APPENDIX F

SEISMIC AND STATIC STABILITY

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF ATTACHMENTS

F.1   INTRODUCTION

This appendix presents the methods, input and results of the slope stability analyses for the disposal cell and surrounding natural slopes at the Sequoyah Fuels Corporation (SFC) Facility near Gore, Oklahoma. These analyses are an update of stability analyses in the Preliminary Design Report (Reclamation Plan Appendix C). The slope stability analyses were conducted according to applicable stability criteria under both static and seismic conditions, including geotechnical stability criteria in NRC (2000). Liquefaction analyses were conducted according to procedures outlined in Youd et al., (2001).

Slope stability analyses were performed using limit equilibrium methods with the aid of the computer program SLOPE/W (GEO-SLOPE, 1999). The SLOPE/W program calculates factors of safety by a variety of limit equilibrium methods. Spencer's method was used for these analyses because it considers both force equilibrium and moment equilibrium in the factor of safety calculation.

F.2   CRITICAL CONDITIONS AND GEOMETRY

Slope stability analyses are typically conducted under scenarios that represent the critical conditions for construction and operation. For the disposal cell, these conditions include: (1) the period during cell construction, and (2) the long-term period after cell construction.

Construction period. Key factors during construction are development of excess porewater pressures in foundation, berm or cover materials due to equipment or fill placement, or displacement of low-strength fill materials (such as sludges) in response to covering fill placement. The foundation materials (unsaturated soils and underlying sedimentary rock) are not susceptible to development of excess porewater pressures. Disposed materials will be placed and compacted in a manner to minimize void spaces and future settlement. Due to these construction methods and surrounding perimeter soils within the cell, conditions during cell construction were not analyzed for slope stability.

Long-term period. The long-term period after cell construction was analyzed along a critical section of the disposal cell slope. Long-term, steady-state material properties and porewater pressure conditions were used to represent this area.

The critical cross-section location used in the analysis is shown on Figure F.1, and the geometry of the section is shown on Figures F.2. This critical cross-section was selected because it represents the longest slope of the disposal cell. In addition, the analysis of this section was compared to the results of an

infinite slope scenario (slope length much longer than thickness of critical layer). Analyzing the infinite slope scenario minimizes any stabilization effects of a passive resistive wedge at the base of the slope.

Analysis conditions. The cell profile for the cross-section was based on a reclamation cover thickness of 10 feet, underlain by a textured synthetic liner, contaminated site soils and foundation soils. The foundation soil layer was assumed to be 10 feet thick, based on site boring logs (discussed in Appendix A). The thickness of the contaminated site soils was determined based on the topography shown on Figure F.1.

Slope stability analyses were performed by calculating factors of safety along circular failure surfaces as well as block and fully specified wedge failure surfaces. Circular failure surface analysis was conducted by targeting deeper, full slope failures. Small, shallow surface failures were not considered. Wedge failure surfaces were specified to occur along the synthetic liner. In both cases, a number of failure surfaces were analyzed to find the lowest factor of safety.

Most analyses were run assuming the soils are not saturated. Some analyses were performed assuming two feet of water on top of the synthetic liner. This represents an unlikely scenario, since the sands above the liner likely will drain any precipitation off the liner. However, the analyses are presented to check the sensitivity of the cover stability to pore water pressures.

F.3   MATERIAL PROPERTIES

Materials properties used in SLOPE/W for cover soil, contaminated site soils and foundation materials were based on typical values for the materials present at the site (discussed in Appendix A). Material properties are discussed below and summarized in Table F.1.

Cover material. A multi-layered cover system is proposed. This cover system will consist of (from bottom to top): (1) a two-foot compacted clay layer, (2) a textured synthetic liner, (3) an 18-inch layer of sand, (4) a five-foot subsoil zone, and (5) an 18-inch topsoil layer. On side slopes, the topsoil layer will be reduced to 9 inches thick, and overlain with 9 inches of rock mulch. Potential construction materials for disposal cell cover system include soils and weathered sedimentary rock from on-site sources, and rock from off-site sources. The cover is modeled as being the predominant subsoil zone, underlain by the synthetic liner. From geotechnical testing of a sample of the material for the subsoil zone, (documented in ESCI, 1998), the silty clay portion is a low-plasticity clay with a plasticity index of 17. The dry unit weight is approximately 110 pcf (Appendix A). Based on the general relationship between plasticity

index and shear strength in Holtz and Kovacs (1981), the effective angle of internal friction (for a material with a plasticity index of 17) is 32 degrees. In the stability analyses, the cover materials were conservatively represented by a dry unit weight of 110 pcf, an effective angle of internal friction of 30 degrees, and no cohesion.

Synthetic Liner and Compacted Clay Liner Interface. The critical surface is the interface between the textured geomembrane and compacted clay liner. The texturing on the geomembrane, under light to moderate loadings, will act to force a failure into the compacted clay. An average plasticity index value for the borrow material used to construct the clay liner is 17. Based on the Holtz and Kovacs (1981) relationship, this correlates with an internal friction angle of the clay of 32 degrees. An effective angle of internal friction of 28 degrees was used to conservatively represent the soil/synthetic liner interface. The synthetic liner/clay interface was represented in the analyses as a one-foot thick layer with a dry unit weight of 60 pcf, typical of synthetic liner material.

Foundation materials. Foundation materials in the site area are primarily terrace deposits consisting of silts, sandy silts, silty clays, sandy gravelly clays, silty sandy clays and clays which overlie shale and sandstone units. A dry unit weight of 110 pcf was used for these materials, due to the higher density and gravel content of these materials relative to the potential cover materials. An angle of internal friction of 30 degrees with no cohesion was used to represent the shear strength of these materials.

Contaminated soils. Contaminated site soils are expected to consist of a mixture of soils, construction debris (such as concrete and structural materials and sediments). This material will be placed with a specified compactive effort to minimize voids, thus a dry unit weight of 120 pcf was used to account for the fill materials and compaction. Shear strength was represented by an effective friction angle of 32 degrees with no cohesion.

Table F.1   Material Parameters Used in Stability Analyses

Material Type	Dry Unit Weight, γ (pcf)	Angle of Internal Friction, ϕ (degrees)	Cohesion, c (psf)
Cover Soil	110	30	0
Synthetic Liner	60	28	0
Contaminated Site Soils	120	32	0
Foundation Materials	110	30	0
Alluvial Soils on Natural Slopes	110	25	0

F.4   SEISMIC ANALYSIS AND SEISMICITY

Stability analyses under seismic conditions were conducted as pseudo-static analyses, with a design seismic coefficient applied to each cross-section. The design seismic coefficient is 67 percent of the peak horizontal acceleration (U.S. Department of Energy, 1989).

Analysis approach. If the materials in a structure are saturated and of low density or susceptible to significant loss of shear strength, an evaluation of the potential for liquefaction of these materials is conducted. The structure is then analyzed for slope stability based on a liquefied or reduced shear strength condition. If the materials in the structure are not susceptible to liquefaction or loss of shear strength, an analysis of the structure from seismic-induced accelerations is conducted. This consists of a stability analysis under an equivalent constant acceleration (described in Seed, 1979) or an evaluation of seismic-induced deformations (described in Makdisi and Seed, 1978). The equivalent, constant acceleration used in these analyses is the seismic coefficient, which is a fraction of the maximum seismically-induced acceleration anticipated at the site during the design period.

