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Performance of a Stone Column Supported Embankment

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The proposed expansion of ramps connecting Interstate Route 664 with Interstate Route 64 at Hampton, Virginia involved numerous high embankments and bridge structures over marshlands. Potential problems of embankment stability and excessive long term, post construction settlements were further complicated by very strict environmental constraints on acceptable construction methods. The solution chosen was stabilization of the in situ soils by the installation of stone columns.

A description is given of stone column design, construction, field embankment instrumentation, and embankment performance for the first two years of operation.

Four theories for predicting settlements of stone column reinforced ground are briefly reviewed. Calculated settlements of the embankment are then compared with the measured settlements. Although the settlements predicted by each method differ, they generally give good results.

INTRODUCTION

Construction of the interchange expansion at Hampton, Virginia connecting Interstate I-64 with I-664 involved numerous high embankments and bridge structures constructed over very soft marshland deposits. Approximately 134,000 ft. (40,900 m) of stone columns were used to support portions of the interchange embankments. Important factors in deciding to reinforce the ground with stone columns included (1) strict environmental constraints, (2) the presence of Newmarket Creek immediately adjacent to one interchange ramp, and (3) achieving acceptable post construction settlements without delaying the project. Stone columns were selected over (1) excavation and replacement and (2) surcharging due primarily to environmental and economic considerations.

Before construction of the interchange, a long term, vertical load test program was conducted to verify the design principles. This test program, which has been described in detail by Goughnour and Bayuk (1979a), gave valuable information concerning ultimate column load, group settlements, pore pressure development and stress concentrations in the stone columns.

INTERCHANGE CONSTRUCTION

A plan view of the I-64/I-664 interchange is shown in Figure 1. The major portion of the interchange is located in a shallow tidal marsh area having a ground surface elevation of approximately +2 ft. (0.6 m) above mean sea level. Brush up to 8 ft. (2.4 m) in height is present.

Stone columns were placed under portions of the east and west bound lanes of I-64, and portions of Ramps A, B, C and D (Figure 1).

The embankments placed above the stone column improved ground varied in height from 7 to 28 ft. (2.1 to 8.5 m). All embankments were constructed on a 2 (horizontal) to 1 (vertical) side slope.

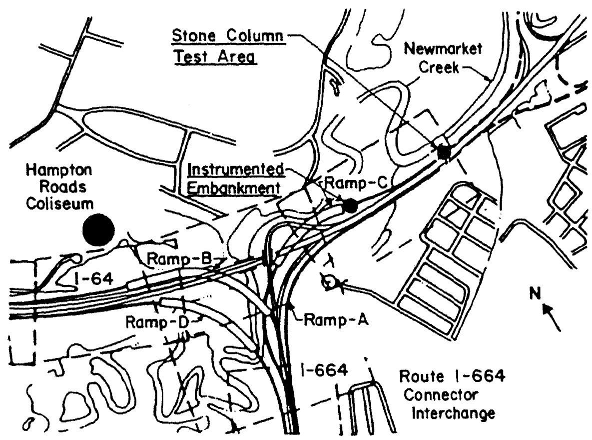

Figure 1. General Location Plan.

SUBSURFACE CONDITIONS

A firm to very stiff "marsh mat" 2 to 4 ft. (0.6 to 1.2 m) thick occurs at the surface in the vicinity of the interchange. Immediately beneath the marsh mat, 10 to 16 ft. (3 to 5 m) of erratic marine deposits were encountered including very soft brown silts with sand and very soft to firm, dark gray and blue clays with very thin seams of fine sand and silt. Organics were often present.

This stratum was underlain at a depth of 10 to 16 ft. (3 to 5 m) by loose to very firm clayey and silty sands, fine to medium sands, and fine sandy clays. The median value of the undrained shear strength in the upper 10 to 16 ft. (3 to 5 m), as determined by field vane shear tests, was between about 500 and 600 psf (24 to 29 kN/m^2), while the median value for the softer zones was about 380 psf (18 kN/m^2).

