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Stantec

Exhibit 14
Properties of Soil-Cement and Stabilized Foundation

Purpose:

- Determine design material properties for the soil-cement to be formed during construction.
- Determine equivalent parameters for the stabilized foundation, consisting of a 3D structure of soil-cement walls.

Methods:

- Review laboratory data from samples acquired in the deep mixing pilot test.
- Review the literature for typical parameters for soil-cement materials.
- Engineering evaluations to select appropriate design parameters.

Results:

- Tabulations of stiffness and strength parameters for the specified soil-cement.
- Tabulations for equivalent properties of the stabilized foundation.

Calculation Performed by: Stantec Consulting Services, Inc.	
Prepared by: AFR 15-Nov-10	*Reviewed by:* YW 16-Nov-10
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Properties of Soil-Cement and Stabilized Foundation

1. Introduction

Closure of the Kingston dredge cell will include stabilizing the ground in place beneath a new perimeter berm. The stabilized foundation will be designed to support the ash landfill in the design earthquake, where extensive liquefaction is expected. The foundation soils will be treated using in situ ground improvement techniques. Soil mixing will be used to form panels or walls of cemented soil and ash that enclose cells of untreated, liquefiable soil and ash. This will be accomplished using specialty construction equipment to inject and mix cementitious grout with the in-place soils, to form walls of stabilized soil-cement or “soilcrete”. Depending on the machinery employed, the walls will be formed by overlapping columns or as continuous panels.

Mechanical properties (strength and stiffness) of the cured soil-cement are discussed here in Section 3. The work of Filz and Navin (2005) is particularly helpful in establishing these design parameters; they surveyed the literature and evaluated data from several deep mixing projects, including more than 2,000 data points from one project in Virginia.

Two-dimensional engineering analyses require a set of properties that represent the composite behavior of the untreated soil and the mixed soil-cement in the stabilized perimeter foundation. The derivation of composite properties is discussed in Section 4 and Exhibit 22.

2. 2010 Deep Mixing Pilot Test

2.1. Scope of Pilot Test

In July and August of 2010, TVA conducted a field demonstration test (pilot test) of deep soil mixing at the Kingston dredge cell site. The primary purpose of this test was to demonstrate the feasibility of mixing cemented soil elements in place, in the site-specific ash and soils. The pilot test was completed by Remedial Construction Services, L.P. (RECON) of Houston, Texas.

Eleven overlapping, soil-cement columns were constructed to rock, at a location inside the northern perimeter of the site. The columns were constructed with a 5-ft diameter auger and paddle mixing tool, which rotated about a vertical axis for insertion, mixing, and withdrawal in each column.

Columns were built using mixing rates of 7.5% or 10% cement by dry weight. In the pilot test, the target strength (unconfined compression) for the cured soil cement mix was 150 psi at 28 days. Wet grab samples were retrieved from the columns at the completion of mixing, and then formed into cylindrical test specimens. Additional test specimens were obtained a month later by drilling and coring the cured columns.

2.2. Strength Data from the Pilot Test

Strength data from the Kingston deep mixing pilot test are provided here in appendices. Laboratory data from uniaxial (unconfined) compression tests, triaxial compression tests, and splitting tensile strength tests are tabulated in Appendix A. The laboratory test results for the wet grab specimens are presented in Appendix B, while data for the cored specimens are provided in Appendix C.

The unconfined compression test data are summarized in the first two tables of Appendix A, for wet grab and core samples. Of 19 tests on wet grab samples that had cured 28 days, only one specimen achieved the specified strength of 150 psi. For samples that had been mixed at both 7.5% and 10% cement, the average 28-day strength was just 86 psi. Thirteen additional wet grab samples that were tested at 56 to 59 days showed an average strength of 118 psi. All eight of the cored samples, tested at 51 to 59 days after construction, had unconfined compressive strengths greater than the required minimum of 150 psi. The average strength of the cored specimens was 399 psi.

The results of the unconfined compression tests on the retrieved core samples exhibit substantial variation, even though the amount of data is limited. In general this variation makes drawing conclusions from these tests difficult. However, the results of the core tests clearly show strengths that are higher than those from the wet grab samples. Any of the following possible causes, or combinations of them, could lead to this condition:

(1) The wet grab samples may have undergone inadequate curing. Cementitious materials typically gain strength quickly at first, then at a slower rate over time. In many of the wet grab samples the rate of strength gain between 56 and 84 days is higher than the rate between 28 and 56 days. This suggests that some change in the curing conditions or process may have taken place, leading to the increase in strength gain.

(2) The wet grab sampler may not have retrieved representative samples of the soil-cement columns. In some cases, compression tests on wet grab samples gave a lower strength than in earlier tests, even though the strengths typically increase over time. This suggests that either the wet grab sample was not homogenous, or the soil-cement column was not homogenous in the area sampled.

(3) Shape and size effects may have skewed the results of the core tests. The core specimens were typically smaller in diameter and height than the wet grab specimens. Smaller specimens of identical materials often produce higher strengths, because the smaller size contains fewer low-strength areas that initiate failure. Furthermore the ratio of specimen height to specimen diameter is not identical between the cores and wet grab specimens. This ratio can affect the results of unconfined compression strength tests.

Examination of the unconfined compressive strength tests on the wet grab samples can provide information about the variation in strength with cement dosage, with time, and with depth in the soil-cement column. The wet grab samples were generally retrieved from any of three different depths in the soil-cement column. These three depths correspond roughly with three different layers of subsurface materials at the site: ash, clay, and sand.

The plot in Appendix A shows the average strengths of each depth, for each cement dosage, over time for the strength tests on the wet grab samples. From this plot, the difference in the strength with respect to the depth or soil type seems to be minor. This is an indication of the vertical mixing that took place within each soil-cement column, resulting in relatively consistent strengths in the different soil strata. In general, there is only a minor difference in strength associated with the two different cement dosage rates, although this could be caused by problems with some of the samples. The strengths of the samples at the depth of ash for the 10% dosing rate seem to be lower than what would be expected based on the other data. There appears to be a fairly consistent pattern of the lower dosage rates having a higher early strength, with the higher dosage rates gaining strength more quickly later on, but the difference is not sufficiently large to rule out random variation as the cause. Finally, as noted above, the rate of strength gain for each material increases between 56 and 84 days, which may indicate irregularities in the specimen curing process.

3. Soil-Cement Properties

3.1. Unit Weight

The weight of the added cement would, theoretically, increase the density of the treated soil, but the mixing process introduces water and produces spoils that are removed at the ground surface. Hence, the impact of deep mixing on the overall soil density is difficult to predict. Filz and Navin (2005) observed that deep mixing does not usually have a large effect on unit weights; they discuss case histories where deep mixing has lead to modestly increased and modestly decreased soil unit weights.

Estimates of composite unit weights within the stabilized foundation are needed to compute stresses for the various engineering analyses. The distribution of vertical stresses across the treated and untreated soil will vary due to stress concentrations and vertical shear along the sides of the stabilized walls. These effects are also difficult to quantify, but the impact on the stability analyses are not expected to be significant.

Hence, the treated soil zones are assumed to have the same, unaltered unit weight as the untreated soils. The design parameters are summarized in Table 3.1.

3.2. Unconfined Compressive Strength (f'_{sc})

The project specifications require that the soil-cement in the stabilized walls achieve specified targets for the unconfined compressive strength. Using soil-cement samples obtained by coring the completed walls, the "unconfined compressive strength" (f'_{sc}) is to be measured in accordance with ASTM D1633 with specimens that have cured for 56 days. For the Kingston project, the key requirement for soil-cement strength is:

- Average $f'_{sc} \geq 200$ psi at 56 days

In the deep mixing industry, soil-cement strength specifications are typically written in terms of unconfined strengths measured at 28 days of curing (Filz and Navin 2005). This allows verification of compliance with the specifications about one month after construction, but does not account for the expected increase in soil-cement strength with time after 28 days. One notable exception is the San Pablo Dam project (Kirby et al. 2010), where the design calculations effectively assumed a doubling of the 28-day soil-cement strength within 1 to 5 years.

For the Kingston dredge cell project, the specifications for the mixed soil-cement require compliance based on strengths measured at 56 days. The extended curing time can be accommodated within the project schedule, and should allow for somewhat lower cement contents that achieve the design strength. Any strength gain beyond 56 days is neglected in the design assumptions.

A mean strength of f'_{sc} = 200 psi (28,800 psf) was selected for the Kingston closure project, based on considerations for localized, maximum stresses predicted to result in the design earthquake. The structural capacity of the grid of soil-cement walls was evaluated using a three-dimensional numerical model (see Exhibit 21). The project specifications also allow for variability in the soil-cement strengths, based on a reliability analysis presented in Exhibit 16.

The specified, mean strength of f'_{sc} = 200 psi is compared to the strengths specified in other case history projects in Table 3.2. Based on these past projects, the strength selected for the Kingston project should be readily achievable.

3.3. Shear Strength (S_f)

In the field, the soil-cement walls in the stabilized foundation are subject to failure by shearing along failure planes that are mostly horizontal, but may be inclined. To support the design calculations, the unconfined compressive strength (f'_{sc}) must be related to the strength on the failure plane (S_f). From the literature, however, there appears to be some lack of consensus within the industry for how to relate S_f to f'_{sc} in soil-cement.

In their review of this issue, Filz and Navin (2005) indicate that the typical practice in Europe is to adopt effective stress parameters, with ϕ'_{sc} = 25° to 30° and a range of c'_{sc}, for soil-cement strengths. Suzuki (1982) performed drained triaxial tests on soil-cement and measured ϕ'_{sc} = 26° to 33°, with c'_{sc} = (0.25 to 0.5) f'_{sc}.

In the U.S. and Japan, the design practice appears to more commonly quantify soil-cement strengths using total stress parameters, with ϕ_{sc} = 0 and S_f = (0.33 to 0.5) f'_{sc}. Filz and Navin (2005) indicate that the Japanese may commonly assume S_f = 0.5 f'_{sc}. Suzuki (1982) recommended using S_f = 0.25 f'_{sc} for f'_{sc} < 140 psi, S_f = 0.33 f'_{sc} for 140 < f'_{sc} < 580 psi, and S_f = 0.5 f'_{sc} for f'_{sc} > 580 psi. Wooten et al. (2003) and Wooten and Foreman (2005) suggest that S_f should be (0.33 to 0.5) f'_{sc}. For the design of the Clemson Dams project, they used S_f = 0.42 f'_{sc} without further explanation. Kirby et al. (2010), acknowledging the work of Wooten et al. (2003), pointed to the effect of stress path on shear strength; their design for the San Pablo Dam project assumed S_f = 0.33 f'_{sc}. Filz and Navin (2005), noting the differences of opinion on this issue, recommend using ϕ_{sc} = 0 and S_f = 0.40 f'_{sc}.

These two approaches to characterizing shear strength will result in similar values of S_f for soil-cement in the typical pressure range of interest. For example, the Kingston project requires f'_{sc} = 200 psi (28,800 psf). Assuming S_f = 0.40 f'_{sc}, this results in a shear strength of 11,520 psf. In a saturated soil deposit at a depth of about 40 feet, the vertical effective stress is typically about 2,500 psf. If ϕ'_{sc} = 30° and c'_{sc} = 0.33 f'_{sc}, then the drained shear strength would be $S_f = c'_{sc} + \sigma' \tan\phi'_{sc}$ = (28,800/3) + 2,500 tan(30°) = 11,000 psf. Hence, using total stress or effective stress parameters would result in about the same S_f at this depth.

If the drained friction angle (ϕ'_{sc}) is known, the shearing resistance on the failure plane in an unconfined compression test can be computed directly. Referring to the Mohr's circles for effective and total stress in Figure 3.1, the shear stress on the failure plane is computed as:

$$S_f = \frac{f'_{sc}}{2} \cos \phi'_{sc} \qquad 3.1$$

Data from five sets of consolidated-undrained triaxial compression tests were obtained on samples from the Kingston deep mixing pilot test. The test data are presented in Appendices B and C, and summarized in Table 3.3. Setting aside the rather high friction angle reported for the cored samples, the average friction angle measured in the other four test sets is about ϕ'_{sc} = 33°. Per Equation 3.1, this would correspond to $S_f = 0.42\ f'_{sc}$.

Saito et al. (1980) performed a series of direct shear and unconfined compression tests on a range of soil-cement mixtures, and presented this empirical correlation:

$$\tau_o = 0.53 + 0.37 f'_{sc} - 0.0014 f'^2_{sc} \qquad (\text{for } f'_{sc} \leq 60\ \text{kg/cm}^2) \qquad 3.2$$

where τ_o is the shear strength obtained from a direct shear test at zero normal stress, and τ_o and f'_{sc} are both expressed in pressure units of kg/cm^2. For f'_{sc} = 200 psi = 14.1 kg/cm^2, this empirical correlation would indicate a shear strength of 5.5 kg/cm^2, or $S_f = 0.39\ f'_{sc}$.

Filz and Navin (2005) reasoned that the design strength for soil-cement should represent the post-peak, residual shearing resistance of brittle soil-cement. Strain-softening materials are subject to progressive failure, wherein excessive stresses may initiate failure in localized areas, which then leads to stress redistributions and failure in adjacent areas. Filz and Navin recommend using a lower strength as a means to protect against progressive failures. For the Kingston project, however, the design is based on peak strengths for the soil-cement, with detailed 3D numerical analyses and adequate safety factors used to avoid initiating a progressive failure.

In consideration of the state of the practice, the published research reviewed here, and the test results from the field pilot test, the following relationship will be assumed for the Kingston design:

$$S_f = 0.40\ f'_{sc} \qquad 3.3$$

For the required, mean f'_{sc} = 200 psi = 28,800 psf, the corresponding shear strength of the soil-cement is then S_f = 11,520 psf.

3.4. Tensile Strength (σ_T)

Filz and Navin (2005) report that Japanese data show the tensile strength (σ_T) of deep-mixed soil-cement is typically in the range of 10% to 20% of f'_{sc}.

In the deep mixing pilot test, the tensile strength of 10 test specimens was measured by the splitting test method (ASTM D3967). The results are tabulated in Table 3.4. Ignoring the one outlier, the average measured tensile strength was 8.5 psi.

Focusing on just the wet grab samples that had cured 28 days, the laboratory tests gave a mean tensile strength of 7.8 psi. For the wet grab samples tested in unconfined compression at 28 days, the average unconfined compressive strength was 85.8 psi (Section 2). Hence, the data from the pilot test program shows that, on average, the tensile strength is about 9.1% of unconfined compressive strength.

A tensile strength of σ_T = 0.10 f'_{sc} (σ_T = 20 psi = 2,880 psf) will be assumed in the design calculations. This value corresponds to the lower end of the expected range for soil-cement mixtures, based on the literature, and is substantiated by measurements from the field pilot test.

3.5. Secant Elastic Moduli

Filz and Navin (2005) found that values of the secant Young's modulus (E) vary from about 75 to 1,000 times f'_{sc}, and Poisson' ratios (ν) in the range of 0.25 to 0.50 have been reported. Filz and Navin chose to use ν = 0.45 and E/f'_{sc} = 300 in their study.

For the specified strength of f'_{sc} = 200 psi and a modulus ratio of 300, the secant Young's modulus would be E = 8.6 x10^6 psf. Poisson's ratios in the range of 0.32 to 0.37 were estimated for the untreated soils in the perimeter foundation (see Exhibit 9). Adding cement will result in a stiffer material, so a value of ν = 0.3 was assumed.

Elastic parameters are required for the *FLAC3D* model of the stabilized foundation. The *FLAC* model requires input of shear (G) and bulk (K) moduli for each material. Based on the assumptions above, these parameters are then:

$$G = \frac{E}{2(1+\nu)} = \frac{8.6 \times 10^6\ psf}{2(1+0.3)} = 3.3 \times 10^6\ psf \qquad 3.4$$

$$K = \frac{E}{3(1-2\nu)} = \frac{8.6 \times 10^6\ psf}{3(1-2\cdot 0.3)} = 7.2 \times 10^6\ psf \qquad 3.5$$

These estimated values, which represent the stiffness under static loading conditions, were used for the mixed soil-cement in all subsurface strata of the foundation.