Seismicity. The site seismicity was reviewed in terms of: (1) general regional data, and (2) site specific data, as discussed below.

Based on general seismicity information, the site is within a region of low seismicity. This region is classified as a Zone 1 area in U.S. Army Corps of Engineers (1982), with a recommended seismic coefficient of 0.025 g (where g is the acceleration of gravity). The region is classified as a Zone 1 area in IBCO (1991), with a recommended seismic coefficient of 0.075 g. In addition, the United States Geological Society (USGS) National Seismic Hazard Mapping Project estimates a peak horizontal acceleration of 0.09 g to have a 2% exceedance in 50 years (2,475-year return period).

Site area seismicity was reviewed from local publications, National Earthquake Information Center (NEIC) earthquake database search, and local geomorphic structure information. Annual seismology data in Oklahoma is compiled by the Oklahoma Geological Survey (Lawson and Luza, 1983, and Luza and Lawson, 1993). This data shows activity of low magnitude, with epicenters primarily in the central and south-central portion of the state. The potential for seismic accelerations at the site was evaluated by considering 1) historical earthquake events, 2) capable faults in the area, and 3) probabilistic analysis of earthquake events not associated with known faults.

A review of recorded or documented seismic activity within a 300-mile radius of the site was conducted from data compiled by the National Earthquake Information Center (NEIC) of the U.S. Geological Survey. The data was compiled from prior to 1811 through April 2003. The results were compared with data published by the Oklahoma Geological Survey from 1900 to 1998 compiled in Lawson and others (1979), Luza and Lawson (1993), and subsequent annual Oklahoma Earthquake Catalog publications. The events that produced the greatest vibratory ground motions at the site, using attenuation relationships developed by Atkinson and Boore (1995), are summarized in Table F.2.

Table F.2   Summary of Seismic Events Producing Accelerations

Rank	Date	Richter Magnitude	Distance from Site		Peak Ground Acceleration (g)	Location
			(mi)	(km)		
1	Jun 20, 1926	4.2	12	19	0.061	Sequoyah County, OK
2	Oct 22, 1882	5.5	116	186	0.013	South-central OK
3	Apr 27, 1961	4.1	43	69	0.013	South-eastern OK
4	May 2, 1969	4.6	71	114	0.012	Central OK
5	Oct 8, 1915	3.4	22	36	0.011	Central-eastern OK
6	Mar 31, 1975	2.9	14	22	0.010	Central-eastern OK
7	Jan 11, 1961	3.8	48	77	0.008	South-eastern OK
8	Dec 16, 1811	7.2	263	424	0.008	New Madrid MO, a.m.
9	Oct 30, 1956	4.0	63	101	0.007	North-eastern OK
10	Dec 16, 1811	7.0	263	424	0.007	New Madrid MO, p.m.
11	Jun 1, 1939	4.3	82	132	0.007	Central OK
12	Jun 2, 1977	4.3	83	133	0.007	Central-western AR
13	Sep 6, 1997	4.5	96	155	0.007	South-eastern OK

Capable faults. Based on geologic investigations, only two documented faults of Quaternary age, the Meers fault and the Humboldt fault zone, are located within 200 miles of the site. The Meers fault, also referred to as the Thomas fault and the Meers Valley fault, is located in southwestern Oklahoma in the Frontal Wichita fault system that is the boundary between the Anadarko basin and the Wichita Mountains. It is the only significant fault within a 200-mile radius of the site with positive documentation of Quaternary tectonic movement. The fault is approximately 54 km (34 miles) long, with the closest section of the fault approximately 306 km (190 miles) from the site. Based on the length of fault, the maximum credible earthquake (MCE) associated with the Meers fault is approximately Richter magnitude 7.2.

The Humboldt fault zone is a north-northeasterly trending complex set of faults that bound the eastern margin of the Nemaha uplift in Nebraska, Kansas, and Oklahoma. The fault zone and the adjacent uplift are known based on drill-hole data from the region. Because the faults are only known from subsurface data, details of the fault slip and fault patterns are limited. Although convincing surficial evidence of

large, prehistoric earthquakes is absent in the area, a regional seismograph network indicates that the structures are currently tectonically active. Based on the length of the fault segments in the Humboldt fault zone, Steepes and others (1990) suggest that infrequent magnitude 6 or greater earthquakes could occur. The nearest part of the fault zone to the site is close to Oklahoma City, approximately 140 miles from the site.

Site investigations performed in 1998 concluded that there are no active faults within 5 miles of the site (Van Arsdale, 1998). In addition, other geologic investigations (Shannon and Wilson, 1975) conducted for the Black Fox Nuclear Power Plant concluded that faults within the tectonic provinces surrounding the site have no evidence of having been active since before Cretaceous time. Although there is no evidence of active faults within 200 miles of the site, specific documentation of every fault meeting the length requirements of 10 CFR 100 Appendix A was not found. Instead, faults that (if considered active) have the capability of producing peak ground accelerations at the site of greater than 0.27 g were considered further. Of these 23 faults, positive documentation was found (Van Arsdale, 1998, Shannon and Wilson, 1975) to show that these faults are not considered capable faults per the criteria set forth in 10 CFR 100.

Random Earthquake Analysis. Two evaluations of the random earthquake event were used to determine the design event for earthquakes not associated with identifiable faults, as is the case for most U.S. earthquakes east of the Rocky Mountains. In both approaches, tectonic provinces are established to group regions with similar seismological characteristics. It is assumed that the spatial distribution of earthquakes is uniform across the province. Within the province, historical data of earthquake events are evaluated and magnitude-frequency plots are generated. The first evaluation of the random earthquake used a semi-probabilistic method. Various areas surrounding the site were modeled as generating a 10,000-year event. The event was applied at the mean distance of the area from the site, and attenuated to the site using Atkinson and Boore (1995) relationships. The peak acceleration associated with the 10,000-year event was estimated to be 0.27 g.

In addition, another analysis using the U.S. Bureau of Reclamation (USBR) program *mrs* (LaForge, 2005) was conducted. The USBR approach is a probabilistic approach that incorporates the probability of seismic magnitude, location of event, and attenuation characteristics. Results from this analysis are considered more rigorous for two reasons. First, it estimates the 10,000-year acceleration at the site, while the semi-probabilistic model estimates the 10,000-year earthquake, and then attenuates the ground motion to the site. Second, the mrs program is able to incorporate variability in the attenuation relationships and the magnitude-frequency relationships. The semi-probabilistic model uses the mean

values for these parameters. The results from the *mrs* analysis estimate that the mean peak acceleration associated with a 10,000-year event is 0.16 g.

