IUC

INTERNATIONAL URANIUM (USA) CORPORATION

June 30, 2006

VIA E-MAIL AND OVERNIGHT DELIVERY

Mr. Dane L. Finerfrock
Director
Division of Radiation Control
Department of Environmental Quality
168 North 1950 West
P.O Box 144850
Salt Lake City, UT 84114-4850

Re: Cell 4A Lining System Design Report, Response to DRC Request for Additional Information – Round 2 Interrogatory, Cell 4A Design.

Dear Mr. Finerfrock:

We are responding to your June 14, 2006 letter, requesting additional information following on the Cell 4A Lining System Design.

For ease of review, the Division of Radiation Control's ("DRC's") questions are repeated below in italics with International Uranium (USA) Corporation's ("IUSA's") responses following each question.

IUSA has previously responded to questions 2, 3, 4, 9, 11, 14, and 15.

1. *__Radiation Survey Report and Demonstration__*

IUSA has informally submitted revised cleanup and verification procedures to DRC and followed up with a teleconference on June 19, 2006, and a meeting at DRC offices on June 20th to work through the critical issues for final verification. IUSA and DRC have agreed on cleanup criteria of 5/15 pCi per gram Ra_{226} plus U_{NAT} of 30 pCi per gram in soil. IUSA will submit, under separate cover, justification for sampling frequency based on categories of low, medium and high possibilities for presence of residual contamination in the Cell 4A area.

5. __*Liner System Chemical Resistance*__ *– quantitative evaluation that addresses the long-term resistance of all the liner system components to the tailings cell solution, or the results of liner compatibility studies to demonstrate the long-term resistance of the liner materials.*

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Due to its excellent resistance to degradation by a wide range of chemicals, among other factors, HDPE geomembrane is the most widely used type of geomembrane. The reaction of geomembranes to chemicals has probably been studied more than any other liner degradation mechanism (Koerner et al., 1990). In accelerated chemical compatibility testing of geomembranes conducted in the laboratory and in field investigations of geomembranes that have been installed as long as several decades, polyethylene geomembranes have been found to have good resistance to a wide variety of chemicals, including aliphatic and aromatic hydrocarbons, chlorinated and oxygenated solvents, crude petroleum solvents, alcohols, organic and inorganic acids, heavy metals, and salts (Matrecon, Inc., 1988; Brady et al., 1994; Koerner, et. al., 1990; Koerner, 1999; Hsuan et al., 1991; Eith and Koerner, 1998; Koerner and Hsuan, 2002), which is why HDPE is commonly used for containing pure chemicals in laboratory bottles. Leachate containing a relatively large amount of organic solvent can lead to an increase in the rate of oxidation of an HDPE geomembrane (Koerner and Hsuan, 2002). However, this is not an issue for the White Mesa Mill, as synthetic organic chemicals are found at only trace amounts.

GCLs contain clay minerals that may react with certain chemicals. The clay minerals in GCLs are primarily montmorillonite, a mineral that has a high swelling capacity, which provides for chemical reactivity and attenuation. A number of researchers have addressed the issue of GCL compatibility with leachate and leachate constituents (Shan and Daniel, 1991; Rad et al., 1994; Ruhl and Daniel, 1997; Petrov et al., 1997; Thiel and Criley, 2005). They found that the hydraulic conductivity of a GCL is highly dependent on the hydrating liquid and the applied effective stress during permeation. GCLs that are hydrated with water and subjected to confining stress do not exhibit large increases in hydraulic conductivity when permeated with organic constituents, unless the permeating solution is a pure organic liquid with a low dielectric constant (e.g., acetone).

6. <u>Additional GCL Data</u> – *that the GCL will resist damage/degradation due to exposure to the leachate and freeze/thaw action. Include data on the hydration of the GCL and the potential impact of freeze/thaw on the GCL in the exposed portion of the liner system.*

The performance of the bentonite clay component of the GCL is derived from the ability of the bentonite to hydrate (absorb water). Bentonite clays have been shown to absorb water from adjacent soils with moisture contents as low as 1% (Daniel, et. al. 1992). Based on the construction records for the clay liner and dike construction (Appendix D of Design Report), the average dike soil moisture content was approximately 13.0% and the average clay liner soil moisture content was approximately 18.6%. As illustrated in Attachment A (Daniel 1992), a soil with a moisture content of 10% will allow the bentonite component of the GCL to reach a moisture content of approximately 140% at approximately 15 days. After approximately 45 days, bentonite adjacent to soil with moisture content higher than 10% will hydrate to a moisture content of between 150% and 200%. Based on the construction schedule and the anticipated regulatory approval time, the GCL will be hydrated long before the cell is placed in service.

The liner system that will remain exposed during the winter, and therefore subject to freezing, will not be covered with waste materials. When additional waste material is added to the Cell, the increase in the surface elevation will change very gradually. As the surface elevation increases, the exposed liner system will become covered and will then be insulated from the freezing effects. Once insulated, the liner system components will thaw and the bentonite component of the GCL will self heal.

Furthermore, the GCL will not likely be exposed to waste materials due to the following conditions:

- The head on the primary geomembrane portion of the side slope that may experience the freeze/thaw will be very small;
- The primary geomembrane combined with the secondary geomembrane provide two levels of protection of the GCL; and
- The leak detection system underlying the primary geomembrane will not allow head to develop on the secondary geomembrane, which will preclude potential migration through the secondary geomembrane into the GCL.

The effect of freeze/thaw on GCLs has been demonstrated by numerous studies. Nelson & Associates (Nelson 1993) demonstrated that a GCL subjected to a minimal normal stress during hydration and more than 10 freeze/thaw cycles exhibited no appreciable change to the permeability of the GCL.

In contrast, compacted clay liner materials subjected to freezing will develop ice lenses that, upon thawing, will leave voids in the soil matrix thereby increasing the permeability of the clay liner. La Plante, et. al. show that clay soils exposed to more than 20 freeze/thaw cycles can exhibit a permeability increase of more than one order of magnitude (LaPlante, 1992). To correct the problem, the clay soil would potentially need to be reworked and re-compacted. Therefore, a GCL is less susceptible to damage from freeze/thaw cycles and more protective of the environment.

7. *<u>Construction and Operational Loading</u> – Detailed procedures that cover installation of the cell and operation of the cell.*

Tailings/waste deposits will be pumped into the cell alone the north, northwest and east sides of the cell and will be discharged into or below the standing water surface, which is anticipated to be at least 20 feet deep prior to beginning solids placement in the cell. The tailings slurry will, upon reaching the quiescent liquid in the cell, disperse and allow the solids to settle out of the slurry, creating a gradual build-up along the base of the cell. The tailings discharge pipes will be placed so that at no time will the tailings solids impact directly on the liner material. Once the cell lining system is installed and accepted as complete, traffic into the cell will be restricted to foot traffic or low ground pressure vehicles, such as a one-person ATV. After completion, at no time will construction or earthmoving equipment or pickup trucks be allowed on to the cell liner. All operations will be conducted from the dike crests surrounding the cell. Once process solutions are introduced to the cell, access to foot traffic and ATV's will be restricted due to safety concerns surrounding the acidic nature of the solutions.