3.6. Dynamic Moduli

Representative values of the bulk and shear moduli for soil cement under dynamic load are needed for the dynamic *FLAC* analysis. Lacking a direct correlation for soil cement mixes within the literature, these values were estimated by considering typical values for soil. The maximum shear modulus (G_{max}) was thus assumed to be about 50 times the drained shear modulus. The bulk modulus was calculated using the method described in Exhibit 9.

Table 3.1. Assumed Unit Weights for Mixed Soil-Cement

Material Name	Void Ratio, e	γ_d (pcf)	γ_m (pcf)	γ_{sat} (pcf)
Stabilized Foundation Ash	0.85	78	100	107
Stabilized Foundation Clay	0.58	107	--	130
Stabilized Foundation Sand	0.62	104	--	128

Table 3.2. Specifications used in Case History Deep Mixing Projects

Project	Pattern of Deep Mixed Walls	Specified Unconfined Compressive Strength
Jackson Lake Dam	Honeycomb	400 – 600 psi
Clemson Upper and Lower Diversion Dams	Comb	400 psi
Sunset North Basin	Grate	300 psi
Tuttle Creek Dam	Transverse Walls	300 psi
San Pablo Dam	Grate	450 psi
Kingston Dredge Cell	*Grate*	*200 psi*

Table 3.3. Data from Consolidated-Undrained Triaxial Compression Tests on Samples Acquired from the Deep Mixing Pilot Test

Depth of Sample (feet)	Cement Mix	Sample Type	Cure Period (days)	c'_{sc} (psi)	ϕ'_{sc} (degs)
12	7.5%	Wet Grab	28	17.9	34.1
12	10%	Wet Grab	28	14.0	31.3
35	7.5%	Wet Grab	28	11.2	32.0
35	10%	Wet Grab	28	10.9	32.8
24	10%	Core	56	0.0	64.2

Table 3.4. Tensile Strengths (σ_T) Measured in Splitting Tensile Tests (ASTM D3967) on Samples Acquired from the Deep Mixing Pilot Test

Depth of Sample (feet)	Cement Mix	Sample Type	Cure Period (days)	σ_T (psi)
12	10%	Wet Grab	28	9.0
12	10%	Wet Grab	28	10.5
12	7.5%	Wet Grab	28	9.0
12	7.5%	Wet Grab	28	9.1
35	7.5%	Wet Grab	28	5.6
35	7.5%	Wet Grab	28	5.9
35	10%	Wet Grab	28	6.5
35	10%	Wet Grab	28	6.9
15	7.5%	Core	56	83.1
55	7.5%	Core	56	13.7

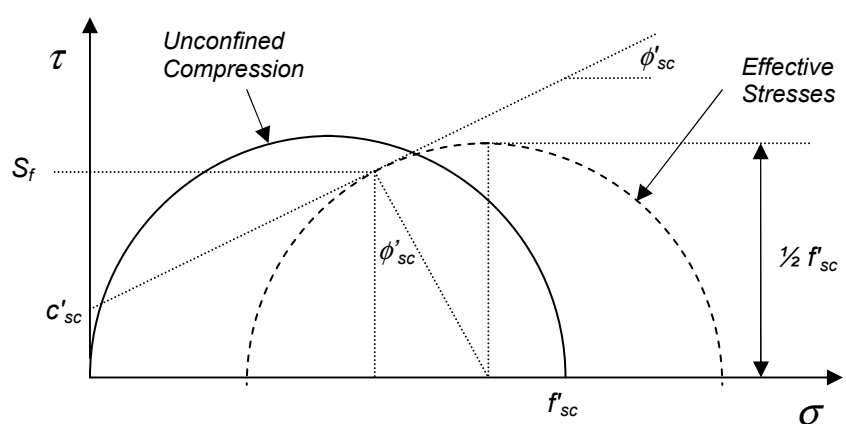

Figure 3.1. Relationship between Strength on the Failure Plane (S_f) and Unconfined Compressive Strength (f'_{sc})

4. Composite Properties for Treated and Untreated Soils

This section outlines the methods and assumptions used to estimate properties for the composite behavior of the stabilized and untreated soils in the foundation.

Vertical sections through the closed dredge cell perimeter were evaluated for stability using conventional, limit equilibrium, slope stability analyses and an advanced, dynamic numerical model (in the computer code *FLAC*). In each, the behavior of the three dimensional pattern of soil-cement panels must be represented in two dimensions. Material parameters are thus needed that will result in a predicted mechanical response of the stabilized berm foundation in two dimensions (2D) that is equivalent to the response in three dimensions (3D).

The equivalent 2D parameters must represent the composite behavior of both the soil-cement walls and the untreated soils (between the walls) beneath the berm footprint. As described in Exhibit 22, a three-dimensional model (in *FLAC3D*) was used to determine equivalent shear moduli for the 2D model. The 3D model was also used to evaluate the internal, structural integrity of the soilcrete panels (see Exhibit 21). However, because of the complexity and computational effort that would be required, dynamic calculations were not attempted with the 3D model.

In estimating the composite stiffness and strength parameters for the stabilized foundation, the soil-cement elements are assumed to maintain their full pre-earthquake strength (no softening under earthquake load). The soil between the cemented panels is assumed to experience increased pore pressure and strength loss due to dynamic loading; the saturated ash and silty sands are assumed to be fully liquefied at the start of seismic loading. This assumption is conservative. To model the progressive development of liquefaction in the untreated soils between the panels, a prohibitively complex 3D dynamic simulation would be required.

4.1. Composite Strength

Composite strength parameters, representing the zone of stabilized foundation soils, were computed based on the percentage of the horizontal area that is treated (area replacement ratio approach). A weighted average was computed from the strengths of the soil-cement and untreated materials, with different composite strengths computed for each ash or soil layer in the foundation. The resulting strength parameters are appropriate as inputs for both conventional slope stability analyses and the *FLAC* model.

The derivation below for the weighted average or composite strength in the treated foundation zone assumes that the potential shear planes are primarily horizontal. Stability analyses for the conceptual closure plan demonstrated that this assumption was valid, with critical failure surfaces that cut through the base of the perimeter berm. For a horizontal shear plane, the shearing resistance is a function of the vertical stress.

In each of the soils, the shearing resistance is appropriately characterized using values of c and ϕ, for either drained or undrained conditions (see Exhibit 13). The undrained strength of some soil layers has been characterized using c/p ratios; assuming horizontal shearing planes (where the normal consolidation stress equals σ'_v), the strength of these soils can be approximated using $c = 0$ and $\phi = \tan^{-1}(c/p)$. Likewise, the residual strength along horizontal shearing planes in the liquefied soils can be approximated with $c = 0$ and $\phi = \tan^{-1}(S_{us}/\sigma'_v)$.

The error associated with this approximation for the soil strength is negligible, given the small values of ϕ, *c/p,* and S_{us}/σ'_v. Using this formulation, instead of a constant shear strength with $\phi = 0$, allows for consideration of the modestly increased shear resistance at greater depths.

Given strength parameters for each soil (*c* and ϕ) and the soilcrete (f'_{sc}), the weighted average strength is computed based on the fraction of the horizontal plane that cuts through soilcrete or untreated soil. The area replacement ratio (*ARR*) is the fraction of the foundation footprint that has been treated:

$$Area\ Repalcement\ Ratio = ARR = \frac{A_{sc}}{A_t} = \frac{Area\ Treated\ with\ Soil{-}Cement}{Total\ Area\ in\ Plan} \qquad 4.1$$

The total or composite shearing resistance along a horizontal plane is:

$$\left(S_{avg} \cdot A_t\right)_{composite} = (S \cdot A)_{untreated} + \left(S_f \cdot A_{sc}\right)_{soil-cement} \qquad 4.2$$

where S_{avg} is the weighted average or equivalent strength, *S* and *A* are the strength and plan area of untreated soil, and S_f and A_{sc} are the strength and plan area of the soil-cement (soilcrete). Substituting appropriate parameters gives:

$$S_{avg} = (c + \sigma \tan\phi)\left(\frac{A_t - A_{sc}}{A_t}\right) + \left(S_f\right)\left(\frac{A_{sc}}{A_t}\right) \qquad 4.3$$

or:

$$S_{avg} = (c + \sigma \tan\phi)(1 - ARR) + (0.4 f'_{sc})(ARR) \qquad 4.4$$

The equivalent, weighted average strength parameters for shearing along horizontal planes through the stabilized foundation soils (soil-cement walls and untreated soil between) are then:

$$c_{avg} = (1 - ARR)c + (0.4 \cdot ARR \cdot f'_{sc}) \qquad 4.5$$

$$\phi_{avg} = \tan^{-1}[(1 - ARR)\tan\phi] \qquad 4.6$$

Equivalent, weighted average strength parameters, computed using Equations 4.5 and 4.6, are provided in Table 4.1 through Table 4.3 for the three foundation soil layers and area replacement ratios ranging from 0.20 to 0.30.

When assigning composite strengths, some authors express concern for strain incompatibility between the stiff soil cement and the softer untreated soil. That is, larger strains are required to develop the strength of the untreated soil than within a soil-cement column, such that one cannot rely upon the strength of the soil and soilcrete to develop at the same point in time. In the Kingston design, however, the untreated soils are mostly assumed to liquefy in the design earthquake. Because of the seismic loading, the residual shear resistance of the soil is developed concurrently with loading on the soil-cement walls. Hence, for this particular design problem, strain incompatibility is not a consideration.

4.2. Composite Stiffness

Composite, 2D stiffness parameters for the 3D structure of soil-cement walls were obtained using *FLAC3D* and are documented in Exhibit 22.

Table 4.1. Composite Strength Parameters for the Stabilized Foundation Ash, for Soil-Cement Strength of f'_{sc} = 200 psi

Area Replacement Ratio (ARR)	Drained, Effective Stress Parameters for Static, Long Term Conditions		Undrained, Total Stress Parameters for Static, Short Term Conditions		Undrained, Total Stress Parameters for Seismic Conditions*	
	ϕ'_{avg} (deg.)	c'_{avg} (psf)	ϕ_{avg} (deg.)	c_{avg} (psf)	ϕ_{avg} (deg.)	c_{avg} (psf)
0	**25**	**0**	**10**	**0**	**3.4**	**0**
20%	20.5	2304	8.0	2304	2.7	2304
21%	20.2	2419	7.9	2419	2.7	2419
22%	20.0	2534	7.8	2534	2.7	2534
23%	19.8	2650	7.7	2650	2.6	2650
24%	19.5	2765	7.6	2765	2.6	2765
25%	19.3	2880	7.5	2880	2.6	2880
26%	19.0	2995	7.4	2995	2.5	2995
27%	18.8	3110	7.3	3110	2.5	3110
28%	18.6	3226	7.2	3226	2.5	3226
29%	18.3	3341	7.1	3341	2.4	3341
30%	18.1	3456	7.0	3456	2.4	3456

* Equivalent parameters for liquefied soil with S_{us}/σ'_v = 0.06

Table 4.2. Composite Strength Parameters for the Stabilized Foundation Clay, for Soil-Cement Strength of f'_{sc} = 200 psi

Area Replacement Ratio (ARR)	Drained, Effective Stress Parameters for Static, Long Term Conditions		Undrained, Total Stress Parameters for Static, Short Term Conditions		Undrained, Total Stress Parameters for Seismic Conditions	
	ϕ'_{avg} (deg.)	c'_{avg} (psf)	ϕ_{avg} (deg.)	c_{avg} (psf)	ϕ_{avg} (deg.)	c_{avg} (psf)
0	**32**	**0**	**24**	**0**	**20**	**0**
20%	26.6	2304	19.6	2304	16.2	2304
21%	26.3	2419	19.4	2419	16.0	2419
22%	26.0	2534	19.2	2534	15.8	2534
23%	25.7	2650	18.9	2650	15.7	2650
24%	25.4	2765	18.7	2765	15.5	2765
25%	25.1	2880	18.5	2880	15.3	2880
26%	24.8	2995	18.2	2995	15.1	2995
27%	24.5	3110	18.0	3110	14.9	3110
28%	24.2	3226	17.8	3226	14.7	3226
29%	23.9	3341	17.5	3341	14.5	3341
30%	23.6	3456	17.3	3456	14.3	3456

Table 4.3. Composite Strength Parameters for the Stabilized Foundation Sand, for Soil-Cement Strength of f'_{sc} = 200 psi

Area Replacement Ratio (ARR)	Drained, Effective Stress Parameters for Static, Long Term Conditions		Undrained, Total Stress Parameters for Static, Short Term Conditions*		Undrained, Total Stress Parameters for Seismic Conditions**	
	ϕ'_{avg} (deg.)	c'_{avg} (psf)	ϕ_{avg} (deg.)	c_{avg} (psf)	ϕ_{avg} (deg.)	c_{avg} (psf)
0	**30**	**0**	**12**	**1000**	**3.4**	**0**
20%	24.8	2304	9.7	3104	2.7	2304
21%	24.5	2419	9.5	3209	2.7	2419
22%	24.2	2534	9.4	3314	2.7	2534
23%	24.0	2650	9.3	3420	2.6	2650
24%	23.7	2765	9.2	3525	2.6	2765
25%	23.4	2880	9.1	3630	2.6	2880
26%	23.1	2995	8.9	3735	2.5	2995
27%	22.9	3110	8.8	3840	2.5	3110
28%	22.6	3226	8.7	3946	2.5	3226
29%	22.3	3341	8.6	4051	2.4	3341
30%	22.0	3456	8.5	4156	2.4	3456

* Undrained parameters tabulated for static, short-term conditions are applicable only for $\sigma \geq 2{,}740$ psf. At lower stress levels, use the drained strength parameters.