Seismic coefficient. For materials that do not liquefy or lose shear strength with seismic shaking, seismic slope stability is analyzed by a pseudo-static approach. This consists of the application of an equivalent horizontal acceleration or seismic coefficient to the structure being analyzed (described in Seed, 1979). The seismic coefficient represents an inertial force due to strong ground motions during the design earthquake, and is represented as a fraction of the maximum expected seismic acceleration at the site (typically at the base of the structure). The coefficient for calculating seismic coefficient is typically 0.5 to 0.7 of the maximum expected acceleration. The 0.5 value typically represents operational conditions (a relatively short period of time), and the 0.7 value represents post-reclamation conditions (a relatively long period of time). This strategy has been adopted in review of uranium tailings facility design and documented in DOE (1989). The following table summarizes the peak horizontal accelerations and associated seismic coefficients resulting from the various seismic sources considered.

Table F.3   Summary of Seismic Events Producing Accelerations at Site

Seismic Source	Peak Horizontal Acceleration	Seismic Coefficient (g)
Historical earthquake causing largest accelerations at site (June 20, 1926 Sequoyah County Earthquake, magnitude 4.2)	0.061	0.04
Maximum credible earthquake associated with known active Meers fault	0.019	0.01
Maximum credible earthquake associated with known active Humboldt zone	0.017	0.01
Maximum credible earthquake associated with all other faults meeting length requirements of 10 CFR 100, Appendix A that are lacking positive documentation to demonstrate they are not active faults	0.27	0.18
Maximum credible earthquake associated with active faults per Black Fox report	0.00	0.00
10,000-year random earthquake event using semi-probabilistic approach	0.27	0.18
10,000-year random earthquake event using *mrs* probabilistic approach	0.16	0.11

From the data summarized above, the peak anticipated horizontal acceleration at the site is conservatively estimated to be 0.27 g, with a corresponding seismic coefficient of 0.18 g. The results of the pseudo-static analyses are summarized below.

F.5   STABILITY OF NATURAL SLOPES

The stability of natural slopes downslope of the disposal cell was analyzed to evaluate the impact of the disposal cell on these slopes.

The natural slopes in the area are less than 10 percent and covered by a thin veneer of terrace materials. As shown in Figures 7-2 through 7-8 of the Hydrogeological and Geochemical Site Characterization Report (SMI, 2001), the slopes have less than 10 feet of alluvial soils. Because of this relatively thin layer of alluvial soils, the construction of the disposal cell has only a small impact on the stability of the native slopes. Borrow material investigation reports on the south borrow area include HC (1980), PSI (1990) and GGH (1997). This borrow material is typical of the clay soils found in the area. Laboratory testing data indicate the plasticity index varies from approximately 10 to 30 percent. Correlations between plasticity index and shear strength of normally consolidated clays indicates the average shear strength is between 28 and 32 degrees, and plus or minus one standard deviation shear strengths are between 25 and 38 degrees (Bowles, 1988, and Lambe and Whitman, 1969). Stability analyses of the natural slopes have been evaluated by looking at a Section CS-1 cut through the west slope of the disposal cell. This section goes through both the longest slope of the disposal cell cover and the head of Stream 005. Although in the previous analyses, the foundation materials were modeled with an angle of internal friction of 30 degrees, for the specific analysis of the stability of the natural slopes, the alluvium was conservatively modeled as having shear strength of 25 degrees.

F.6   LIQUEFACTION ANALYSIS

The potential for liquefaction of the foundation underlying the disposal cell was evaluated using procedures as outlined in Youd et al. (2001). Vertical profiles were evaluated for sections containing both the maximum and minimum height of disposed material, and variable subgrade materials.

Current conditions under the disposal cell footprint consist of approximately 10 feet of alluvial soils overlying bedrock. It is likely much of the alluvial soils will be removed during soil clean up. Excavated areas will be brought to subgrade elevations by compacting granular material. Therefore, the subgrade is assumed to consist of either ten feet of alluvium, or ten feet of compacted granular material. The

reclamation plan stipulates the bottom of the clay liner be a minimum of 2 feet above the maximum observed groundwater table. Therefore, the groundwater table is conservatively assumed to occur eight feet above the base of bedrock.

Profile 1 represents the maximum height in the disposal cell. From the top of cover down, the layers are as follows: 10 feet of compacted cover, 25 feet of disposed material, 3 feet of compacted clay liner, 2 feet of unsaturated native alluvium, and 8 feet of saturated native alluvium.

Profile 2 represents a minimum height of in the disposal cell along the side slope of the disposal cell where the disposed material pinches out. From the top of cover down, the layers are as follows: 10 feet of compacted cover, 3 feet of compacted clay liner, 2 feet of unsaturated native alluvium, and 8 feet of saturated native alluvium.

Profiles 3 and 4 are the same as Profiles 1 and 2, respectively, with the exception that it is assumed all native alluvium has been replaced with relatively clean, compacted granular material.

Material properties in Table F.4 are assumed typical values. Alluvium soils from geotechnical investigations (HC 1980, PSI 1990, and GGH 1997) are described as being low-plasticity clayey soils with a minimum of 25 percent fines and plasticity indexes between approximately 10 and 30 percent.

Table F.4   Material Properties and Thicknesses

Material	Bulk Density	Profile 1	Profile 2	Profile 3	Profile 4
Cover	110	10 feet	10 feet	10 feet	10 feet
Disposed Material	120	25 feet	0 feet	25 feet	0 feet
Clay Liner	115	3 feet	3 feet	3 feet	3 feet
Alluvium	105	10 feet	10 feet	0 feet	0 feet
Granular Backfill	112	0 feet	0 feet	10 feet	10 feet

The factor of safety against liquefaction is given by the following equation

$$FS = \frac{CRR_{7.5}}{CSR} MSF$$

Where:

CRR = Cyclic Resistance Ratio,
CSR = Cyclic Stress Ratio, and
MSF = Magnitude Scaling Factor.

CRR values can be correlated to various field parameters of the foundation materials, including standard penetration tests, cone penetration, and shear-wave velocities. No data has been collected regarding the density of the alluvial soils, therefore a very conservative SPT corrected blow counts of approximately 3 is assumed. This corresponds to a CRR of 0.1. The compacted granular material is assumed to be medium to dense, with a SPT corrected blow count of greater than 15, corresponding to a CRR of 0.17.

CSR is calculated by the following equation:

$$CSR = 0.65 * \frac{a_{max}}{g} * \frac{\sigma_{vo}}{\sigma'_{vo}} * r_d$$

Where:

a_{max}/g = ratio of the peak horizontal acceleration to the acceleration of gravity,
σ_{vo} and σ'_{vo} = total and effective vertical overburden stresses, respectively, and
r_d = stress reduction coefficient.

MSF is a scaling factor to account for scaled effects of earthquakes of magnitudes different than 7.5 and can be estimated by the following equation:

$$MSF = \frac{10^{2.24}}{M_w^{2.56}}.$$

The design earthquake is for a magnitude 4.4 event occurring 5.7 km from the site.

Table F.5 lists the various parameters calculated for the different profiles.

