not completely flexible, as previously described, is believed to account for these time dependent stress adjustments.

Shear stress and effect on principal stress directions - The indicated shear stress at the column-soil interface (Fig. 6) was computed by considering the variation in vertical stress increase in the stone from one increment to the next, multiplied by the column cross sectional area, divided by the outside surface area of the column between the centers of these two elements. This magnitude of shear stress is considered to apply at mid-distance between the centers of the two elements under consideration. The largest of these resulting shear stresses for all calculations was less than 200 psf (9.6 kN/m²).

In the original derivation it was assumed that the principal stress directions are vertical and horizontal. Therefore, the maximum theoretical shear stress within the stone is equal to one-half of the difference between the vertical and horizontal stresses on the stone column. As indicated by Fig. 6 this is about 1000 to 1500 psf (47.9 to 71.9 kN/m²). Therefore, Mohr's circle, representing stress conditions within the stone, is only slightly altered when considering the low shear stress at the column-soil interface.

Equal strain assumption - Although a peak shear stress near 200 psf (9.6 kN/m²) was found at the column-soil interface, the overall average for the entire stone column depth was about 100 to 125 psf (4.8 to 6.0 kN/m²). These values of peak and overall average computed shear stresses, when compared with measured values of soil strength (see Fig. 2) indicate the possibility of only very limited local yielding at peak shear stress locations and therefore, justifies the equal strain assumption for this case. Vautrain (1977) states that the settlements of the columns and the ground are of the same amplitude. This further supports the equal strain assumption for stone column analysis.

Stress distribution between stone and in situ soil - Fig. 7 presents the ratio, at consolidated equilibrium, of the vertical stress in the stone to the average vertical stress in the clay, as a function of depth. Various assumed values of K_{comp} and $\bar{\phi}_s$ are considered, and dashed lines indicate the depths at which the stone columns are unyielded. The computed results, compared with observed values of 2.6 to 3.0 from pressure cell readings, appear to indicate that a value of less than 38° (Engelhardt and Golding, 1976) would be appropriate for $\bar{\phi}_s$.

However, it must be pointed out that these load cells were located well up into the working platform where bulging of the stone columns is so limited by the relatively incompressible sand as to remain in an elastic condition. Since the equations used here were derived under the assumption that the in situ soil behaves as ideal clay, the equations are not appropriate for conditions which existed at the elevation of the load cells. Therefore, the indicated value for $\bar{\phi}_s$ is not applicable. It is of interest to note that Vautrain (1977) reports that the vertical stresses measured at the columns are 2 to 3 times as high as those measured in the natural ground. Also, Vautrain mentioned the presence of surface fills incapable of absorbing a large quantity of stone, and it is likely that the presence of these relatively stronger surface fills was reflected in his readings. In future tests, the writers recommend that load cells be located in the soft yielding soils which are being treated.

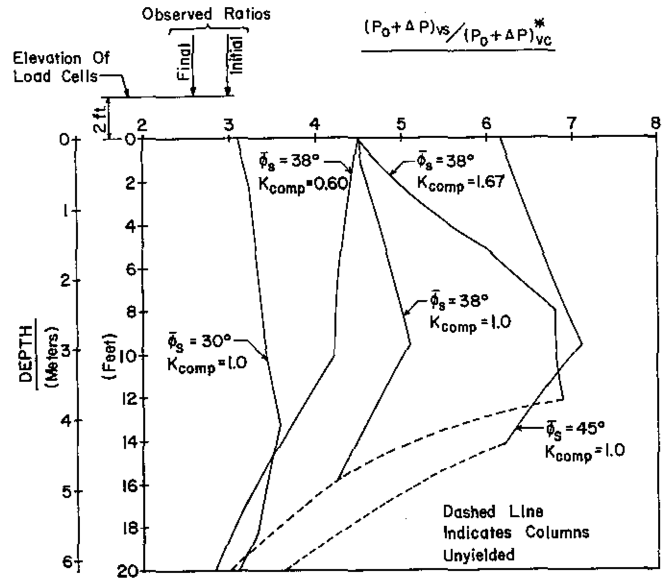

Fig. 7 - Computed stress distribution between stone and in situ soil.

Settlement - The effect of various parameters on computed ultimate settlement are presented in Fig. 8. For comparison purposes, calculations using Boussinesq's stress distribution were also included. The effects of varying $\bar{\phi}_s$ and K_{comp} are as expected. Use of reasonable values for these parameters produce the correct order of magnitude of settlement at the center of the stone column group.

