FOUNDATION STABILITY ANALYSES

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Tennessee Valley Authority (TVA)

Project: Kingston Fossil Plant Gypsum Disposal Facility **Project/Proposal #:** GR3731 **Task #:** 06

Title of Computations: *Foundation Stability Analyses*

Computation Package: ______________________

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FOUNDATION STABILITY ANALYSES

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PURPOSE

The purpose of this calculation package is to evaluate the static and seismic slope stability of the proposed gypsum disposal facility at the Kingston Fossil Plant (hereafter referenced as KIF gypsum disposal facility). For these analyses, potential slip surfaces passing through the gypsum material and underlying native foundation soils are considered.

METHOD OF ANALYSIS

Static Stability Analysis:

Slope stability analyses were performed using the simplified Bishop method [Bishop, 1955] for the circular search method for potential slip surfaces, and the Spencer method [Spencer, 1973] for block surfaces as implemented in the computer program SLIDE [2003]. The program was used to generate potential slip surfaces and calculate the factor of safety for each of these surfaces. SLIDE identifies the slip surface with the lowest factor of safety. Information required for the analyses include:

- the geometry of the gypsum disposal facility at the cross section location;
- the subsurface soil stratigraphy at the cross section location;
- the material properties for gypsum, subgrade fill, and subsurface materials;
- the water level within the gypsum stack; and
- the groundwater table elevation along the cross section location.

Analyses were performed for an interim construction phase representing the top elevation of the wet stack gypsum material (approximate Elevation 900 ft mean sea level (msl)); and for the final build out phase representing the top of dry stack gypsum material (approximate Elevation 985 ft msl). Both drained and undrained analyses were performed.

Seismic Stability Analysis:

Seismic slope stability analyses were performed using a procedure consistent with the guidance document prepared by the U.S. Environmental Protection Agency [USEPA, 1995]. The procedure is as follows:

- Estimate the maximum horizontal acceleration (MHA) in lithified earth material and the peak horizontal acceleration at the ground surface (PGA) for the site.
    - Based on the most recent current USGS seismic hazard map (2002), the MHA is 0.25g.
    - The PGA is conservatively assumed equal to the MHA (i.e., 0.25g).
- Estimate the peak horizontal acceleration of the potential sliding mass. This value is assumed to be equal to the PGA.
- Perform pseudo-static slope stability analyses of potentially critical cross sections to evaluate the yield acceleration. Yield acceleration is the acceleration value which produces a calculated pseudo-static factor of safety equal to one.
    - If the calculated yield acceleration exceeds the peak horizontal acceleration of the potential sliding mass (equal to PGA), it is concluded that permanent seismic deformations will not occur.
    - If the calculated yield acceleration is less than the PGA, it is concluded that permanent seismic deformations will occur and their magnitude is evaluated in the following step.
- Estimate the magnitude of the permanent seismic deformation using a seismic deformation analysis.
    - The ratio of yield acceleration to PGA is used with relationships presented by Hynes and Franklin [1984] and to estimate the magnitude of permanent seismic deformation. These relationships were based on analyses performed using the Newmark [1965] method of seismic deformation analysis and several hundred recorded time histories for earthquakes from around the world as well as six synthetic time histories, representing earthquakes up to 7.7 in magnitude. The "modified mean + one standard deviation curve" developed by GeoSyntec considers data associated with only large earthquakes, and therefore, is more conservative and is used herein.

For the pseudo-static slope stability analyses described, the computer program SLIDE [2003] was used. The analyses were performed using the simplified Bishop method [Bishop, 1955] for circular potential slip surfaces and the Spencer method [Spencer, 1973] for block surfaces.

Design Water Levels Within Disposal Facility

The gypsum material at the KIF gypsum disposal facility will be sluiced in up to Elevation 900 ft msl; therefore, the interim construction stability was evaluated assuming a water level within the gypsum stack to be at Elevation 900 ft msl (thus assuming no drainage has occurred). Under final configuration (i.e., wet and dry stack configuration), it is assumed that the water level within the gypsum stack will reduce as waters are removed via the internal drainage system. Analyses to estimate the water level within the KIF gypsum disposal facility at different time periods are presented in the calculation package titled "*Seepage Analysis*." According to this calculation package, and neglecting the effect of the central drainage corridor, the water level within the gypsum stack is calculated to drop by approximately 40 ft after five years. Considering that it will take more than 10 years to reach the maximum elevation of the dry stack material and since the beneficial effects of the central drainage corridor are neglected (i.e., assuming drainage only occurs through the perimeter drainage system), a 40 ft drop in the water level in the gypsum stack is considered to be a conservative assumption.

Target Factors of Safety:

The target calculated factor of safety for static stability analyses is 1.5.

The criterion for seismic stability is based on calculated permanent deformation. Based on the limiting seismic slope stability design criteria of the Tennessee Division of Solid Waste Management a division of the Tennessee Department of Environment and Conservation (TDEC) [TDEC, 1993], "*No landfill shall be acceptable if the predicted seismic induced deformations within the waste fill exceed one-half the thickness of the clay liner component of the liner system.*" Since there is no liner mandated for this facility, the 3-ft thick layer of geologic buffer (compacted clay) may be considered to be the clay liner component and therefore the maximum acceptable calculated permanent seismic deformation is 1.5 ft (18 inch).

CROSS SECTIONS ANALYZED

Two cross sections (i.e., Cross Section A and Cross Section B) were analyzed. The location of the cross sections with respect to the final cover system of the KIF gypsum disposal facility features is shown in Figure 1. The cross section geometries at each location (including dry stack and wet stack gypsum, coarse gypsum, soil stratigraphy, water table, and piezometric surface within the dry stack material) are shown in Figures 2 and 3. Each cross section is considered critical since the maximum waste height and grade is obtained at these locations.

SITE STRATIGRAPHY

Information on the site stratigraphy used in these analyses is summarized in MACTEC [2005], MACTEC [2006], and TVA [2005]. The top of bedrock elevations were obtained from a contour map developed from a series of site investigations that included soil borings, CPT soundings, and GeoProbe soundings performed at the site as presented in TVA [2005]. Current ground elevations were obtained from the Kingston Fossil Plant topographic map provided by TVA. Nearby borings were projected to the cross section to develop the thicknesses of the compressible native material along the cross section. This native material was subdivided into two groups based on the Standard Penetration Test (SPT) blow count and water content of the material. A description of the subsurface stratigraphy and the corresponding material properties are presented in the following section.

MATERIAL PARAMETERS

Information on the material parameters used in these analyses is obtained from MACTEC [2004], MACTEC [2005], and MACTEC [1995]. Material parameters used for the stability analyses are summarized in Table 1.

Gypsum

Samples of gypsum are not yet available from the Kingston Fossil Plant. Material from the Cumberland Fossil Plant is considered representative of the material that will be produced at the Kingston Fossil Plant once the scrubber is brought online. For design purposes, material properties of the Cumberland gypsum are used herein.

- *Dry Stack Gypsum*: The dry placed gypsum material will be dewatered at the plant before it is transported to the KIF gypsum disposal facility. This material will be placed at elevations above approximately 900 ft msl. Material properties for the dry stack gypsum are provided in the report titled *Use of Coal Combustion By-Products as Engineered Fills* prepared by MACTEC [1995]. According to this report, consolidated undrained (CU) triaxial tests were performed on specimens remolded to approximately 95 percent of the Standard Proctor maximum density at or near optimum moisture content. Based on these test results, an effective stress friction angle of 38 degrees was reported. For the stability analysis described herein, a friction angle of 35 degrees and a zero cohesion intercept was selected.