The tailings, consisting of medium to fine sands with silt and some clay, will segregate upon entering the liquids in the cell. The sands will drop out of solution soonest while the silt and clay fraction will be suspended longer and tend to drop out of solution further from the discharge pipe. Based on a review of existing operations in the existing cells at the facility, we have assumed the cell will be filled to approximately one-half of full height with liquids (approximately 20 feet) and that tailings may extend up to approximately 5 feet above liquid levels in the cell. In the modeling, the phreatic surface (liquid surface) is assumed to apply to only the waste and liner materials, since the composite liner system essentially eliminates infiltration of liquids into the underlying subgrade/foundation. Although tailings placed from a fluid deposition process are anticipated to develop a very shallow slope (i.e. beach) that is likely 10H:1V (Horizontal to Vertical) to 20H:1V or shallower, the analyses assumed a maximum slope inclination of 7H:1V. Based on these input parameters, along with the anticipated liner interface shear strength, a factor of safety value of 1.3 is obtained for a very small, shallow failure at the toe of the interim slope of the tailings. The slope stability analyses are presented in Attachment B.

8. <u>Settlement Evaluation</u> – *to evaluate anticipated settlement of the liner at the cell bottom and sideslopes under static conditions of the final cover system.*

   Cell 4A was constructed by removal of natural overburden soils and approximately 10 to 20 feet of undisturbed Dakota Sandstone upstream of the compacted fill creating the dikes. The majority of the Cell 4A tailings and cap material will be supported by the Dakota Sandstone formation, except for the small vertical component on the dike slopes. The estimated differential settlement of the dikes after placement of the waste materials within Cell 4A is approximately 2.4 inches from the top of the slope to the base of the slope. This value is conservatively estimated based on conservative modulus of elasticity values for the underlying materials. Based on this differential settlement, a potential strain of 0.16% will develop within the side slope liner system geosynthetic materials. This strain is well below the typical strain values that HDPE geomembrane, geonet, and GCL materials can withstand. The estimated differential settlement calculation is presented in Attachment C.

9. <u>Dike Stability</u> – *evaluation of June 9, 2006 submittal underway.* .

No response needed.

10. <u>Basis for Assumed Ground Acceleration</u> – *including submittal of the basis for the 0.10 g seismic loading used in the current dike stability analysis for the 3H:1V sideslopes.*

DRC states that no justification for the 0.10 g seismic loading was provided. Contrary to this statement, IUSA previously submitted the original Cell 4A Design documents providing a Seismic Risk Analysis for the dike design of Cell 4A. A copy of the Seismic Risk Analysis (17 pages) was included as Attachment I in the May 26 response to the 1st Round Interrogatories. Section 1.3.4, "Potential Earthquake Hazards to Project" of this submittal details the justification for the 0.10 g seismic loading.

DRC has provided IUSA with two additional references detailing a probabilistic seismic analysis for the region surrounding the Moab Title II tailings site ("Wong"). IUSA finds nothing in the Wong analysis specific to the Moab site to contradict the conclusions and basis for the design parameters presented in the Cell 4A Design documents. The Wong analysis for the Moab site lists peak horizontal accelerations of 0.05, 0.07, 0.14 and 0.18g based on return periods of 500, 1000, 5000 amd 10,000 years respectively. Wong recommended the seismic design criteria for the Moab site to be based on a return period of 10,000 years. This recommendation is stated to be very conservative based on the fact that the Moab site is located adjacent to the Colorado River and could cause a release into a major water source. The Moab site was considered to be a higher risk than other Title II sites.

40 CFR 192.02 and Appendix A requires 1,000 return periods, which would have resulted in a peak horizontal acceleration of 0.07g for the Moab site. Because of the proximity of the Moab site to the Colorado River the more conservative assumption is warranted. The White Mesa Cell 4A Design risk analysis looked at potential fault systems close to the actual site. Various studies cited in the report recommend peak horizontal ground accelerations ranging from 0.04 to 0.07g based on return periods of 50 to 1000 years. This is not inconsistent with the values for the Moab site assuming the more realistic return periods. The original Cell 4A Design Report used a more conservative value for peak horizontal accelerations of 0.10g for design basis.

12. <u>Leachate Monitoring, Operations, Maintenance, and Reporting Plan</u> – *that includes anticipated flow rates and maximum flow rates in the leachate collection layer. This is to include a demonstration that the tailings sands will settle out and function properly as a slimes drain layer without clogging and that the collection pipes are properly located and have the ability to remove the tailings solution in a reasonable time and manner. This plan shall also include the demonstration of the Action Leakage Rate and proposed response actions should the Action Leakage Rate be exceeded.*

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The slimes drain system is designed to provide a means to drain the fine fraction (slimes) component of the tailings. As the tailings are deposited within the cell, the sands will fall out of solution rather quickly and in close proximity to the discharge pipe outlet at the northern perimeter of the cell. The slimes, which consist of mostly silts with some clays, will stay in suspension longer and will therefore settle out of solution later and further from the discharge pipe. These slimes are anticipated to deposit within the sump area of the cell, furthest from the discharge pipe location(s) along the northern perimeter of the cell.

The slimes drain system consists of two components; strip drain composite laterals, and a polyvinyl chloride (PVC) pipe and gravel header. These two components will provide a means of draining the slimes, when necessary, while the sand fraction of the tailings will self drain.

The strip drain composite consists of a geotextile wrapped around a high density polyethylene core. This geotextile wrap has an apparent opening size (AOS) that provides for retention of the slimes (silt fraction) that is anticipated. Also, this geotextile provides a flow rate that is much greater than the flow that could emanate from slimes material (either silt or sand).

The PVC pipe will be bedded in non-calcareous gravel. A woven slit film geotextile will be added to the external surface of the gravel to provide retention of the slimes material and sufficient flow rate through the geotextile into the gravel and PVC pipe. Drawings 5 and 6 have been revised, and attached to this submittal, to indicate the type of geotextile and the installation criteria.

13. <u>*Action Leakage Rate*</u> *– additional information, including a computation of different Action Leakage Rates that correlate to the range of liquid levels that are anticipated in the cell during operation, and an appropriate factor of safety, as needed, to account for uncertainties associated with the manner of installation of the geonet in the cell.*

Attachment D presents a graphical representation of the varying factor of safety values and Action Leakage Rates (ALRs) associated with different head conditions in Cell 4A. It is important to note that the ALR calculation is based on the worst case condition (i.e. longest drainage path within the leak detection system layer), which represents less than 5% of the lined area.