The embankment fill for the east approach to the Ramp C bridge was located in the vicinity of some of the poorest soils encountered along the route. Near Ramp C the very soft soils were about 8 to 9 ft. (2.4 to 2.7 m) in thickness. The lowest two undrained shear strengths measured in this area (and on the site) were 140 and 180 psf (6.7 to 8.6 kN/m^2) at depths of 3 and 6 ft. (0.9 and 1.8 m), respectively.

SOIL PROPERTIES

The highly compressibile gray and blue clays (CH) had void ratios varying from about 1.5 to 2.6, low wet unit weights of about 85 pcf (13.3 kN/m^3), and water contents around 110%. The liquid limit of these soils was around 118, and the plastic limit 39, with a corresponding liquidity index of 0.75 to 0.90. The Compression Index, C_c, varied from 0.9 to 1.1 as summarized in Table I. Effective stress strength parameters 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 (2.4 kN/m^2) and ϕ = 26°.

	C_c	e_o	γ_w (pcf)	w %	DEPTH (ft.)	DESCRIPTION
1	1.06	2.6	84.2	110	--	Dark gray organic clay (OH)
2	1.07	2.6	87.6	109	--	Dark gray highly plastic clay (CH)
3	0.86	1.5	84	112	7-9	Gray silty clay
4	0.27	0.8	117	39	7-9	Gray silty clay
5	0.203	0.75	122.5	30	17-19	Gray silty & fine sand with shell fragments
6	0.050	0.93	128	50	22-24	Silty sand with some clay

Table I - Summary of Soil Properties on Total Project.

The less compressible silty sands (SM) organic silts (ML-OL) and low plasticity clay (CL) had liquid limits typically varying fro 17 to 45, and plastic limits varying from 0 t 27; many of the sample tested were nonplastic The void ratio typically varied from about 0. to 1.0, and wet weight from 115 to 128 pcf (1 to 20 kN/m^3), with water contents of 30 t 40%. The Compression Index, C_c, varied fro about 0.05 to 0.3.

Peak and remolded undrained shear strength were obtained by field vane tests. The media value of sensitivity for the site, taken a the ratio of peak to remolded shear strength was about 2. The sensitivity varied fro approximately 1 to 3. According to th classification system of Bjerrum (1954), thes soils fall within the insensitive (1-2) t moderately sensitive (2-4) range. Ston column experience has been limited to site having sensitivities not exceeding about (Baumann and Bauer, 1974).

STONE COLUMN DESIGN AND CONSTRUCTION

Approximately 134,000 linear ft. (40,900 m) c stone columns were placed beneath 6,300 linea ft. (1920 m) of interchange embankment. Th stone columns were constructed using a equilateral triangular pattern with sid dimensions varying from 5 to 8 ft. (1.5 to 2. m).

Column spacing

The column spacing selected depended on th height of embankment and column locatic within the embankment. Zone A was the centra part of the embankment, and Zone C wa that part of the side slope $\leqslant$ 12 ft. (3.7 m in height (see Figure 2). Zone B was inter mediate to Zones A and C. In low embankment Zone B did not exist. This was the case a the instrumented sections. In Zone A th limiting design criterion was settlement while in Zones B and C stability considera- tions determined column spacing. Figure illustrates how design spacings were chose for Zone A.

The design curve in Figure 3 was based on th results of the full scale, long term, vertica load test (Goughnour and Bayuk, 1979a), ar was intended to limit long term settlements t slightly over 1 ft. (0.3 m). Column spacing in zone B varied between 6 and 8 ft. (1.8 ar 2.4 m). Zone C spacing was 8 ft. (2.4 m) i all cases.

Installation

A working platform of fine to medium sand wi some silt approximately 3 ft. (0.9 m) thic was first constructed to permit moving th crane and vibrator over the site.

The stone columns were installed using a 1 in. (406 mm) diameter, 100 hp electri vibrator approximately 7 ft. (2.1 m) long The vibrator operated at 1800 rpm's, creatir 20 tons (18.2 mt) of centrifugal force in th horizontal direction. A more detaile description of the equipment and technique used has been given elsewhere (Goughnour and Bayuk, 1979a).