- *Coarse Gypsum*: Coarse grained gypsum is a by-product of the rim-ditch method of sluiced material placement. Coarser grained gypsum settles out in or near the rim ditch and is scooped out to form the perimeter dikes. Relatively undisturbed samples representing a coarser grained sluiced gypsum material at the Cumberland Fossil Plant were obtained by MACTEC [2004]. Based on a three-point consolidated undrained (CU) triaxial test a friction angle of 40 degrees was obtained and is used in the analyses presented herein.

- *Fine Gypsum*: Fine grained gypsum is also a by-product of the rim-ditch method of sluiced material placement, however the finer grained material travels further from the discharge point towards the center of the gypsum pond than the coarser material. Like the coarser grained gypsum, undisturbed samples representing the fine grained gypsum were obtained at the Cumberland Fossil Plant by MACTEC [2004]. Shear strength parameters were estimated based on a three-point CU triaxial test assuming failure occurs where the shear induced excess pore pressures are zero. Based on these results, the effective stress shear strength parameters used in the analyses presented herein are an effective stress friction angle of 30 degrees and a zero cohesion intercept. An undrained shear strength ratio (S_u/σ_{vo}') of 1.5 was selected.

Soil Fill/Subgrade

On site material will be used to construct the initial soil berm around the gypsum pond and the subgrade fill. Standard Proctor tests were run on 17 samples of native material from depths ranging from 6 to 12.5 ft. The unit weight of the soil fill material was selected as 95 percent of the average of the maximum dry unit weights resulting from the Standard Proctor tests. Effective stress properties for the soil berm and subgrade material are average values from three, three-point CU triaxial tests performed on remolded samples taken from depths ranging from 6 to 10 ft. Based on these results, the effective stress shear strength parameters used in the analyses presented herein are an effective stress friction angle of 30 degrees and a zero cohesion intercept.

Geologic Buffer

The geologic buffer effective stress properties for the geologic buffer have been estimated from averaging typical peak drained strengths for CL, MH, and CH soils as presented by Duncan and Wright [2005]. The effective stress shear strength parameters used in the analyses presented herein are estimated as an effective stress friction angle of 24 degrees and a zero cohesion intercept.

Native Soil

The onsite native material is primarily classified as a medium stiff to stiff silty clay. The average blow count of the material onsite ranges from 6 to 20 blows per foot (bpf). Approximately one-half of the borings encountered a "soft" material, classified by Standard Penetration Test (SPT) N values less than or equal to 4 bpf. This soft material ranged in thickness from 0 to 20 ft along the cross sections selected for the stability analyses and occurred just above the bedrock material. For the analyses performed herein, drained and undrained shear strength parameters were selected for two layers of foundation material (i.e., N>4 and N≤4). Triaxial tests summarized in MACTEC [2006] and CPT soundings summarized in TVA [2005] were used to develop the short and long term shear strength of the native material.

- N>4:
  - (*Undrained shear strength for analyses where gypsum disposal facility is at Elevation 900 ft msl*)

    CU and unconsolidated undrained (UU) triaxial tests were performed on eight samples obtained from depths ranging from 13 to 41 ft below ground surface. This triaxial data in combination with data from ten Cone Penetration Test (CPT) soundings performed across the site to depths of 42 ft were used to estimate native soil undrained shear strength. The undrained shear strength can be estimated from the measured tip resistance according to the following equation developed by Schmertmann [1978]:

$$S_u = \frac{q_c - \sigma_{vo}}{N_k}$$

where N_k is a normalizing factor that ranges from 12 to 19 and is related to the plasticity index of the in situ material. For the native soil at the KIF gypsum disposal facility, an N_k factor of 19 was chosen to calibrate the calculated CPT undrained shear strength data to the undrained shear strength developed from the triaxial test data.

The undrained shear strength data were plotted versus effective confining pressure to develop an undrained shear strength profile for the native material (i.e., a best fit linear trend line through the data as shown in Figure 4) resulting in the following equation:

$$S_u = 1{,}792 \text{ psf} + 0.27 * \text{Confining Pressure (psf)}$$

Conservatively assuming that the native soil is saturated, and a unit weight of 120 pcf, an undrained strength profile with depth can be estimated using the following equation:

$$S_u = 1{,}792 \text{ psf} + 15.6 * \text{depth}$$

where depth is measured in feet below the pre-construction ground surface (i.e. at the elevation of the top of the native material).

- (*Undrained shear strength for analyses where construction of gypsum disposal facility is above Elevation 900 ft msl up to Elevation 985 ft msl*)

  For analyses with gypsum placement above Elevation 900 ft msl (dry stack material placement), it was assumed that the native material would experience some improvement in undrained shear strength due to consolidation which will occur as a result of the weight of the previously place wet stack material. Based on a construction period of 14.5 years (i.e., assuming 10 ft of wet stack gypsum would be placed per year), the native soil will experience approximately 50 percent consolidation, and a corresponding increase in effective confining pressure at the approximate time when placement of the dry stack material is anticipated to commence. This improved undrained shear strength is evaluated in three zones under the wet stack loading: (i) beneath the maximum gypsum height of 900 ft msl; and (ii) two zones beneath the side slope. No shear strength improvement was assumed beneath the toe of the slope. Calculations to evaluate improvements in undrained strengths due to the consolidation of the native material under the weight of the wet stack gypsum are provided in Attachment A.

For the drained analyses, an average effective stress friction angle of 34 degrees was used based on triaxial testing results.

- N≤4: No triaxial tests were performed on native material with SPT blow counts of less than or equal to 4 bpf. Four of the ten CPT soundings performed at the site encountered the soft native material. The average undrained shear strength of this material was developed from these CPT soundings and is 800 psf for the analyses performed herein. The improved undrained shear strength of the "soft" native material was not developed.

For the drained analyses, an effective stress friction angle for the soft native material was estimated based on the plasticity index (PI) using the following relationship [Mitchell, 1976]:

$$\sin \phi'_{cv} \approx 0.8 - 0.094 \ln(PI)$$

Considering an average PI of the soft native material to be 43, an effective stress friction angle of 26.5 degrees was calculated. A friction angle of 25 degrees was used for the analyses described herein.

Bedrock

Due to the anticipated high shear strength of the bedrock, the top of bedrock elevation is considered the lower limit for the potential critical slip surface therefore; reasonable cohesion, friction angle, and unit weight values were selected as required by the computer simulation.

RESULTS

Table 2 summarizes the results of the static slope stability analyses for both left and right potential slip surface directions (i.e., towards or away from the Clinch River). Analyses were performed for Cross Section A-A' at the interim wet stack material height of 900 ft msl and the final dry stack material height of 985 ft msl. As shown for Cross Section A-A', the critical geometry (i.e., the lowest calculated factor of safety) is the maximum height of dry stack gypsum of 985 ft msl. Therefore, for Cross Section B-B', the interim geometry of wet stack gypsum material height of 900 ft msl was not investigated. For Cross Section B-B', analyses were performed for the final dry stack gypsum material height of 980 ft msl.

Table 3 summarizes the results of the seismic slope stability analyses performed for the KIF gypsum disposal facility. Seismic slope stability analyses were performed for the final maximum height of gypsum of 985 ft msl. The calculated displacements were selected based on the modified Hynes and Franklin (1984) chart as shown in Figure 5, where the "modified mean + one standard deviation curve" developed by GeoSyntec was used for this analysis. Associated output files and figures from SLIDE are presented at the end of this package in Attachments B through E.