Within the ALR calculation, partial factor of safety values are applied to account for chemical and creep issues. The overall factor of safety, 1.3 minimum, does not account for these partial factor of safety values. Accounting for the partial factor of safety values, the global factor of safety would be approximately 3.64. In addition, the geonet thickness, a key factor in the ALR calculation, is specified to be a minimum value. Therefore, the actual geonet installed in Cell 4A will likely have a thickness greater than the minimum requirement of 300 mils. This provides an additional factor of safety.

If you have any additional questions please feel free to contact me at 303 389-4160.

Very truly yours,

Harold R. Roberts
Vice President - Operations

cc: Ron F. Hochstein, IUSA
Greg Corcoran, GeoSynec

References

Brady, K.C., McMahon, W., and Lamming, G. (1994). "Thirty Year Ageing of Plastics", *Transport Research Laboratory*, Report 11, E472A/BG, ISSN 0968-4093.

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Rad, N.S., Jacobson, B.D., and Bachus, R.C. (1994). "Compatibility of Geosynthetic Clay Liners with Organic and Inorganic Permeants", *Proceedings of 5th International Conference on Geosynthetics*, Singapore, pp. 1165-1168.

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Attachment A

EFFECTS OF PARTIAL WETTING OF GUNDSEAL ON STRENGTH AND HYDROCARBON PERMEABILITY

Prepared for
Gundle Lining Systems, Inc.
19103 Gundle Road
Houston, TX 77073

Prepared by

David E. Daniel and Hsin-yu Shan
The University of Texas
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Austin, TX 78712

February 21, 1992

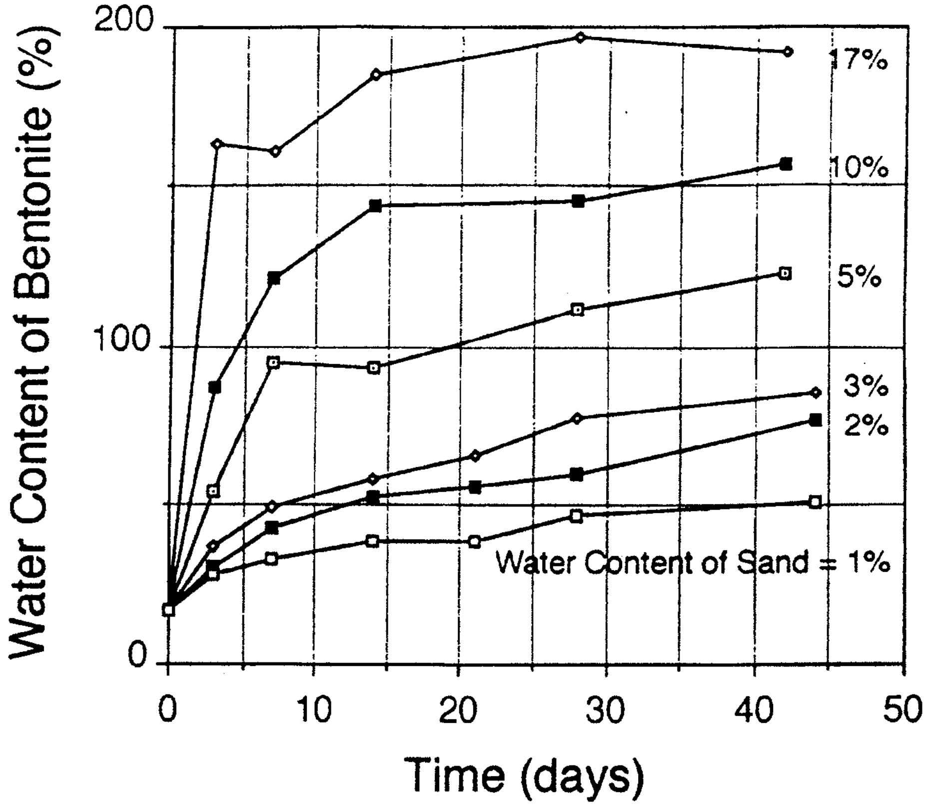


Figure 6. Water Content Versus Time for Samples of Gundseal Placed in Contact with Sand at Various Water Contents.

Attachment B

Written by: Jane Soule Date: 06 / 6 / 22 (YY MM DD) Reviewed by: Greg Corcoran Date: 6/23/06

Client: IUC Project: White Mesa Mill Cell 4A Project/Proposal No.: SC0349 Task No.: 01/04

SLOPE STABILITY ANALYSES
CELL 4A – INTERIM CONDITIONS
WHITE MESA MILL
BLANDING, UTAH

OBJECTIVE

This calculation includes static slope stability analyses for the interim waste/tailings slopes associated with operation of Cell 4A at the White Mesa Mill facility located in Blanding, Utah.

The purpose of the stability analyses is to evaluate operational conditions required to maintain a minimum factor of safety of approximately 1.3 for interim slope conditions based on the proposed design of the cell and its liner system.

METHODOLOGY

Two-dimensional static slope stability analyses were performed using the computer program SLOPE/W 2004 (Version 6.17) developed by Geo-Slope International Ltd. (2004). The results of the slope stability analyses are based on Spencer's Method of Slices for moment and force equilibrium by assuming a constant interslice shear force function. The analyzed slopes were kinematically modeled using either circular or linear/circular sliding surfaces.

For each condition analyzed, the program will search for the sliding surface that produces the lowest factor of safety. Factors of safety are defined as the ratio of the shear forces/moments resisting movement along a sliding surface to the forces/moments driving the instability.

Due to the relatively uniform geometry of Cell 4A, one cross-section was selected for analyses, Section A-A' (see Figure 1). Section A-A' is a north-south cross section with berm slopes inclined at approximately 3:1 (Horizontal:Vertical) and a base grade sloping southwest at approximately 1 percent. Cell 4A will be constructed with the following liner system on both the bottom area and side slopes:

- Slimes Drain System (Cell bottom only);
- 60 mil smooth HDPE geomembrane (Primary Liner);
- Geonet Drainage Layer (Leak Detection System);
- 60 mil smooth HDPE geomembrane;
- Geosynthetic Clay Liner (GCL); and } (Composite Secondary Liner)
- Prepared Subgrade.

