

August 28, 1998

Mr. Joseph Holonich  
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Office of Nuclear Materials Safety & Safeguards  
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Dear Mr. Holonich:

Attached are our responses to the July 17, 1998 request for information from the Nuclear Regulatory Commission on the Reclamation Plan for the White Mesa Mill. We have performed additional analyses and calculations in support of several of these responses, and hope that we have thoroughly responded to all the issues raised by the NRC. We also have incorporated in our responses the information and specifics discussed during our meeting of June 12, 1998. Many of the more complicated questions were discussed at that time, and we have developed our responses based on our best understanding of the technical issues and NRC's request, and in certain cases, where we felt it was appropriate, proposed alternatives.

If you have any questions, please feel free to contact me.

Very truly yours,



Harold R. Roberts  
Executive Vice President

HRR/pl  
Enclosures

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Responses to NRC Letter Dated July 17, 1998  
Request for Additional Information on the Reclamation Plan for the White Mesa  
Uranium Mill

Source Material License SUA-1358  
Docket No. 40-8681

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High Level Waste and Uranium Recovery Projects Branch  
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2 White Flint North, Mail Stop T7J9  
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Re: White Mesa Uranium Mill  
Response to NRC's Questions Dated July 17, 1998  
Site Reclamation Plan

Dear Mr. Holonich:

The following is in response to comments from the Nuclear Regulatory Commission (NRC) regarding the White Mesa Uranium Mill Reclamation plan. Each comment from the NRC (in italics) is followed by the response.

**Geotechnical and Radon Barrier Design**

*1 The proposed random fill material requires additional characterization.*

*IUSA states that the random fill material to be used consists of clay, silt, sand, and gravel. The material, which has been stockpiled onsite, contains isolated pockets of clay (CL type) and varying amounts of sandstone cobbles (from 75 to 300 millimeters [mm] in size) and boulders (larger than 300 mm in size). IUSA states that it may screen out the cobbles and boulders prior to placing the material on the disposal cell. It is not clear from IUSA's response what it proposes as the maximum size of particles in random fill to be placed in the disposal cell.*

*If placed in a disposal cell, a strongly heterogeneous random fill can significantly affect the performance of the radon barrier, making complex and difficult the estimation of differential settlement and the potential for cover cracking.*

*Therefore, IUSA should specify the maximum particle size of random fill material to be placed in the disposal cell. Quality control (QC) procedures to ensure the separation of*

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*undesirable materials and to ensure material specifications are met should be provided. If stones and large particles, such as boulders and cobbles, are used, their potential effects on differential settlement, cover cracking potential, preferential infiltration pathways, and the potential for cover erosion should be analyzed and documented.*

### Response #1

International Uranium (USA) Corporation ("IUSA") suggests that two new terms, **platform fill** and **frost barrier fill**, be used to distinguish the random fill placed directly over tailings to form a working platform from the random fill placed above the clay layer to provide frost and shrink-swell protection.

### Platform Fill

An initial lift of 3-4 feet of random fill will be placed to form a stable working platform (platform fill) over tailings for subsequent controlled fill placement. This initial lift will be placed by pushing the material across the tailings in increments, slowly enough that the underlying tailings are displaced as little as possible. This initial lift cannot be compacted through its entire thickness because the underlying tailings will not support the weight and vibrations of compaction equipment. We understand that this situation is well-known to the NRC from the other Title II sites (e.g. Homestake Grants, Quivera).

The maximum particle size in the initial lift could be as large as the lift thickness. Because settlement originates in the tailings, not the cover, the maximum particle size in the platform fill has no effect on settlement. Both differential and total settlement are primarily functions of tailings compressibility. The effect of the platform fill is to provide normal stress to drive settlement, and the difference between unit weights of various materials randomly mixed in the platform fill will not be large enough to produce meaningful differentials in normal stress.

The top surface (top 1.0 feet) of the platform fill will be compacted to 90% maximum dry density per ASTM D 698. If large rock protrudes into this part of the platform fill from lower portions, smaller fill material will be placed to bridge the protrusion before compaction.

Examination of the platform fill that has already been placed is the best way to evaluate its performance and the validity of the foregoing response. IUSA has already placed a significant amount of platform fill in Cell 2 and a portion of Cell 3. Through field observation the fill has exhibited excellent stability and should continue to perform as predicted above.



Frost Barrier Fill

Random fill placed above the clay cover (the *frost barrier fill*) will be placed in 12-inch lifts, with particle size limited to 8 inches, or 2/3 the uncompacted lift thickness. This maximum particle size conforms to standard earthwork practice. IUSA plans to prepare and apply this random fill in frost-barrier cover construction as follows:

- Random fill borrow will be excavated by loader or scraper from stockpiles and, if oversize material is found, it will be removed by whatever means is appropriate for the earthmoving equipment in use (e.g., rock rake or grader blade).
- The random fill material will be loaded into scrapers or trucks, hauled to cover locations, and spread in lifts of not more than 12 inches uncompacted thickness.
- Oversize material will be stockpiled for possible use as riprap.

Source verification QC procedures for frost-barrier borrow material will consist of:

- Visual inspection of placed random fill at each location of field density testing.
- Particle size analyses (ASTM D 422) for all minus 3-inch material, with oversize material saved and weighed separately. One test per 5,000 cubic yards.

Visual inspection will include observation of particle sizes and manual measurement of orthogonal dimensions of any observed particle that appears to exceed eight inches in any one of the three dimensions. If the intermediate dimension exceeds eight inches, the particle will be removed from the lift.

2. *A QC test procedure and appropriate test frequencies are required to ensure that only CL and CH clays will be used for cover construction*

*Soil profiles provided by IUSA show that layers of acceptable types of clay (CL and CH) are overlain by clays with undesirable properties (SC, SM, and ML). The thicknesses of undesirable clay layers have considerable spatial variation. These clays need to be separated from the acceptable clay types to ensure that the radon barrier will have desirable radon containment properties. An adequate QC plan with an acceptable sampling program is necessary to ensure that required tests and any corrective action are completed to ensure that acceptable clay types are used for cover construction.*

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*The test frequencies proposed by IUSA are inadequate to ensure that only clays with desirable properties are used in the radon barrier. The NRC Staff Technical Position on testing and inspection plans (NRC, 1989) provides guidelines for the frequencies of QC tests deemed acceptable for disposal cell construction.*

## Response #2

IUSA should clarify here that the term "clay", as used in the 1996 Titan Environmental report, does not mean only soils classified as CL or CH according to the Unified Soil Classification System. In the report, that term was not intended to be limited so narrowly that it would exclude the use of other clay-bearing soils, such as clayey sand (SC) and clayey gravel (GC), in the clay portion of the cover.