SUMMARY AND CONCLUSIONS

The stability of the KIF gypsum disposal facility was evaluated with respect to static and seismic foundation stability. The most critical cross sections with respect to foundation stability were analyzed. Results indicate that the minimum static stability factor of safety for a potential slip surface through the gypsum and foundation soils is 1.60, which is greater than the target factor of safety.

Results indicate that the minimum yield acceleration for slip surfaces through the waste and the foundation soils is 0.155 g. For the analyses considered herein, the maximum calculated permanent deformation evaluated by the modified Hynes and Franklin (1984) chart is 1.97 inch (as shown in Figure 5) which is less than half the clay liner thickness (18 inch) as prescribed by the TDEC Earthquake Evaluation Guidance document. Therefore, the calculated permanent seismic deformations are considered acceptable.

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TABLES

Table 1. Summary of Material Properties.

Material	Unit Weight	Effective Stress		Undrained Strength S_u	
	(pcf)	Cohesion (psf)	Friction Angle	For Placement of Gypsum *up to* Elevation 900 ft (psf)	For Placement of Gypsum *above* Elevation 900 ft (psf)
Dry Stack Gypsum	107	0	35	-	-
Coarse Gypsum	90	0	40	-	-
Fine Gypsum	100	0	30	$S_u/\sigma_{vo}'=1.5$	$S_u/\sigma_{vo}'=1.5$
Soil Fill	117	0	30	-	-
Geologic Buffer	117	0	24	-	-
Native Soil (N>4)	120	0	34	1,792+15.6* depth	Varies (See Attachment A)
Native Soil (N≤4)	105	0	25	800	800
Bedrock	155	10,000	30	-	-

Table 2. Summary of Static Foundation Stability Analyses.

Cross Section	Potential Slip Surface Direction	Search Method	Drained/ Undrained(1)	Maximum Height of Gypsum (ft msl)	Factor of Safety	Figure	File Name
A-A'	Left	Circle	Undrained	900	2.20	B-1	Cross Section A-A'_1a
A-A'	Left	Block	Undrained	900	1.93	B-2	Cross Section A-A'_1a_block
A-A'	Left	Circle	Undrained	985	1.98	B-3	Cross Section A-A'_2
A-A'	Left	Block	Undrained	985	1.67	B-4	Cross Section A-A'_2block
A-A'	Left	Circle	Drained	900	2.28	B-5	Cross Section A-A'_1a_drained
A-A'	Left	Circle	Drained	985	1.64	B-6	Cross Section A-A'_2_drained
A-A'	Right	Circle	Undrained	985	2.13	B-7	Cross Section A-A'_2_right
A-A'	Right	Block	Undrained	985	2.12	B-8	Cross Section A-A'_2_right_block
A-A'	Right	Circle	Drained	985	1.92	B-9	Cross Section A-A'_2_drained_right
B-B'	Left	Circle	Undrained	980	2.31	C-1	Cross Section B-B_1
B-B'	Left	Block	Undrained	980	1.88	C-2	Cross Section B-B_1_block
B-B'	Left	Circle	Drained	980	1.60	C-3	Cross Section B-B_1_drained
B-B'	Right	Circle	Undrained	980	2.70	C-4	Cross Section B-B_1_right
B-B'	Right	Block	Undrained	980	2.62	C-5	Cross Section B-B_1_right_block
B-B'	Right	Circle	Drained	980	2.05	C-6	Cross Section B-B_1_drained_right

Notes: (1) For all analyses, the coarse gypsum, soil fill, and dry stack gypsum were modeled as drained materials. For analyses indicated as "Undrained", only the fine gypsum and native material (foundation soils) were modeled as undrained materials.

Table 3. Summary of Seismic Foundation Stability Analyses.

Cross Section	Drained/ Undrained(1)	Minimum Yield Acceleration a_y	Design Peak Acceleration a_{max} (PGA)	$(a_y)/(a_{max})$	Calculated Displacement (inch)	Figure	File Name
A-A'	Undrained	0.175g	0.25g	0.7	1.67	D-1	Cross Section A-A'_2block_seis
A-A'	Drained	0.17g	0.25g	0.68	1.7	D-2	Cross Section A-A'_2_drained_seis
B-B'	Undrained	0.18g	0.25g	0.72	1.65	E-1	Cross Section B-B_1_block_seis
B-B'	Drained	0.155g	0.25g	0.62	1.97	E-2	Cross Section B-B_1_drained_seis

Notes: (1) For all analyses, the coarse gypsum, soil fill, and dry stack gypsum were modeled as drained materials. For analyses indicated as "Undrained", only the fine gypsum and native material (foundation soils) were modeled as undrained materials.

FIGURES

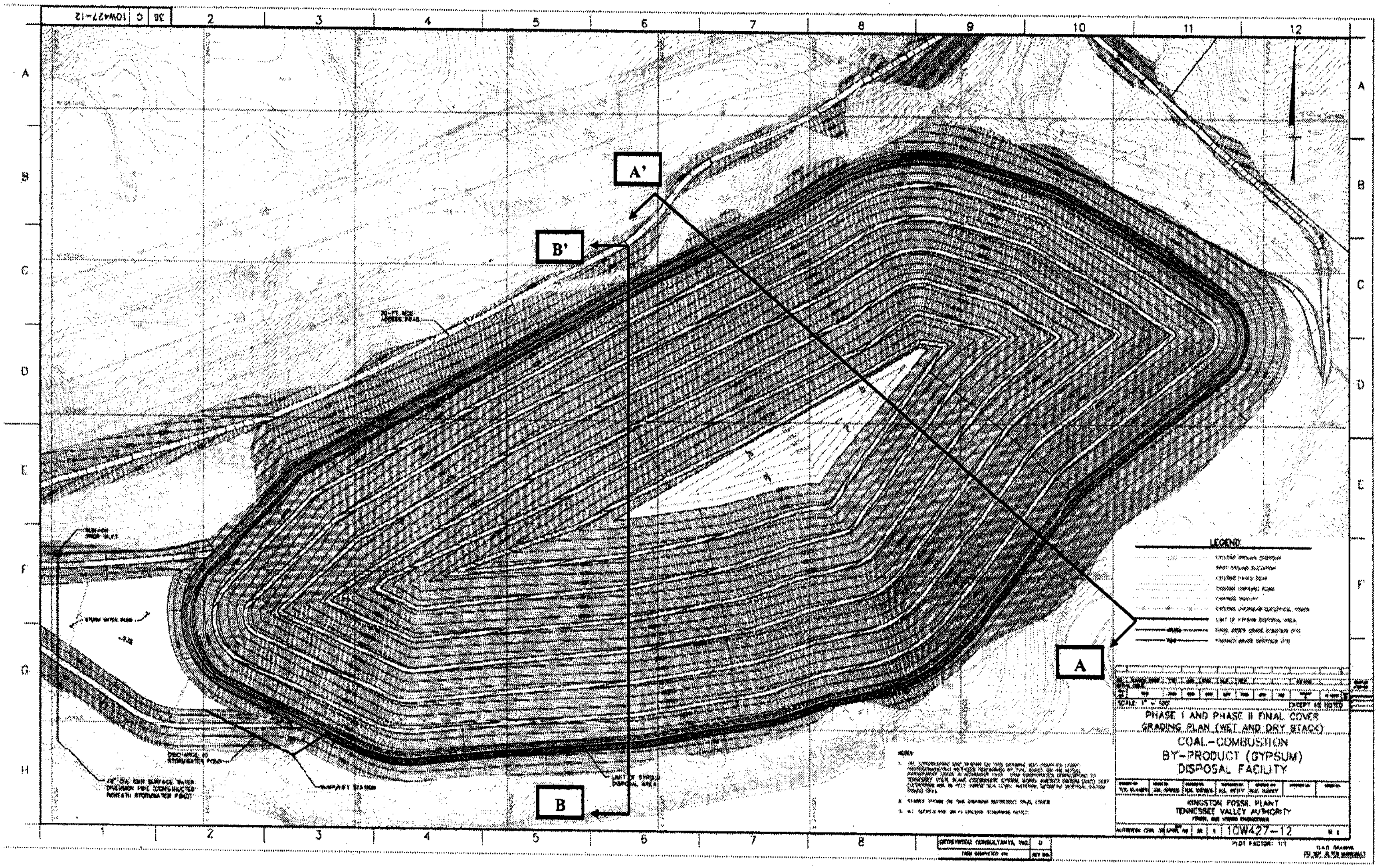


Figure 1: Location of the Analyzed Cross Sections on Final Cover Grading Plan.