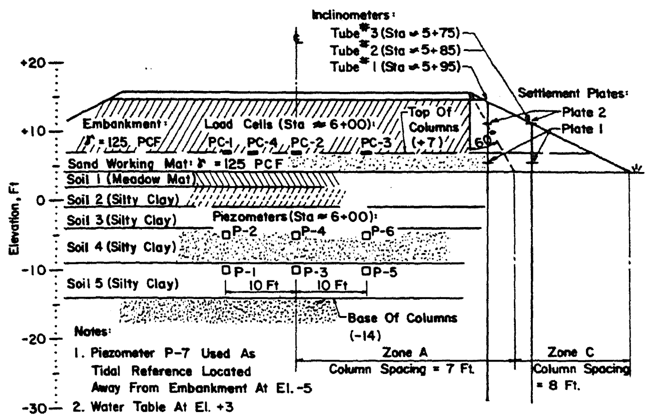

Figure 2. Arrangement of Instrumentation.

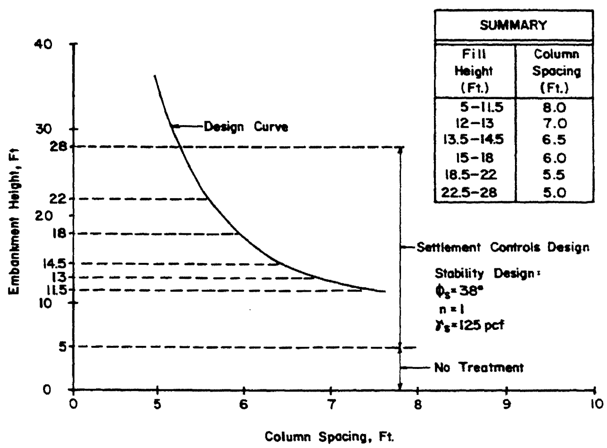

Figure 3. Embankment Height Versus Column Spacing.

The stone columns were constructed using the vibro-replacement (wet) method of construction (Barksdale and Bachus, 1983).

Stone Characteristics

The stone columns were formed using a special run, crushed angular granite having the following typical gradation:

Sieve Size (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

Using AASHTO Test Method T19-74 Hernandez (1983) found the Hampton stone to have a minimum density of 96 pcf (15 kN/m^3) and a maximum density of 108 pcf (17.0 kN/m^3). A stone consumption of 0.65 tons/ft. (1.94 mt/m) was determined during column construction for the preliminary load tests (Goughnour and Bayuk, 1979a). For an assumed relative density of 100%, the back-calculated column diameter is 3.9 ft. (1.2 m), which is slightly larger than the 3.6 ft. (1.1 m) diameter reported previously (Goughnour and Bayuk, 1979a) for an assumed density of 125 pcf (19.6 kN/m^3).

The angle of internal friction, ϕ_s, of the stone was determined to be 45^o using a large triaxial cell and 4 in. (102 mm) diameter specimens. Particles greater than 1.5 in. (38.1 mm) were scalped and replaced on a weight basis by stone retained on the 1.0 in. (25.4 mm) sieve. Large scale lateral load tests (run like a direct shear test) have been performed at Jourdan Road Terminal (Parsons, Brinckerhoff, 1980) and large scale direct shear laboratory tests at WES (Ehrgott, 1977). The results of these tests indicate that the direct shear ϕ of unscalped stone is in the range of 45 to 53^o.

INSTRUMENTATION

Instrumentation, including inclinometers, load cells, piezometers, and settlement plates was installed in the east approach fill to the Ramp C bridge (Figure 2). The face of the east abutment of the Ramp C bridge is located at station 6 + 41, and can be correlated with stationing shown on Figure 2.

Three inclinometer tubes were installed in the right side slope to approximately a 40 ft. (12.2 m) depth. In conjunction with each of the slope indicator tubes, plates were installed at elevations +11.5 ft. (+3.5 m) and +5.8 ft. (1.7 m), for settlement measurements by means of an Idel Radiosonde.

The piezometers were Sinco Model No. 57481 and were placed at elevations -5 ft. (-1.5 m) and -10 ft. (-3 m) as shown. Sinco Model No. 51482 load cells were placed at about elevation +7 ft. (2.1 m). Load cells PC-1, PC-2, and PC-3 were placed on top of separate stone columns, while PC-4 was placed between columns.