** Equivalent parameters for liquefied soil with $S_{us}/\sigma'_v = 0.06$

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Appendix A

Deep Mixing Pilot Field Test: Laboratory Data Summaries

Unconfined Compressive Strength (psi) - Wet Grab Samples

Depth and Strata	Curing Period (days)	10% Mix Design, Column #				7.5% Mix Design, Column #				
		1A	2A	3	5A	7	8	9	10	11
12 feet Ash	7	30	65	30	40	75	_	70	50	55
	14	55	50	50	50	90	_	85	50	55
	28	65	75	45	65	160	_	135	75	95
	56	_	90	100	_	_	_	_	_	110
	59	_	_	_	60	_	_	_	_	_
	84	190	_	_	_	_	_	220	180	_
35 feet Clay	7	_	40	35	70	50	_	60	_	50
	14	_	40	60	90	60	_	70	_	45
	28	_	50	80	120	65	_	90	_	55
	56	_	55	200	_	105	_	_	_	65
	59	_	_	_	165	_	_	_	_	_
	84	_	_	_	_	_	_	225	_	_
45 feet Silty Sand	7	_	55	45	60	_	45	_	45	_
	14	_	70	65	80	_	70	_	70	_
	28	_	80	85	110	_	90	_	90	_
	56	_	125	190	_	_	125	_	_	_
	59	_	_	_	145	_	_	_	_	_
	84	_	_	_	_	_	_	_	190	_

Unconfined Compressive Strength (psi) - Core Samples

Strata	Depth (feet)	Curing Period (days)	10% Mix Design Column #		7.5% Mix Design Column #	
			1A	2A	8	10
Ash	12.0	51			385	190
	12.0	56	415			
	12.0	58		230		
Clay	24.0	51				605
	26.0	56	705			
	33.5	51				475
Silty Sand	42.5	51				190

Strengths from Triaxial Compression Tests

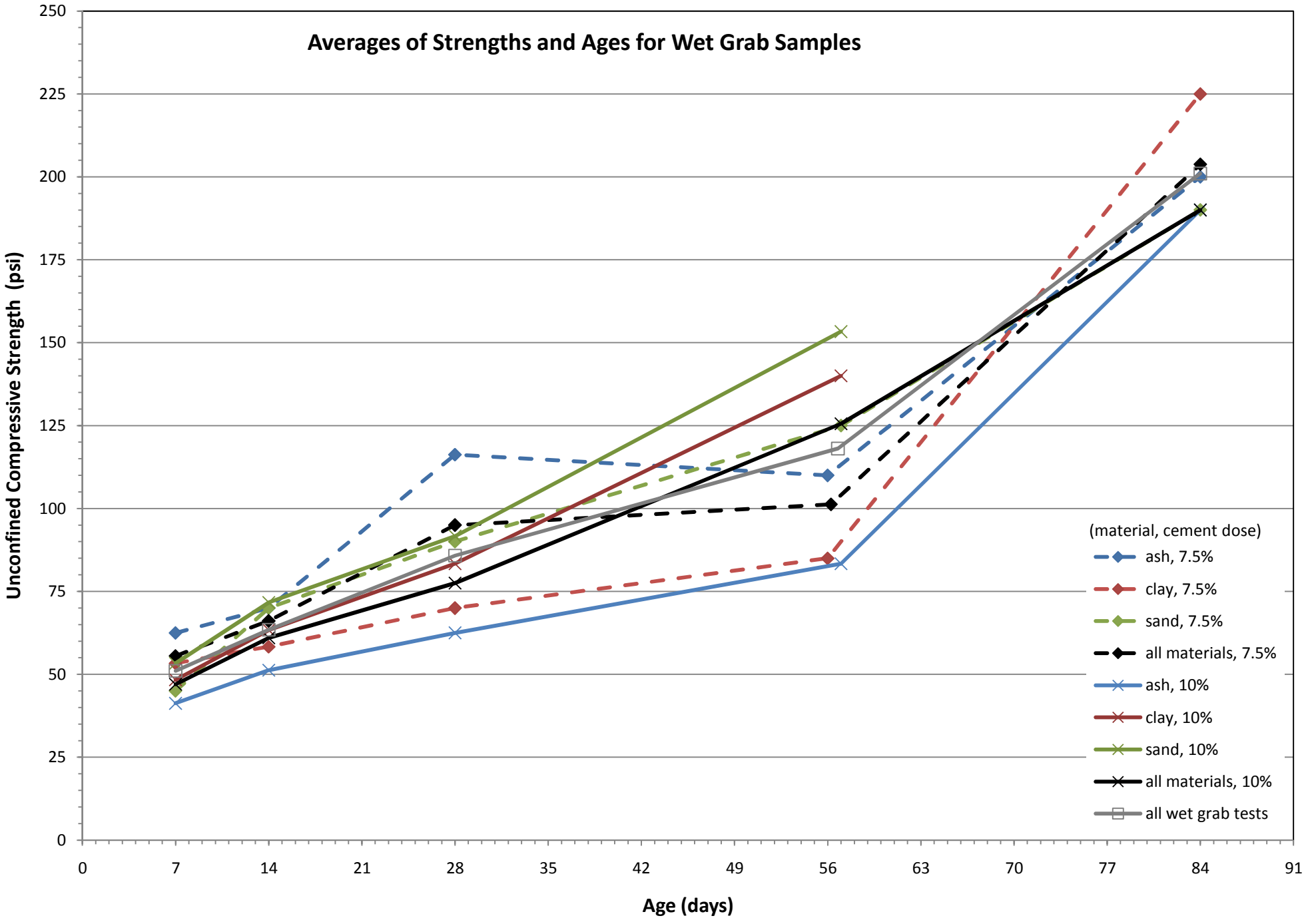
Strata	Depth (ft)	Sample Type and Cure	10% Mix Design		7.5% Mix Design
			Column 1A	Column 4	Column 8
Ash	12	Wet Grab 28-day		c' = 14 psi ϕ '= 31.3 deg	c' = 17.9 psi ϕ '= 34.1 deg
Ash	24	Core 56-day	c' = 0 psi ϕ '= 64.2 deg		
Clay	35	Wet Grab 28-day	c' = 10.9 psi ϕ '= 32.8 deg		c' = 11.2 psi ϕ '= 32.0 deg

Axial Strains (%) at Peak Deviator Stress in Triaxial Compression Tests

Strata	Depth (ft)	Sample Type and Cure	10% Mix Design		7.5% Mix Design
			Column 1A	Column 4	Column 8
Ash	12	Wet Grab 28-day		4.0 4.0 8.7	2.8 12.0 7.0
Ash	24	Core 56-day	2.7 2.6		
Clay	35	Wet Grab 28-day	2.8 10.3 8.5		5.3 8.7 8.6

Splitting Tensile Strengths (psi)

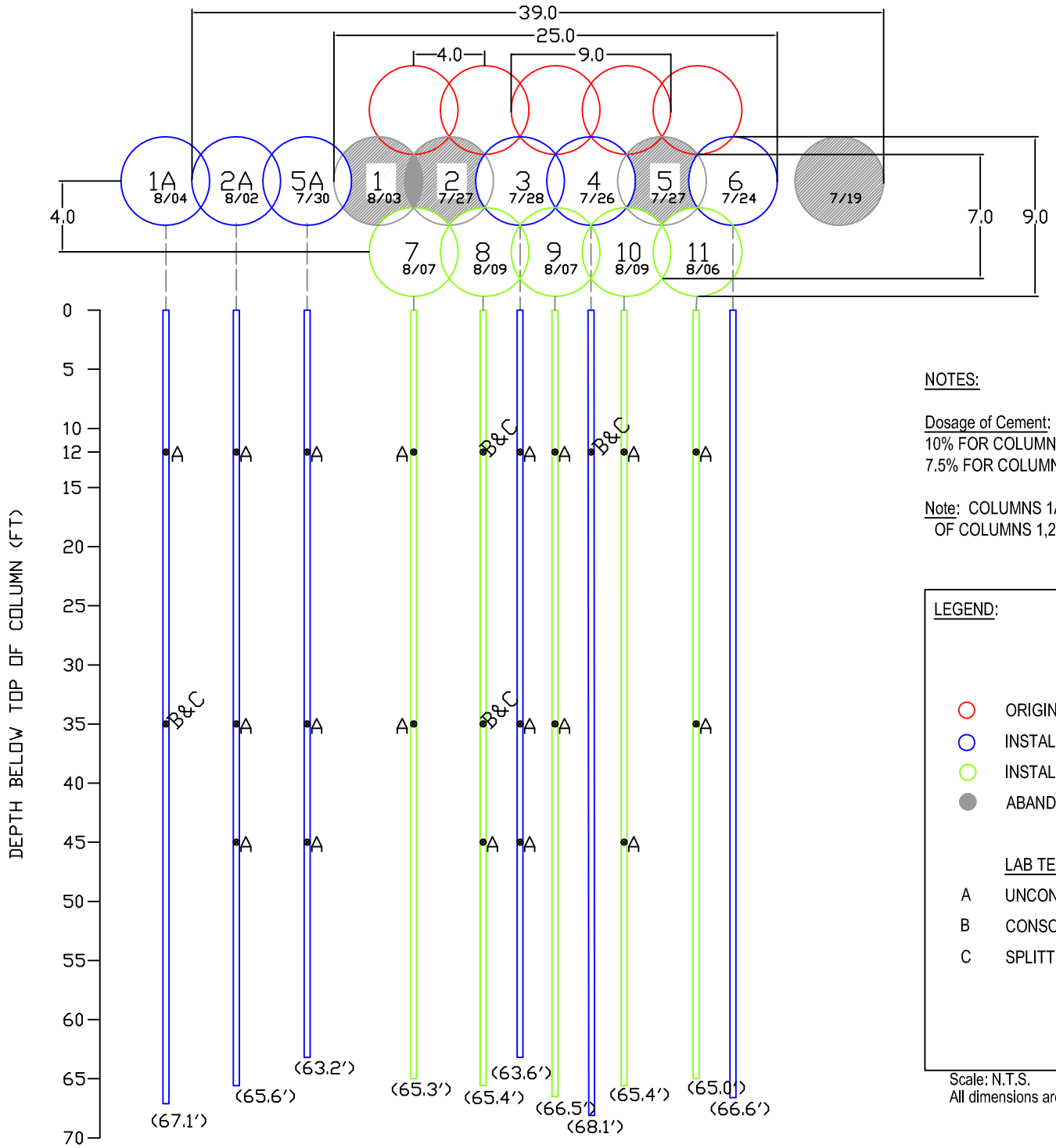
Strata	Depth (ft)	Sample Type and Cure	10% Mix Design		7.5% Mix Design
			Column 1A	Column 4	Column 8
Ash	12	Wet Grab 28-day		9.0 10.5	9.0 9.1
Ash	15	Core 56-day			83.0
Clay	35	Wet Grab 28-day	6.5 6.9		5.6 5.9
Silty Sand	55	Core 56-day			13.7


Averages of Strengths and Ages for Wet Grab Samples
Unconfined Compressive Strength (psi)
Age (days)
0
25
50
75
100
125
150
175
200
225
250
0
7
14
21
28
35
42
49
56
63
70
77
84
91
(material, cement dose)
ash, 7.5%
clay, 7.5%
sand, 7.5%
all materials, 7.5%
ash, 10%
clay, 10%
sand, 10%
all materials, 10%
all wet grab tests

Appendix B

Deep Mixing Pilot Field Test: Laboratory Data on Wet Grab Samples

DSM LAYOUT AND WET GRAB SAMPLING



NOTES:

Dosage of Cement:
10% FOR COLUMNS 1A,2A,5A,3,4&6
7.5% FOR COLUMNS 7,8,9,10&11

Note: COLUMNS 1A,2A&5A WERE INSTALLED IN PLACE OF COLUMNS 1,2&5.

LEGEND:

- ORIGINAL PROPOSED LOCATION OF COLUMNS 7 TO 11
- INSTALLED DSM COLUMN (ROW 1)
- INSTALLED DSM COLUMN (ROW 2)
- ABANDONED COLUMNS

LAB TESTS

- A UNCONFINED COMPRESSIVE STRENGTH (Cu)
- B CONSOLIDATED UNDRAINED (CU) TRIAXIAL TEST
- C SPLITTING TENSILE STRENGTH

Scale: N.T.S.
All dimensions are in feet unless noted otherwise.

Figure No.
1.0

MACTEC

DEEP SOIL MIXING
UNCONFINED COMPRESSIVE STRENGTH
Wet Grab Samples

Report Date: 10/29/2010
Project Name: TVA KIF – Deep Soil Mixing
Project Number: 3043101040
Sample Location: Column # 1A
Sample Cast Date: 8/4/2010

Sample ID.	Date Tested	Sample Age (Days)	Average Diameter of Sample (in)	Sample Height (in)	Sample Area (in^2)	Moisture at Time of Test (%)	Maximum Load (lbs)	Compressive Strength (psi)
1A-A-1	8/11/2010	7	2.990	5.60	7.022	42.4	225	30
1A-A-2	8/18/2010	14	2.995	5.10	7.045	43.1	370	55
1A-A-3	9/1/2010	28	3.005	5.30	7.092	42.7	460	65
1A-A-4	10/27/2010	84	3.015	5.20	7.139		1340	190

Notes:

1- All tests were performed in accordance with ASTM D1633
2- Moisture values not given above indicate they are not ready yet

Tested By: RCR 10/27/2010
Checked By: HAB 10/29/2010 HAB

MACTEC

DEEP SOIL MIXING
UNCONFINED COMPRESSIVE STRENGTH
Wet Grab Samples

Report Date: 9/28/2010
Project Name: TVA KIF – Deep Soil Mixing
Project Number: 3043101040
Sample Location: Column # 2A
Sample Cast Date: 8/2/2010

Sample ID.	Date Tested	Sample Age (Days)	Average Diameter of Sample (in)	Sample Height (in)	Sample Area (in^2)	Moisture at Time of Test (%)	Maximum Load (lbs)	Compressive Strength (psi)
2A-A-1	8/9/2010	7	2.990	5.90	7.022	35.2	465	65
2A-A-2	8/16/2010	14	3.000	5.90	7.069	33.9	340	50
2A-A-3	8/30/2010	28	3.000	5.90	7.069	34.6	535	75
2A-A-4	9/27/2010	56	3.010	5.90	7.116		645	90
2A-A-5	8/9/2010	7	2.990	5.90	7.022	31.7	265	40
2A-A-6	8/16/2010	14	3.000	6.00	7.069	32.4	295	40
2A-A-7	8/30/2010	28	2.995	5.90	7.045	31.8	355	50
2A-A-8	9/27/2010	56	2.995	5.90	7.045		375	55
2A-A-9	8/9/2010	7	2.990	4.95	7.022	34.9	370	55
2A-A-10	8/16/2010	14	3.010	5.80	7.116	34.9	490	70
2A-A-11	8/30/2010	28	3.000	5.90	7.069	33.1	560	80
2A-A-12	9/27/2010	56	3.005	5.85	7.0922		890	125

Notes:

1- All tests were performed in accordance with ASTM D1633
2- Moisture values not given above indicate they are not ready yet

Tested By: RCR 9/27/2010
Checked By: HAB 9/28/2010 HAB

MACTEC

DEEP SOIL MIXING
UNCONFINED COMPRESSIVE STRENGTH
Wet Grab Samples

Report Date: 9/28/2010
Project Name: TVA KIF – Deep Soil Mixing
Project Number: 3043101040
Sample Location: Column # 5A
Sample Cast Date: 7/30/2010

Sample ID.	Date Tested	Sample Age (Days)	Average Diameter of Sample (in)	Sample Height (in)	Sample Area (in^2)	Moisture at Time of Test (%)	Maximum Load (lbs)	Compressive Strength (psi)
5A-A-1	8/6/2010	7	3.000	5.90	7.069	29.6	275	40
5A-A-2	8/13/2010	14	3.005	4.90	7.092	29.8	340	50
5A-A-3	8/27/2010	28	2.985	5.90	6.998	29.9	445	65
5A-A-4	9/27/2010	59	3.005	5.95	7.092		415	60
5A-A-5	8/6/2010	7	3.015	5.95	7.139	32.7	505	70
5A-A-6	8/13/2010	14	2.995	6.00	7.045	32.4	645	90
5A-A-7	8/27/2010	28	3.000	5.90	7.069	32.3	865	120
5A-A-8	9/27/2010	59	3.015	5.95	7.139		1195	165
5A-A-9	8/6/2010	7	3.000	5.80	7.069	32.0	420	60
5A-A-10	8/13/2010	14	3.015	5.95	7.139	31.8	565	80
5A-A-11	8/27/2010	28	3.000	5.50	7.069	32.9	785	110
5A-A-12	9/27/2010	59	3.015	5.95	7.139		1050	145

Notes:

1- All tests were performed in accordance with ASTM D1633
2- Moisture values not given above indicate they are not ready yet

Tested By: RCR 9/27/2010
Checked By: HAB 9/28/2010 HAB

MACTEC

DEEP SOIL MIXING
UNCONFINED COMPRESSIVE STRENGTH
Wet Grab Samples

Report Date: 9/23/2010
Project Name: TVA KIF – Deep Soil Mixing
Project Number: 3043101040
Sample Location: Column # 3
Sample Cast Date: 7/28/2010

Sample ID.	Date Tested	Sample Age (Days)	Average Diameter of Sample (in)	Sample Height (in)	Sample Area (in^2)	Moisture at Time of Test (%)	Maximum Load (lbs)	Compressive Strength (psi)
3-A-1	8/4/2010	7	3.000	5.90	7.069	37.8	200	30
3-A-2	8/11/2010	14	2.995	5.90	7.045	40.1	350	50
3-A-3	8/25/2010	28	3.000	5.90	7.069	37.9	310	45
3-A-4	9/22/2010	56	3.000	5.90	7.069		720	100
3-A-5	8/4/2010	7	3.005	5.90	7.092	39.8	245	35
3-A-6	8/11/2010	14	3.000	5.90	7.069	39.3	430	60
3-A-7	8/25/2010	28	3.010	5.90	7.116	38.9	575	80
3-A-8	9/22/2010	56	3.015	5.90	7.139		1420	200
3-A-9	8/4/2010	7	3.005	5.90	7.092	37.5	305	45
3-A-10	8/11/2010	14	3.000	5.85	7.069	37.5	455	65
3-A-11	8/25/2010	28	3.005	5.90	7.092	37.7	600	85
3-A-12	9/22/2010	56	3.005	5.90	7.092		1350	190