Tailings/waste deposits are expected to be pumped into the pond below the water surface where the tailings will settle out creating a gradual build-up of solids along the base of the cell. The tailings will be pumped into the cell from north to south, beginning at the splash pad locations located along the northern slope of the pond. Based on a review of existing operations in the existing cells at the facility, we have assumed the pond will be filled to approximately one-half of full height with liquids (approximately 20 feet) and that tailings may extend up to approximately 5 feet above water levels in the pond. In the modeling, the phreatic surface (water surface) is assumed to apply to only the waste and liner materials, since the composite liner system prevents infiltration of liquids into the underlying subgrade/foundation. Although tailings placed from a fluid deposition process are anticipated to develop a very shallow slope (i.e. beach), tailings are assumed to be placed at a maximum slope inclination of 7H:1V. Figure 2 shows a cross section of the assumed operational conditions.

MATERIAL PARAMETERS

Based on a review of potential liner interfaces, the likely critical interface has been identified as the smooth HDPE geomembrane and the geosynthetic clay liner. Based on our experience and available laboratory data (see Attachment 1), this interface has been estimated to have a shear strength of approximately 8 degrees.

Based on existing operations at the site, tailings/waste deposits are anticipated to be primarily fine sands with silt and some clay. Based on our experience, we have estimated a total unit weight of 125 pounds per cubic foot (pcf) and a friction angle of 25 degrees, with no cohesion, for these materials.

Due to the low interface strength of the liner system, failures are not anticipated to extend beneath the liner system into the foundation (Dakota Sandstone). As such, the foundation system has been modeled as bedrock within the slope stability program (i.e., impenetrable) to allow slip surfaces within the liner system.

STATIC STABILITY RESULTS/RECOMMENDATIONS

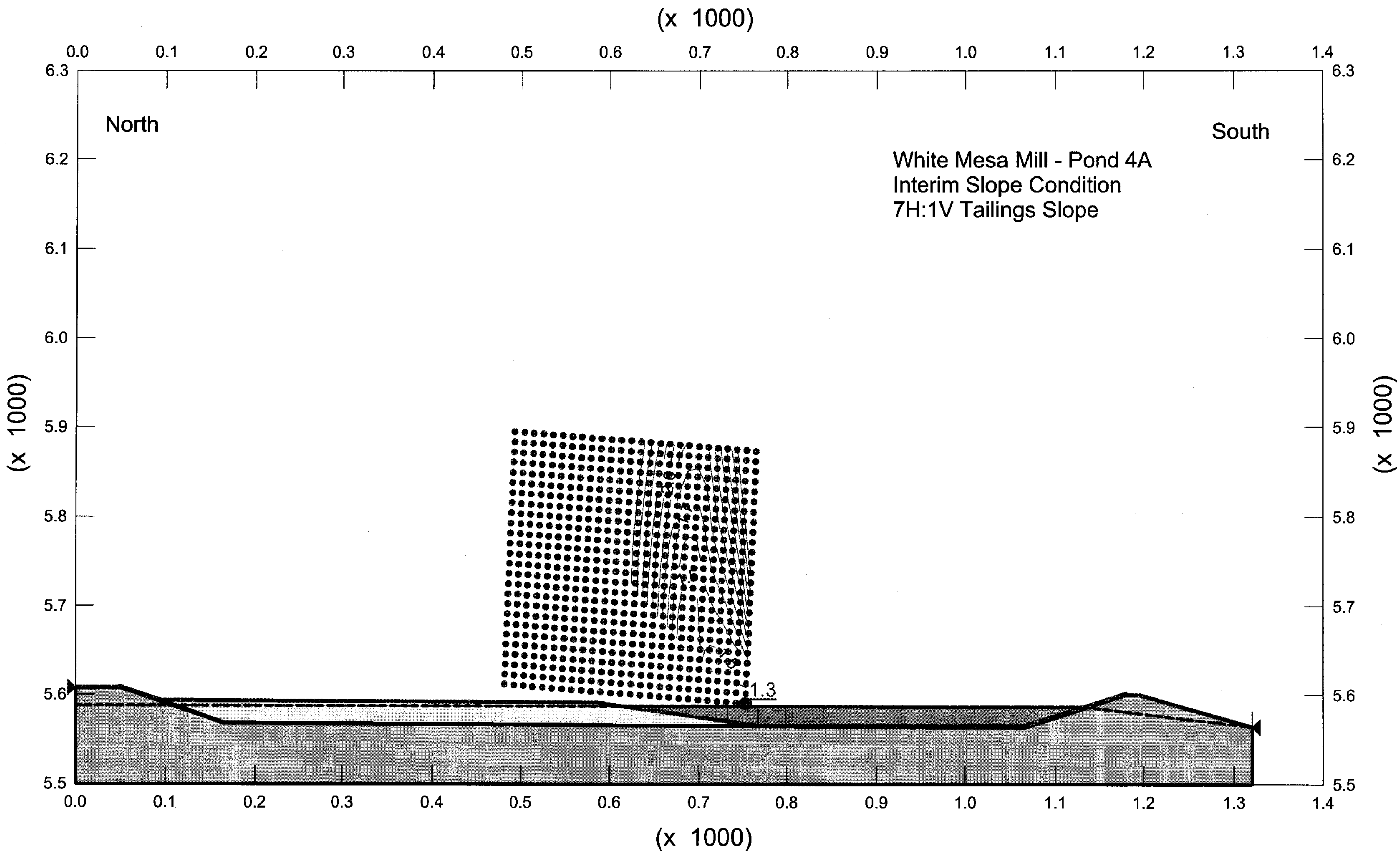
As discussed above, one cross-section was analyzed which represents typical operating conditions for Cell 4A.

Numerous potential failure surfaces were performed to evaluate various slip surface geometries and to identify the critical slip surface. The results of this analysis indicate a minimum static factor of safety of approximately 1.3 assuming waste slopes inclined at approximately 7H:1V, as shown in Figure 2. This failure surface is a very small, shallow failure at the toe of the interim tailings slope. Much higher factor of safety values are obtained for larger failure surfaces.

We recommend that interface testing of the liner system be performed during construction quality assurance to verify that the interface friction angles used in this analysis for the liner system are met or exceeded. Further, we recommend that operations at the site limit the tailings/waste deposits slope to inclinations of 7H:1V or flatter.

REFERENCES

GeoSlope International, LTD (2004) SLOPE/W Version 6.17.



Note: Phreatic surface is applied to the liner and waste materials only.