Written by: JFR Date: 5/5/2006 Reviewed by: PJS Date: 5/5/2006

Client: TVA Project: Kingston Fossil Plant Project/Proposal No.: GR3731 Task No.:

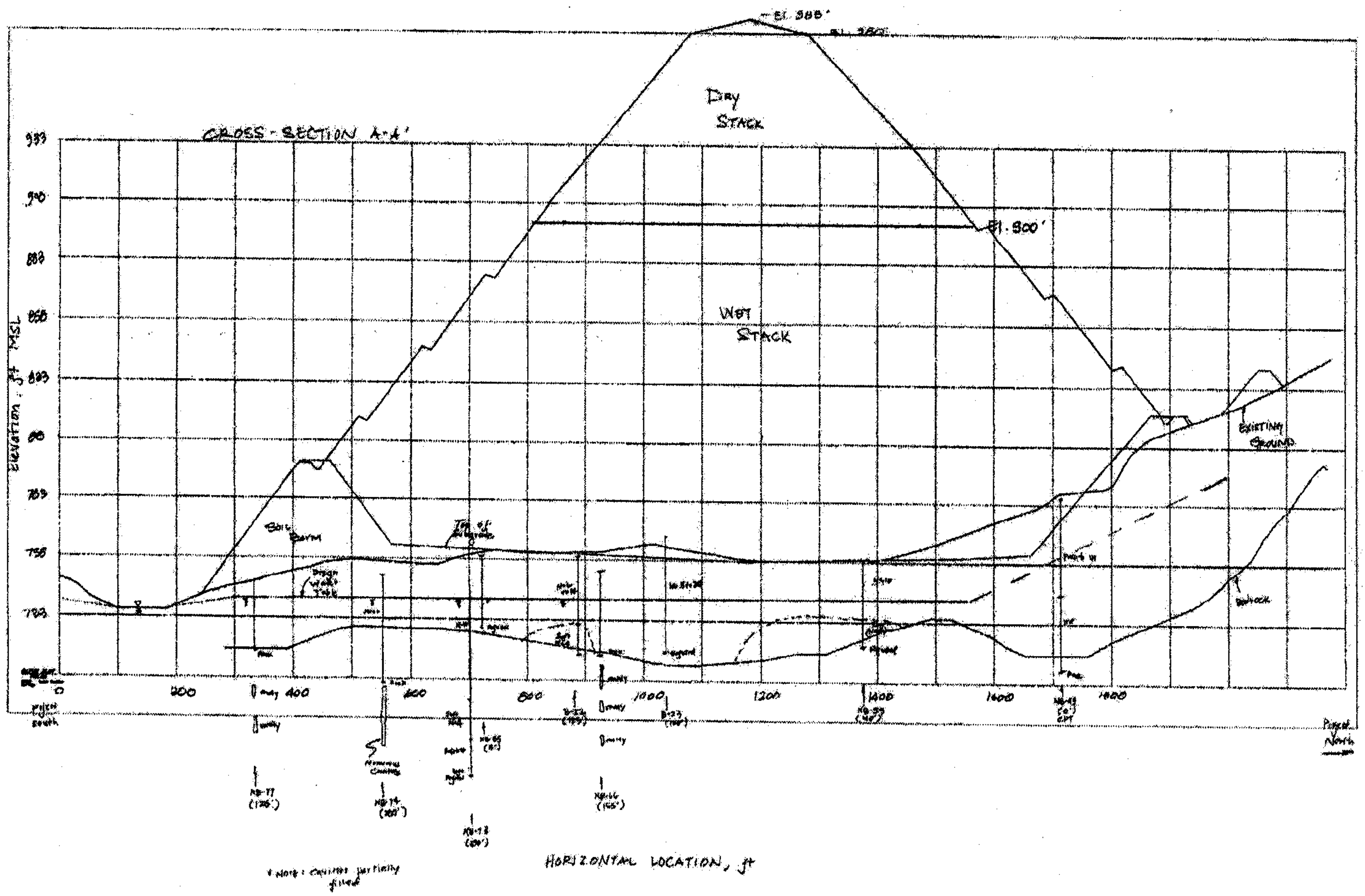


Figure 2: Stratigraphy of Analyzed Cross Section A-A'