Notes:

1- All tests were performed in accordance with ASTM D1633
2- Moisture values not given above indicate they are not ready yet

Tested By: RCR 9/22/2010
Checked By: HAB 9/23/2010 HAB

MACTEC

DEEP SOIL MIXING
UNCONFINED COMPRESSIVE STRENGTH
Wet Grab Samples

Report Date: 10/5/2010
Project Name: TVA KIF – Deep Soil Mixing
Sample Location: Column # 7
Project Number: 3043101040
Sample Cast Date: 8/7/2010

Sample ID.	Date Tested	Sample Age (Days)	Average Diameter of Sample (in)	Sample Height (in)	Sample Area (in^2)	Moisture at Time of Test (%)	Maximum Load (lbs)	Compressive Strength (psi)
7-A-1	8/14/2010	7	3.010	5.90	7.116	38.3	535	75
7-A-2	8/21/2010	14	3.010	5.90	7.116	37.8	655	90
7-A-3	9/4/2010	28	3.015	5.90	7.139	37.3	1140	160
7-A-4		spare						
7-A-5	8/14/2010	7	3.000	5.90	7.069	36.7	345	50
7-A-6	8/21/2010	14	3.010	5.95	7.116	37.1	420	60
7-A-7	9/4/2010	28	3.000	5.95	7.069	37.0	475	65
7-A-8	10/2/2010	56	3.01	5.95	7.116	36.7	755	105

Notes:

1- All tests were performed in accordance with ASTM D1633

Tested By: RCR 10/02/2010
Checked By: HAB 10/05/2010 HAB

MACTEC

DEEP SOIL MIXING
UNCONFINED COMPRESSIVE STRENGTH
Wet Grab Samples

Report Date: 10/6/2010
Project Name: TVA KIF – Deep Soil Mixing
Sample Location: Column # 8
Project Number: 3043101040
Sample Cast Date: 8/9/2010

Sample ID.	Date Tested	Sample Age (Days)	Average Diameter of Sample (in)	Sample Height (in)	Sample Area (in^2)	Moisture at Time of Test (%)	Maximum Load (lbs)	Compressive Strength (psi)
8-A-9	8/16/2010	7	2.995	5.90	7.045	31.2	330	45
8-A-10	8/23/2010	14	2.995	5.90	7.045	33.3	490	70
8-A-11	9/6/2010	28	3.000	5.90	7.069	33.1	650	90
8-A-12	10/5/2010	57	3.000	5.90	7.069		895	125

Notes:

1- All tests were performed in accordance with ASTM D1633
2- Moisture values not given above indicate they are not ready yet

Tested By: RCR 10/05/2010
Checked By: HAB 10/06/2010 HAB

MACTEC

DEEP SOIL MIXING
UNCONFINED COMPRESSIVE STRENGTH
Wet Grab Samples

Report Date: 11/1/2010
Project Name: TVA KIF – Deep Soil Mixing
Project Number: 3043101040
Sample Location: Column # 9
Sample Cast Date: 8/7/2010

Sample ID.	Date Tested	Sample Age (Days)	Average Diameter of Sample (in)	Sample Height (in)	Sample Area (in^2)	Moisture at Time of Test (%)	Maximum Load (lbs)	Compressive Strength (psi)
9-A-1	8/14/2010	7	3.015	5.95	7.139	39.5	485	70
9-A-2	8/21/2010	14	3.010	5.95	7.116	39.3	595	85
9-A-3	9/4/2010	28	3.015	5.95	7.139	39.2	965	135
9-A-4	10/30/2010	84	3.015	5.95	7.139		1560	220
9-A-5	8/14/2010	7	3.010	5.95	7.116	41.4	430	60
9-A-6	8/21/2010	14	3.010	5.95	7.116	39.5	500	70
9-A-7	9/4/2010	28	3.010	5.95	7.116	38.9	625	90
9-A-8	10/30/2010	84	3.015	5.90	7.139		1610	225

Notes:

1- All tests were performed in accordance with ASTM D1633
2- Moisture values not given above indicate they are not ready yet

Tested By: RCR 10/30/2010
Checked By: HAB 11/01/2010 HAB

MACTEC

DEEP SOIL MIXING
UNCONFINED COMPRESSIVE STRENGTH
Wet Grab Samples

Report Date: 11/3/2010
Project Name: TVA KIF – Deep Soil Mixing
Project Number: 3043101040
Sample Location: Column # 10
Sample Cast Date: 8/9/2010

Sample ID.	Date Tested	Sample Age (Days)	Average Diameter of Sample (in)	Sample Height (in)	Sample Area (in^2)	Moisture at Time of Test (%)	Maximum Load (lbs)	Compressive Strength (psi)
10-A-1	8/16/2010	7	3.000	5.90	7.069	36.2	340	50
10-A-2	8/23/2010	14	2.995	5.30	7.045	36.8	335	50
10-A-3	9/6/2010	28	2.995	5.90	7.045	37.0	535	75
10-A-4	11/1/2010	84	3.005	5.90	7.092		1265	180
10-A-5	8/16/2010	7	2.990	5.90	7.022	36.3	320	45
10-A-6	8/23/2010	14	2.995	5.90	7.045	36.8	505	70
10-A-7	9/6/2010	28	3.000	5.85	7.069	36.9	625	90
10-A-8	11/1/2010	84	3.000	5.50	7.069		1330	190

Notes:

1- All tests were performed in accordance with ASTM D1633
2- Moisture values not given above indicate they are not ready yet

Tested By: RCR 11/01/2010
Checked By: HAB 11/03/2010 HAB

MACTEC

DEEP SOIL MIXING
UNCONFINED COMPRESSIVE STRENGTH
Wet Grab Samples

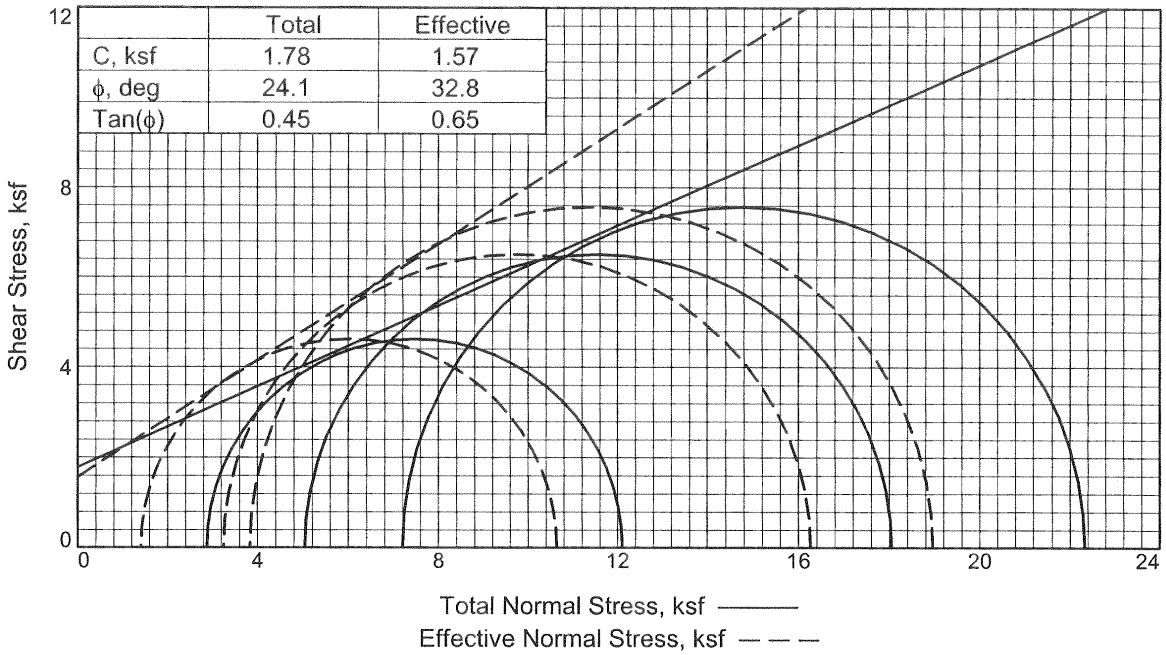
Report Date: 10/5/2010
Project Name: TVA KIF – Deep Soil Mixing
Project Number: 3043101040
Sample Location: Column # 11
Sample Cast Date: 8/6/2010

Sample ID.	Date Tested	Sample Age (Days)	Average Diameter of Sample (in)	Sample Height (in)	Sample Area (in^2)	Moisture at Time of Test (%)	Maximum Load (lbs)	Compressive Strength (psi)
11-A-1	8/13/2010	7	3.005	5.50	7.092	34.0	395	55
11-A-2	8/20/2010	14	2.990	5.90	7.022	33.3	370	55
11-A-3	9/3/2010	28	3.010	6.00	7.116	32.5	665	95
11-A-4	10/1/2010	56	3.010	5.95	7.116	33.9	785	110
11-A-5	8/13/2010	7	3.000	5.90	7.069	34.3	355	50
11-A-6	8/20/2010	14	3.005	5.90	7.092	36.0	335	45
11-A-7	9/3/2010	28	3.005	5.50	7.092	35.3	380	55
11-A-8	10/1/2010	56	3.010	5.95	7.116	35.8	480	65

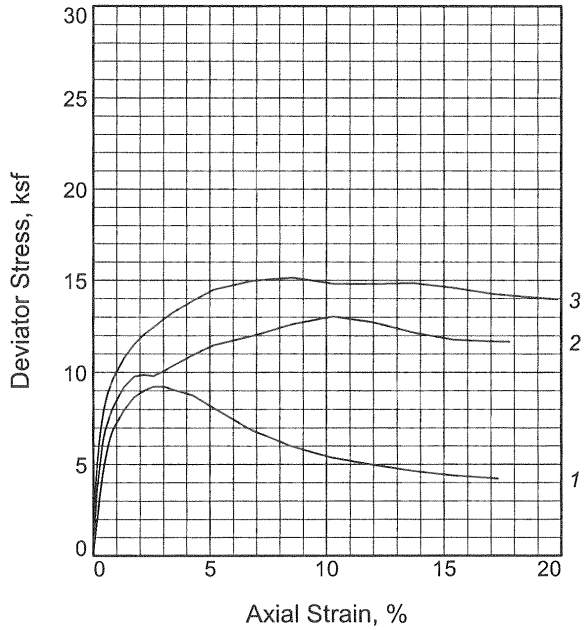
Notes:

1- All tests were performed in accordance with ASTM D1633

Tested By: RCR 10/01/2010
Checked By: HAB 10/05/2010 HAB



	Total	Effective
C, ksf	1.78	1.57
ϕ, deg	24.1	32.8
Tan(ϕ)	0.45	0.65



Sample No.		1	2	3
Initial	Water Content, %	38.7	37.1	36.5
	Dry Density, pcf	77.9	78.9	79.9
	Saturation, %	99.6	98.2	99.3
	Void Ratio	0.9380	0.9127	0.8881
	Diameter, in.	3.01	3.01	3.00
	Height, in.	5.94	5.91	5.92
At Test	Water Content, %	37.4	36.3	34.9
	Dry Density, pcf	79.3	80.4	81.8
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.9039	0.8779	0.8443
	Diameter, in.	2.99	2.99	2.98
	Height, in.	5.90	5.87	5.87
Strain rate, in./min.		0.03	0.03	0.03
Back Pressure, psi		40.00	40.00	40.00
Cell Pressure, psi		60.00	75.00	90.00
Fail. Stress, ksf		9.2	13.0	15.1
Total Pore Pr., ksf		7.2	7.5	9.1
Ult. Stress, ksf				
Total Pore Pr., ksf				
$\bar{\sigma}_1$ Failure, ksf		10.6	16.3	19.0
$\bar{\sigma}_3$ Failure, ksf		1.4	3.3	3.8

Type of Test:
CU with Pore Pressures

Sample Type: Wet grab-field molded

Description: Dark gray cement stabilized soil

Specific Gravity= 2.418

Remarks: Date tested: 9-1-10

Client: Tennessee Valley Authority (TVA)

Project: TVA KIF DSM DEMO PROGRAM

Source of Sample: Column 1A **Depth:** 35'

Sample Number: 1A 5-7

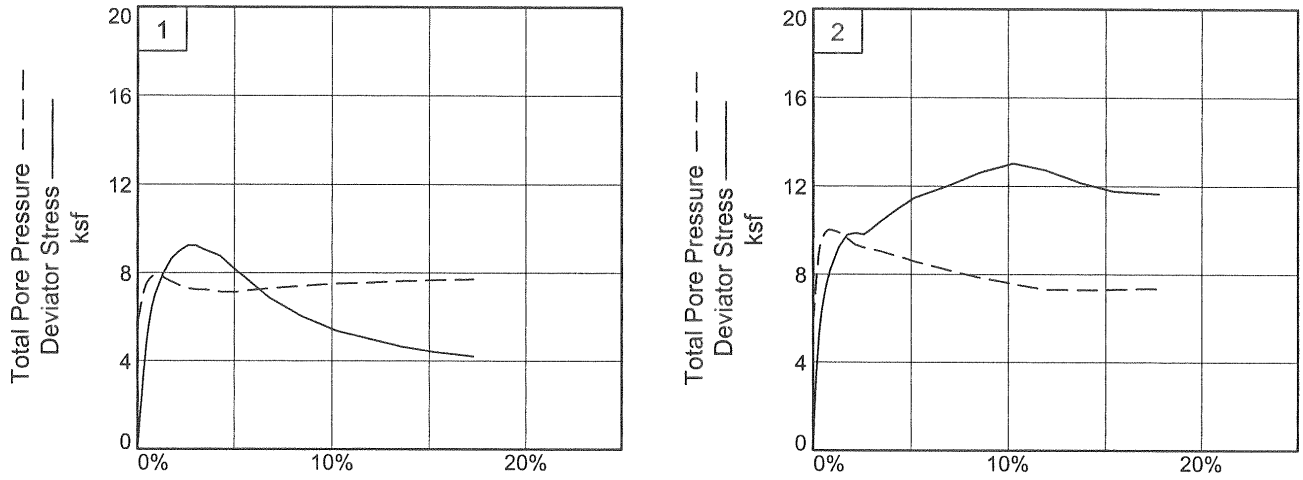
Proj. No.: 3043-10-1040 **Date Sampled:** 8-4-10