Figure 2

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA
INTERFACE W/ GEOMEMBRANE

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Residual Friction Angle (deg)[4]	Apparent Peak Cohesion (psf)	Comments
GA	09-04-92	W/60-mil sm. HDPE	0.5 - 1 - 2 - 4 - 10	Hydrated	0.02	8	7	0	
		W/60-mil text. HDPE	0.5 - 1 - 2 - 4 - 10	Hydrated	0.02	28	28 / 6	29	bi-modal residual
GSC	12-08-94	W/60-mil text. HDPE	7.5 - 15 - 30	Hydrated	0.04	18	17	175	Diff. membrane
		W/60-mil text. HDPE	7.5 - 15 - 30	Hydrated	0.04	16	12	345	manufacturers
GSC	12-16-94	W/60-mil text. HDPE	1 -3 - 6 - (15)	Hydrated	0.04	25 (19)	10	100	(lower d at 15 psi)
AGP	07-12-95	W/80-mil Text. HDPE	14 - 28 - 69 - 104	Hydrated	0.04	18	8	192	
		W/80-mil Text. HDPE	14 - 28 - 69 - 104	Dry	0.04	30	14	0	
AGP	11-30-95	W/ Text. HDPE	10 - 26 - 38	Dry	0.08	30.2	13.3	0	
GSC	03-12-96	W/ 30 mil PVC	2 - 4 - 6	Hydrated	0.04	17	17	24	
GSC	05-29-96	NW/80mil Text. HDPE	140	Hydrated	0.04	19	5	475	Consol 24 hrs @140 psi
AGP	11-08-96	NW/60mil Text. HDPE	1.4 - 3.5 - 7.0	Hydrated	0.04	34.8	22.7	149	
		W/60 mil Text. HDPE	1.4 - 3.5 - 7.0	Hydrated	0.04	28.8	20.4	83	
GSC	01-08-97	NW/60mil Text. HDPE	14 - 40 - 70	Hydrated	0.04	17	9	255	
TRI	4-15-97	NW/60mil Text. HDPE	14 – 28 – 56	Hydrated	0.04	21.9	10.8	722	
Emcon	6-16-97	NW/40mil Text. LLPE	1.0 – 2.0 – 3.5	Hydrated	0.04	32.0	18.5	111	
		NW/40mil Text. LLPE	1.0 – 2.0 – 3.5	Hydrated	0.04	37.5	27.1	118	
TRI	10-15-97	NW/ 60mil Text.HDPE	3.5 – 7 – 14	Hydrated	0.04	20.3	18.6	278	
		NW/ 60mil Text.HDPE	.35 - .7 – 1.4	Hydrated	0.04	36.6	25.8	2	
TRI	12-01-97	NW/60mil Text. HDPE	0.35 – 0.7 – 1.4	Hydrated	0.001	25.6	23.3	54	Residual @ 4"
		NW/60mil Text. HDPE	3.5 – 7 – 14	Hydrated	0.001	23.2	17.8	85	"
Emcon	04-06-98	NW/60mil Text. HDPE	14 – 28 – 70	Hydrated	0.04	24.1	12.2	290	

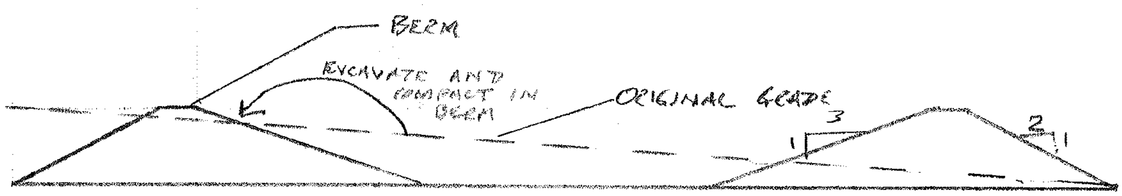
Attachment C

Written by: S. FITZWILLIAM Date: 22 / 06 / 06 (DD MM YY) Reviewed by: [illegible] Date: 23 / 06 / 06 (DD MM YY)

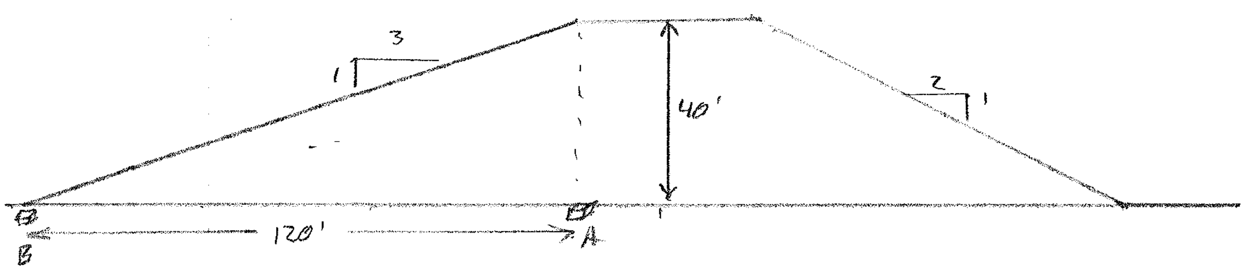
Client: IUC Project: CELL 4A Project/Proposal No.: SC0349 Task No: 01 / 04

SETTLEMENT EVALUATION OF BERMS

SUBJECT: EVALUATE THE DIFFERENTIAL SETTLEMENT UNDER THE LOADING FROM THE BERMS AT THE PERIMETER OF THE CONTAINMENT PONDS. TO ASSESS THE POTENTIAL EFFECT ON THE CLAY LINER



EVALUATE SETTLEMENT UNDER CENTER OF BERM VS. INSIDE OF BERM ON 3:1 SLOPE



FROM BERM COMPACTION TESTS USE $\gamma_{max} = 125\,pcf$ (FILL)

$$P, \text{LOAD} = \gamma_{max} H = (125\,pcf)(40\,ft) = 5{,}000\,psf$$

THE FOUNDATION FOR THE BERM CONSIST OF FOUNDATIONAL SOIL CONSISTING OF SILTY AND CLAYEY COARSE TO FINE SAND

Written by: S. FITZWILLIAM Date: 22/06/06 Reviewed by: GTC Date: 23/06/06

Client: IUC Project: ______ Project/Proposal No.: ______ Task No: ______

EVALUATE THE ELASTIC STRAIN OF THE FOUNDATION DUE TO LOADING UNDER THE BERM

STRESS-STRAIN MODULUS, E_s ⇒ FROM BOWLES, 4th ED. (ATTACHMENT A)

$E_s = 1{,}500$ ksf

USE BOUSSINESQ CASE FOR INFLUENCE UNDER A TRIANGULAR LOAD AS PRESENTED IN DM7.1-171 (ATTACHMENT B)