Experience with radon barriers elsewhere demonstrates that mixtures of clay and sand make good cover soils and can have some advantages over clay-only covers. Clay-sand soil is easier to handle and moisture-condition, compacts more readily and to higher densities than clay soil that has little or no sand content, and is less susceptible to cracking. SC and GC soils also have hydraulic conductivities commonly in the  $10^{-6}$  to  $10^{-7}$  cm/sec range and diffusion coefficients in the  $10^{-2}$  to  $10^{-3}$  cm<sup>2</sup>/s range, both of which are more than adequate for the required functions of the tailing covers at the White Mesa site. Therefore, it would appear unnecessary to limit the clay layer soil to CL and CH material only, and IUSA would propose that SC and GC soils should be included as acceptable soils.

The source verification QC program for clay-layer borrow material proposed below is consistent with that previously approved by the NRC for Cell 4A clay base construction. It will include:

- Soil classification - particle size analysis per ASTM D 422 and Atterberg limits per ASTM D 4318, one test for every 5,000 cubic yards (cy) of excavated borrow soil.
- Maximum density and optimum moisture content per ASTM D 698, one test for every 10,000 cy of borrowed soil.

We note that the radon flux emanating from the existing interim cover (platform fill), consisting of random fill, already satisfies the 20 pCi/m<sup>2</sup>/s limit. One foot of "clay" cover (clay layer) plus two feet of random fill (frost barrier) will provide additional attenuation in excess of that already achieved, as documented in Appendix B of the 1996 Titan report. Therefore, the conservatism in the design substantially reduces the dependence of cover performance on the QC program, allowing IUSA to employ testing frequencies somewhat less than those in NRC's guidance, we believe are strictly applicable only to situations where the design satisfies only the minimum requirements.

IUSA understands that clay borrow from the Section 16 source has not been used previously in construction on the site. In order to provide additional confidence that the Section 16 soils will

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satisfy the requirements for the clay layer, IUSA will commit to an initial source verification testing program as follows:

The first 50 classification samples will be collected across the candidate borrow soil horizons at a frequency of one sample per 1,000 cy. This represents about 20 % of the total borrow volume from Section 16. After the first 50 soil classifications are performed on the clay borrow source, testing frequency will be reduced to one test for each 5,000 cy of borrow if the results of the first 50 classifications are CH, CL, SC, or GC. IUSA will notify NRC of its findings before implementing this change.

3. *The test hole locations used to estimate the properties of mill tailings and cover materials should be identified. The standards and procedures used to determine the material properties also should be specified.*

*IUSA should provide a map(s) showing the locations of the test holes from which tailings and cover material characterization samples were obtained. The locations to be identified should include those samples described in Appendix A of the reclamation plan and in Attachment A of IUSA's December 16, 1997 response. IUSA also should specify the standards and procedures used to assess the material properties.*

### **Response #3**

IUSA is including a copy of "Cell 4 Design-Tailings Management System, Appendix B" (previously submitted to NRC in 1988) as Attachment 1. This document contains a compilation of the drill hole logs, test pit logs, soil classification data and a map showing the location of the pits and drill holes located on the White Mesa Mill site. This data has been used to characterize the soils used in the construction of the cell dike and to classify the material that has been stockpiled for use in the cover construction. Some of the data included in this Appendix was used and included (Chen 1978, Chen 1979 and Dames and Moore 1978) in the Titan, Tailing Cover Design, White Mesa Mill (10 96) in Appendices A and G, without map. Additional data on soils stockpiled for later use and used in dike construction can be found in the Construction Reports submitted to NRC for Cells 1-I & 2 (March 1, 1982), Cell 3 (March 4, 1983), and Cell 4A (1990). Soil classification data, test pit locations and drill hole locations for the clay borrow site located in Section 16 were included with the "Letter Report Section 16 Clay Material Test Data, D'Appolonia, March 8, 1982", provided as Attachment 1 to IUSA's comments dated December 16, 1997. Sample UT-1 was obtained in 1996 by Titan and included in Appendix A to the Cover Design and was taken out of Test Pit 2 (TP-2) shown on Figure 1 in the D'Appolonia Section 16 Report. The map is included here as Attachment 2.

There are no drill holes in the tailing. Any samples taken for classification purposes of the tailing are taken by hand shovel excavation.

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Sample Composite (2,3, & 5) with radiological results from Rogers & Associates Engineering Corporation, shown in Appendix A of Titan's Cover Design is a composite sample of grab samples from random fill stockpiles RF-2, RF-3 and RF-5 as shown on the map in Attachment 1 to these comments. Site 1 and Site 4 samples are from Clay stockpile C-1 and Random fill stockpile RF-4 as shown on the map in Attachment 1 to these comments. All of these samples were taken from shallow pits (<5 feet deep) excavated in these stockpiles.

4. *Tests using multiple samples should be used to quantify adequately and account for the intrinsic variation of material properties in IUSA's analyses.*

*The intrinsic variability of the material properties should be characterized by conducting measurements of each property using several samples. Such a process appears not to have been followed. For example, the Atterberg Limits tests, Standard Proctor test, and permeability test were carried out using only one sample (UT-1). Moreover, the location from which this sample was obtained has not been provided. Similarly, supporting information concerning the samples used for determining the material properties employed in the slope stability analyses is missing also. Moreover, the hydraulically-placed tailings show significant heterogeneity that must be taken into account in settlement analyses.*

*IUSA should conduct and document tests using multiple samples to adequately estimate the intrinsic variation of material properties, including the heterogeneity of the tailing properties (see NRC, 1978). The location of Sample UT-1 should be specified, as well as the additional samples used for determining the parameters for the slope stability analyses. Technical justification should be provided to support the contention that the material property values used in the slope stability analyses are appropriate and acceptably consider the intrinsic variability.*

*REFERENCE: NRC, April 1978, "Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants," Regulatory Guide 1.138.*

#### **Response #4**

We understand from this comment that the NRC appears to be concerned about the stability of the impoundment dikes. The stability of the dikes, which were designed to contain liquid (hydrostatic stress at maximum pond level), was addressed in the mill permit application and License Amendments approved by NRC in 1979, 1982, and 1990.