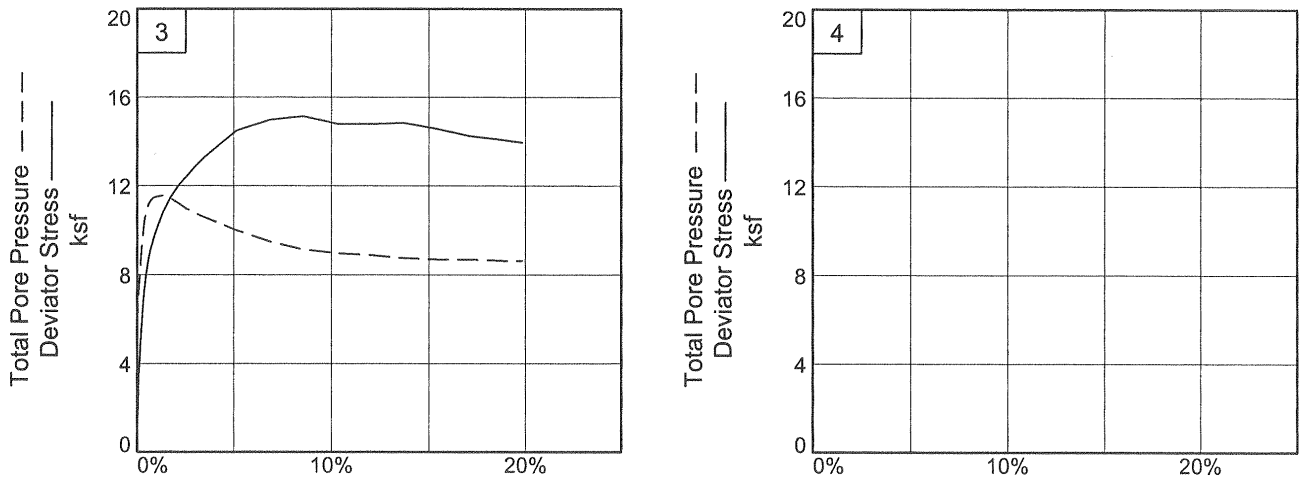
TRIAXIAL SHEAR TEST REPORT

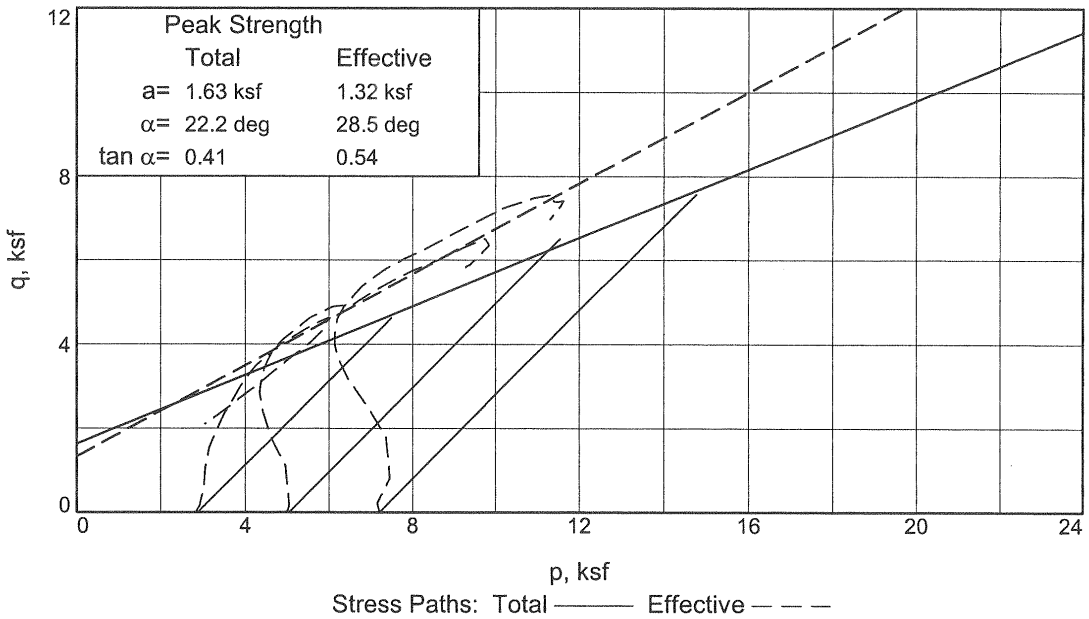
MACTEC ENGINEERING AND CONSULTING, INC.

Figure _____

Tested By: FB

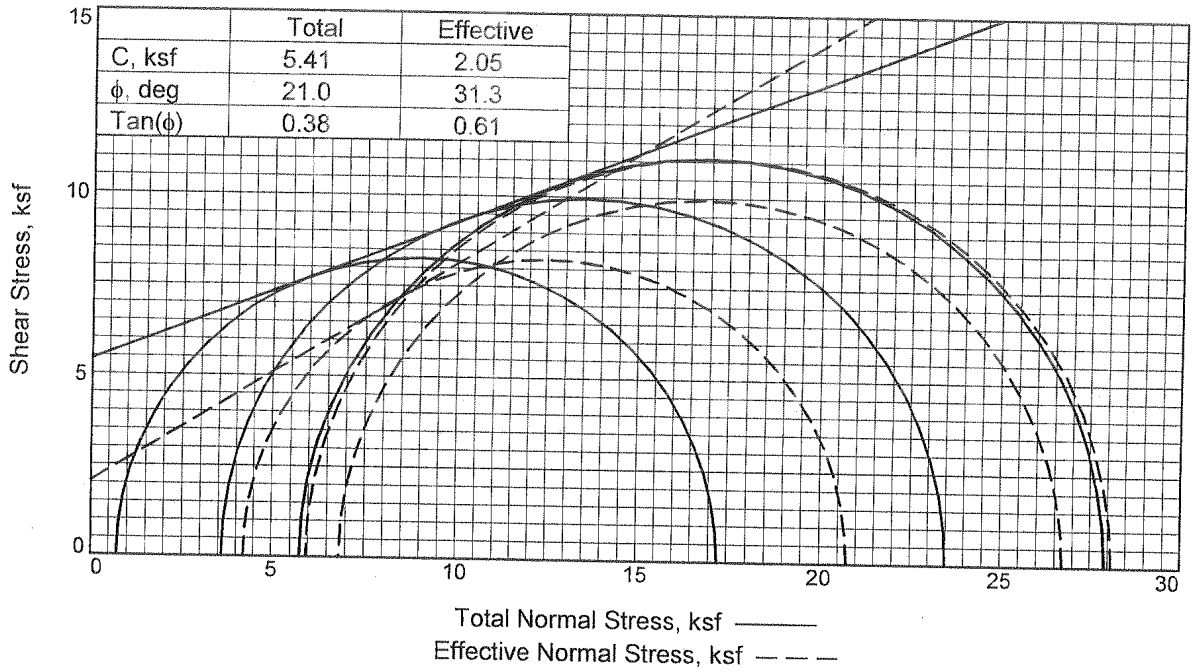




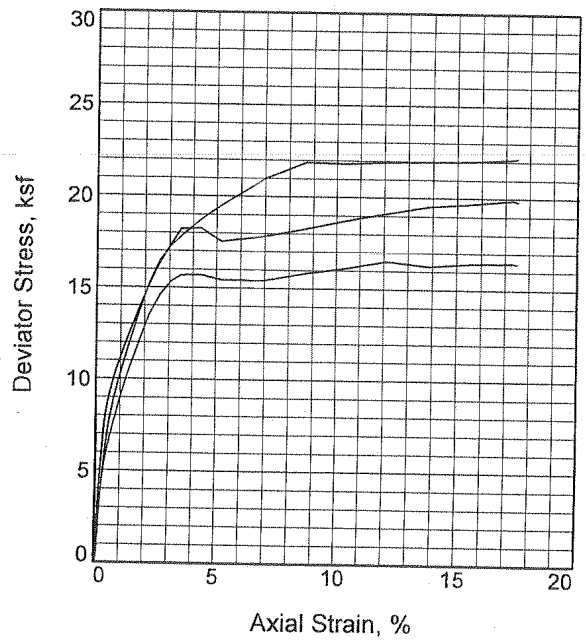


Client: Tennessee Valley Authority (TVA)
Project: TVA KIF DSM DEMO PROGRAM
Source of Sample: Column 1A **Depth:** 35' **Sample Number:** 1A 5-7
Project No.: 3043-10-1040 **Figure** _____ **MACTEC Engineering and Consulting, Inc.**

Tested By: FB



	Total	Effective
C, ksf	5.41	2.05
ϕ, deg	21.0	31.3
Tan(ϕ)	0.38	0.61



Sample No.		1	2	3
Initial	Water Content, %	30.2	28.3	28.5
	Dry Density, pcf	86.2	87.8	87.8
	Saturation, %	98.5	96.5	97.0
	Void Ratio	0.7333	0.7021	0.7028
	Diameter, in.	3.01	3.01	3.01
	Height, in.	5.81	5.84	5.81
At Test	Water Content, %	29.8	28.1	28.0
	Dry Density, pcf	87.2	89.3	89.5
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.7145	0.6733	0.6704
	Diameter, in.	3.00	3.00	2.99
	Height, in.	5.79	5.80	5.77
Strain rate, in./min.		0.01	0.01	0.01
Back Pressure, psi		40.00	40.00	40.00
Cell Pressure, psi		45.00	65.00	80.00
Fail. Stress, ksf		16.5	19.9	22.2
Total Pore Pr., ksf		2.3	2.5	5.6
Ult. Stress, ksf				
Total Pore Pr., ksf				
$\bar{\sigma}_1$ Failure, ksf		20.7	26.7	28.1
$\bar{\sigma}_3$ Failure, ksf		4.2	6.8	5.9

Type of Test:
CU with Pore Pressures
Sample Type: Wet grab-field molded
Description: Dark Gray Cement Stabilized Soil

Specific Gravity= 2.394
Remarks: Date Tested: 8-23-10
Initial moisture based on mass increase during saturation and final moisture.

JAX, FL. _____

Client: Tennessee Valley Authority (TVA)

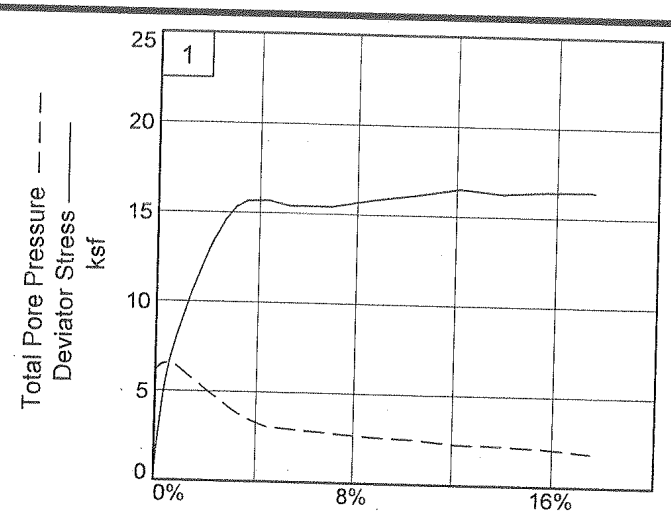
Project: TVA DSM DEMO PROGRAM

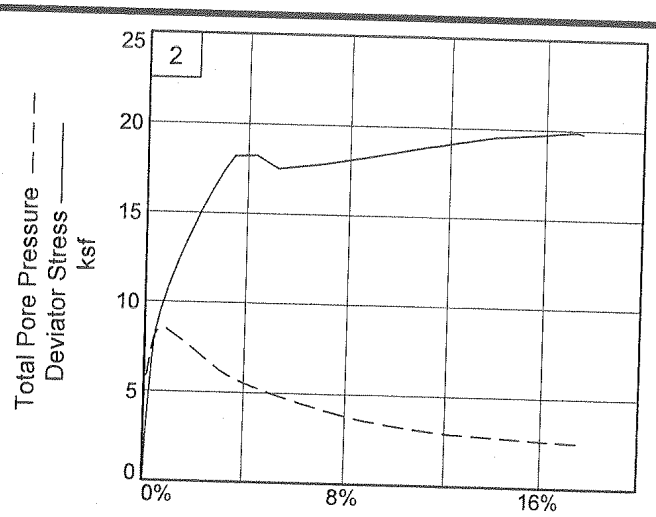
Source of Sample: Column #4 **Depth:** 12.0'
Sample Number: 4B 1-3
Proj. No.: 3043-10-1040 **Date Sampled:** 7-26-10

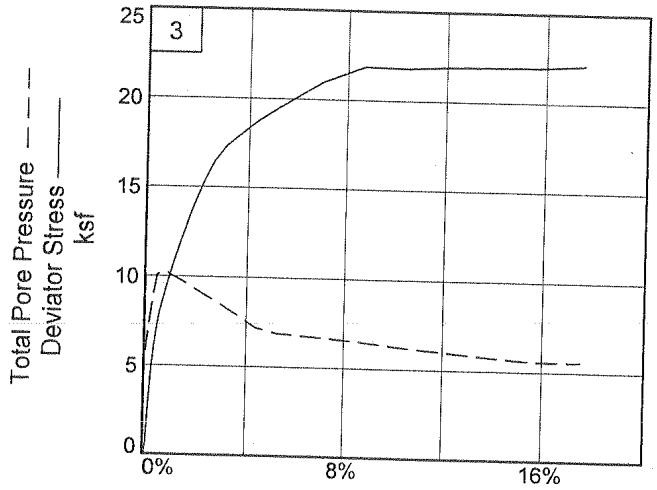
TRIAXIAL SHEAR TEST REPORT

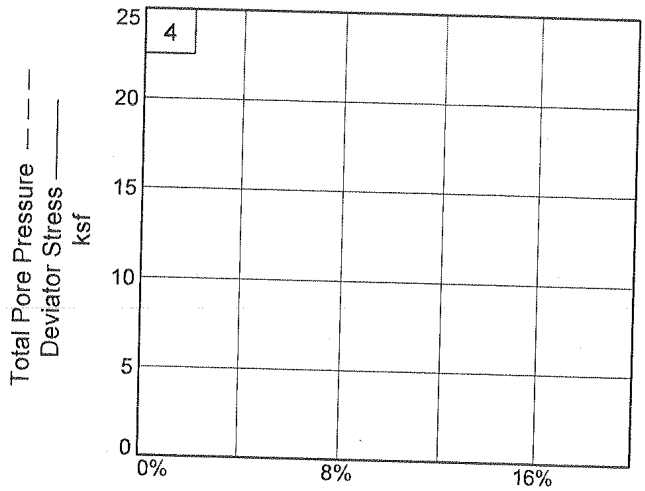
MACTEC ENGINEERING AND CONSULTING, INC.

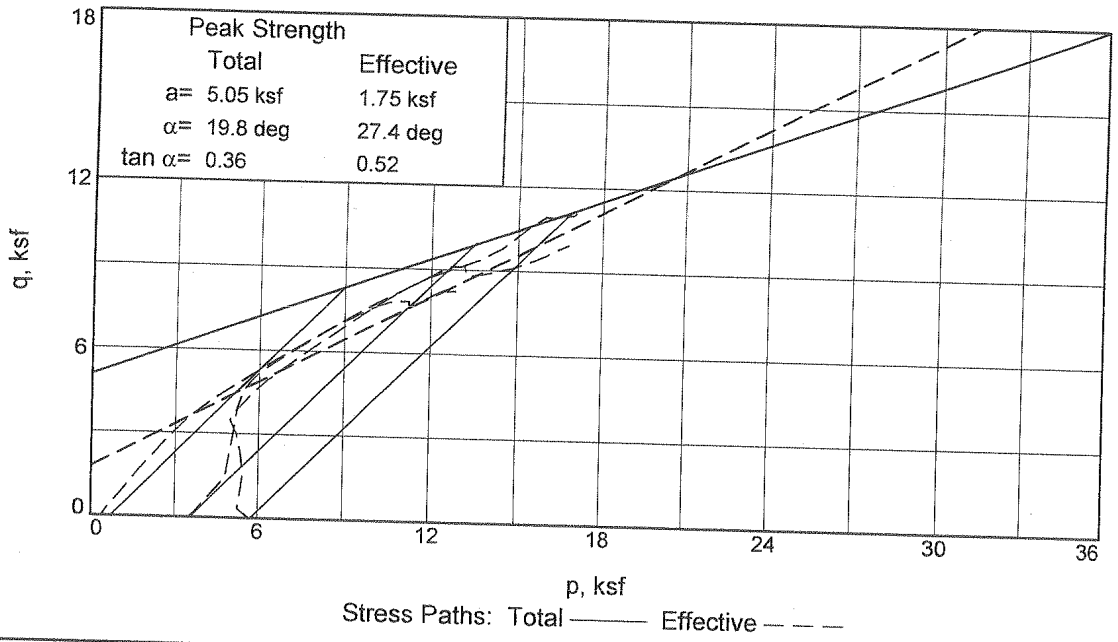
Tested By: MC/FB Rajni Sukhwani
RAJNI SUKHWANI