SEE SPREADSHEET #1 ATTACHED

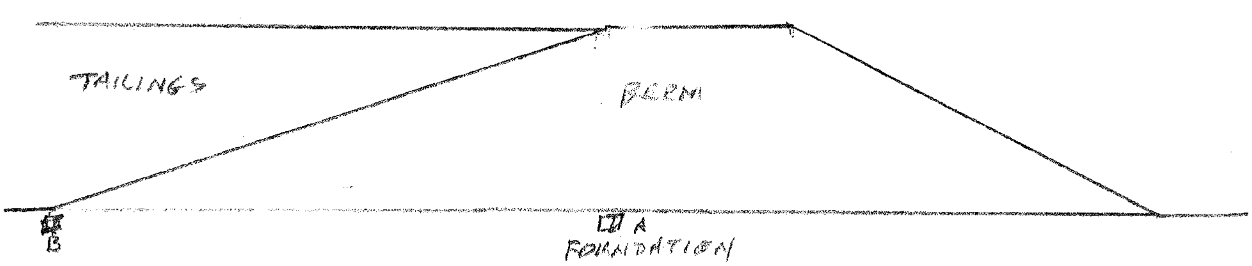
SETTLEMENT UNDER A = 4.4 in
" " B = 2.5 in

$\Delta S = 1.9$ in

LINER IS $[(120')^2 + (40')^2]^{1/2} = 126.5$ ft LONG

STRAIN IN LINER IS $\dfrac{1.9}{(126.5)(12''/ft)} = 0.13\%$

SUBJECT 2: EVALUATE THE SETTLEMENT UNDER THE BERM UPON FILLING OF THE POND



Settlement due to load under the embankment

B = 120
L = 1000
P = 5000

Under top of berm - Point A

z ft	n = B/z	m = L/z	I	4I	σz = 4I*P ksf	Es ksf	ΔH ft	S in
5	24	200	0.24	0.96	4.8	1500	5	0.192
10	12	100	0.235	0.94	4.7	1500	5	0.188
15	8	66.66667	0.23	0.92	4.6	1500	5	0.184
20	6	50	0.22	0.88	4.4	1500	5	0.176
40	3	25	0.2	0.8	4	1500	20	0.64
60	2	16.66667	0.175	0.7	3.5	1500	20	0.56
80	1.5	12.5	0.158	0.632	3.16	1500	20	0.5056
100	1.2	10	0.14	0.56	2.8	1500	20	0.448
150	0.8	6.666667	0.108	0.432	2.16	1500	50	0.864
200	0.6	5	0.085	0.34	1.7	1500	50	0.68
							Total	4.4376

Under toe of slope - Point B

z ft	n = B/z	m = L/z	I	4I	σz = 4I*P ksf	Es ksf	ΔH ft	S in
5	24	200	0.08	0.32	1.6	1500	5	0.064
10	12	100	0.08	0.32	1.6	1500	5	0.064
15	8	66.66667	0.08	0.32	1.6	1500	5	0.064
20	6	50	0.08	0.32	1.6	1500	5	0.064
40	3	25	0.08	0.32	1.6	1500	20	0.256
60	2	16.66667	0.08	0.32	1.6	1500	20	0.256
80	1.5	12.5	0.08	0.32	1.6	1500	20	0.256
100	1.2	10	0.08	0.32	1.6	1500	20	0.256
150	0.8	6.666667	0.078	0.312	1.56	1500	50	0.624
200	0.6	5	0.07	0.28	1.4	1500	50	0.56
							Total	2.464

SPREADSHEET #1

Written by: S. FITZWILLIAM Date: __ / __ / __ Reviewed by: GTC Date: 23 / 06 / 06

Client: IUC Project: CELL 4A Project/Proposal No.: SCO 349 Task No: 01/04

UPON FILLING THE POND THE FOUNDATION UNDER B MAY SETTLE DIFFERENTIALLY COMPARED WITH THE THE TOP OF THE BERM AT POINT A.

E_s = 1,000 ksf BERM (ATTACHMENT A)
γ_{max} = 125 pcf

γ_{max} = 125 pcf TAILINGS

E_s = 1,500 ksf FOUNDATION SOIL

LOAD OVER B = (125 pcf)(40 ft) = 5,000 psf

LOAD OVER A = 5,000 psf AT A DISTANCE OF 120 FT

UNIFORM LOADING (AERIAL FILL) UNDER B
ASSUME B = 200 ft FOR INFINITELY LONG FOOTING, USE DM7.1-167 (ATTACHMENT C)

SEE ATTACHED SPREADSHEET # 2

DIFFERENCE IN SETTLEMENT BETWEEN B AND A IS 8.45 − 6.01 = 2.44 in

STRAIN IN LINER $\frac{2.44}{(124.5)(12)}$ = 0.16 %

Settlement due to load from tailing in pond

B = 200
L = 1000
P = 5000

Under top of berm - Point B

z ft	z/B	I	$\sigma z = I*P$ ksf	Es ksf	ΔH ft	S in
5	0.025	0.99	4.95	1500	5	0.198
10	0.05	0.98	4.9	1500	5	0.196
15	0.075	0.96	4.8	1500	5	0.192
20	0.1	0.92	4.6	1500	5	0.184
40	0.2	0.9	4.5	1500	20	0.72
60	0.3	0.85	4.25	1500	20	0.68
80	0.4	0.8	4	1500	20	0.64
100	0.5	0.75	3.75	1500	20	0.6
150	0.75	0.6	3	1500	50	1.2
200	1	0.52	2.6	1500	50	1.04
300	1.5	0.4	2	1500	100	1.6
400	2	0.3	1.5	1500	100	1.2
					Total	8.45

Under toe of slope - Point A (120/200 = 0.6B from center)

z ft	z/B	I	$\sigma z = I*P$ ksf	Es ksf	ΔH ft	S in
5	0.025	0.9	4.5	1500	5	0.18
10	0.05	0.8	4	1500	5	0.16
15	0.075	0.75	3.75	1500	5	0.15
20	0.1	0.7	3.5	1500	5	0.14
40	0.2	0.6	3	1500	20	0.48
60	0.3	0.5	2.5	1500	20	0.4
80	0.4	0.5	2.5	1500	20	0.4
100	0.5	0.45	2.25	1500	20	0.36
150	0.75	0.4	2	1500	50	0.8
200	1	0.33	1.65	1500	50	0.66
300	1.5	0.29	1.45	1500	100	1.16
400	2	0.28	1.4	1500	100	1.12
					Total	6.01

SPREADSHEET #2

JOSEPH E. BOWLES

FOUNDATION ANALYSIS AND DESIGN

FOURTH EDITION

ATTACHMENT A (1/2)

TABLE 2-7 Typical range of values for the static stress-strain modulus E_s for selected soils

Field values depend on stress history, water content, density, etc.

	E_s	
Soil	**ksf**	**Mpa**
Clay		
Very soft	50–250	2–15
Soft	100–500	5–25
Medium	300–1000	15–50
Hard	1000–2000	50–100
Sandy	500–5000	25–250
Glacial till		
Loose	200–3200	10–150
Dense	3000–15 000	150–720
Very dense	10 000–30 000	500–1440
Loess	300–1200	15–60
Sand		
Silty	150–450	5–20
Loose	200–500	10–25
Dense	1000–1700	50–81
Sand and gravel		
Loose	1000–3000	50–150
Dense	2000–4000	100–200
Shale	3000–300 000	150–5000
Silt	40–400	2–20

The *modulus of subgrade reaction* k_s is defined as the ratio of stress to deformation as shown on Fig. 2-37*c*. The units of k_s are the same as unit weight.