Stone columns in this vicinity were installed in mid to late January, 1979. Embankment construction commenced approximately July 1, 1979. Estimated parameters for the soil strata indicated in Figure 2 are given in Table II.

Slope indicator results from inclinometer No. 2 shown on Figure 4 are typical. The initial reading was taken 30 days after the start of embankment construction when the embankment height was at about elevation +11.5 ft. (3.5 m). The maximum lateral movement was about 4.5 inches (11.4 cm) which appears to have occurred mostly between elevation -8 ft. (-2.4 m) and elevation -2 ft. (-0.6 m). This corresponds to soils 3 and 4 (Figure 2). Lateral movement appears to have stabilized after about 500 days in all inclinometers. Settlement results for Tube 2 shown on Figure

5 are also typical. The total settlement was approximately 1.15 ft. (0.35 m), and very little additional settlement appears to have occurred after about 450 days. Settlement at Tube 1 was 1.35 ft. (0.41 m) and at Tube 3 was 1.25 ft. (0.38 m).

SOIL NO.	THICK (ft.)	γ_w (pcf)	$C_c/1+e_o$	e_o	C_c	C_v (ft^2/day)
1	2	100	0.15	1.0	0.3	0.10
2	3	85	0.35	1.5	0.875	0.02
3	3	85	0.30	2.7	1.11	0.02
4	4	87	0.28	0.75	0.49	0.02
5	5	120	0.05	0.90	0.095	0.10

Table II - Estimated Soil Properties at Instrumented Embankment Section.

The results of total pressure cell readings were somewhat erratic, as shown on Figure 6. No pattern is discernable between the cells located on top of columns and PC-4 located between columns. The total pressure cells were located on top of the 3 ft. (0.9 m) sand platform, where the difference between compressibilities of the stone columns and the intervening sand platform material is not appreciable. This factor may have had some influence on high pressure readings between columns.

Pore pressure results are shown for piezometers P-3, P-4 and P-7 on Figure 7. Piezometer P-3 was located at elevation -10 ft. (-3 m) and P-4 was at -5 ft. (-1.5 m); both were located between columns. Excess pore pressures dissipated rapidly and remained well below the maximum fill pressure of about 13 psi (90 kN/m^2). A maximum pore pressure of almost 4 psi (28 kN/m^2) occurred in P-3 when the fill height was about 7.5 ft. (2.3 m).

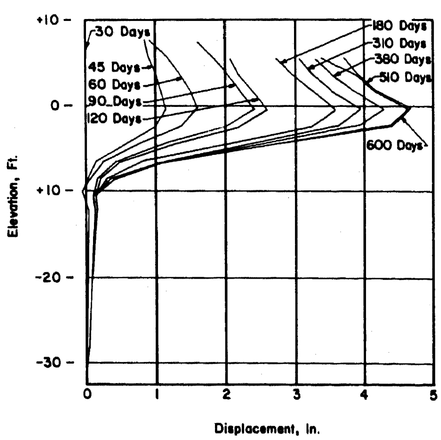

Figure 4. Typical Inclinometer Results - Tube 2.

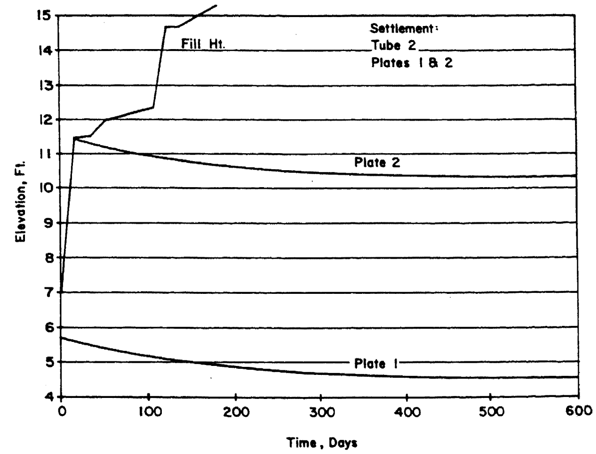

Figure 5. Settlement as Recorded from Tube 2.