Table F.5   Parameters Used to Evaluate Liquefaction Potential

Parameter	Profile 1	Profile 2	Profile 3	Profile 4
a_{max}/g	0.27	0.27	0.27	0.27
σ_{vo} (pcf)	5495	2195	5565	2565
σ'_{vo} (pcf)	4996	1996	5066	2066
r_d	0.90	0.95	0.90	0.95
CSR	0.174	0.183	0.174	0.207
$CRR_{7.5}$	0.1	0.1	0.17	0.17
MSF	3.91	3.91	3.91	3.91
FS	2.25	2.14	3.82	3.21

Even using very conservative numbers, the potential for liquefaction of the foundation materials underlying the disposal cell is minimal.

F.7   DISCUSSION OF ANALYSIS RESULTS

The results of stability analyses for the critical cross-section and the infinite slope analyses are presented in Table F.6. These values represent the lowest calculated factor of safety from a number of individual failure surfaces. All calculated factors of safety were significantly above the NRC recommended values of 1.5 for static and 1.1 for pseudo-static analysis. In addition, potential for liquefaction of the foundation material is minimal. SLOPE/W input and output for each scenario are presented in Attachment F.1.

Table F.6   Stability Analysis Result Summary

Condition	Circular Failure Surface	Block Specified Wedge Failure Surface	Infinite Slope Block Failure Surface	Minimum Factor of Safety Criteria
Static	2.98	2.97	2.66	1.5
Pseudo-static	1.51	1.46	1.35	1.1
Static with two feet of water above synthetic liner	---	2.59	---	1.5
Pseudo-static with two feet of water above synthetic liner	---	1.30	---	1.1
Natural Slopes, Static	2.48			1.5
Natural Slopes, Pseudostatic	1.33			1.1

F.8   REFERENCES

Algermissen, S.T., D.M. Perkins, P.C. Thenhaus, S.L. Hanson, and B.L. Bender, 1982. "Probabilistic Estimates of Maximum Acceleration and Velocity in Rock in the Contiguous United States," *USGS Open-File Report 82-1033.*

Atkinson, G.M., and Boore, D.M., 1995. Ground-Motion Relations for Eastern North America, Bulletin of the Seismological Society of America, Vol. 85, No. 1, pp 17-30, February.

Earth Science Consultants, Inc. (ESCI), 1998. "Calculation Brief, RADON Analysis, Case I and Case II Scenarios, Sequoyah Fuels Corporation, Gore, Oklahoma, Project No. 4881-04." Prepared for SFC, December 9.

GEO-SLOPE International Ltd, 2001. *Slope/W, Version 5, User's Guide,* Calgary, Alberta.

Holtz, Robert D. and William D. Kovacs, 1981. *Introduction to Geotechnical Engineering,* Prentice-Hall, Inc., Englewood Cliffs, NJ.

International Commission on Large Dams (ICOLD), Committee on Seismic Aspects of Dam Design, 1989. "Selecting Seismic Parameters for Large Dams." *Bulletin 72,* ICOLD, Paris.

International Conference of Building Officials (ICBO), 1991. *Uniform Building Code,* 1991 Edition.

LaForge, R.C. 2005. Probabilistic Hazard Curves for Peak horizontal Acceleration, Sequoyah Fuels Site, Oklahoma. Submitted to MFG, Inc. April.

Lambe, T. William and Robert V. Whitman, 1969. *Soil Mechanics*, Massachusetts Institute of Technology, John Wiley & Sons, NY.

Lawson, J.E., and K.V. Luza, 1983. "Oklahoma Earthquakes 1983." *Oklahoma Geology Notes*, Oklahoma Geological Survey, University of Oklahoma, pp. 32-42.

Lawson, J.E., K.V. Luza, R.L. DuBois, and P.H. Foster, 1979. "Inventory, Detection, and Catalog of Oklahoma Earthquakes." *Oklahoma Geological Survey*. University of Oklahoma.

Luza, K.V. and J.E. Lawson, 1993. "Oklahoma Seismic Network." *NUREG/CR-6034*, U.S. Nuclear Regulatory Commission, July.

Makdisi, F.I. and H.B. Seed, 1978. "Simplified Procedures for Estimating Dam and Embankment Earthquake-Induced Deformations." *Journal of the Geotechnical Engineering Division*, ASCE, Vol. GT7, pp. 849-867, April.

Ramelli, A.R., D.B. Slemmons, and S.J. Brocoum, 1987. "The Meers Fault: Tectonic Activity in Southwestern Oklahoma." *NUREG/CR-4582*, U.S. Nuclear Regulatory Commission, March.

Schnabel, P.B. and H.B. Seed, 1973. "Acceleration in Rock for Earthquakes in the Western United States." *Bulletin of the Seismological Society of America*, vol. 63, no. 2, pp. 501-516. April.

Seed, H.B., 1979. "Considerations in the Earthquake-Resistant Design of Earth and Rockfill Dams," *Geotechnique*, Vol. 29, No. 3, pp 215-263.

Sequoyah Fuels Corporation (SFC), 1998. "Site Characterization Report." Prepared December 15.

Shannon and Wilson, Inc., 1975. Geotechnical Investigations, Black Fox Station, Rogers County, Oklahoma. To Black and Veach for Public Service Company of Oklahoma. April.

Steepes, D.W., Bennett, B.C., Park,, C., Miller, R.D., and Knapp, R.W., 1990. Microearthquakes in Kansas and Nebraska 1977-1989. U.S. Nuclear Regulatory Commission, NUREG/CR-5629 RA.

Strachan, C., and D. van Zyl, 1988. "Leach Pads and Liners," *In Introduction to Evaluation, Design and Operation of Precious Metal Heap Leaching Projects*, Society of Mining Engineers, Littleton, CO.

Trifunac, M.D., and A.G. Brady, 1976. "Correlations of Peak Acceleration, Velocity, and Displacement with Earthquake Magnitude, Distance, and Site Conditions." *Earthquake Engineering and Structural Dynamics*, vol. 4, pp. 455-471.

U.S. Army Corps of Engineers (COE), 1982. "Engineering and Design Stability for Earth and Rockfill Dams," *EM 1110-2-1902*.

U.S. Department of Energy (DOE), 1989. "Technical Approach Document: Revision II, Uranium Mill Tailings Remedial Action Project," U.S. Department of Energy, Washington D.C.

U.S. Nuclear Regulatory Commission (NRC), 1998. "Sequoyah Fuels Corporation (SFC) Site Evaluation of Faults and Faulting: Input to Safety Evaluation Report." Note to James Shepherd, Sequoyah Fuels Corporation, from Philip Justus, NRC, December 3.

U.S. Nuclear Regulatory Commission (NRC), 2000. "Standard Review Plan for the Review of a Reclamation Plan for the Mill Tailings Sites Under Title II of the Uranium Mill Tailings Radiation Control Act." *NUREG-1620*. Division of Waste Management, May.

Van Arsdale, R. 1998. Paleoseismologic Analysis of the Carlile Fault in Sequoyah County, Oklahoma. Report prepared for Sequoyah Fuels Corporation (SFC), March 6, 1998.