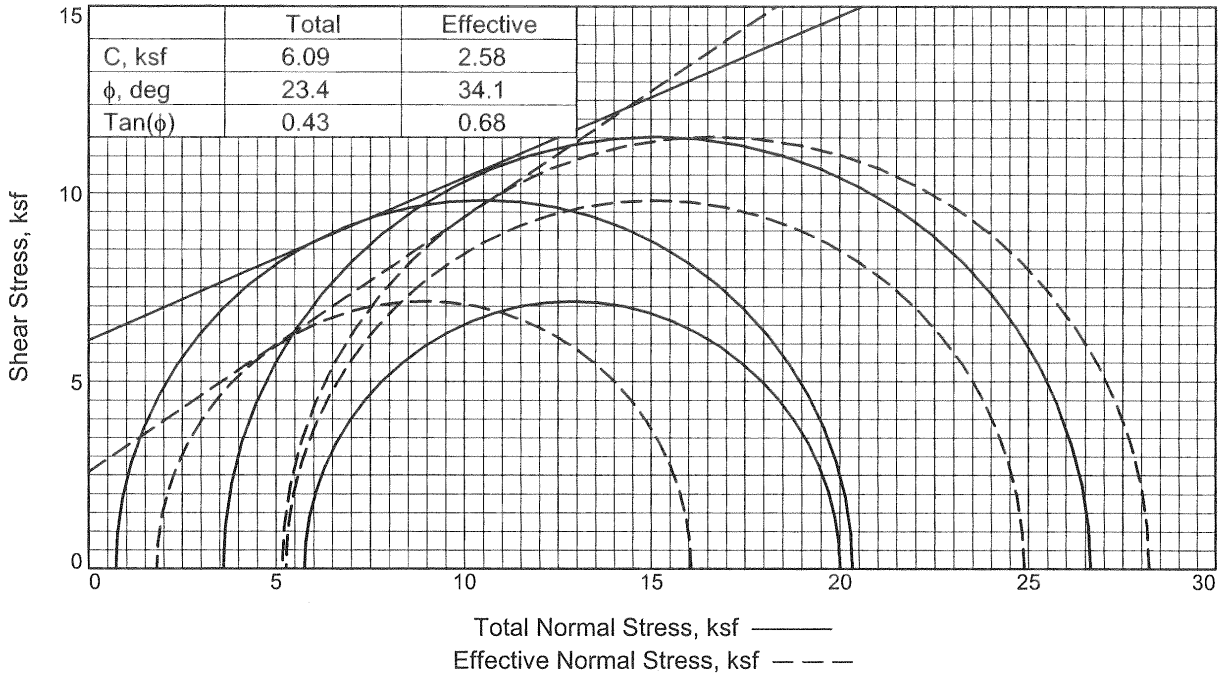




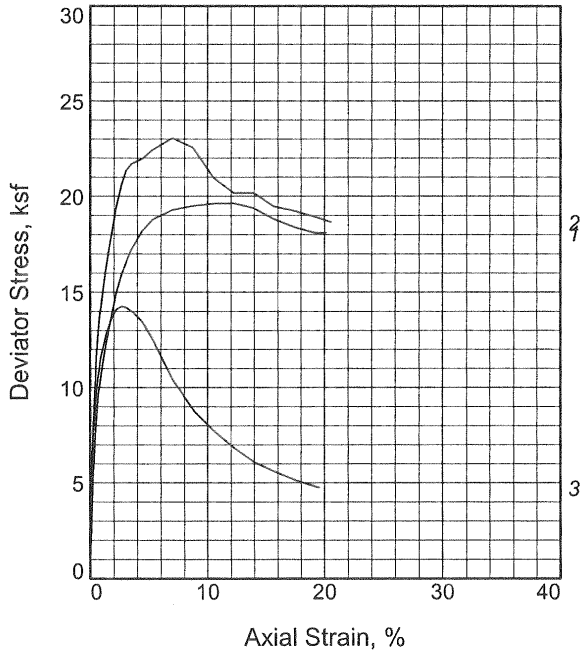
Stress Paths: Total ——— Effective – – –

Client: Tennessee Valley Authority (TVA)
Project: TVA DSM DEMO PROGRAM
Source of Sample: Column #4 **Depth:** 12.0' **Sample Number:** 4B 1-3
Project No.: 3043-10-1040 **JAX, FL.** _____ **MACTEC Engineering and Consulting, Inc.**

Tested By: MC/FB Rajni Sukhwani
RAJNI SUKHWANI



	Total	Effective
C, ksf	6.09	2.58
ϕ, deg	23.4	34.1
Tan(ϕ)	0.43	0.68



Sample No.		1	2	3
Initial	Water Content, %	34.5	31.7	32.4
	Dry Density, pcf	82.3	84.8	84.5
	Saturation, %	99.6	97.9	99.2
	Void Ratio	0.8387	0.7835	0.7907
	Diameter, in.	3.00	3.00	3.00
	Height, in.	5.75	5.78	5.78
At Test	Water Content, %	34.5	31.4	28.9
	Dry Density, pcf	82.4	85.9	88.9
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.8358	0.7623	0.7017
	Diameter, in.	3.00	2.99	2.95
	Height, in.	5.74	5.76	5.69
Strain rate, in./min.		0.03	0.03	0.01
Back Pressure, psi		40.00	40.00	40.00
Cell Pressure, psi		45.00	65.00	80.00
Fail. Stress, ksf		19.6	23.0	14.2
Total Pore Pr., ksf		1.2	4.2	9.7
Ult. Stress, ksf				
Total Pore Pr., ksf				
$\bar{\sigma}_1$ Failure, ksf		24.9	28.2	16.1
$\bar{\sigma}_3$ Failure, ksf		5.3	5.2	1.8

Type of Test:
CU with Pore Pressures

Sample Type: Wet grab-field molded

Description: Dark gray cement stabilized soil

Specific Gravity= 2.424

Remarks: Date tested: 9-7-10 Note: Sample No. 3 failed without explanation at approximately 50% of anticipated load. Calculations are based on results obtained from Samples No. 1 and No. 2

Figure _____

Client: Tennessee Valley Authority (TVA)

Project: TVA KIF DSM DEMO PROGRAM

Source of Sample: Column 8 **Depth:** 12'

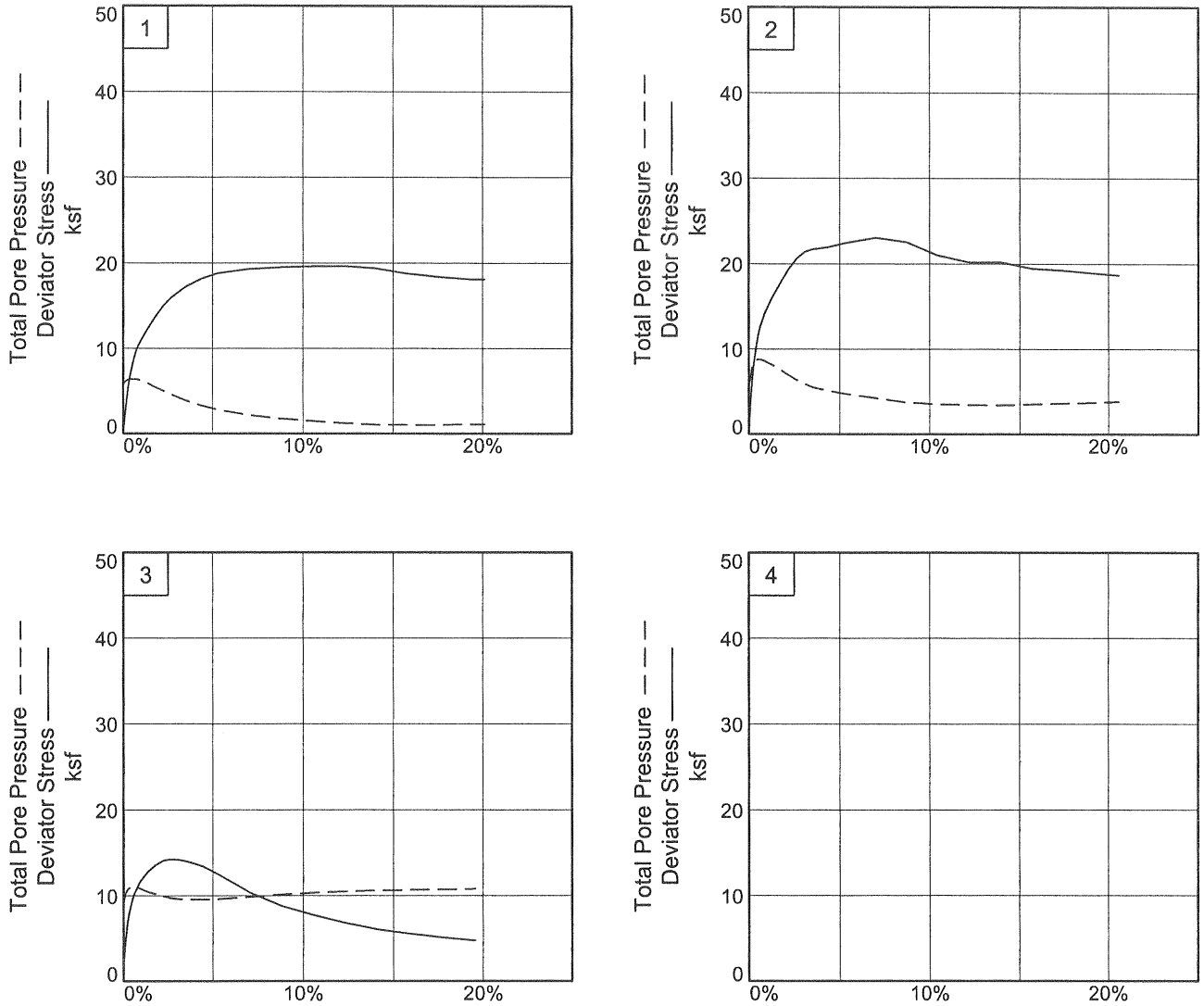
Sample Number: 8-B 1-3

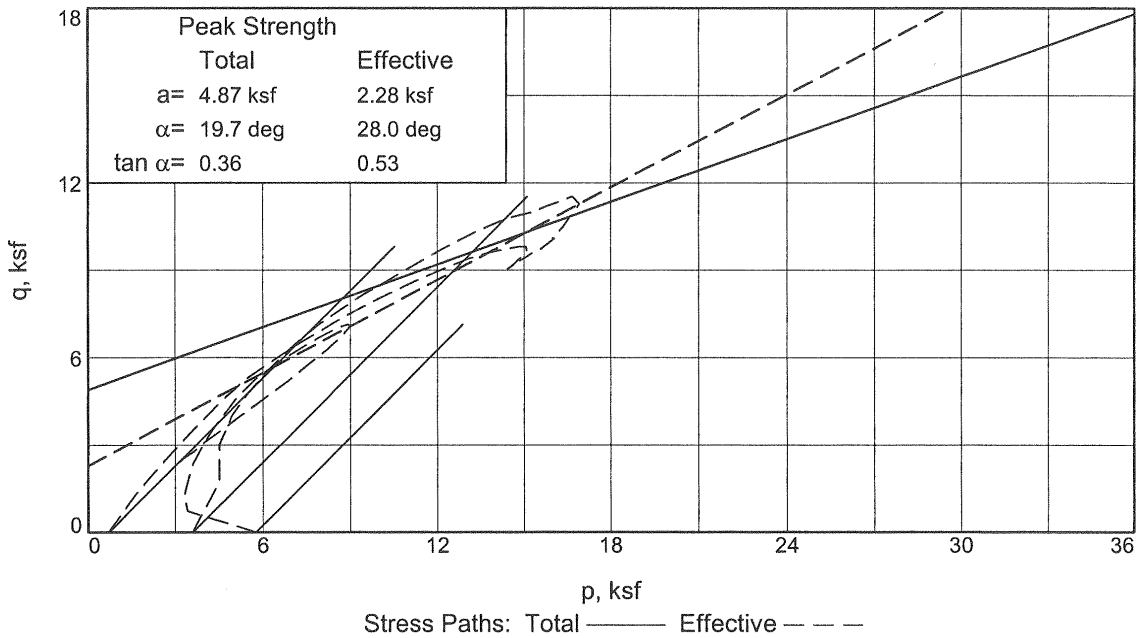
Proj. No.: 3043-10-1040 **Date Sampled:** 8-9-10

TRIAXIAL SHEAR TEST REPORT

MACTEC ENGINEERING AND CONSULTING, INC.

by MCL



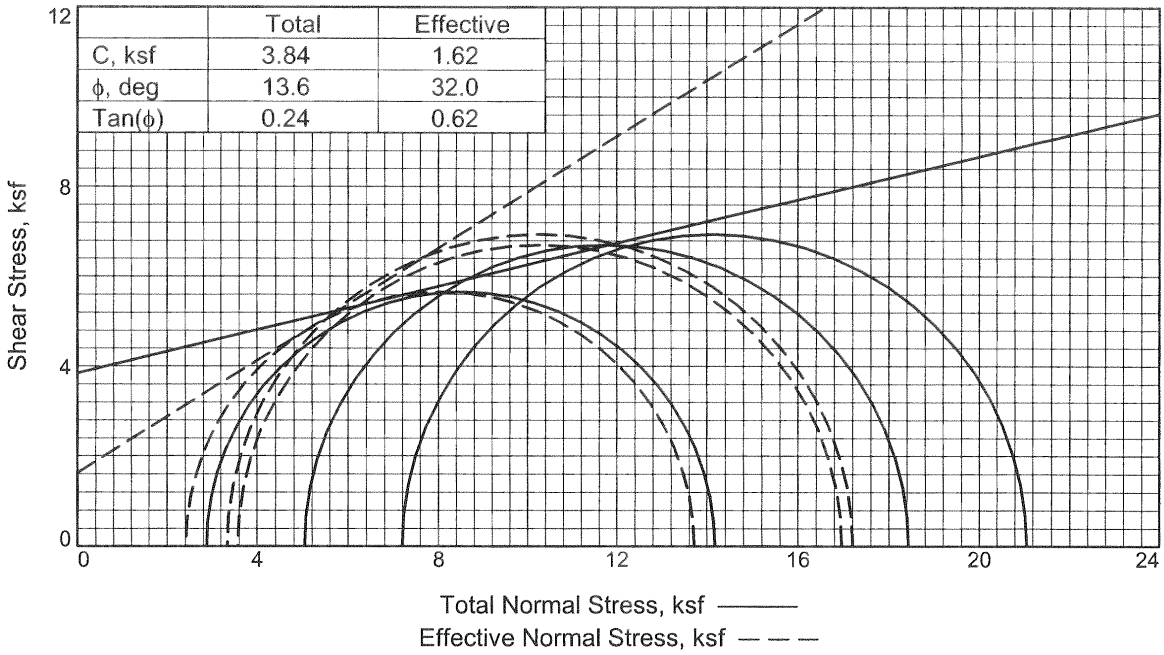


Client: Tennessee Valley Authority (TVA)

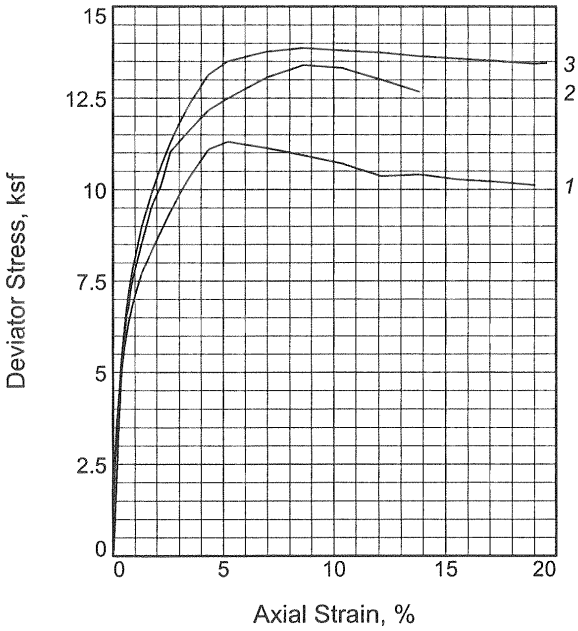
Project: TVA KIF DSM DEMO PROGRAM

Source of Sample: Column 8 **Depth:** 12' **Sample Number:** 8-B 1-3

Project No.: 3043-10-1040 **Figure** _____ **MACTEC Engineering and Consulting, Inc.**