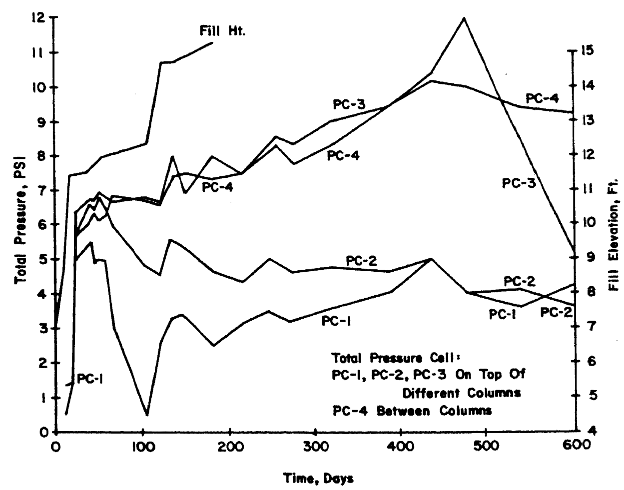

Figure 6. Result of Total Pressure Cell Readings.

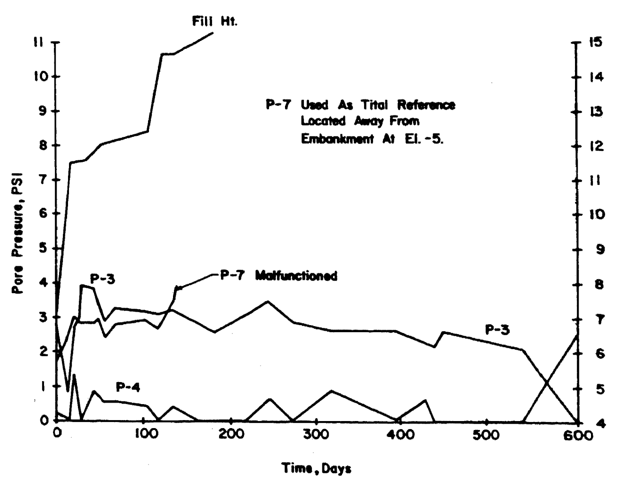

Figure 7. Typical Results of Pore Pressure Measurements.

SETTLEMENT PREDICTIONS

The ability of stone columns to reduce settlement under vertical load is well established, and this method has been applied in a wide variety of soil types. In this section the settlement of the embankment fill at settlement Tube 1 is predicted using the Japanese equilibrium method, a method presented by Priebe (1976), an incremental method (Vibroflotation method), and a finite element method.

Stone column construction in the Ramp C area was accomplished very quickly after placement of the 3 ft. (0.9 m) thick sand blanket working platform. Fill placement and settlement measurements began 5 to 5 1/2 months later. Therefore, an appreciable portion of the settlement due to the weight of the sand blanket had taken place before settlement measurements commenced. In order to account for this, the ultimate settlement was computed for the total embankment load including the sand blanket. Time-settlement calculations were also made for loading of the sand blanket only, and the amount of settlement at 5 1/2 months estimated. This estimated settlement was subtracted from the ultimate settlement calculated for the total embankment. Time-settlement calculations considered combined radial drainage to the stone columns and vertical soil drainage (Barksdale and Bachus, 1983). The estimated percent consolidation at 5 1/2 months was about 60%, and correlated very closely with behavior actually observed under embankment loading (see Figure 5).

Settlement Tube 1 was located 46 ft. (13.7 m) from the Ramp C bridge. The fill height at this location was 8.7 ft. (2.7 m) plus the 3 ft. (0.9 m) sand blanket. The vertical stress applied to the original ground surface was assumed to have the same shape as the embankment, and have a maximum value of 1588 psf (76 kN/m^2). This allowed for an additional 1 ft. (0.3 m) of fill added to the embankment to compensate for settlement. No shear stress was assumed to exist at the surface of the original ground. The stress in the layers due to the embankment was calculated at the center of each stratum (Figure 2) using Boussinesq stress distribution theory. Under total embankment load the stress increment ranged from 1421 psf (68 kN/m^2) at the center of the top layer to 1224 psf (58.6 kN/m^2) in the lower layer. Under sand blanket only loading these stress magnitudes were 375 psf (18 kN/m^2) and 348 psf (16.7 kN/m^2), respectively. An alternate more conservative approach would be to assume that no spreading of stress occurs.