The shear modulus G' (and may be subscripted) is defined as the ratio of shear stress to shear strain. It is related to E_s and μ as

$$G_s' = \frac{s}{\epsilon_s} = \frac{E_s}{2(1 + \mu)} \tag{b}$$

The shearing strain ϵ_s is the change in right angle at any corner of an element as in Fig. 2-37*b* such that

$$\epsilon_s = \text{angle } BCD - \text{angle } B'C'D' \tag{c}$$

Another concept occasionally used is the volumetric strain, defined as

$$\epsilon_v = \frac{\Delta V}{V} = \epsilon_1 + \epsilon_2 + \epsilon_3 \tag{d}$$

ATTACHMENT A (2/2)

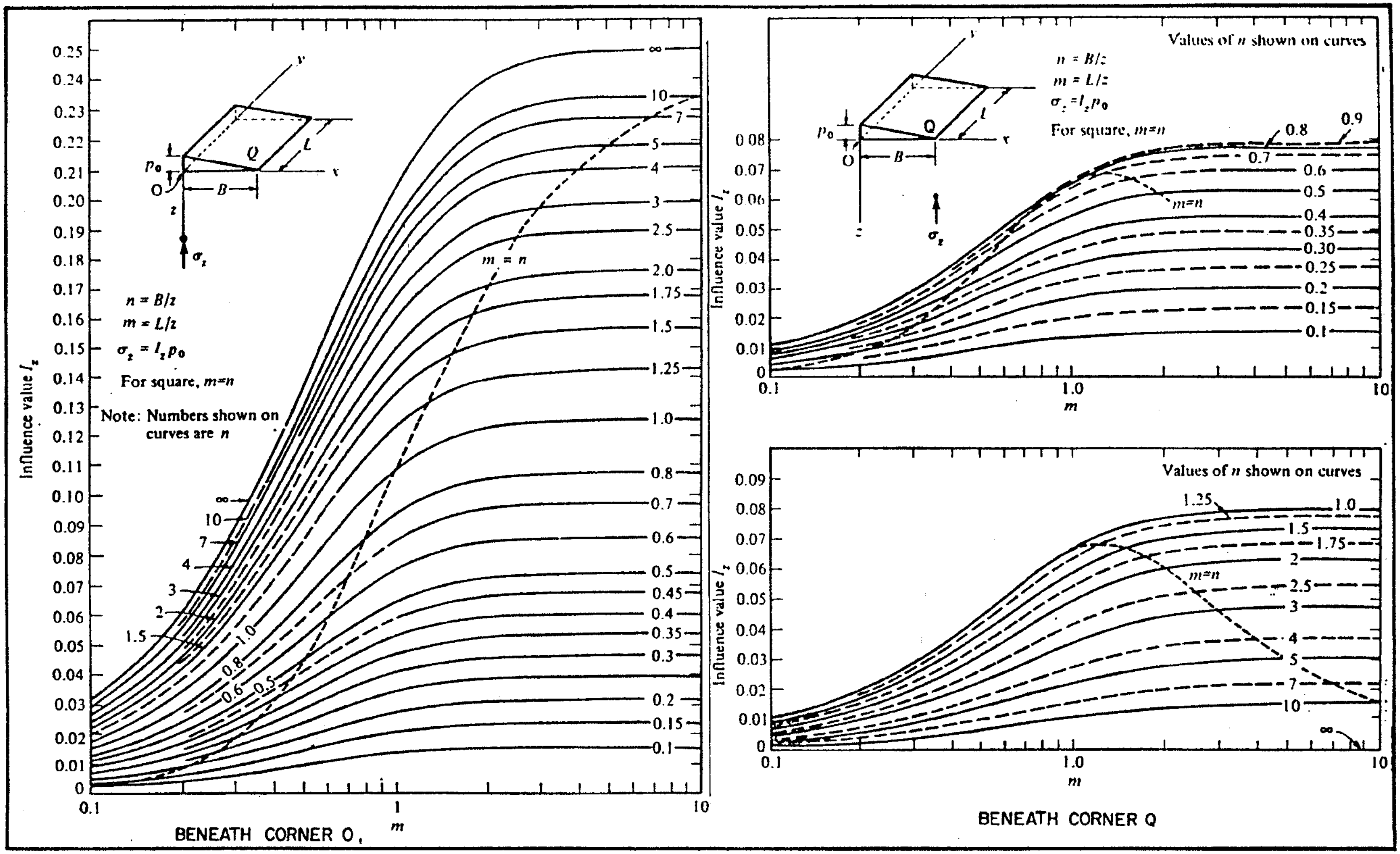
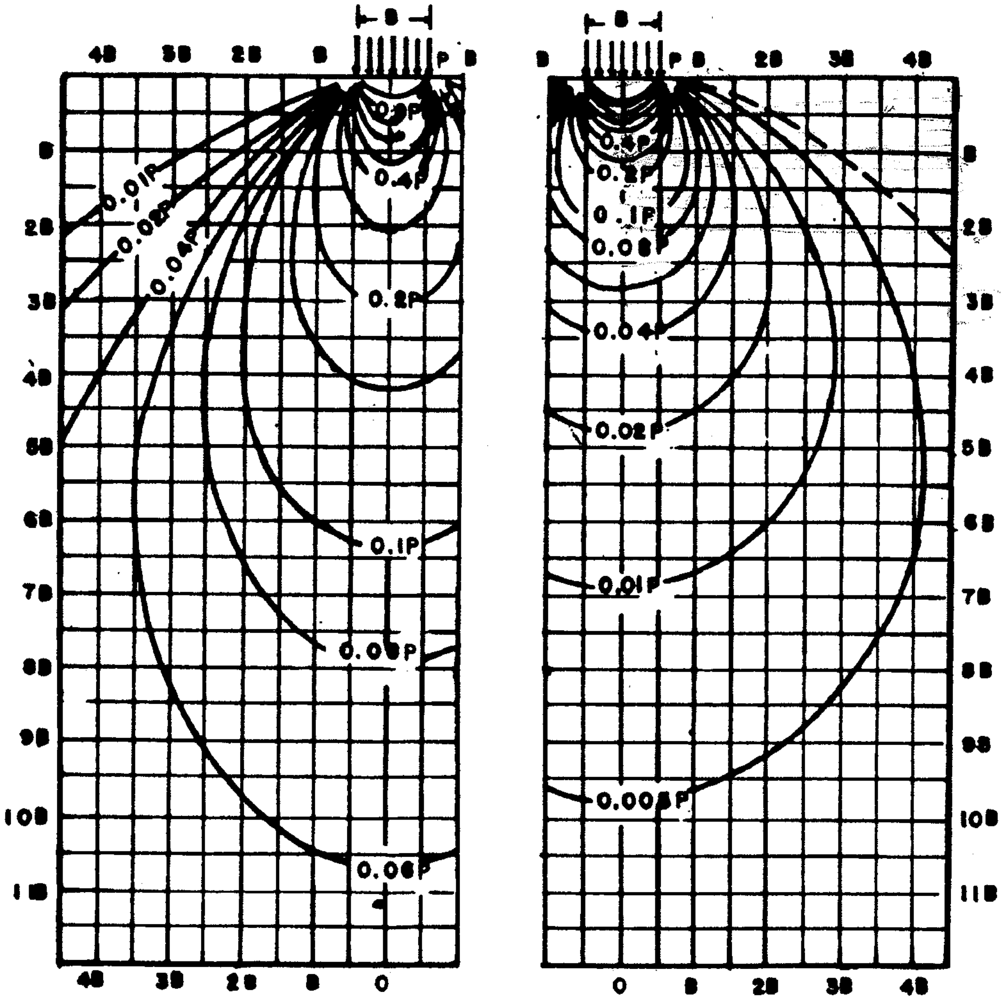