	Total	Effective
C, ksf	3.84	1.62
ϕ, deg	13.6	32.0
Tan(ϕ)	0.24	0.62



Sample No.		1	2	3
Initial	Water Content, %	32.9	31.4	31.7
	Dry Density, pcf	83.3	85.2	84.7
	Saturation, %	98.6	99.0	98.5
	Void Ratio	0.8052	0.7657	0.7760
	Diameter, in.	3.01	3.00	3.00
	Height, in.	5.83	5.85	5.86
At Test	Water Content, %	32.3	30.3	30.5
	Dry Density, pcf	84.6	86.9	86.7
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.7784	0.7314	0.7362
	Diameter, in.	2.99	2.98	2.98
	Height, in.	5.80	5.81	5.82
Strain rate, in./min.		0.03	0.03	0.01
Back Pressure, psi		40.00	40.00	40.00
Cell Pressure, psi		60.00	75.00	90.00
Fail. Stress, ksf		11.3	13.4	13.9
Total Pore Pr., ksf		6.2	7.2	9.6
Ult. Stress, ksf				
Total Pore Pr., ksf				
$\bar{\sigma}_1$ Failure, ksf		13.7	17.0	17.2
$\bar{\sigma}_3$ Failure, ksf		2.4	3.6	3.3

Type of Test:
CU with Pore Pressures

Sample Type: Wet grab-field molded

Description: Dark gray cement stabilized soil

Specific Gravity= 2.410

Remarks: Date tested: 9-6-10 Note: Sample 2 terminated at 15% strain due to puncture in membrane

Figure _____

Client: Tennessee Valley Authority (TVA)

Project: TVA KIF DSM DEMO PROGRAM

Source of Sample: Column 8 **Depth:** 35.0'

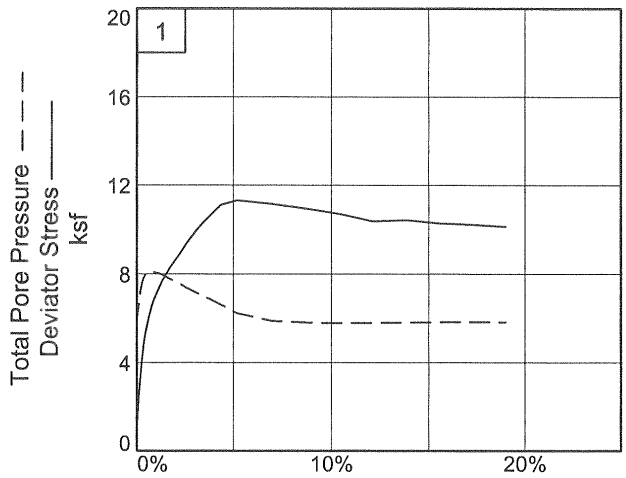
Sample Number: 8-B 5-7

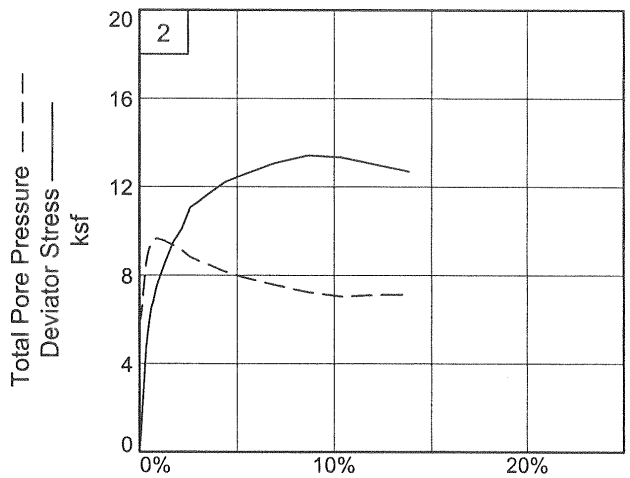
Proj. No.: 3043-10-1040 **Date Sampled:** 8-9-10

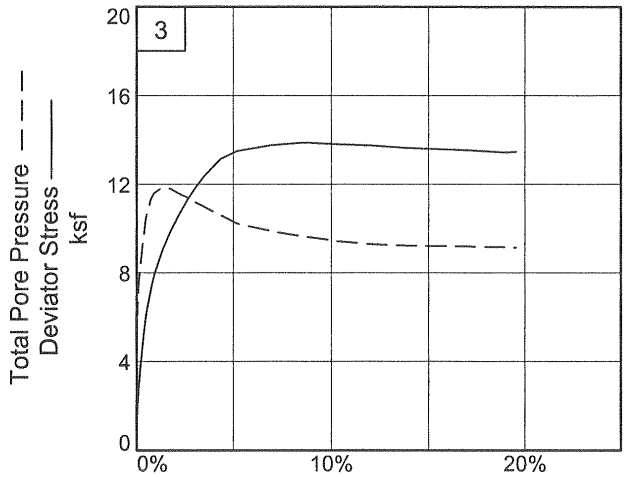
TRIAXIAL SHEAR TEST REPORT

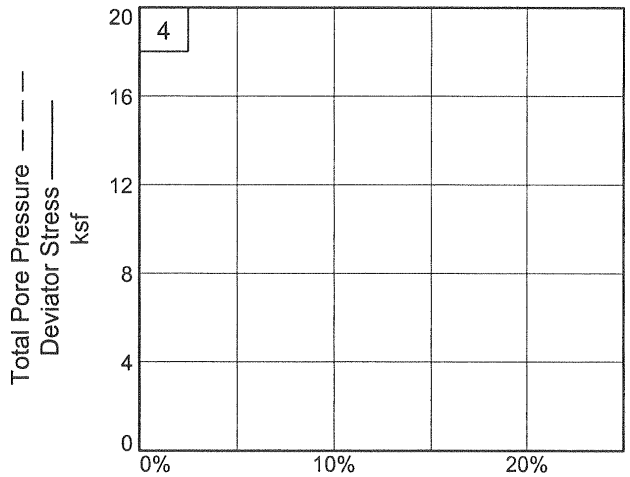
MACTEC ENGINEERING AND CONSULTING, INC.

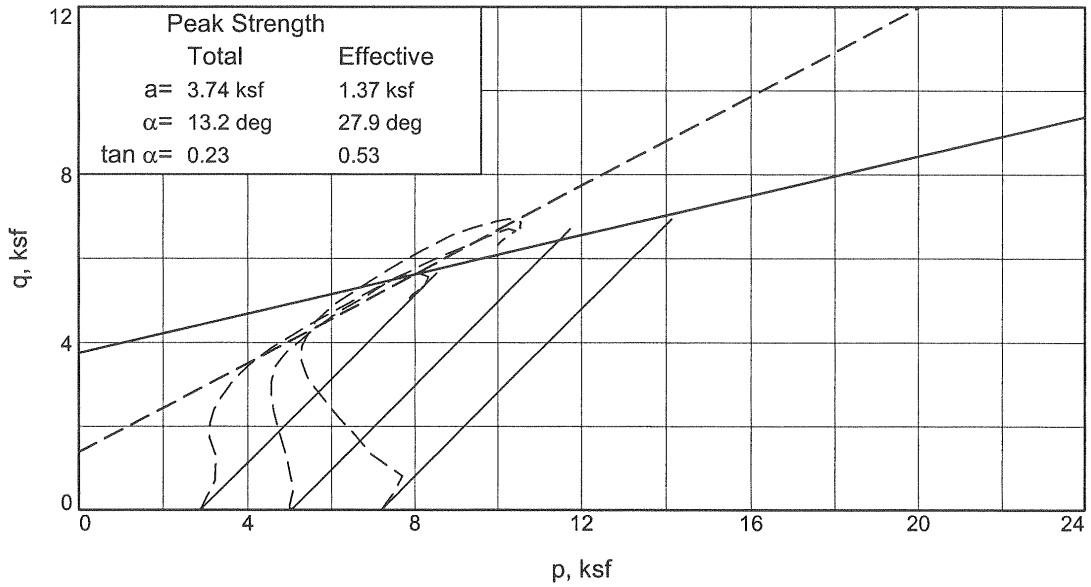
Tested By: FB











Client: Tennessee Valley Authority (TVA)
Project: TVA KIF DSM DEMO PROGRAM
Source of Sample: Column 8 **Depth:** 35.0' **Sample Number:** 8-B 5-7
Project No.: 3043-10-1040 **Figure** _____ **MACTEC Engineering and Consulting, Inc.**

Tested By: FB

MACTEC

REPORT OF SPLITTING TENSILE STRENGTH OF SOIL CEMENT SPECIMENS

CLIENT: Tennessee Valley Authority (TVA)

PROJECT NO.: 3043-10-1040

PROJECT: TVA KIF DSM Demo Program

SAMPLE LOCATION: Column #1A

DATE TESTED: 9/1/2010

DATE CAST: 8/4/2010

SPECIFIED MINIMUM COMPRESSIVE STRENGTH (psi) N/A

SAMPLE NO.	DATE TESTED	LENGTH (in.)	DIAMETER (in.)	ULTIMATE LOAD (lbs.)	L:D	SPLITTING TENSILE STRENGTH (psi)
1A-C-3	9/1/10	5.83	3.02	180	1.93	6.5
1A-C-4	9/1/10	5.83	3.01	190	1.94	6.9

GENERAL NOTES

Tested in accordance with ASTM C-496

RESPECTFULLY SUBMITTED

Kelly Marshall

MACTEC

REPORT OF SPLITTING TENSILE STRENGTH OF SOIL CEMENT SPECIMENS

CLIENT: Tennessee Valley Authority (TVA)

PROJECT: TVA DSM Demo Program
SAMPLE LOCATION: Column #4

PROJECT NO.: 3043-10-1040

DATE TESTED: 8/23/2010

SPECIFIED MINIMUM COMPRESSIVE STRENGTH (psi) N/A

SAMPLE NO.	DATE TESTED	LENGTH (in.)	DIAMETER (in.)	ULTIMATE LOAD (lbs.)	L:D	SPLITTING TENSILE STRENGTH (psi)
4-C-1	8/23/10	6.09	3.02	260	2.02	9.0
4-C-2	8/23/10	6.05	3.02	300	2.01	10.5

GENERAL NOTES

Tested in accordance with ASTM C-496

RESPECTFULLY SUBMITTED

Rajni Sukhwani

Rajni Sukhwani, E.I.

MACTEC

REPORT OF SPLITTING TENSILE STRENGTH OF SOIL CEMENT SPECIMENS

CLIENT: Tennessee Valley Authority (TVA)

PROJECT NO.: 3043-10-1040

PROJECT: TVA KIF DSM Demo Program

SAMPLE LOCATION: Column #8-Depth 12'

DATE TESTED: 9/3/2010

DATE CAST: 8/9/2010

SPECIFIED MINIMUM COMPRESSIVE STRENGTH (psi) N/A

SAMPLE NO.	DATE TESTED	LENGTH (in.)	DIAMETER (in.)	ULTIMATE LOAD (lbs.)	L:D	SPLITTING TENSILE STRENGTH (psi)
8-C-1	9/3/10	5.77	3.00	246	1.92	9.0
8-C-2	9/3/10	5.81	3.00	250	1.93	9.1

GENERAL NOTES

Tested in accordance with ASTM C-496

RESPECTFULLY SUBMITTED,

Kelly Marshall

MACTEC

REPORT OF SPLITTING TENSILE STRENGTH OF SOIL CEMENT SPECIMENS

CLIENT: Tennessee Valley Authority (TVA)

PROJECT NO.: 3043-10-1040

PROJECT: TVA KIF DSM Demo Program

SAMPLE LOCATION: Column #8-Depth 35'

DATE TESTED: 9/3/2010

DATE CAST: 8/9/2010

SPECIFIED MINIMUM COMPRESSIVE STRENGTH (psi) N/A

SAMPLE NO.	DATE TESTED	LENGTH (in.)	DIAMETER (in.)	ULTIMATE LOAD (lbs.)	L:D	SPLITTING TENSILE STRENGTH (psi)
8-C-3	9/3/10	5.81	3.01	155	1.93	5.6
8-C-4	9/3/10	5.81	3.01	162	1.93	5.9

GENERAL NOTES

Tested in accordance with ASTM C-496

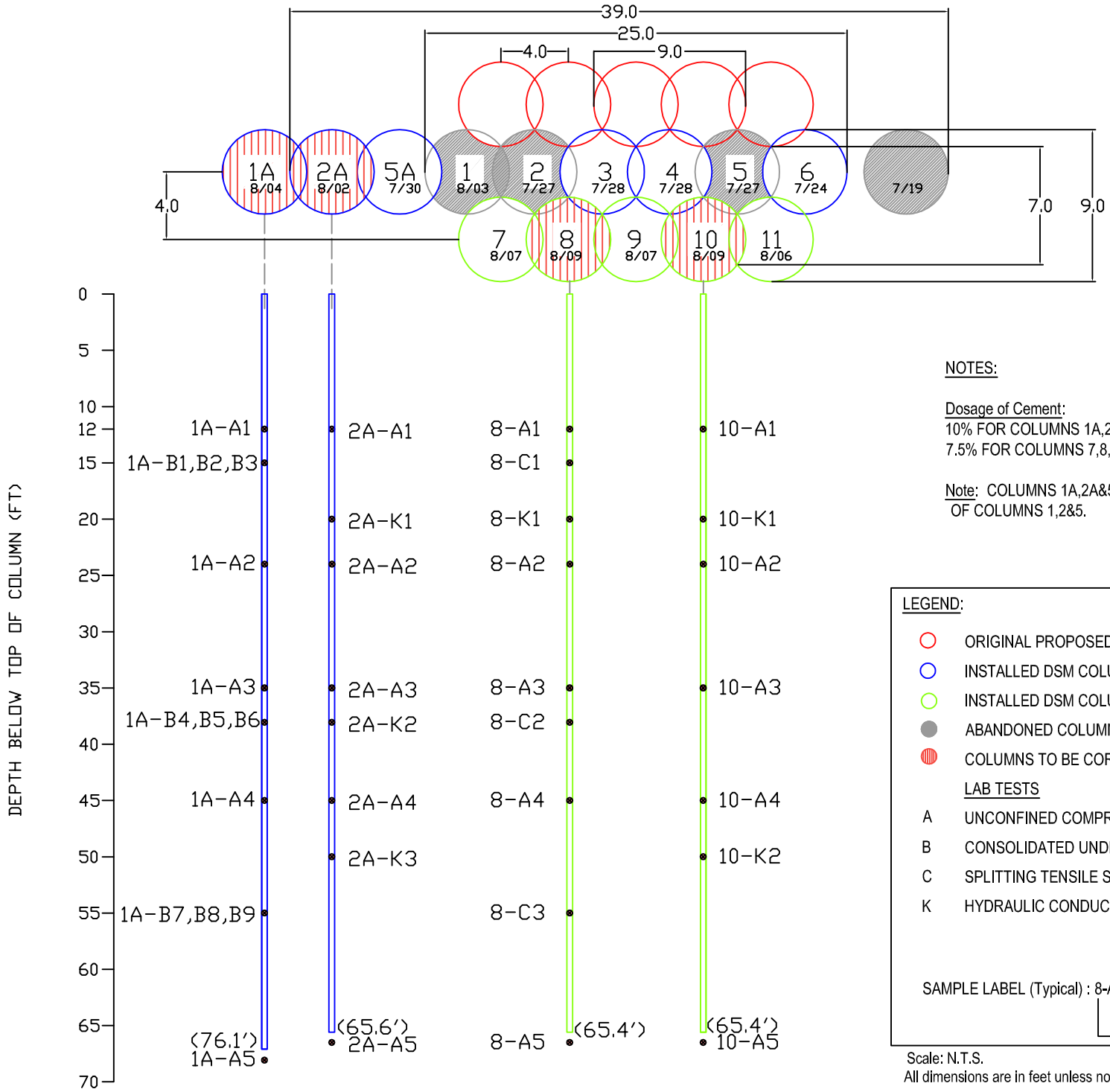
RESPECTFULLY SUBMITTED

Kelly Marshall

Appendix C

Deep Mixing Pilot Field Test: Laboratory Data on Cored Samples

DSM LAYOUT AND CORING SCHEDULE



NOTES:

Dosage of Cement:
10% FOR COLUMNS 1A,2A,5A,1,2,3,4,5&6
7.5% FOR COLUMNS 7,8,9,10&11

Note: COLUMNS 1A,2A&5A WERE INSTALLED IN PLACE OF COLUMNS 1,2&5.