Settlement Tube 1 was near the transition from the 7 ft. (2.1 m) to 8 ft. (2.4 m) stone column spacing (Figure 3). Therefore, an intermediate area replacement ratio (a_s = 0.262) was used. The soil properties summarized in Table II and the strata shown in Figure 2 were used for the settlement analyses.

Equilibrium method

The equilibrium method is the approach usually used in Japan to estimate the settlement of soft ground stabilized with sand compaction piles (Aboshi, et al., 1979). This method can also be readily used for stone column improved ground. A "unit cell" is defined as a stone column and the tributary area of soft soil surrounding it. Assume a uniform surface loading extends a considerable distance in every direction. Also, assume symmetry of load, geometry and stiffness within each unit cell. For these conditions the surface loading applied over a unit cell will remain in the unit cell, and the shear stresses on the boundaries of the cell will be zero. The unit cell can be physically modeled as a cylindrical-shaped container having a frictionless, rigid exterior wall symmetrically located around the stone column.

The stress concentration factor, n, is defined as the stress in the stone column, σ_s, divided by the stress in the surrounding clay, σ_c ($n=\sigma_s/\sigma_c$). For equilibrium of forces to exist within the unit cell, the stresses are as follows:

$$\sigma_c = \sigma/[1+(n-1)a_s] = \mu_c\sigma \tag{1a}$$

$$\sigma_s = n \cdot \sigma/[1+(n-1)a_s] = \mu_s\sigma \tag{1b}$$

where σ is the average stress applied over the unit cell, and a_s is the area replacement ratio, defined as the area of the stone column to the total area within the unit cell.

Once an n value has been estimated based on experience the value of μ_c, and thus the stress in the clay, can be computed by equation (1a). Conventional one-dimensional consolidation theory (or other technique) can then be used to estimate settlement in the clay. In Japan n is usually assumed to be between 3 and 5 for sand compaction piles (Barksdale, 1981). Barksdale and Bachus (1983) have recommended using an n of 4 to 5 for stone columns settlement calculations.

A stress concentration factor of 4 was selected as being appropriate, and the resulting μ_c value was 0.560. The estimated ultimate settlement under full embankment loading using this approach was 27.9 in. (709 mm). Under sand blanket loading the estimated ultimate settlement was 12.7 in. (323 mm), with 7.6 in. (193 mm) having occurred in 5 1/2 months. Thus, the estimated settlement for conditions of field measurement was 27.9 - 7.6 = 20.3 in. (516 mm).

Priebe method

The method proposed by Priebe (1976) uses the unit cell model also. The stone column is assumed to be in a state of plastic equilibrium under a triaxial stress state. The soil within the unit cell is idealized as an elastic material. Since the stone column is assumed to be incompressible, the change in volume within the soil is directly related to vertical shortening of the cylindrical column which forms the basis of the derivation. The radial deformation of the elastic soil is determined using an infinitely long, elastic hollow cylinder solution. The elastic cylinder of soil, which has a rigid exterior boundary coinciding with the boundary of the unit cell, is subjected to a uniform internal pressure. Other assumptions made in the analysis include (1) equal vertical settlement of the stone and soil, (2) uniform stresses in the two materials, and (3) end bearing on a rigid layer. The design relationship developed by Priebe (1976) is given in Figure 3 of his original paper. The ratio of settlement of untreated to treated ground, S/S_t, is given as a function of the area replacement ratio, a_s, and angle of internal friction of the stone, ϕ_s.

Using this figure with $a_s = 0.26$ and $\phi_s = 45^o$, S/S_t was found to be about 3.2. Calculations were performed using a conventional one dimensional consolidation analysis with stress levels as indicated previously. The computed settlement was then divided by the factor 3.2 to obtain the settlement of treated ground. In this case the estimated ultimate settlement under full embankment loading was 11.7 in. (298 mm). Under the sand blanket the estimated ultimate settlement was 6.0 in. (152 mm) with 3.5 in. (89 mm) having occurred in 5 1/2 months. The estimated settlement for conditions of field measurement was 11.7 - 3.5 = 8.2 in. (208 mm).