This comment also appears to question the database for characterization of borrow soils to be used in construction. Although the cover design document contained data only on a sample for UT-1, substantial information has been developed on the Section 16 borrow source and from test drilling by Chen and Associates and Dames and Moore submitted previously to the NRC. A field and

laboratory investigation of the borrow area in Section 16 was performed by D'Appolonia in 1982, and the report of that study was submitted to the USNRC most recently on December 16, 1997. The investigation included 6 test borings and 3 test pits located as shown on Figure 1 of that report. Sample UT-1 was taken from the D'Appolonia test pits (TP-2) as shown on Figure 1 - Attachment 2 to this submittal.

Technical justification of the material property values used in the slope stability analysis is based on both the results of soil testing conducted previously and on conservative procedures used to quantify the input parameters for the stability model. Specifically:

- **Earthfill and Dike** - This is random fill that consists mostly of mixed-grain soils and cobbles up to 8-inch size. This material is variable, ranging from sandy soils (SC, SM) to clay soils (CL, CH) with enough larger particles to possibly be GP to GC in places. The input parameters used for this material are conservative for slope stability analysis, because zero cohesion and a friction angle of 30 degrees are used. Zero cohesion is conservative because the testing by Chen and Associates and others show the random fill material has a substantial minus-200 fraction and samples typically have low to moderate plasticity. These properties would justify a classification of the fines as CL-ML to CL and some cohesion value between zero and 1000 psf (ref: NAVFAC DM-7, Table 9-1). Materials placed to 95% maximum dry density typically have an angle of internal friction of 31-34 degrees (NAVFAC DM-7, Table 9-1) and, along potential failure surfaces, a secant friction angle of over 40 degrees (Terzaghi, Peck, and Mesri; 1996, Fig 19.4). The intrinsic variability of the random fill will fall in a range that is accounted for by the relatively low strength parameters used in the analysis.
- **Foundation** - Drill logs show that the foundation soils are locally-derived alluvium and weathered bedrock (sandstone with some claystone) consisting of SM to ML soils with some CL and CH. This material is the source for most of the random fill and, therefore, has similar properties except that both the unit weight and the friction angle are slightly less to account for the difference in compaction, the latter being uncompacted but undisturbed. The same justification for conservatism provided above applies to the foundation soil.
- **Bedrock** - It is more accurate to characterize intact rock strength in terms of unconfined compressive strength and fractured rock in terms of shearing resistance along fracture surfaces, but for inclusion in a soil slope stability model the parameters of cohesion and friction angle are used. Typical values of compressive strength for weathered, porous sandstone are above 5,000 psi, or 720,000 psf (Krynine and Judd, *Principles of Engineering Geology and Geotechnics*, 1957). For rock under 10 feet of cover, this translates into an equivalent of cohesion of 720,000 psf with zero friction angle or cohesion of 718,000 psf with a friction angle of 60 degrees. In fractured bedrock the cohesion portion of strength may be lost but the secant friction angle, which includes components for dilation resistance

and resistance to pushing of grains, remains and would likely be at least 45-50 degrees. In any case, the parameters for bedrock used in the analysis are extremely conservative and show that no part of a failure surface would pass through the bedrock.

It is clear that the intrinsic variability of the foregoing properties will fall in a range for which the selected values provide a conservative limit.

Concerning the issue of tailing heterogeneity, the comment appears to be asking for specific exploration and test data on physical properties of tailings. Such data are not available and collection of such data is in our opinion infeasible. Hydraulically placed tailings in active or recently active impoundments are mostly saturated, soft or easily liquefied, and have very low strength. Consequently, they are unable to support the static weight and vibrations imposed by drilling and sampling equipment. In circumstances where sampling has been attempted from structures within the impoundments that can support equipment (e.g., Homestake Grants sand-fill dikes), the samples were disturbed and not demonstrably representative of the tailings. Therefore, while IUSA agrees that multiple samples and three-dimensional characterization of tailings may be an ideal objective, it would not appear to be a technically feasible objective.

An alternative, IUSA suggests the following approach to:

- Observe (monitor) settlements. Settlements reflect the cumulative effects of tailing variability. The cumulative effects, rather than the variations themselves, are what is more critical to reclamation and long-term cover performance.
- Use empirical data from other Title II tailing impoundments that have already been reclaimed to predict tailings behavior. Reclaimed impoundments with similar tailing thicknesses and tailing placement methods might provide data that would support better prediction of settlement, liquefaction, etc. than standard soil-engineering tests performed on disturbed, perhaps non-representative samples from the White Mesa impoundments.

For the reasons listed above, IUSA believes that it would be unproductive to conduct tests on samples taken from the White Mesa tailing slurry or impoundments. The mill has, and will in the future, handle ores from many different mines and host rock formations. To date the mill has processed ores from over 130 individual mines located in Utah, Colorado and Arizona. Therefore, the tailings will vary in percent fines, mineralogical composition, and particle shape, all of which affect the properties and behavior of the deposited tailings. The alternative approach proposed above is especially appropriate for the White Mesa Mill, which, in contrast to most Title II sites, is active and will remain so for several decades. For sites that are inactive and in the process of reclamation, predictive calculations may be necessary, but, at White Mesa, IUSA has the time and opportunity to conduct observation of real behavior, eliminating the need for predictive calculations.