Youd, TL, Idriss, IM, Andrus, RD, Arango, I., Castro, G., Christian, JT, Dobry, R., Finn, WD, Harder, L, Hynes, ME, Ishihara, K, Koester, J, Liao, S, Marcuson, W, Martin, G, Mitchell, J, Moriwaki, Y, Power, M, Robertson, P, Seed, R, and Stokoe, K., (2001). Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Geoenvironmental Engineering, Vol, 127, No. 10, October.
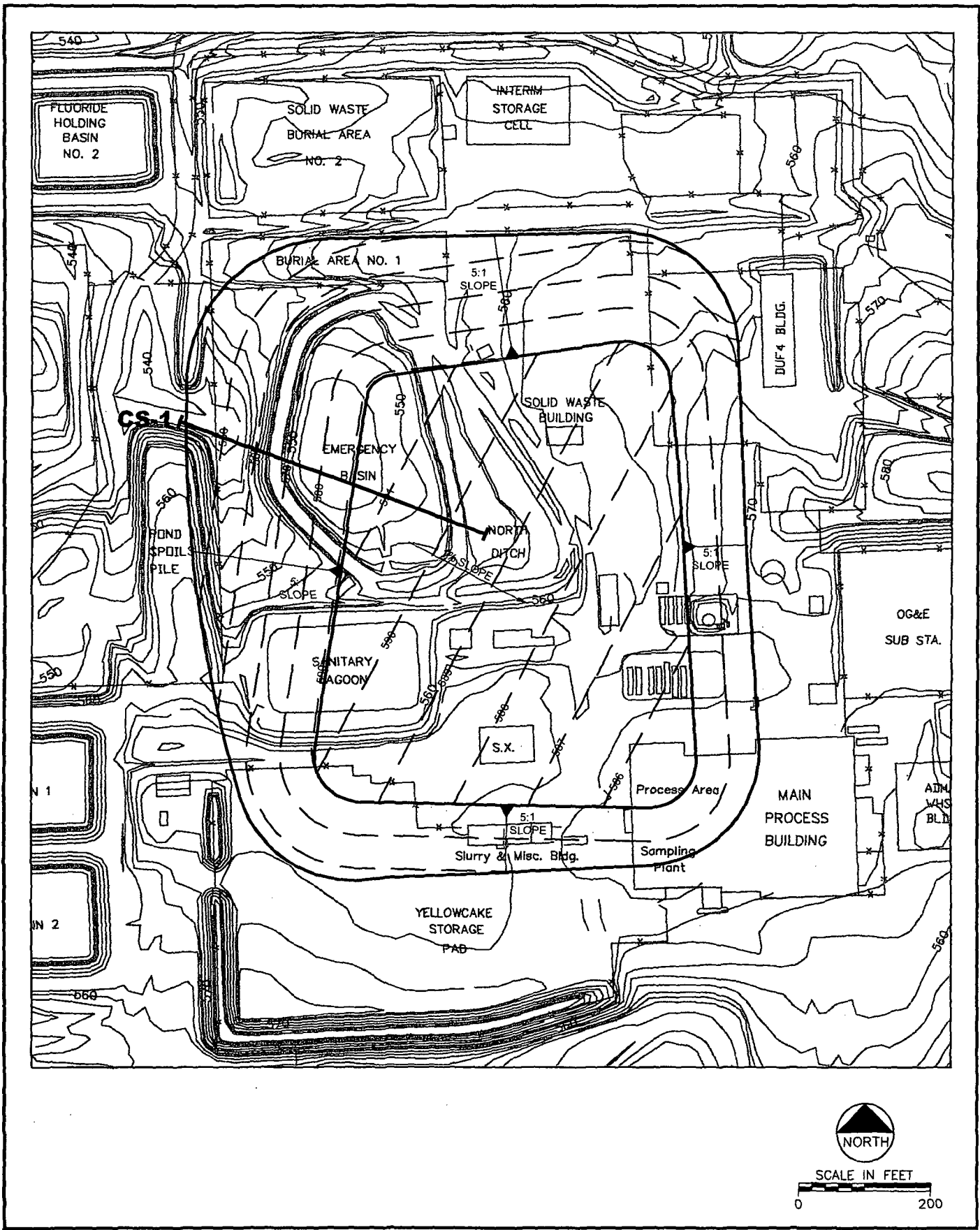
MFG, Inc. *consulting scientists and engineers*	FIGURE F.1 CRITICAL CROSS-SECTION LOCATION	Date: APRIL 2005
		Project: 100734
		File: XSEC-LOC-1.DWG

Elevation (feet) vs **Distance (feet)**

Cover Soil

Textured HDPE Liner

Contaminated Materials and Soil

Foundation Soils

MFG, Inc.
consulting scientists and engineers
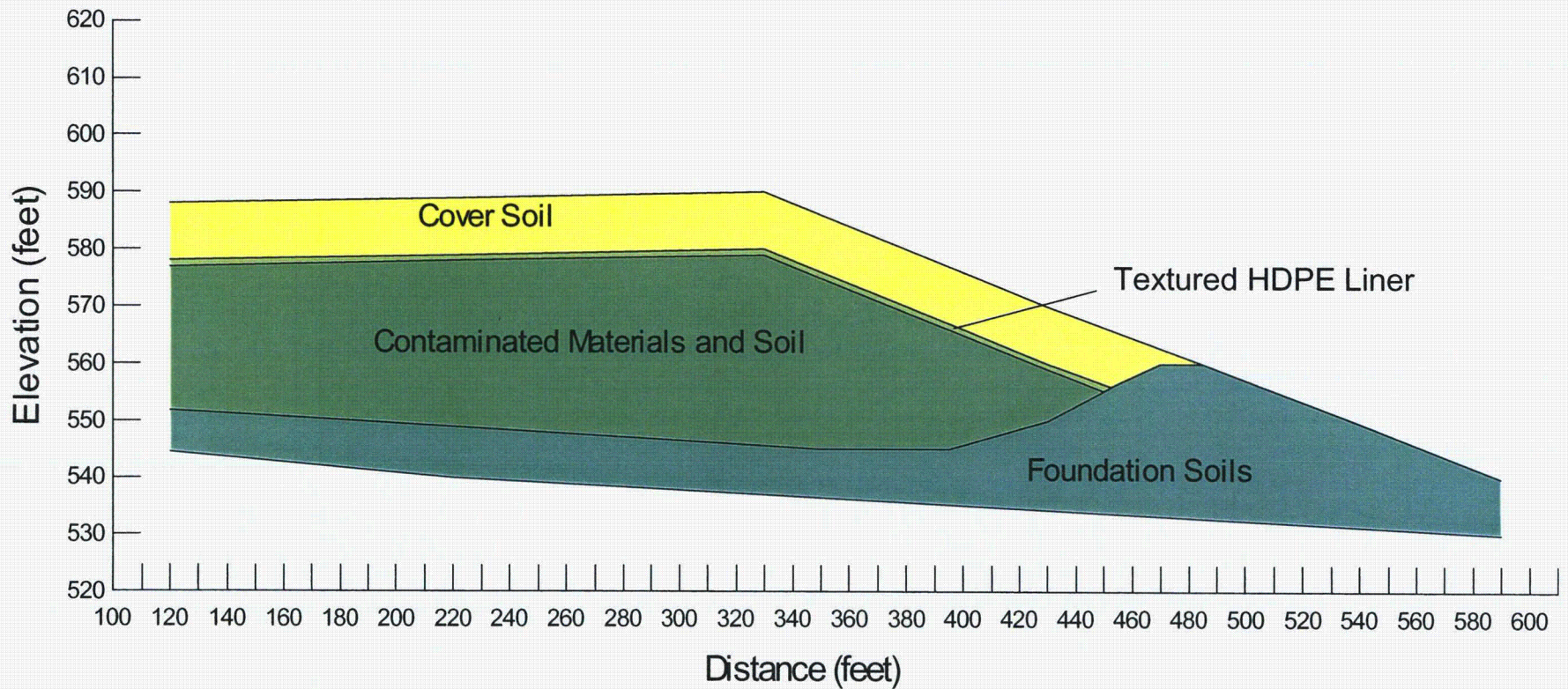
FIGURE F.2
Critical Cross-Section 1 (CS-1) Geometry