Vibroflotation method

The incremental approach (Goughnour and Bayuk, 1979b) analyzes individual disc shaped elements of the unit cell. The column material is assumed to be elasto-plastic, and incompressible in the plastic state. The soil confined within the unit cell is assumed to have a nonlinear elastic behaviour following an effective stress path which depends on the vertical and the radial strains ε_v and ε_r and on the problem geometry. When the replacement ratio approaches 1 the ratio, K, of the radial to the vertical effective stresses approaches $1/K_o$. During loading the effective stress path is assumed to be bilinear, and the K coefficient varies between K_o and $1/K_o$.

The stress ratio, n, is a function of the replacement factor, a_s, the instantaneous value of K, and the confining pressure applied to the column by the in situ soil.

In order to account for changing confining pressure with depth, the analysis is performed by successively considering vertical increments of the unit cell, i.e. disc shaped elements of thickness H. Soil properties as well as stress conditions can be varied from one element to the next. The effects of both vertical and radial compression of the soil within each element are considered, and conventional soil consolidation parameters are used directly.

The analysis follows two steps: first, the column is considered to be in a contained plastic state of equilibrium, and all the volume change is accommodated by the soft compressible soil. Then, the column is assumed to be linearly elastic and its vertical strain is calculated. The actual vertical strain at any level is the larger of those calculated for the two stages.

A complete summary of this theoretical approach along with design curves suitable for use in hand calculations of settlement in stone column treated soft ground has been presented elsewhere (Goughnour, 1983).

A K_o value of the in situ material was estimated as 0.5, based on the measured ϕ value of 26^o. The friction angle of the stone was taken as 45^o. The ultimate settlement under total embankment loading was computed as 19.3 in. (490 mm). Under sand blanket loading the estimated ultimate settlement was 7.7 in. (196 mm), with 4.7 in. (119 mm) occurring in 5 1/2 months. Thus, the estimated settlement was computed as 19.3-4.7 = 14.6 in. (371 mm).

Finite element analysis

Design curves developed by Barksdale and Bachus (1983) based on nonlinear, finite element theory (Barksdale, et al., 1982) were used for settlement predictions. In developing the finite element method, the unit cell concept previously described was used.

In soft clays not reinforced with stone columns, lateral bulging can increase the amount of vertical settlement beneath a fill by as much as 50% (Schwab, Broms, and Funegard, 1976). To approximately simulate lateral bulging effects in stone column improved ground, a soft boundary was placed around the unit cell to allow lateral deformation. The soft boundary was 1 in. (25 mm) thick and had a modulus of 12 psi (83 kN/m^2). The soft layer surrounding the unit cell gave lateral deformations similar to those observed at Jourdan Road Terminal (Munfakh, 1983).

In developing the design curves, a uniform loading was applied to a relatively rigid, 3 ft. (0.9 m) thick sand distribution blanket. The blanket was located above stone columns having length to diameter ratios of 5, 10, and 20. Stone replacement ratios of 0.10, 0.25 and 0.35 were used. The clay was assumed to be elastic-plastic with a shear strength of 400 psf (19 kN/m^2). The stone was assumed to have stress-strain properties similar to the gravel used at Santa Barbara. The angle of internal friction of the stone was taken to be 42^o. A coefficient of at-rest earth pressure, K_o, of 0.75 was used for both the stone and soil.

The finite element analysis was developed assuming the total load within the unit cell does not vary with depth (i.e., spreading of stress does not occur). The modulus of elasticity of the clay was also assumed constant with depth. At the Hampton site both of these quantities were found to vary with depth. Therefore, appropriate average values of the modulus and loading were used in the analysis.

Soil Strata 1 through 4 are soft and highly compressible clays, while Strata 5 is a relatively incompressible silty sand (refer to Table II and Figure 2). Because of the large difference in compressibility (and hence drained elastic modulus), the stone columns were assumed only to extend downward 12 ft. (3.7 m) to the bottom of the soft clay (Strata 4). Settlement in the underlying silty sand strata was calculated separately using elastic methods and added in.