5. *Technical support is required for the contention that the slope stability analysis is conservative.*

*The sketch of the cross section for Dam No. 4 provided with Attachment 3 of IUSA's December 16, 1997 response, shows the top of the bedrock to be inferred from borehole logs (which were not provided with the reclamation plan). This bedrock surface is closer to the toe of the slope than the hypothetical surface used in the slope stability analysis. IUSA justifies this inconsistency by stating that the analysis presented with a bedrock surface lower than actual would be conservative. A technical rationale is required for this statement.*

*Assessments conducted by the NRC staff suggest that the actual mode of failure may be quite different from the circular failure surface assumed by IUSA in the hypothetical case presented in the reclamation plan (a plot of the critical failure surface was not provided with the reclamation plan). The assessments suggest that the critical failure surface may not be circular and a portion of the failure surface may be bounded by the bedrock surface. Consequently, the failure of the slope may be determined by the cohesion and friction of the interface between the foundation material and the bedrock.*

*Therefore, IUSA should conduct and document in the reclamation plan an appropriate suite of analyses to determine the effects of bedrock close to the toe of the slope, specifically with respect to the mode of failure. In addition, the appropriateness of the assumption of a circular failure profile should be demonstrated.*

*In its analyses, IUSA also should:*

- investigate and analyze other potential failure modes using other analysis methods (e.g. the wedge method [Lambe and Whitman, 1979]).*
- address the potential for tensile crack formation, and if significant, analyze the possible effect on water infiltration and on the integrity and function of the cover.*
- measure (in the laboratory) or estimate (based on available published information) the properties of the bedrock interface. Estimated values should be justified, and a sensitivity analysis should be conducted to demonstrate the conservativeness of the assumed values.*
- include the effects of potential seismicity at the site (a horizontal ground acceleration of 0.12g) in the slope stability analysis.*

- indicate the measured location of the phreatic surface and, if appropriate, account for the effects of this surface in the slope stability analysis; and
- assess the potential impacts from earthquake-induced pore pressures.

REFERENCE: *Lambe, T.W., and R.V. Whitman, "Soil Mechanics, SI Version," New York, NY: John Wiley and Sons, 1979*

### Response #5

IUSA suggests that the NRC please review the Cell 4 Design which was submitted to the NRC February 8, 1989 with revisions submitted January 10, 1990. In this document a complete stability analysis was presented and accepted for the stability of the slope of the Cell 4 Dam. The embankment stability analysis is addressed in section 3.4 of the design document that includes analysis for both static and seismic loading at two embankment sections. The stability analysis assumed that the tailings were saturated and were completely fluid. It also assumed that the liner had completely failed and that the steady state seepage condition had been reached. It was based on these conditions that the statement concerning the conservatism of the analysis was made. Cell 4A was approved to operate as designed and constructed by Amendment 20 to the Source Materials License SUA-1358 on March 1, 1990.

In the White Mesa Mill Reclamation Plan submitted February 28, 1997, the Plan calls for the Cell 4A dike to be breached. In view of this, we assume that analysis of the continued stability of the dike after reclamation would not add to the Plan.

6. *Inconsistencies between figures and borehole logs should be resolved so that the position of the bedrock surface can be identified.*

*There appears to be inconsistency among the figures in Appendix G and the borehole logs from Chen & Associates given in that appendix. Without a discussion of the borehole logs and the figures, it is not possible to determine how the position of the bedrock surface was inferred from the borehole logs. For example, Figure 1 of Appendix G shows that the bedrock surface was encountered in two boreholes - Chen #29 and Chen #77. The borehole logs provided in Appendix G do not include these boreholes. If these two boreholes belong to the series of holes shown in Figure 3 (BH-15 through BH-28), then the boreholes named Chen #29 and Chen #77 cannot intersect the boundary of Cell 4. If these holes belong to the series of borehole logs given in Appendix G (Hole 1 through Hole 75), then information about borehole Chen #77 is missing.*

*Also, the surface elevation contours given in Figure 3 do not match the borehole logs provided. For example, according to the contours in the figure, the collar elevation of BH-*



*24 should be between 5570 feet and 5580 feet. However, according to the borehole logs, BH-24 is between 5601 and 5609 feet.*

*Therefore, IUSA should resolve the inconsistencies among the figures and the borehole logs and provide the missing information.*

#### **Response #6**

The inconsistency of drill hole data comes from an incomplete data set being provided. Drill holes designated with a BH are holes drilled by Dames and Moore while the holes drilled by Chen and Associates have no prefix. The attached copy of Appendix B from the Cell 4 design submitted to the NRC in August of 1988 (Attachment 1) provides a better explanation of drill hole series, locations, elevations and drill logs. The map included with this attachment provides an overall view of the site that better explains the locations of the various drill hole series, with corresponding collar elevations.

Concerning the specific example pointed out in this comment, the collar elevation indicated by the map contours (before the cover stockpiles were placed) for the Chen series hole 24 is between 5600 and 5610 which corresponds with the drill log collar elevation of 5609. Drill hole BH-24 (Dames & Moore drill hole 24) has a map collar elevation between 5570 and 5580, with the drill log indicating a collar elevation of 5573. The confusion comes from having two series of drill holes with similar numbering systems and only having a piece of the map without the legend. The attached information should clarify the earlier confusion.

*The proposed frequencies of QC tests for controlling the quality of the construction of the final disposal cell should be modified.*

*The NRC Staff Technical Position (STP) on testing and inspection plans (NRC 1989) provides guidelines for the frequencies of QC tests deemed acceptable by the staff for disposal cell construction. IUSA is proposing to perform these QC tests with frequencies significantly less than those recommended in the STP.*

*The objective of the recommended frequencies for the different tests is to ensure that acceptable construction quality of the disposal cells can be achieved given the importance to public health and safety and required long life, as specified in 10 CFR Part 40, Appendix A, Criterion 6(i). The frequencies of testing recommended by the NRC staff are consistent with standard industry practice (e.g., the U.S. Departments of the Army, Navy, and Air Force, the U.S. Bureau of Reclamation) and have been adopted by licensees at other UMTRCA Title II sites.*

*While the NRC staff recognizes that construction methodologies and material testing frequencies should reflect site-specific conditions, the staff considers it inappropriate at this stage in the reclamation process to commit to the proposed frequencies absent actual*