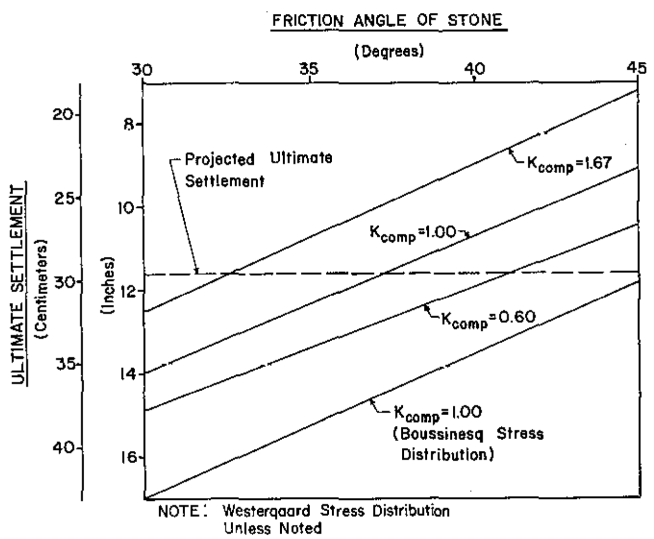

Fig. 8 - Effect of various factors on computed ultimate settlement at center of loaded area.

The results of time-settlement calculations compared with observed time-settlement data for the group load test are presented in Figs. 9 and 10. These calculations were based on a $\bar{\phi}_s$ equal to 38°, and K_{comp} equal to 0.6. In the absence of appropriate test data, the horizontal permeabilities of the various in situ soil strata were taken as three times that of their respective vertical permeabilities. Fig. 9 applies to the center of the loaded area and presents also the effects of various assumed smear factors, $k_r(s-1)/k_s$ (for definition, refer to Scott, 1963). Fig. 10 applies to the corners of the loaded area and a smear factor of 2.5 was assumed. For comparison purposes, time-settlement curves, predicted without stone columns, are included in both figures.

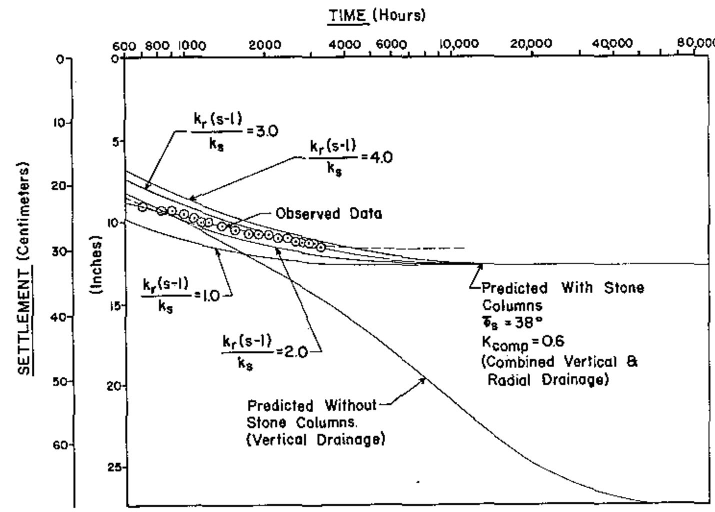

Fig. 9 - Settlement at center of loaded area vs. log of time.

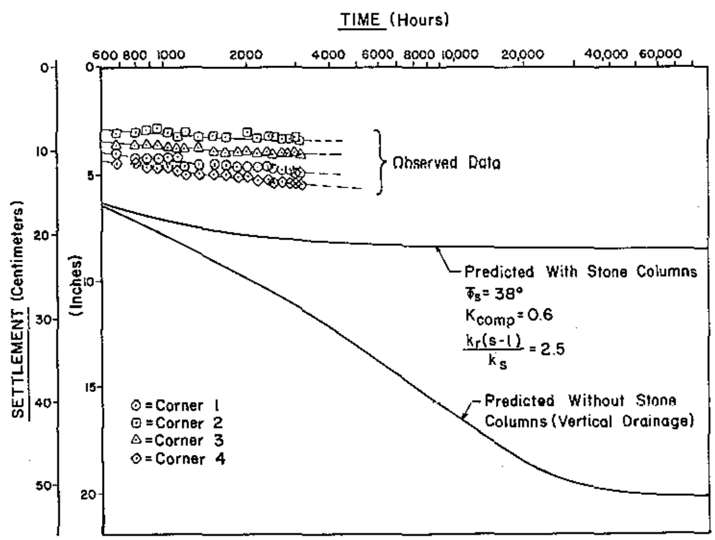

Fig. 10 - Settlement at corners of loaded area vs. log of time.