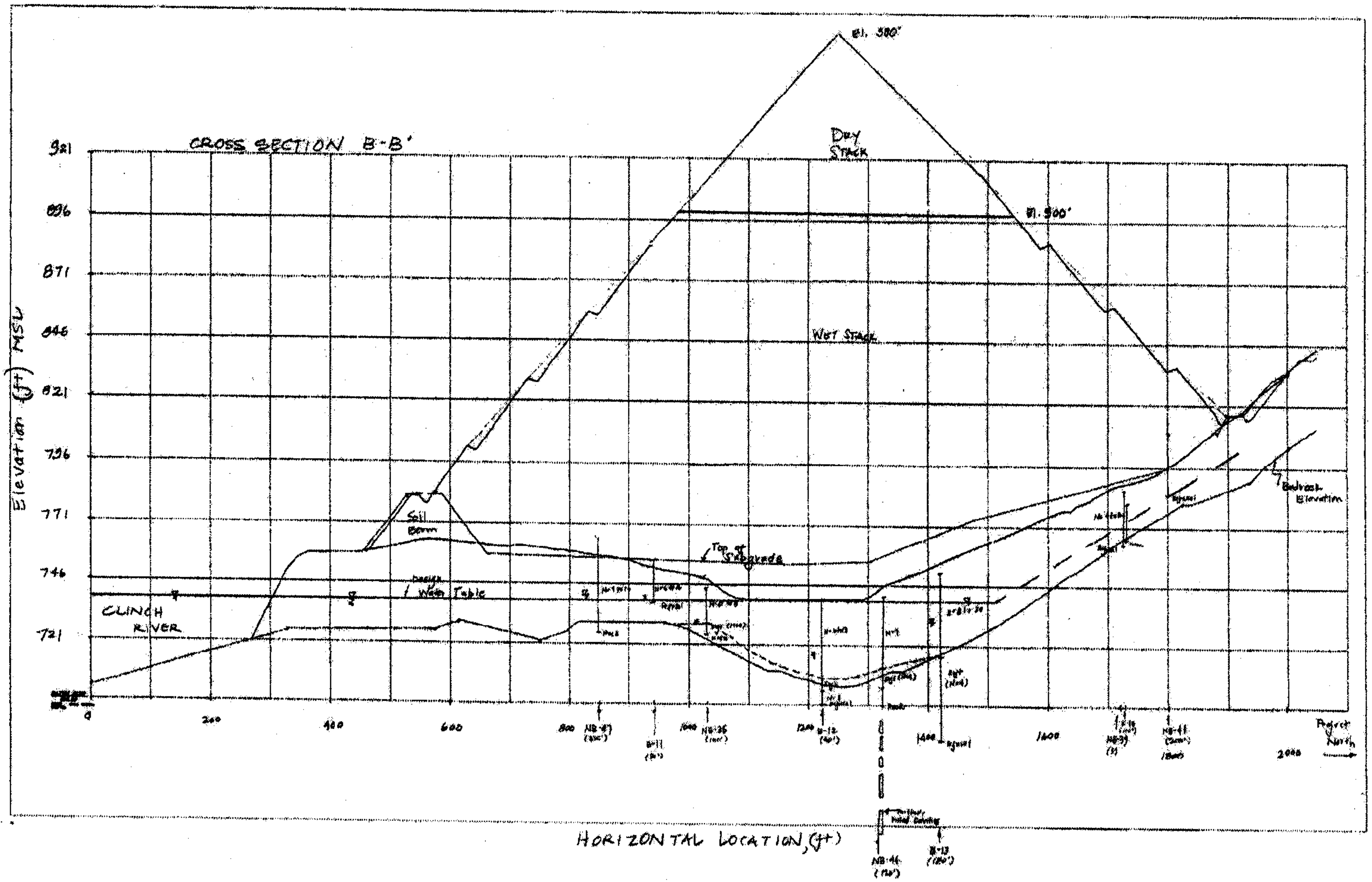
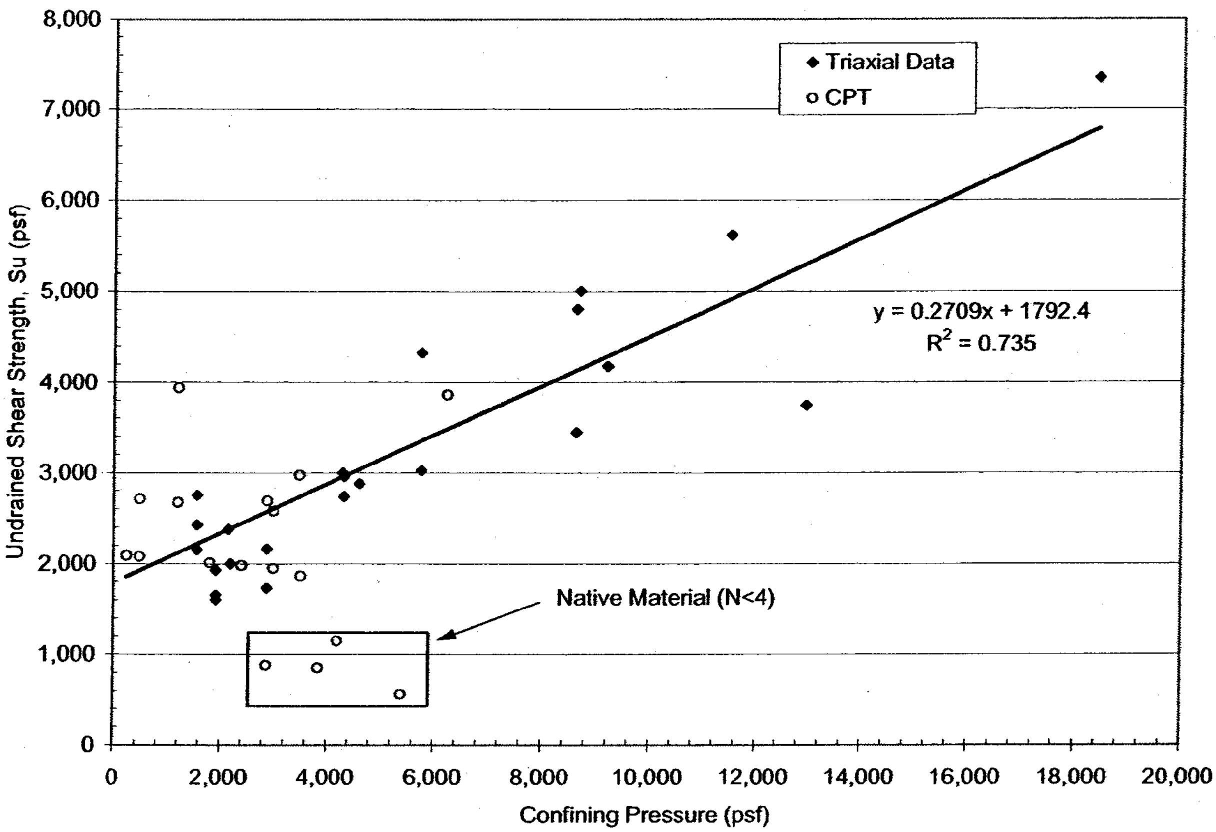


Figure 3: Stratigraphy of Analyzed Cross Section B-B'

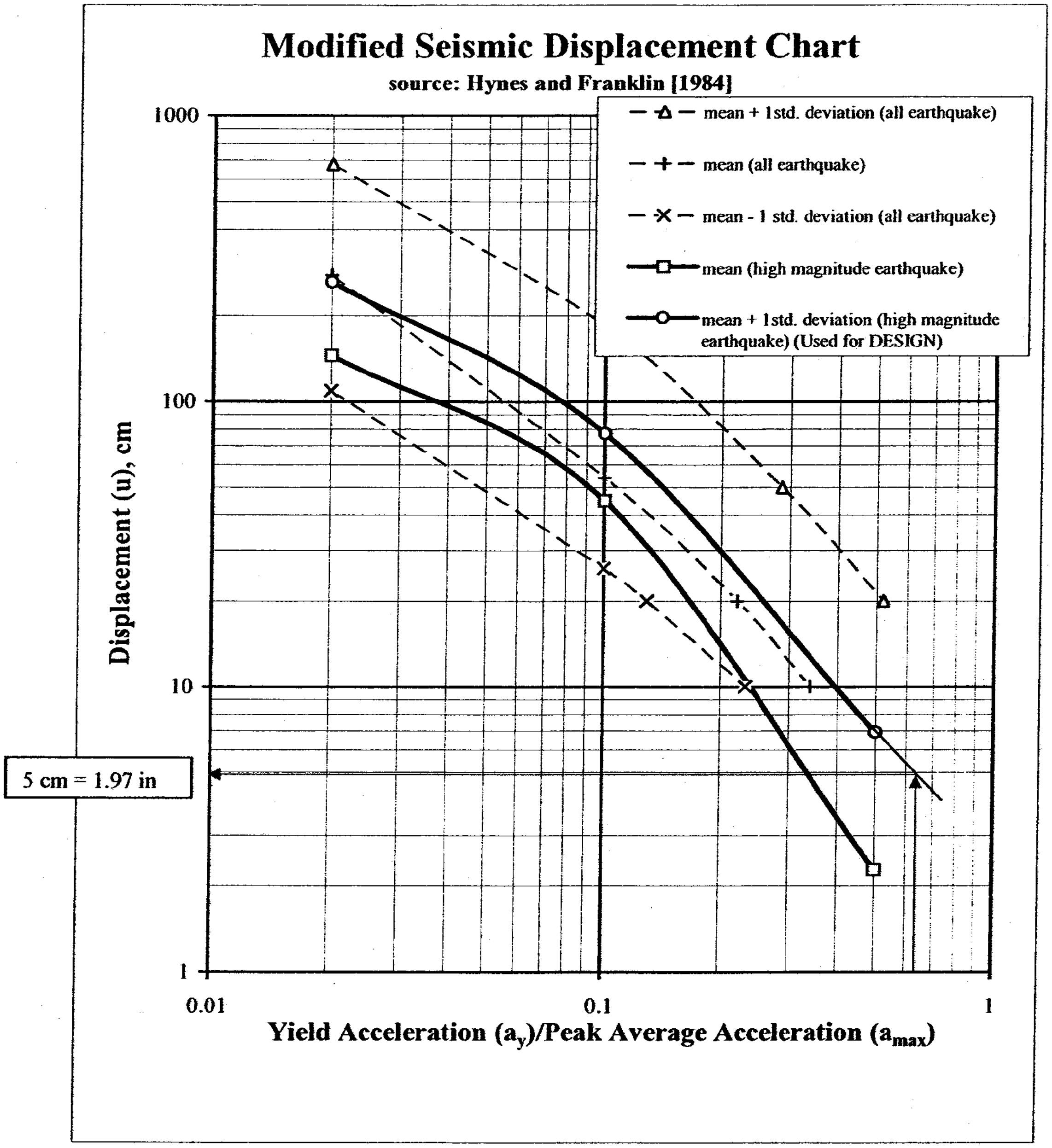


Notes:

1-Undrained shear strengths were derived from the CPT soundings based on the method developed by Schmertmann [1978]. An N_k factor of 19 was chosen to calibrate the CPT data to the triaxial data.

2-Undrained shear strengths based on CPT soundings in the soft native material (N<4) are indicated. An average undrained shear strength of 800 psf was chosen based on these data.

Figure 4. Determination of Undrained Shear Strength Profile for Foundation Material.



Note: The example shown is for the Cross Section B-B' seismic stability analysis as shown in Figure E-2.

Figure 5. Selection of Calculated Displacement.

ATTACHMENT A

IMPROVEMENT IN UNDRAINED SHEAR STRENGTH IN NATIVE SOIL

Written by: JFR Date: 06/05/05 (YY MM DD) Reviewed by: ______ Date: __/__/__ (YY MM DD)

Client: TVA Project: KIF - Peninsula Project/Proposal No.: GR3731 Task No: 06

Improvement in Undrained Shear Strength Due to Wet Stack Loading and Consolidation of Native Material

Purpose: Evaluate the improvement in undrained shear strength of Native material after placement of Wet Stack gypsum.

Procedure: ① Verify that 145 ft (average height of wet stack material) can be placed instantaneously assuming the undrained shear strength of the native material is:

$$S_u = 1{,}792 + 15.6 * \text{depth}$$

② Calculate build rate and time to reach top of wet stack

③ Determine % consolidation of native material after placement of full height of wet stack material.

④ Determine new in situ effective stress at middle of native material layer given % consolidation for each of four zones under wet stack gypsum.

⑤ Determine new undrained shear strength given updated confining pressure ($\sigma_{vo}'|_{new}$). and use this calculated undrained strength at the midheight of the layer for slope stability analyses.

Calculations:

① Filename: Cross Section A-A'_1a_block

Static foundation stability analysis for cross section A-A' demonstrates that 145 ft of wet gypsum load can be placed instantaneously. (See Table 2) Resulting FS = 2.18.

Written by: JFR Date: 06 / 05 / 05 Reviewed by: ______ Date: __/__/__
YY MM DD

Client: TVA Project: KIF- Peninsula Project/Proposal No.: GR3731 Task No: 06

(2) Assuming a production rate of 492,800 ton/yr and a dry unit weight of gypsum of 67 pcf, 544,831 cy of gypsum will be produced each year, or 337.6 ac·ft. Assuming an average footprint for Phase I of 34 ac, this is approximately 10 ft of gypsum per year. To place 145 ft of wet stack material will take 14.5 yr.

(3) Average Cv:

Given three consolidation tests on Native material (N>4), at a load of 14,500 psf (i.e., 145 ft * 100 pcf) an average $C_v = 0.0003\ in^2/sec$ is reported. (See Figure A-1).

% Consolidation:

Use Figure 13 of NAVFAC 7.1 p. 7.1-232 to find % consolidation. (See Figure A-2)

- Construction Time = 14.5 yr = t_o
- Thickness of Compressible Layer = 45 ft

$$T_o = \frac{C_v t_o}{H_{DR}^2} = \frac{0.0003\ in^2/sec\ (14.5\,yr)(365\,d/yr)(86400\,sec/d)}{(45\,ft \cdot 12\,in/ft)^2}$$

$$T_o = 0.47$$

Also, for $t = 14.5\,yr\,(365\,d/yr) = 5293$ day

$T_o = 0.47$ (same as above – final day of construction)

$U_v = 50\%$ ⇒ percent consolidation

(4) Divide native material beneath wet stack load into four zones. Calculate new effective confining pressure based on 50% consolidation and weight of wet stack material. See Figure A-3

Note that zone 4 does not "see" improvement, ∴ undrained shear strength remains the same.

$$\sigma_{vo}'\Big|_{original} = \frac{45}{2}\,ft\,(120 - 62.4\ pcf) = 1296\ psf$$

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Client: TVA Project: KIF-Peninsula Project/Proposal No.: GR3731 Task No: 06

Zone ①: $\Delta\sigma = h_{gypsum}(\gamma_{gypsum}) = 145\text{ ft}(100\text{ pcf}) = 14{,}500\text{ psf}$

$\sigma_{vo}'|_{new} = 50\%(\Delta\sigma) + \sigma_{vo}'|_{original}$

$= 50\%(14{,}500) + 1296\text{ psf}$

$= 8546\text{ psf}$

Zone ②: $\Delta\sigma = 95\text{ ft}(100\text{ pcf}) = 9500\text{ psf}$

$\sigma_{vo}'|_{new} = 50\%(9500\text{ psf}) + 1296\text{ psf}$

$= 6046\text{ psf}$

Zone ③: $\Delta\sigma = 45\text{ ft}(100\text{ pcf}) = 4500\text{ psf}$

$\sigma_{vo}'|_{new} = 50\%(4500) + 1296\text{ psf}$

$= 3546\text{ psf}$

⑤ Determine new undrained shear strength:

(See Figure A-4 and A-3)

Zone ①: $S_u = 4050$ psf

Zone ②: $S_u = 3400$ psf

Zone ③: $S_u = 2700$ psf

Note: ① Due to similar geometry / loading geometry use same values for Cross Section B-B'

② Mirror image values for critical slip surface search to the right (away from Clinch River)