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*construction data. Therefore, the staff requests that IUSA modify its QC plan to meet the recommendations in the STP. Once IUSA can demonstrate using actual construction data that acceptable quality can be achieved with QC tests less frequent than those specified in the STP, the testing frequencies and associated surety assessment can be modified appropriately.*

#### **Response #7**

The NRC's referenced STP was developed for reclamation of Title I sites, and as such is acknowledged to be very conservative, especially for Title II sites. In 1988 specifications were approved by the NRC for construction of the Cell 4A dikes and clay base. The QC requirements in those specifications are, for embankment (dike) construction:

- Field density and moisture - one test per 1,000 cubic yards (cy) and per lift of fill
- Particle-size analysis and Atterberg limits - one per 5,000 cy
- Standard Proctor tests - one per 10,000 cy

and for the clay base:

- Field density and moisture - one test per 500 cubic yards (cy) and per lift of fill
- Particle-size analysis and Atterberg limits - one per 5,000 cy
- Standard Proctor tests - one per 10,000 cy

IUSA believes that NRC's desire for a demonstration of acceptable quality by actual construction data has already been satisfied by its approval of both the construction QC provisions and the construction results of Cell 4A. IUSA will be using the same borrow material in the platform fill and frost barrier fill that was used in the 4A dike construction therefore, IUSA would ask that this experience and the success that came be considered to enable IUSA to propose as alternate requirements the QC plan proposed here in. More frequent tests, particularly the Standard Proctor test, would be necessary for low volume structural fills, but less important where the fill materials come from relatively homogenous stockpiles and a large volume of material is placed daily.

IUSA proposes that testing of fill materials and in-place density and moisture will be performed by a qualified materials testing service contracted by IUSA or by IUSA staff trained for these duties. In line with the reasoning provided above, IUSA proposes the following QC testing program for the tailing covers:

- a. Particle-size analysis - One test by ASTM Method D 422 (no hydrometer analysis will be run but samples will be washed over a #200 sieve) for each 5,000 cy
- b. Atterberg Limits - Not less than one test per 5,000 cy
- c. In-place density and moisture of compacted fill - One test per 500 cubic yards of clay layer and one test per 1,000 cy of platform fill (top one foot only) and frost barrier fill, using the nuclear density gauge according to ASTM D 2922, with moisture determined per ASTM D 3017. The Sand Cone method, ASTM D 1556, will be used to check density measurements at the rate of one Sand Cone test for every 10 nuclear density tests.
- d. Moisture-density standard - Standard Proctor density test using ASTM D 698, and ASTM Methods D 2216 or D 4643 for moisture content will be performed at a frequency of one test per 10,000 cubic yards of fill placed.

Each field density test will be plotted on an earthwork control grid and recorded on test data sheets that become part of the permanent record of the project.

*1. Additional information and analysis are necessary to address the potential for cover cracking due to liquefaction*

*The tailings properties indicate that the material has the potential for liquefaction, and in its December 16, 1997, response, IUSA acknowledges this potential. The potential for liquefaction must be assessed to demonstrate that any resulting damage would be minor and would not cause cover damage.*

*Therefore, IUSA should evaluate the potential for liquefaction at several locations within the impoundment in order to provide adequate areal coverage. This evaluation should be based on laboratory and/or field tests and pore pressure measurements, if necessary. Methods used for interpreting test data and assessing liquefaction potential should be consistent with current practice in geotechnical engineering (Seed and Idriss, 1971 and 1982; Seed, 1994).*

*As a minimum, potentially liquefiable zones should be identified based on index properties and gradation test results for the maximum credible earthquake assessed in Appendix G of the reclamation plan. If the extent of potentially liquefiable zones is local or minor, the effects on stability, assuming zero material strength, should be assessed and the cover integrity should be demonstrated. However, if the potential for liquefaction is assessed to be significant (e.g., involving the entire impoundment and/or the embankment), mitigation measures or redesign of the tailings ponds and/or the embankments should be proposed.*

REFERENCES: Seed, H.B. and Idriss, I.M., 1971, "A Simplified procedure for Evaluating Soil Liquefaction Potential," *ASCE Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 97, No. SM 9, pp. 1249-1274.

Seed, H.B., and Idriss, I.M., 1982, "Ground Motions and Soil Liquefaction During Earthquake," *Earthquake Engineering Research Institute, Engineering Monograph*, Vol. 5

Seed, R.B., 1994, "Introduction to Evaluation of Potential Liquefaction Hazard - Advances in Earthquake Engineering Practice," *Workshop at the University of California at Berkeley*, May 31-June 4, 1994

### Response #8

IUSA recognizes that the saturated sand and non-plastic fines portions of the tailings have the potential for liquefaction. However, the NRC's request for quantitative assessment of liquefaction potential and resulting cover damage carries with it the assumption that:

- The tailings can be characterized spatially and physically enough to quantify liquefaction potential, and
- The structural response of the cover can be also characterized well enough both spatially and physically to quantify damage, if any, resulting from liquefaction.

In IUSA's responses above to previous questions, it was stated that the tailings in their present and near-future state are very difficult, if possible at all, to drill and sample sufficiently to develop a three-dimensional model of the tailings on which quantitative analyses depend. Nor can undisturbed samples be obtained on which tests can be performed to measure the parameters critical to quantification of liquefaction potential (e.g., density and water content). With any exploration method that penetrates the tailings, there is a risk of also penetrating the liner system, thereby compromising containment of the tailings solution. IUSA is concerned that it is not practicable to obtain the quantitative results sought by the NRC at this time.

A qualitative assessment of liquefaction can be made, based on what is known about the ores, the mill grind, and the method of hydraulic placement of tailings. As stated above, the mill has processed ore from many different mines and ore bodies, so the chemistry and clay content of the tailings will vary spatially and nonuniformly through the impoundments. The tailings were deposited by discharging toward the center of the ponds from points along the dikes. Tailing solids settled out of the slurry from coarser to finer fractions away from the discharge points, resulting in concentration of slimes in the middle of each pond. Nevertheless, in detail there will be considerable variation in this pattern due to the development of deltas and backwater areas between deltas, movement of discharge points over time, and variations in grain-size distributions and clay fractions in the slurry. The variations in any horizon can be understood best by examination of aerial photos

of the ponds, which show the aforementioned features. Through any vertical section the ponds will have interfingering lenses of tailing sands and slimes, making the differentiation of physical properties both intricate and gradational. In summary, IUSA believes that a quantitative assessment would require so many assumptions and generalizations that the results would not be credible.