|	
Date:	April 2005
Project:	P:\100734 SFC\
File:	DIspCD\stability\FIG-F2.doc

ATTACHMENT F.1

SLOPE/W INPUT AND OUTPUT

Description: Sequoyah Fuels
Comments: Disposal Cell - Critical Section 1
File Name: Seq1blockseismic0.18textured.gsz
Last Saved Date: 4/25/2005
Analysis Method: Spencer
Slip Surface Option: Block
Seismic Coefficient: horz: 0.18, vert: 0

1.46
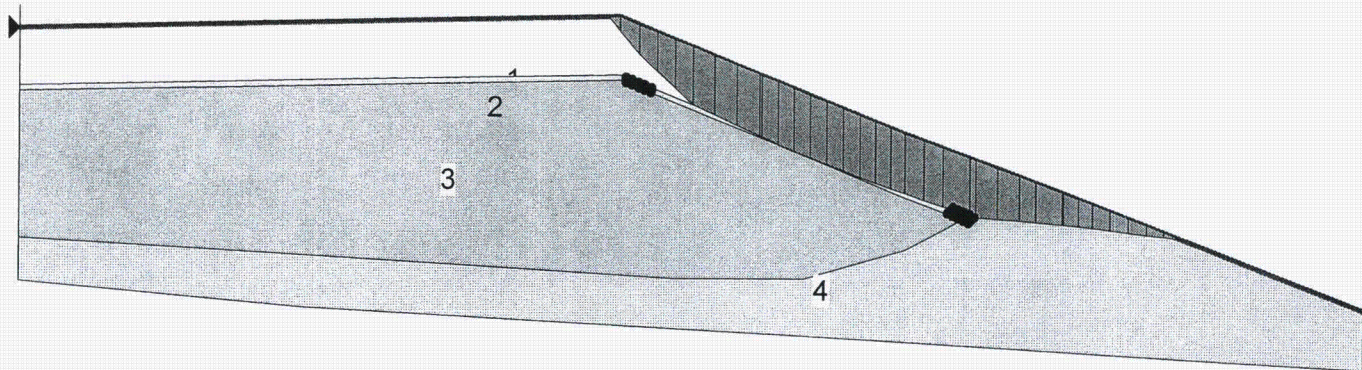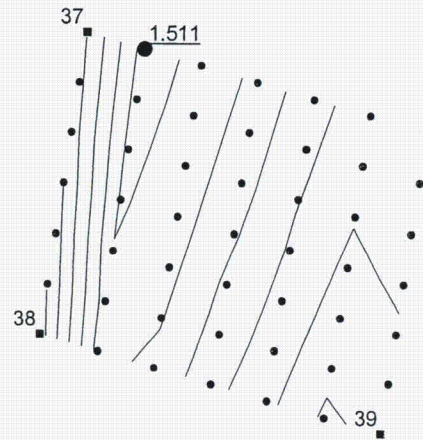
Description: Sequoyah Fuels
Comments: Disposal Cell - Critical Section 1
File Name: Seq1circularseismic0.18textured.gsz
Last Saved Date: 4/25/2005
Analysis Method: Spencer
Slip Surface Option: GridAndRadius
Seismic Coefficient: horz: 0.18, vert: 0