FIGURE 7
Influence Value for Vertical Stress Beneath Triangular Load
(Boussinesq Case)

ATTACHMENT B



a. INFINITELY LONG FOOTING

b. SQUARE FOOTING

SQUARE FOOTING

GIVEN

FOOTING SIZE = 20'X 20'

UNIT PRESSURE P = 2 TSF

FIND

PROFILE OF STRESS INCREASE BENEATH CENTER OF FOOTING DUE TO APPLIED LOAD

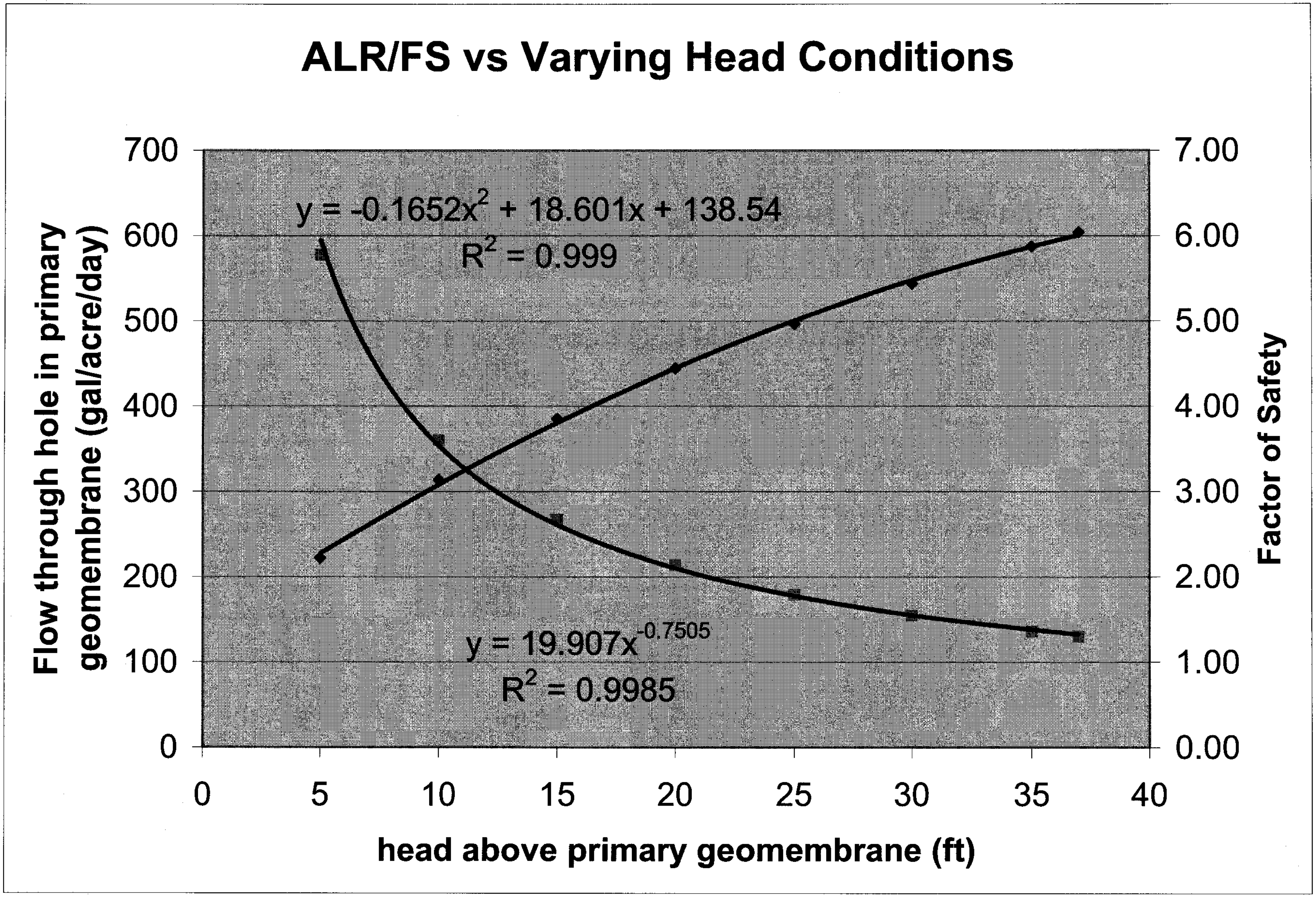
B = 20' P = 2 TSF

Z (FT)	$\frac{Z}{B}$	σ_z TSF		
10	0.5	0.70 X 2	=	1.4
20	1	0.38 X 2	=	0.76
30	1.5	0.19 X 2	=	0.38
40	2.0	0.12 X 2	=	0.24
50	2.5	0.07 X 2	=	0.14
60	3.0	0.05 X 2	=	0.10

FIGURE 3
Stress Contours and Their Application

ATTACHMENT C

Attachment D



ALR/FS vs Varying Head Conditions
y = -0.1652x^2 + 18.601x + 138.54
R^2 = 0.999
y = 19.907x^-0.7505
R^2 = 0.9985
Flow through hole in primary geomembrane (gal/acre/day)
Factor of Safety
head above primary geomembrane (ft)
0
100
200
300
400
500
600
700
0.00
1.00
2.00
3.00
4.00
5.00
6.00
7.00
0
5
10
15
20
25
30
35
40