Overall, the tailings are classified as SC to SM material. They contain some plasticity, a property that resists liquefaction. Fines with some plasticity are probably clays, and these particles are most likely to be concentrated in the center of the ponds. These fines are typically the most compressible portions of the tailings, so settlements can be expected to be greatest over the centers of the impoundments. The tailings are probably sandier (SP to SM) and freer of plastic fines closest to the discharge points along the dikes; consequently, liquefaction potential is likely to be greater near the edges of the impoundments than in the middle. Although, as the ponds begin to fill the discharge points are moved closer to the center areas which further contributes to variability of the contained tailings.

Empirical data and engineering analyses (E. Arango, 1996, "AMagnitude Scaling Factors for Soil Liquefaction Evaluations, *a Journal of Geotechnical Engineering*, Nov. 1996, Attachment 3) show that tailings will not liquefy under seismic loading from events having a magnitude of 4.75 or less at the site. Figures 1.6-3 and 1.6-4 of the Reclamation Plan show that there have been no historic events of this magnitude within 100 miles of the site. Should a seismic event large enough to cause liquefaction occur after the covers are in place, the saturated sand and non-plastic fines portion of the tailings would be expected to liquefy and momentarily lose all strength. The critical factors that would control the damage to the cover are:

- The spatial distribution of liquefaction (variability in liquefaction over the impoundment areas).
- The containment of the liquefied tailings provided by the cover (ability of cover to bend without cracking).
- The rate at which excess pore pressure can dissipate
- The amount of lateral displacement of tailings under the cover during and after liquefaction

It is likely that liquefaction will not be uniform because the tailings are not uniform. Granular, non-plastic material would liquefy but plastic tailings would not. If the excess pore pressures last long enough (an indeterminate time) liquefied sands could be displaced by low-strength plastic fines that shear and move into liquefied zones when they lose lateral confinement. The displaced sands would flow into space vacated by the plastic fines. This process would continue only as long as the excess pore pressures remain; excess pore pressures would probably be relieved first near the dikes, where

pore liquid can migrate into the nonliquefied platform fill, or through cracks in the cover. Whether the process continues to this point depends on how well the cover contains the liquefied tailings, i.e. how flexible and strong the cover is and how much unsaturated tailings exist over saturated tailings. These factors will change over time, improving as the tailings dewater (through evaporation and capillarity) and consolidate.

IUSA recognizes that tailing liquefaction is possible as long as some portion of the non-plastic tailings remain saturated. The impoundment dikes (embankments) are constructed of compacted random fill and are not saturated, therefore, the dikes are not susceptible to liquefaction. These dikes were designed to contain liquid (full hydrostatic pressure) so they will contain completely liquefied tailings.

IUSA is unaware of a technically defensible solution to the liquefaction potential

It is important to note that even a few random cracks in the cover would not be expected to significantly reduce the cover's effectiveness in retarding radon flux. According to Regulatory Guide 3.64, page 3, cracks must be at least 2 cm wide, must be spaced less than 1m apart, and must penetrate at least 75% of the cover thickness to cause the radon flux to double. This scenario is extremely unlikely, given the variability in liquefaction potential over the total areas of the ponds.

9. *Additional information and analysis are necessary in the calculation of the spatial variation of settlement and any potential for cover cracking due to this differential settlement. This calculation requires estimation of the settlement at various locations to provide an adequate areal coverage. Moreover, due to the heterogeneity of the tailings materials, a settlement analysis that takes into account the spatial variability of the material properties is indispensable to developing an adequate settlement monitoring program.*

*In general, monitoring stations or monuments are not placed quickly enough to record the original conditions. Consequently, the estimation of time to 90 percent consolidation (i.e., 1999) from field measurement alone becomes extremely difficult without auxiliary analyses for estimating settlement with location-specific parameter values.*

*Therefore, to provide an adequate areal coverage, IUSA should estimate the settlement due to self-weight and the construction of the cover at several locations. These estimates should be based on the known or measured distribution of materials (e.g., sand, silt, etc.) at each location and laboratory-measured consolidation parameter values. Settlement should be calculated at each settlement platform and monitoring well and, if necessary, pore pressure measurements should be provided to confirm 90 percent consolidation values.*

*IUSA also should calculate the maximum differential settlement and analyze the strain on the clay cover due to the differential settlement. In addition, the capability of the proposed*

*clay to withstand the settlement strain without developing cracks should be demonstrated. These assessments are particularly important considering that the proposed thickness of the clay cover is thin (one foot) when compared to the standard practice in geotechnical engineering (Bennett and Kimbrell, 1991).*

*Finally, additional settlement that may occur as a result of the volume change of tailings during a seismic event (even in the absence of a liquefaction scenario) should be considered in assessing the settlement of the disposal cell cover.*

REFERENCE: Bennet, R.D. and Kimbrell, A.F. 1991. "Recommendations to the NRC for Soil Cover Systems Over Uranium Mill Tailings and Low-Level Radioactive Wastes: Construction Methods for Sealing Penetrations in Soil Covers." NUREG/CR-5432, Vol. 3.

### **Response #9**

HSA's response to comment #8 explains that tailing characterization based on drilling, sampling and testing of tailings is not feasible. Without this spatially distributed quantification of physical properties, it is not possible to perform meaningful calculations of total settlements and differential settlements. Even if a large number of samples could be obtained, they would represent a very small percentage of the total tailings, leaving any analyses still dependent on a substantial amount of engineering judgement. Settlement of the cover can result from:

- Displacement of tailings or cover soils (structural deformations)
- Consolidation of tailings and/or cover soils (volumetric contraction)

Consolidation could cause settlement by reducing porosity and, in turn, the volume of tailings. It is doubtful that calculations of settlement using standard consolidation theory (Terzaghi et al. 1996) would produce meaningful results. Calculations using consolidation theory require quantification of:

- Soil profile, specifically the number, depth and thickness of compressible layers
- Boundary conditions of each compressible layer, whether they are or are not free draining
- Values for the critical properties of all layers: density and void ratio, water content, compression index

Some assumptions are also associated with consolidation theory.