1.511

Description: Sequoyah Fuels
Comments: Disposal Cell - Critical Section 1
File Name: Seq1blockstatictextured.gsz
Last Saved Date: 4/25/2005
Analysis Method: Spencer
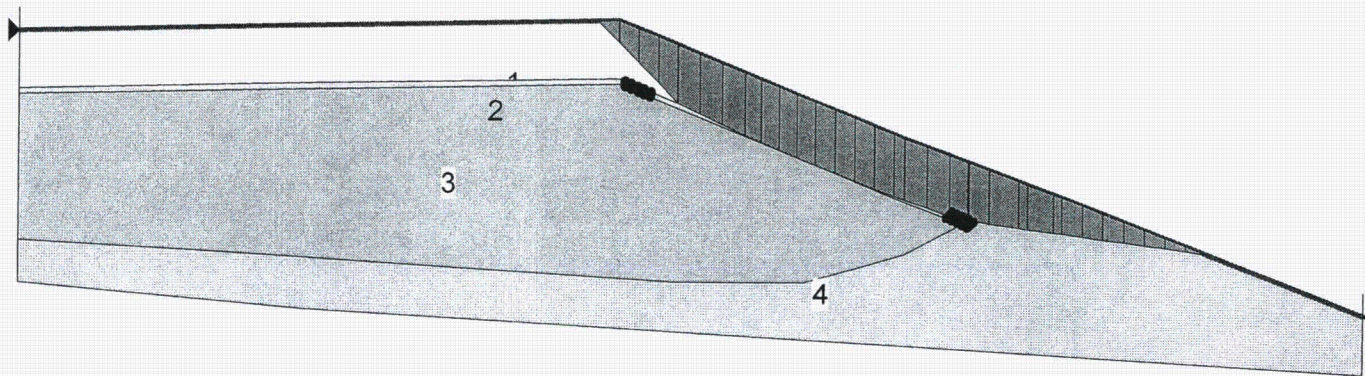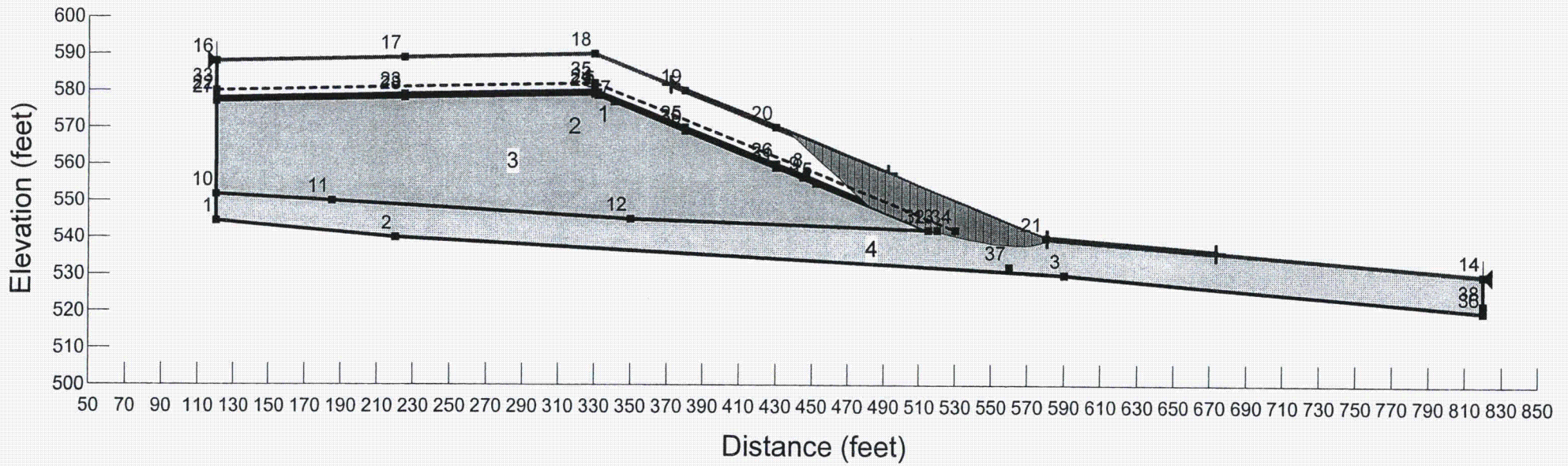Slip Surface Option: Block
Seismic Coefficient: horz: 0, vert: 0
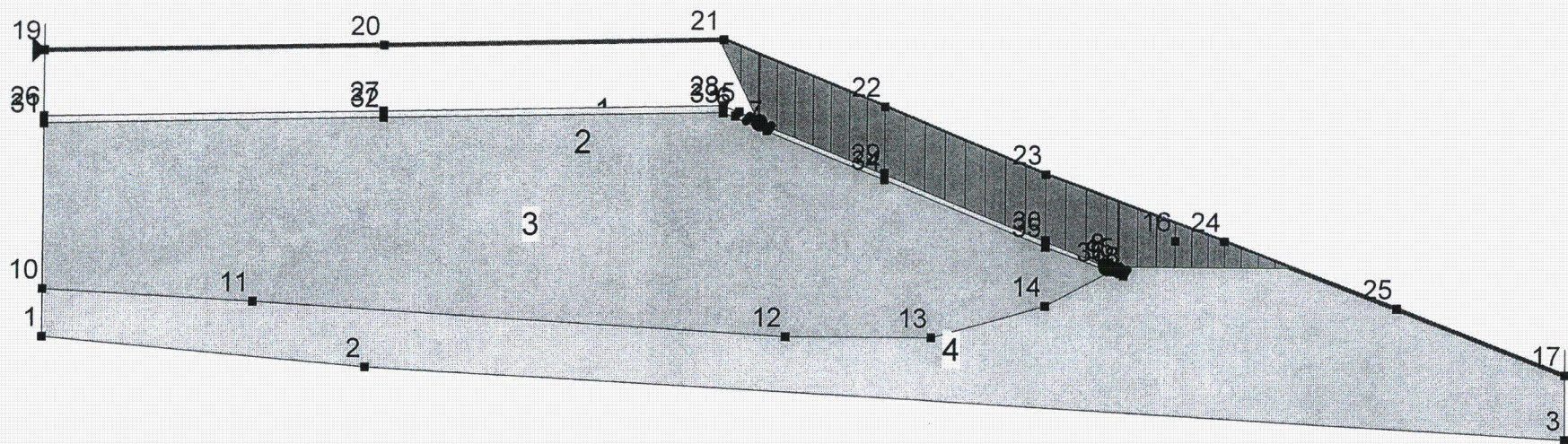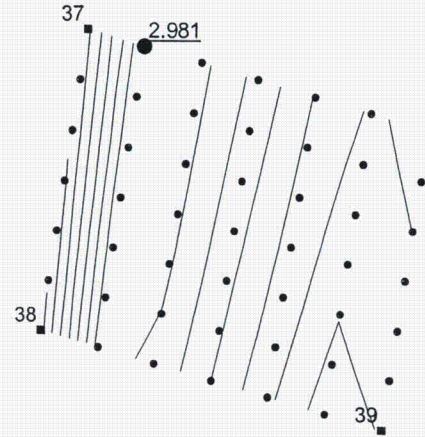
Description: Sequoyah Fuels
Comments: Disposal Cell - Critical Section 1
File Name: Seq1blockstatictextured.gsz
Last Saved Date: 4/25/2005
Analysis Method: Spencer
Slip Surface Option: GridAndRadius
Seismic Coefficient: horz: 0, vert: 0

Description: Sequoyah Fuels
Comments: Disposal Cell - Critical Section 1
File Name: Seq1blockseismic0.18texturedwater.gsz
Last Saved Date: 4/25/2005
Analysis Method: Spencer
Slip Surface Option: Block
Seismic Coefficient: horz: 0.18, vert: 0

1.295
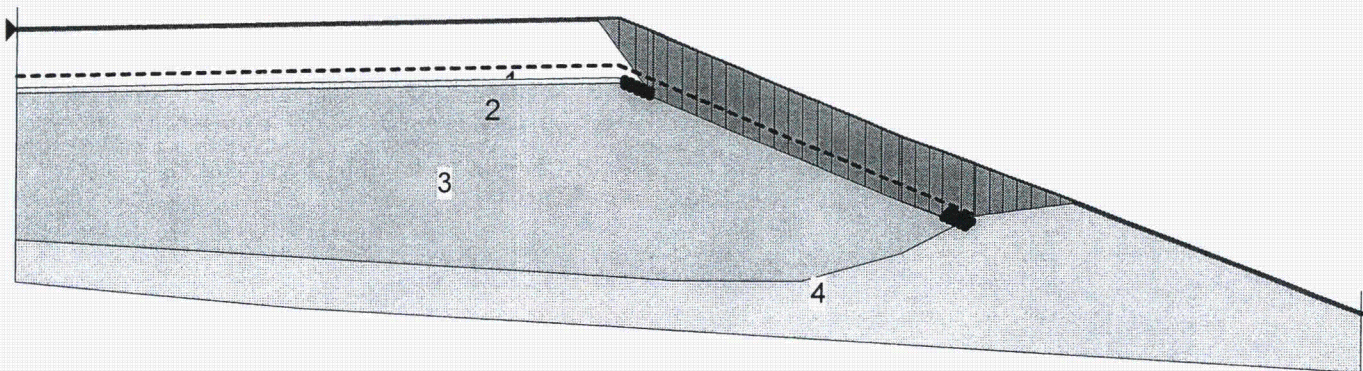
Description: Sequoyah Fuels
Comments: Disposal Cell - Critical Section 1
File Name: Seq1blockstaticwater.gsz
Last Saved Date: 4/25/2005
Analysis Method: Spencer
Slip Surface Option: Block
Seismic Coefficient: horz: 0, vert: 0
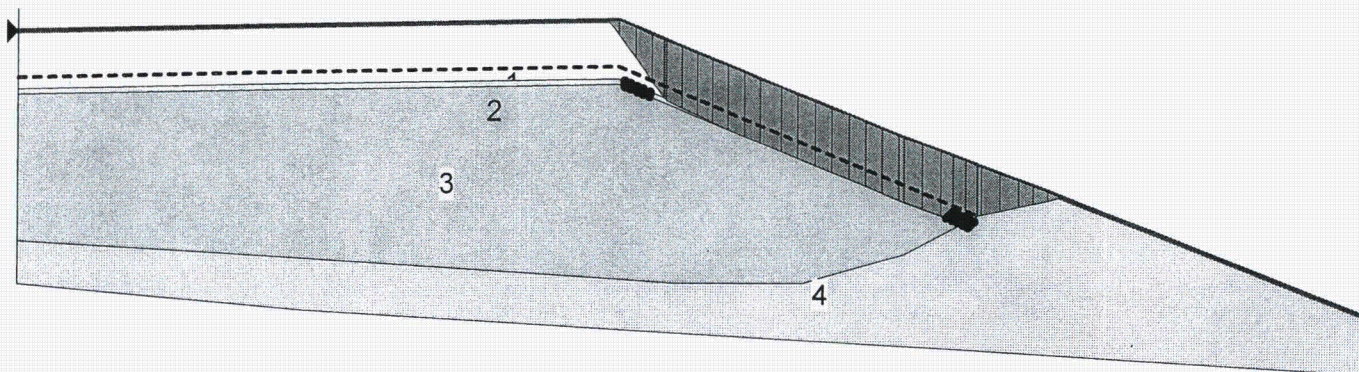
Description: Sequoyah Fuels
Comments: Disposal Cell - Critical Section 1
File Name: Seq1blockseismicyieldtextured.gsz
Last Saved Date: 4/25/2005
Analysis Method: Spencer
Slip Surface Option: Block
Seismic Coefficient: horz: 0.29, vert: 0