The drained modulus of elasticity, E, of Strata 1 through 4 was calculated from the one-dimensional consolidation test results using the relationship (Barksdale and Bachus, 1983)

$$E = \frac{(1+\nu)(1-2\nu)}{0.435(1-\nu)} \left\{ \frac{1+e_o}{C_c} \right\} \sigma_{va} \qquad (2)$$

A drained ν of 0.45 was used in equation (2). The term σ_{va} is the average effective stress during load application. Under the sand blanket and fill the weighed average modulus was 12.3 psi (84.7 kN/m^2) and average vertical stress was 7.6 psi (52.4 kN/m^2). Using Figures 32-34 of Barksdale and Bachus (1983), the calculated long term fill settlement was found to be 17.5 in (444 mm) after settlement readings were begun.

DISCUSSION

The following is a summary of the results of the settlement calculations from the previous section compared with the observed settlement at settlement Tube 1:

	Settlement (ft.)	Settlement (cm)
Measured (600 days)	1.35	41±
Equilibrium method	1.69	52
Priebe method	0.68	21
Vibroflotation method	1.22	37
Finite element method	1.46	44

In all calculations settlement from compression of soil strata deeper than 18 ft. (5.5 m) below the original ground surface was not considered. Also, with the exception of the finite element method, the effect of horizontal movement of soil under the embankment was not included (see Figure 4). It should also be pointed out that although the data contained in Figure 2 and in Table II are the best available, their accuracy is still subject to the usual experimental errors. Thus, the calculation results are subject to the same limitations.

The following remarks concern the practical aspects of applying these methods to future settlement predictions of soft ground reinforced by stone columns:

Equilibrium method:

1. Uses conventional soil mechanics parameters directly in its application, and is no more difficult to apply than conventional settlement analyses. Gives physical feel for the problem.
2. The stress concentration factor, n, must be chosen based on experience. The effects of radial deformation of the in situ material as the column bulges, the stress-strain behavior of the column material, and the increased confining pressure on the column with depth are neglected except through the effect of this factor. Consequently, the user's skill in choosing n is of paramount importance.
3. Nevertheless the method appears to yield reasonable, though somewhat conservative, results for reasonable n values.

Priebe method:

1. Uses conventional soil mechanics parameters directly in its application, and also very easy to apply.
2. Effect of stress-strain behavior of the column material is considered, but in its plastic phase only.
3. Effects of radial compression considered.
4. Appears to under predict settlement for this case.

Vibroflotation method:

1. Uses conventional soil mechanics parameters directly in application.
2. Direct solution of the equations requires programmable calculator or computer, but use of design curves (Goughnour, 1983) yields this method very easy to apply.
3. Considers both radial and vertical compression of in situ soil, both elastic and plastic material behavior, and effect of increased soil confining pressure with depth.
4. Soil properties and stress levels can vary with depth.
5. Excellent results.

Finite element method:

1. Considers both radial and vertical compression of in situ soil, both elastic and plastic material behavior, and effect of increased soil confining pressure with depth.

2. Most versatile and theoretically accurate if computer solution applied to individual problems - can consider changing soil parameters and stress levels with depth.
3. Design curves are easy to apply but versatility is lost:

   (a) necessary to estimate Poisson's ratio in order to compute a drained modulus of elasticity, E, for the clay,

   (b) variations in soil properties or changes in stress level with depth only accommodated by using average values,

   (c) no ability to consider different column material properties.

4. Excellent results.

SUMMARY

A review of the design and construction of stone columns for the support of embankments at the Route I-664 Connector Interchange at Hampton, Virginia has been presented. Instrumentation at the test section has been described and performance for the first two years of operation reported. The stone column supported embankment has performed essentially as expected during this time period.

A review of four methods of predicting settlements of stone column reinforced ground has been presented along with settlement predictions for the embankment test section by each of these methods. Although the settlements predicted by each method differ they generally give good results. Practical aspects of the use of each method have been discussed.

ACKNOWLEDGMENT

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Stone column installation was performed by the Vibroflotation Foundation Company, of Pittsburgh, Pennsylvania.

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