- Permeability remains constant during consolidation
- Void ratio vs effective stress remains a linear relationship independent of time

At the White Mesa mill site it is not possible at this time to quantify the necessary parameters, and the necessary assumptions may not be valid, in part because the parameters cannot be dependably quantified. Furthermore, ores being processed now and in the future are coming from many different sources with widely varying properties so tailings properties may vary accordingly. As an alternative, order-of-magnitude estimates of consolidation and settlement based on volumetric compressibility and empirical data from similar sites can be made, but these have very limited benefit.

As an alternative, HUSA proposes that settlement evaluation based on settlement monitoring is more appropriate for the White Mesa tailings than exploration and testing followed by calculations. Settlement data collected to date are shown in Attachment 4. Additional settlement monuments will be installed on a regular grid, illustrated in Figure 1 of Attachment 4, and measured on a schedule that will provide data to evaluate settlement rates and patterns. From these data, estimates can be made of the amount of settlement to be expected and the time until 90% of primary settlement is achieved.

It is apparent that settlement monuments can be installed only when ground conditions allow, i.e., when the tailing surface is stable enough to support men and equipment needed for monument installation. However, this time will arrive before the time when a drilling rig can be operated at the same place, so settlement data can be collected sooner than samples can be collected for laboratory testing.

The relative merit of HUSA's proposed approach can be understood from the experience of Homestake Mining Company (HMC) at its Grants mill site in predicting and monitoring settlement. In response to a request from the NRC, HMC obtained samples of tailings from the old (inactive since 1962) tailing pile. Despite an experienced driller, good equipment, and "undisturbed" sampling procedures, the samples were clearly disturbed. Slimes were tested for consolidation parameters, and the results were input to a calculation using consolidation theory. That calculation predicted total primary settlement of 3-4 feet for a tailing slime section about 80 feet thick.

Once the large tailing impoundment was recontoured and the slimes covered enough to support workers, 52 monitor points were installed on a regular 300x300-foot grid pattern covering an area 3150 feet by 1250 feet, or about 90 acres. Settlement readings were made initially at least once each week, then about every 20 days. Cumulative settlements were plotted and analyzed for total settlement, rate of settlement, and changes in rate of settlement. During the course of these measurements, another calculation was performed to predict settlement based on test results of samples taken from a drill hole on the large impoundment. Samples from this hole were also clearly



disturbed, and showed interbedded slimes and tailing sand. The calculated total settlement, using test results and consolidation theory, was about three feet. A separate calculation using volumetric compressibility predicted that with worst-case assumptions the total primary settlement could be as much as eleven feet. The actual settlement measured at the nearest monument was more than three feet after less than one year and was over five feet after two years.

This example shows that methods of geotechnical engineering developed for natural soils are not necessarily appropriate for tailings, and that the alternative observational approach proposed by IUSA, relying on settlement measurements rather than predictive calculations, produces site-specific and useful data that can be used in decision-making by both NRC and IUSA.

White Mesa has decades to operate, making the observational approach all the more appropriate; therefore, IUSA proposes to address NRC's concerns about settlement using the observational approach. IUSA will install additional settlement monuments, as shown on Figure 1 of Attachment 4, extending its existing array, over both tailing ponds as fast as ground conditions will allow safe access to and work at each point. The existing monuments will continue to be monitored, with settlements plotted against real time. Monuments installed in the future will conform to the design shown on the accompanying drawing. The monitoring record will include data indicating the location, thickness and date of platform fill placed as interim cover during mill operations. During and after reclamation, the monuments will be monitored on a regular schedule, to be based on observed settlement rates, until 90% of primary consolidation has been achieved, as indicated by the time-settlement curve becoming asymptotic. Alternatively, the end of settlement monitoring may be determined by the annualized rate of settlement, based on the average of the five most recent settlement readings, becoming less than 1% of the total settlement.

We also note that it is not necessary to know the zero point, or start of settlement, for the proposed approach to be used because it does not depend on fixing the initial point of a settlement curve, which is virtually impossible to measure for tailings. The shape and asymptote of the curve are used to evaluate settlement progress, and these are determinable through the course of settlement measurements after monuments are installed.

10. *Additional information addressing the details of disposal cell construction should be provided*

*Details of disposal cell construction are necessary to support NRC's detailed analysis of IUSA's proposed reclamation plans and surety estimate. The minimum required information includes methods, procedures, and requirements for excavating, hauling, stockpiling, and placing contaminated and non-contaminated materials, and other disposal cell materials. The procedures for material placement and compaction should be adequate to achieve the desired moisture content, placement density and permeability. Use of acceptable procedures, such as the recommendations provided in NUREG/CR-5041 (Densen, et al*

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*1987) for gradation, placement, and compaction necessary to achieve design drainage rates and volumes, to prevent internal erosion or piping, and to allow for collection and removal of liquids should be confirmed. Compaction specifications should include restrictions on work related to adverse weather conditions (e.g., rainfall, freezing conditions, etc.). Plans, specifications, and requirements for disposal cell compaction should be supported by field and laboratory tests and analyses to assure stability and reliable performance.*

*A plan for settlement measurement that is satisfactory for producing representative settlement data throughout the disposal cells is required. Settlement measurement stations should be of sufficient coverage and should be strategically placed to yield adequate information for determination of differential and residual settlements. Monitoring monuments should be designed to be durable.*

*The proposed frequency of monitoring should conform to acceptable practice (e.g., NRC, 1989). Procedures and specifications for use of riprap, rock mulch, and for filter placement should be provided and should be consistent with commonly accepted engineering practice and the design specifications (NRC, 1977 and 1982). The construction sequence should be described and demonstrated to be adequate to achieve the intended configuration for the tailings. The proposed time to completion should be reasonably achievable. Appropriate QC provisions should be provided to ensure that the construction will be in accordance with the reclamation plan, and that the appropriate records will be maintained.*

*USA should provide the information necessary to support an elevation of disposal cell construction.*

REFERENCES: Densen et al., 1987, "Recommendations to the NRC for Review Criteria for Alternative Methods of Low-Level Radioactive Waste Disposal," NUREG/CR-5041

NRC, December 1977, "Design, Construction, and Inspection of Embankment Retention Systems for Uranium Mills," Regulatory Guide 3.11, Revision 2

NRC, August 1982, "Rock Riprap Design Methods and their Applicability to Long-Term Protection of Uranium Mill Tailings Impoundments," NUREG-2684

NRC, January 1989, "Staff Technical Position on Testing and Inspection Plans During Construction of DOE's Remedial Action at Inactive Uranium Mill Tailings Sites."