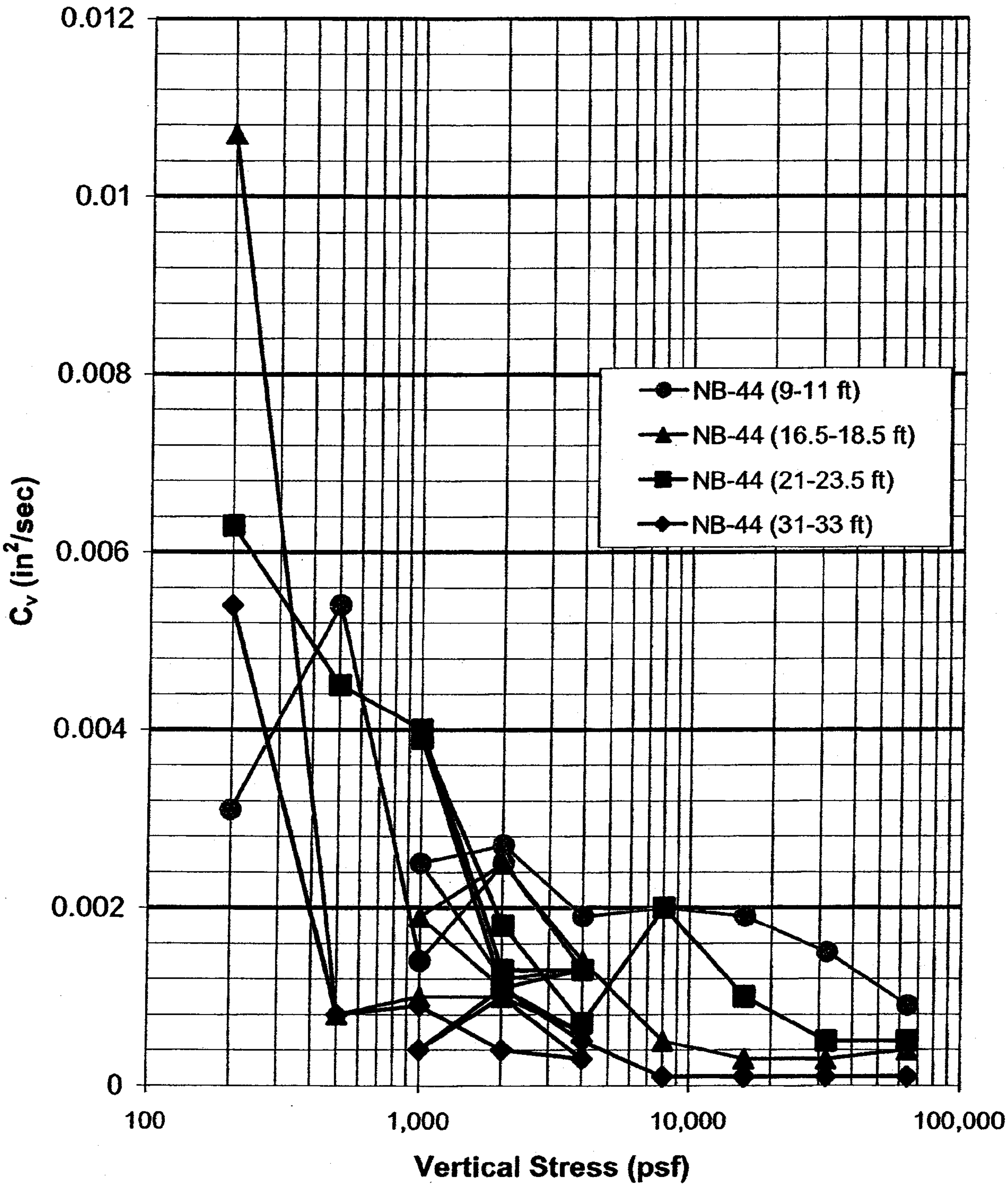
0.012
0.01
0.008
0.006
0.004
0.002
0
C_v (in²/sec)
100
1,000
10,000
100,000
Vertical Stress (psf)
NB-44 (9-11 ft)
NB-44 (16.5-18.5 ft)
NB-44 (21-23.5 ft)
NB-44 (31-33 ft)