Excess pore pressures - Observed excess pore pressures at various depths in the in situ soil are presented in Fig. 11, along with those predicted by calculations. The solid lines in this figure represent the best fit to the observed values and the dashed lines represent the results of the calculations. The smear factor was taken as 2.5, and the A and B pore pressure coefficients were each taken equal to 1. All other parameter values were assumed to be the same as those presentsented in Figs. 9 and 10.

Referring to Fig. 9, it appears that a smear factor of about 2.5 provides a reasonable fit to the observed settlement data. However, in applying this factor there are obviously compensating errors which occur within the soil profile. As indicated by Fig. 11, excess pore pressures in Soil 1 dissipated much slower than predicted and those in Soil 2 dissipated much faster than predicted. The computed rate of pore pressure dissipation is influenced principally by the value of the horizontal permeability of the soil and the values assumed here are probably in error. Although it appears that this method of analyzing time-dependent behavior may be valid, further testing should be performed to establish the proper parameters for design.

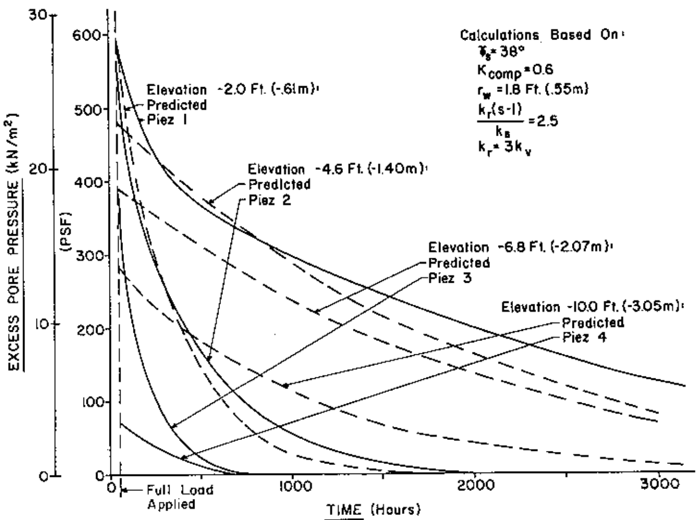

Fig. 11 - Excess pore pressure within in situ soil at various depths.

The measured excess pore pressures which occur at the instant when the load is completely applied give an indication of the stress increase at various depths within the in situ soil. As indicated in Fig. 11, these values agree very well with computed values down to a depth of about 10 ft (3 m) or to a depth equal to about one-half the width of the loaded area. Below this depth measured values are lower than those predicted.

Conclusions

Conclusions presented herein are based upon the findings of this investigation and are limited to the soils tested, the methods of construction used, and test procedures employed.

Observed results of a long term field test on vertically loaded stone columns in soft soil have been presented and compared with those predicted by a theoretical analysis (Goughnour and Bayuk, 1979). Excellent correlation between measured data and predicted stress distributions and ultimate settlements lend support to the analysis.

Reasonable predictions of ultimate settlement and stress distribution between the stone and in situ soil at various depths can be made using soil and stone parameters obtained from standard laboratory tests, K_{comp} values of 0.6 to 1.0, and Westergaard's stress distribution. Although the method appears to be also valid for prediction of time-settlement behavior when using a smear factor of 2.5, further testing should be performed to resolve discrepancies in predicted excess pore pressure dissipation rates within the in situ soil.

Bibliography

BAUMANN V. and BAUER G.E., The performance of foundations on various soils stabilized by the vibro-compaction method, *Canadian Geotechnical Journal*, Vol. III, 1974, pp. 509-530.

ENGELHARDT K. and GOLDING H.C., Field testing to evaluate stone column performance in a seismic area, *Geotechnique*, Vol. XXV, No.1, March, 1975, pp. 61-69.

GOUGHNOUR R.R. and BAYUK A.A., Analysis of stone column - soil matrix interaction under vertical load, *Intern. Conf. on Soil Reinforcement*, March, 1979, Paris.

LAMBE T.W. and WHITMAN R.V., *Soil Mechanics*, John Wiley & Sons, Inc., New York, N.Y., 1969, p. 300.

SCOTT, R.F., *Principles of Soil Mechanics* Addison-Wesley Publishing Co., Inc., Reading, Mass., 1963, pp. 202-203.

THORBURN S., Building structures supported by stabilized ground, *Ground Treatment by Deep Compaction*, The Institution of Civ. Engrs., London, 1976, pp. 83-94.

VAUTRAIN J., Reinforced earth wall on stone columns in soil, *Intern. Symposium on Soft Clay*, Bangkok, July, 1977, pp. 188-194.