LEGEND:

- ORIGINAL PROPOSED LOCATION OF COLUMNS 7 TO 11
- INSTALLED DSM COLUMN (ROW 1)
- INSTALLED DSM COLUMN (ROW 2)
- ABANDONED COLUMNS
- COLUMNS TO BE CORED

LAB TESTS

- A UNCONFINED COMPRESSIVE STRENGTH (Cu)
- B CONSOLIDATED UNDRAINED (CU) TRIAXIAL TEST
- C SPLITTING TENSILE STRENGTH
- K HYDRAULIC CONDUCTIVITY

SAMPLE LABEL (Typical) : 8-A3
- SAMPLE #
- LAB TEST
- COLUMN #

Scale: N.T.S.
All dimensions are in feet unless noted otherwise.

Figure No.
2.0

MACTEC

DEEP SOIL MIXING
UNCONFINED COMPRESSIVE STRENGTH
Core Samples

Project Name: TVA KIF – Deep Soil Mixing

Project Number: 3043101040

Report Date: 10/5/2010

Sampling Date: 9/17/2010

Column Number	Sample ID.	Depth (ft)	Cast Date	Test Date	Sample Age (Days)	Average Diameter of Sample (in)	Sample Height (in)	Sample Area (in^2)	Moisture at Time of Test (%)	Maximum Load (lbs)	Compressive Strength (psi)
1A	1A-A1	12.0	8/4/10	9/29/10	56	2.415	4.75	4.581	48.5	1895	415
1A	1A-A2	26.0	8/4/10	9/29/10	56	2.420	4.65	4.600	43.9	3250	705
1A	1A-A3*	36.5	8/4/10	-	-	-	-	-	-	-	-
1A	1A-A4*	46.5	8/4/10	-	-	-	-	-	-	-	-
1A	1A-A5	68.5	8/4/10	9/29/10	56	2.370	4.75	4.412	29.2	370	85
2A	2A-A1	12.0	8/2/10	9/29/10	58	2.420	4.80	4.600	37.4	1065	230
2A	2A-A2*	24.0	8/2/10	-	-	-	-	-	-	-	-
2A	2A-A3*	33.0	8/2/10	-	-	-	-	-	-	-	-
8	8-A1	12.0	8/9/10	9/29/10	51	2.370	3.70	4.412	41.4	1690	385**
8	8-A2*	24.0	8/9/10	-	-	-	-	-	-	-	-
8	8-A4*	45.0	8/9/10	-	-	-	-	-	-	-	-
8	8-A5	66.5	8/9/10	9/29/10	51	2.360	4.80	4.374	23.2	1125	255
10	10-A1	12.0	8/9/10	9/29/10	51	2.290	4.80	4.119	43.9	785	190
10	10-A2	24.0	8/9/10	9/29/10	51	2.315	4.80	4.209	37.8	2550	605
10	10-A3	33.5	8/9/10	9/29/10	51	2.350	3.85	4.337	45.8	2065	475**
10	10-A4	42.5	8/9/10	9/29/10	51	2.320	4.20	4.227	36.5	810	190**

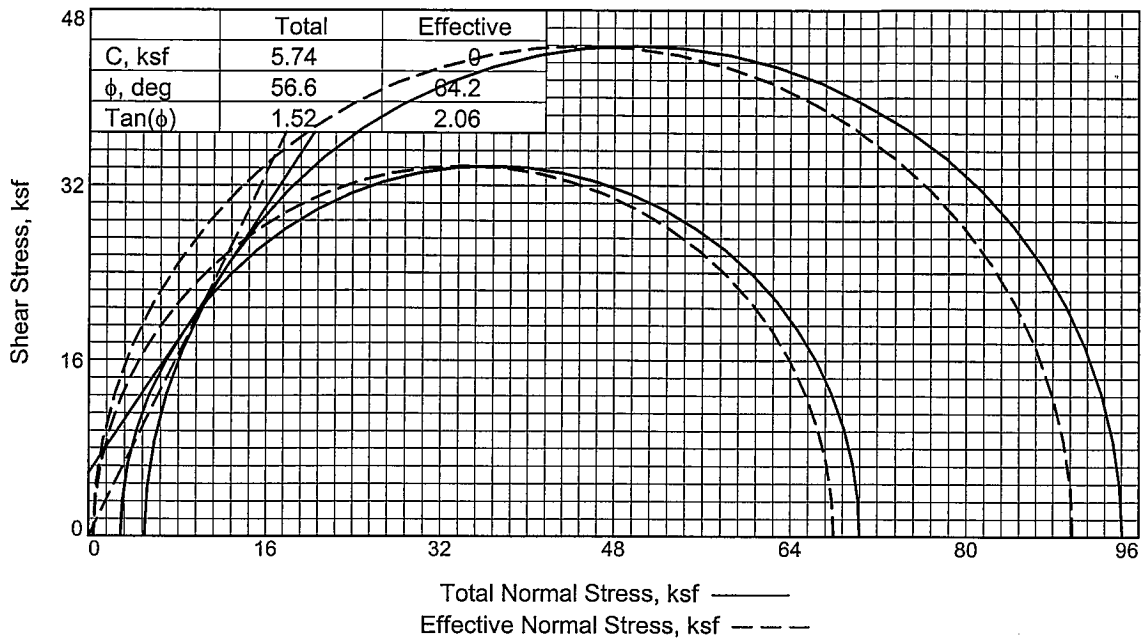
* Sample was not Testable

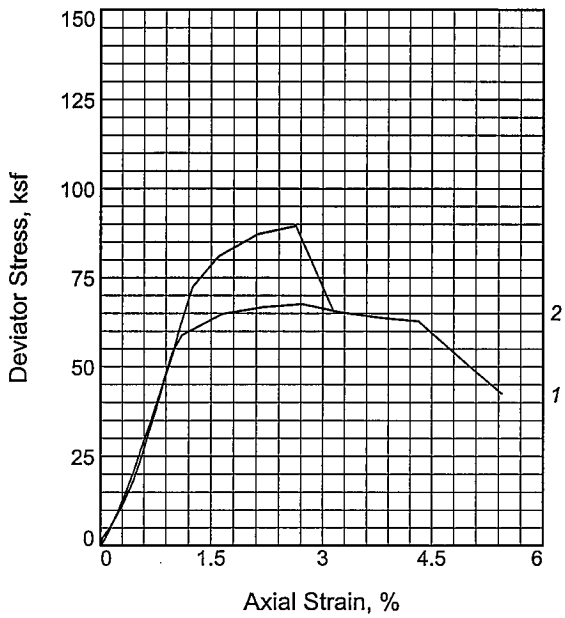
** A correction factor was applied to the compressive strength due to low L/D ratio.

Notes: 1- All tests were performed in accordance with ASTM D1633

Tested By: RCR 10/04/2010

Checked By: HAB 10/05/2010 HAB





Sample No.		1	2
Initial	Water Content, %	45.9	44.8
	Dry Density, pcf	72.0	73.1
	Saturation, %	98.6	99.0
	Void Ratio	1.1600	1.1269
	Diameter, in.	2.42	2.41
	Height, in.	4.72	4.87
At Test	Water Content, %	44.9	44.0
	Dry Density, pcf	73.4	74.2
	Saturation, %	100.0	100.0
	Void Ratio	1.1177	1.0955
	Diameter, in.	2.40	2.40
	Height, in.	4.69	4.84
Strain rate, in./min.		0.03	0.03
Back Pressure, psi		40.00	40.00
Cell Pressure, psi		60.00	75.00
Fail. Stress, ksf		67.5	89.4
Total Pore Pr., ksf		8.2	10.4
Ult. Stress, ksf			
Total Pore Pr., ksf			
$\bar{\sigma}_1$ Failure, ksf		68.0	89.8
$\bar{\sigma}_3$ Failure, ksf		0.5	0.4

Type of Test:
CU with Pore Pressures

Sample Type: Core sample

Description: Dark gray cement stabilized soil

Specific Gravity= 2.49

Remarks: Note: Estimated loads of a third specimen to be tested was determined to be greater than the calibrated range of the equipment and was not tested.

Figure _____

Client: Tennessee Valley Authority (TVA)

Project: TVA KIF DSM DEMO PROGRAM

Source of Sample: Column 1A **Depth:** 23.65'-24.2'

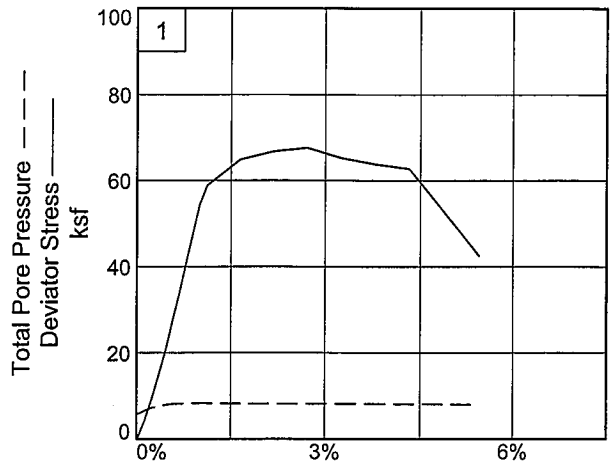
Sample Number: 1A - B1 & B2

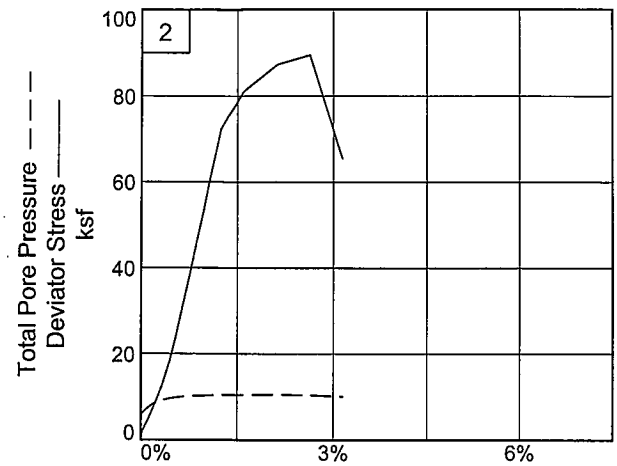
Proj. No.: 3043-10-1040 **Date Sampled:**

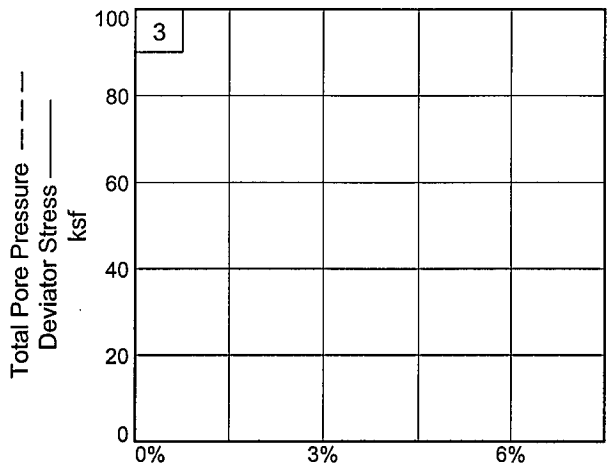
TRIAXIAL SHEAR TEST REPORT

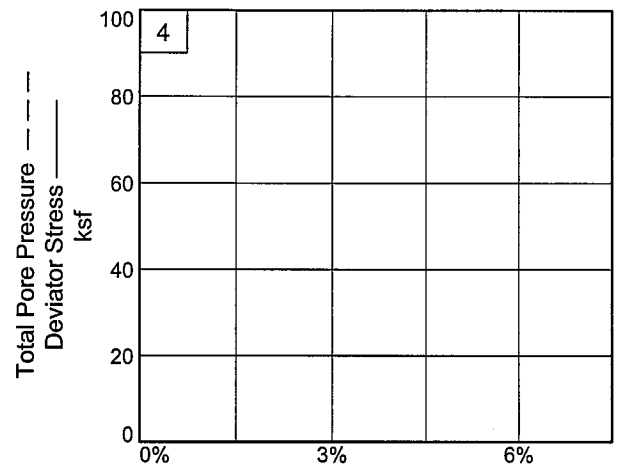
MACTEC ENGINEERING AND CONSULTING, INC.

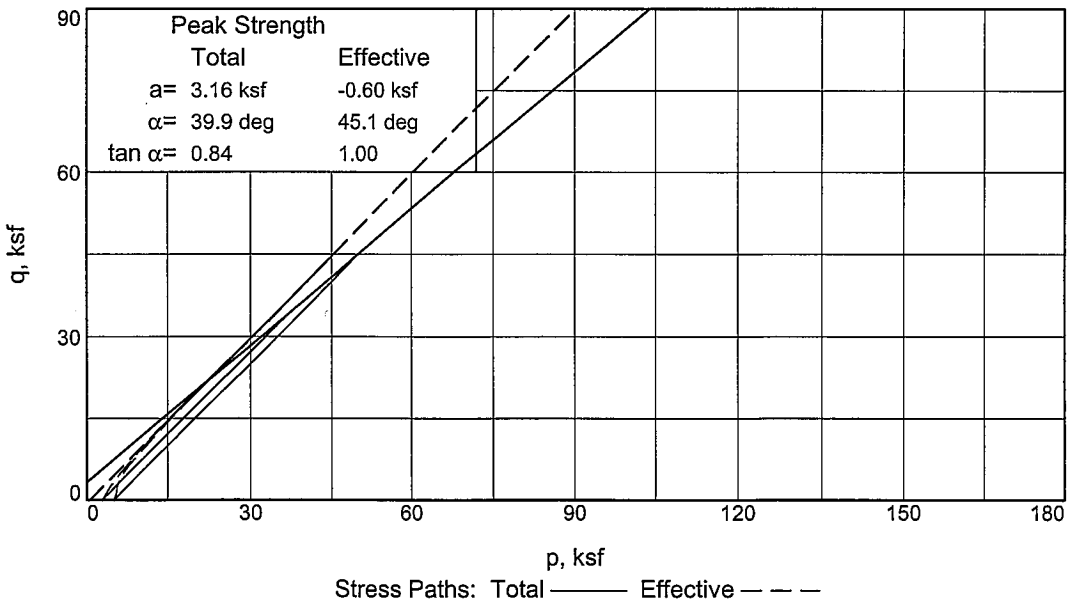
Tested By: FB











Client: Tennessee Valley Authority (TVA)
Project: TVA KIF DSM DEMO PROGRAM
Source of Sample: Column 1A **Depth:** 23.65'-24.2' **Sample Number:** 1A - B1 & B2
Project No.: 3043-10-1040 **Figure** _____ **MACTEC Engineering and Consulting, Inc.**

Tested By: FB

MACTEC

REPORT OF SPLITTING TENSILE STRENGTH OF SOIL CEMENT SPECIMENS

CLIENT: Tennessee Valley Authority (TVA)

PROJECT NO.: 3043-10-1040

PROJECT: TVA KIF DSM Demo Program

SAMPLE TYPE Core Samples

DATE TESTED: 9/29/2010

DATE CAST: 8/9/2010

SPECIFIED MINIMUM COMPRESSIVE STRENGTH (psi) N/A

SAMPLE NO.	DATE TESTED	LENGTH (in.)	DIAMETER (in.)	ULTIMATE LOAD (lbs.)	L:D	SPLITTING TENSILE STRENGTH (psi)
8-C1	9/29/10	4.46	2.37	1380	1.88	83.1
8-C3	9/29/10	4.79	2.39	246	2.00	13.7

GENERAL NOTES

Tested in accordance with ASTM C-496

RESPECTFULLY SUBMITTED

Kelly Marshall