Figure A-1

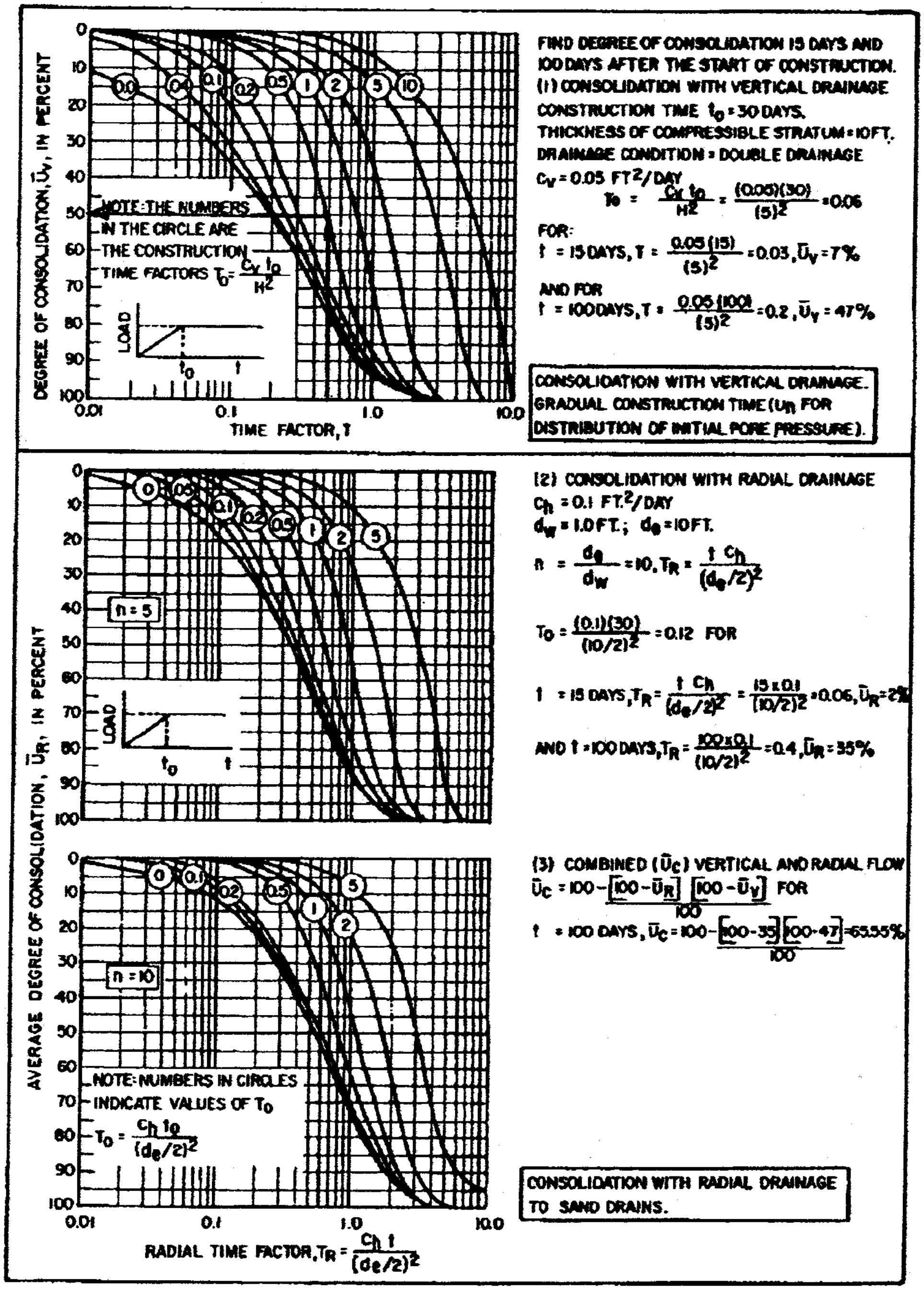

FIGURE 13
Time Rate of Consolidation for Gradual Load Application

Figure A-2

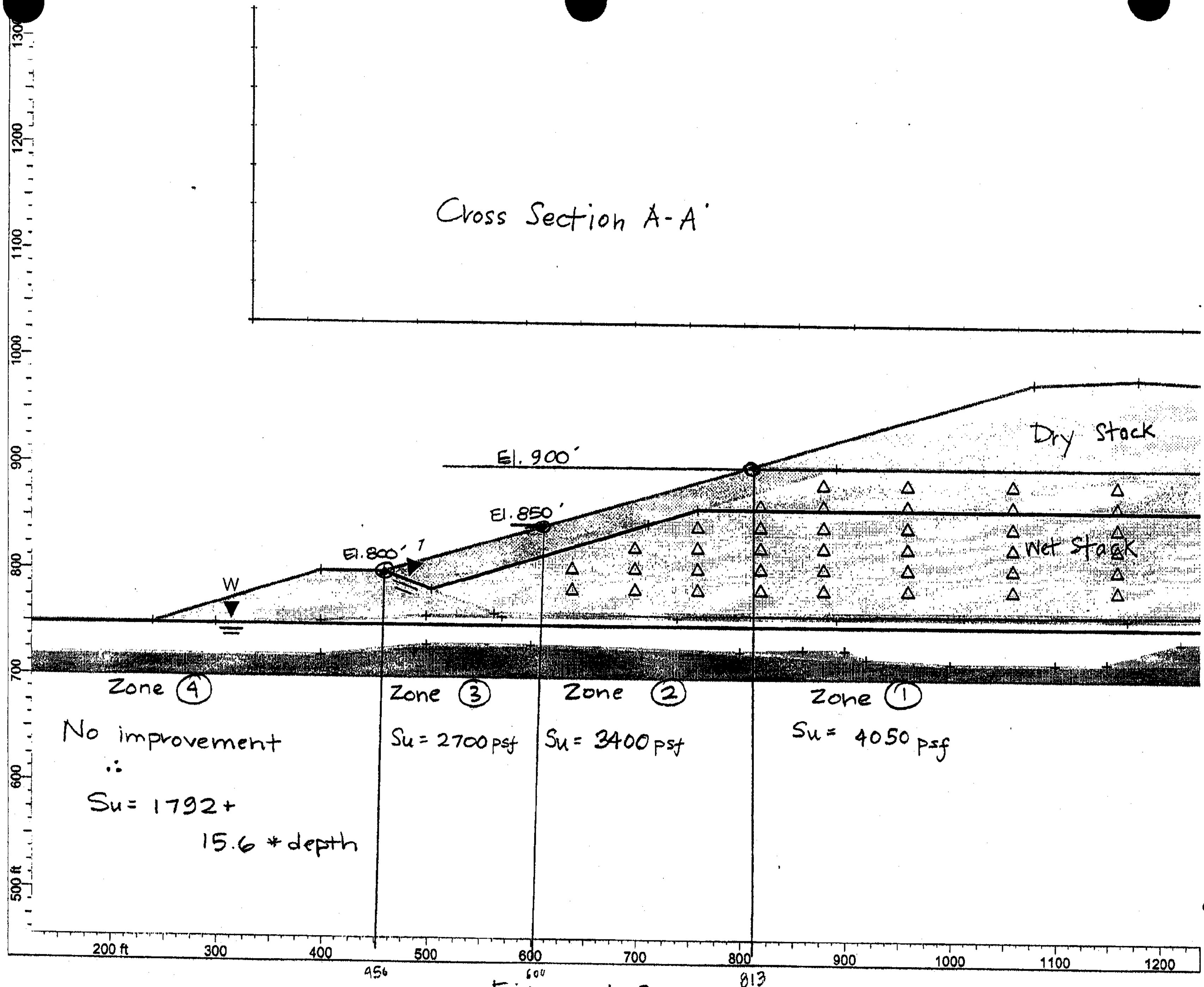
Cross Section A-A'
Dry Stack
Wet Stack
El. 900'
El. 850'
El. 800'
W
Zone (4)
No improvement
∴
Su = 1792 +
15.6 * depth
Zone (3)
Su = 2700 psf
Zone (2)
Su = 3400 psf
Zone (1)
Su = 4050 psf
1300
1200
1100
1000
900
800
700
600
500 ft
200 ft
300
400
500
600
700
800
900
1000
1100
1200
456
600
813

Figure A-3

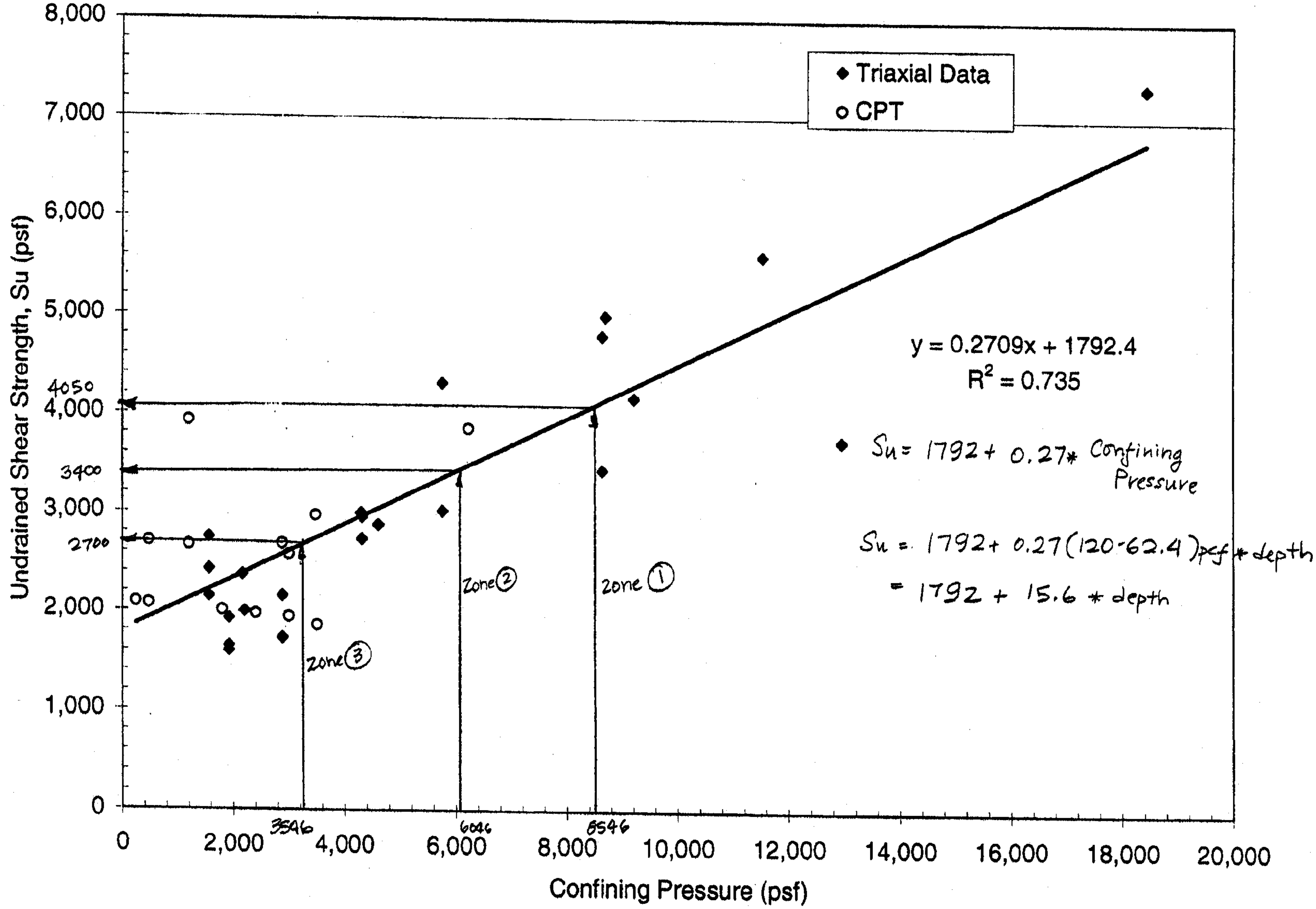

Figure A-4. Undrained Shear Strength (Su) as a function of Confining Pressure.

ATTACHMENT B

SLIDE OUTPUT

CROSS SECTION A-A' – STATIC