1.11

Description: Sequoyah Fuels
Comments: Disposal Cell - Critical Section 1
File Name: Seq1nativeslopesstatic.gsz
Last Saved Date: 12/13/2005
Analysis Method: Spencer
Slip Surface Option: EntryAndExit
Seismic Coefficient: horz: 0, vert: 0

Description: Sequoyah Fuels
Comments: Disposal Cell - Critical Section 1
File Name: Seq1nativeslopesseismic0.18.gsz
Last Saved Date: 12/13/2005
Analysis Method: Spencer
Slip Surface Option: EntryAndExit
Seismic Coefficient: horz: 0.18, vert: 0
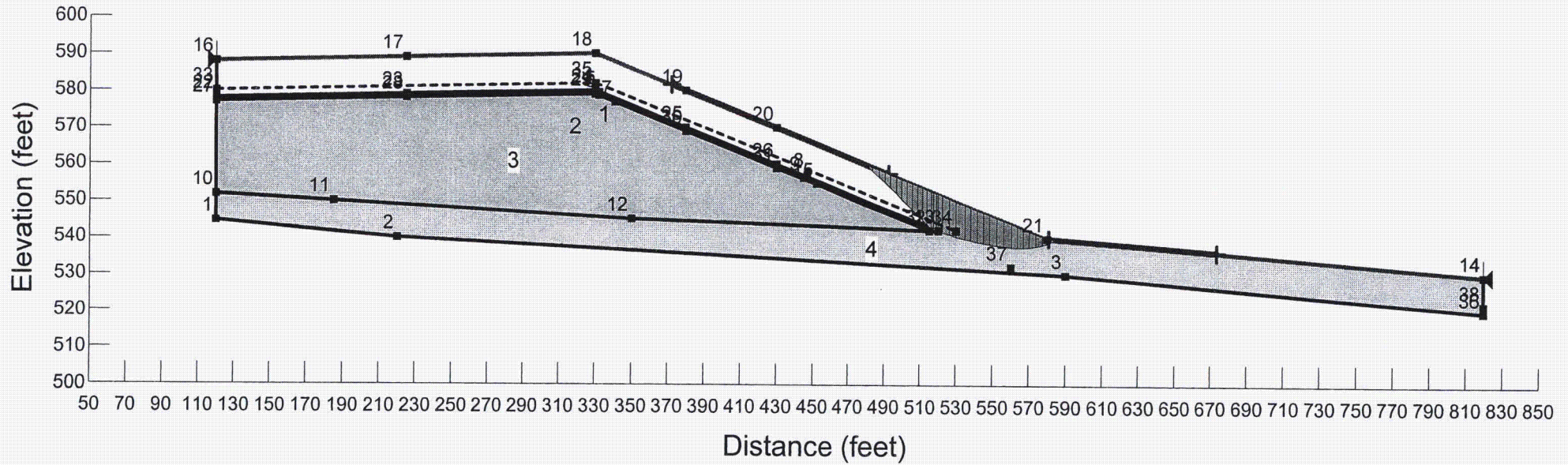
ATTACHMENT F.2

INFINITE SLOPE ANALYSES

Problem: Calculate the Factor of Safety of the cover system assuming infinite slope failure. Also calculate the maximum acceleration (assuming no liner/cover interface strength loss with shaking) allowable to maintain a minimum factor of safety of 1.1. Analyze slope perpendicular to cover at 5H:1V. Critical surface is interface between textured geomembrane and compacted clay liner. Average properties of borrow material for clay liner are CL material with LL = 36, PL = 19 and PI = 17. Assume that under moderate loading conditions (8 feet of cover soil), textured membrane will force failure into clay. Therefore, use typical friction angle of CL material as liner/cover interface strength of 28°.

Solution: Use the following equation

$$FS = \frac{\tan(\phi)}{\tan[\beta + \arctan(k_h)]}$$

where FS= Factor of Safety
ϕ= friction angle of textured liner/compacted clay interface= 28°
β= slope angle of cover=arctan (1/5)
k_h=horizontal seismic coefficient (g)

For static conditions, k_h=0.0 g:

$$FS = \frac{\tan(28)}{\tan\left[\arctan(\frac{1}{5}) + \arctan(0.0)\right]} = 2.66$$

For k_h=0.11g (70 percent of peak horizontal acceleration from LaForge, 2005):

$$FS = \frac{\tan(28)}{\tan\left[\arctan(\frac{1}{5}) + \arctan(0.11)\right]} = 1.68$$

For k_h=0.18g (70 percent of peak horizontal acceleration = 0.27 g):

$$FS = \frac{\tan(28)}{\tan\left[\arctan(\frac{1}{5}) + \arctan(0.18)\right]} = 1.35$$

Calculate maximum acceleration to maintain minimum FS of 1.1 assuming 100% liner/cover interface strength:

$$1.1 = \frac{\tan(28)}{\tan\left[\arctan(\frac{1}{5}) + \arctan(k_h)\right]}$$

k_h=0.27 g (i.e. peak horizontal acceleration = 0.40 g)

Calculate yield acceleration assuming 80% liner/cover interface strength:

$$1.1 = \frac{\tan(23)}{\tan\left[\arctan(\frac{1}{5}) + \arctan(k_h)\right]}$$

k_h=0.20 g (i.e. peak horizontal acceleration = 0.30 g)

References:
LaForge, Roland (2005). Probabilistic Hazard Curves for Peak Horizontal Acceleration, Sequoyah Fuels Site, Oklahoma. Submitted to MFG, Inc. April.