## **Response #10**

Details of the construction of the disposal cells has been provided to the NRC through the following documents:

Cell 1-I and Cell 2 - Construction Report: Initial Phase - Tailing Management System, White Mesa Uranium Project Blanding, Utah - D'Appolonia, submitted to NRC March 1, 1982

Cell 3 - Second Phase Engineering Design - Tailing Management System, White Mesa Uranium Project Blanding, Utah - D'Appolonia, submitted to NRC May 20, 1981

Cell 3 - Construction Report: Second Phase - Tailings Management System - White Mesa Uranium Project - Energy Fuels Nuclear, Inc., submitted to NRC March 4, 1983

Cell 4A - Cell 4 Design - Tailing Management System - White Mesa Project, Blanding, Utah - UMETCO Minerals Corp., submitted NRC February 8, 1989, revised January 10, 1990

Figure 1 of Attachment 4 shows the location of existing and proposed settlement monitoring locations on existing Cells 2 and 3.

Settlement monitors are monitored on a monthly basis utilizing standard surveying practices for vertical control. All elevations are referenced to an established benchmark located on solid ground away from the tailings cells. Settlement for each point is tracked utilizing a spreadsheet program and graphs. Figure 2 of Attachment 4 represents the design of the settlement monitors.

11. *The reclamation plan should address radionuclides other than radium-226 that may require cleanup.*

*The verification survey, as described in Appendix A of the reclamation plan, describes a method to ensure that only radium-226 concentrations will be less than NRC requirements. Other radionuclides, including thorium-230 and uranium, may also be present in the soil. It is noted that IUSA states in Section 3.3.2 of Appendix A of the reclamation plan that "[t]he facility currently monitors soils for the presence of Ra-226, such results being presented in the second semiannual effluent Ra-226-Unit report for each year. Guideline values for these two materials will be determined and will form the basis for the cleanup of the White Mesa Mill..." This statement appears to imply that two radionuclides will be sampled during reclamation. However, if this is the case, the second radionuclide is not identified.*

*IUSA should either (1) provide technical justification for not including radionuclides other than radium-226 in the soil cleanup plan, or (2) describe the methodology that will be used to determine that other radionuclides have been reduced to acceptable levels.*

### Response #11

IUSA presently monitors for both Radium-226 and natural Uranium. It is planned to use both of these radionuclides as a basis for soil cleanup at the site.

12. *The value proposed as the background radium concentration in the soil should be provided*

*In accordance with Appendix A of 10 CFR Part 40, during the final site cleanup, the soil must be reclaimed such that the average radium concentration in the soil does not exceed the background level by more than 5 picocuries per gram (pCi/g) in the upper 15 centimeters (cm) or by more than 15 pCi/g in 15 cm sections below the upper 15 cm. In order to comply with these regulations, a value for the background radium concentration in the soil must be determined. In the reclamation plan, IUSA has provided the historical soil sampling data, but it has not committed to a value for the background radium concentration in soil*

*IUSA should provide the value for background radium concentration in soil that will be used to determine the soil cleanup standards during decommissioning.*

### Response #12

Based on soil sampling data gathered over a 16 year period (Attachment 5) at sample station BHV-3 located upwind and 5 miles west of the White Mesa mill the Radium background concentration in the vicinity of the mill is 0.93 pCi/gram. This value will be used by IUSA as an interim value for the background concentration. Prior to initiating cleanup of wind blown contamination IUSA will conduct a systematic soil sampling program in an area within five miles from the site to determine the average background Radium concentration to be ultimately used for the cleanup.

13. *Technical support should be provided for the scanning rate used for windblown contamination during mill decommissioning*

*On the fifth page of the mill decommissioning calculations, the rate of scanning used for the calculation of the time required to scan for windblown contamination is 0.5 meters per second (m/sec). According to NUREG/CR-5849 (NRC, 1992), the recommended scanning rate for a ground survey is 0.5 m/sec. Of concern to the staff is the potential for IUSA's use of a 0.5 m/sec scanning rate to result in inaccurate surveys.*

*IUSA should adopt the scanning rate recommended in NUREG/CR-5849, or alternately, provide a technical justification that its proposed scanning rate will provide accurate survey results. If the recommended scanning rate is adopted, IUSA should revise its decommissioning costs to reflect the time and associated costs needed to complete these surveys*

REFERENCE: NRC, 1992, "Manual for Conducting Radiological Surveys in Support of License Termination," NUREG/CR-5849.

### Response #13

IUSA has reviewed the calculations in question and finds that although the recommended scanning rate in NUREG/CR-5849 (NRC, 1992) was stated in the assumptions for the calculation an incorrect rate (1.5 meters/second) was actually used. Attachment 6 shows the corrected calculation based on an eight hour work day with an efficiency factor of 80%. This calculation is based on all areas being surveyed on a 30 X 30 meter grid, when in fact the reclamation plan states that the "halo" areas will be surveyed on a 50 X 50 meter grid which gives a more conservative cost. Based on the results of this corrected calculation the cost of the project would increase by \$2,456.00 based on the cost per man hour for this task. This extra cost is less than 0.2% of the total cost of the mill decommissioning. IUSA proposes that this adjustment to the total reclamation cost estimate be made upon final approval of the Reclamation Plan and the associated cost estimate.

14. *Additional information about the location of samp<sup>l</sup> taken to characterize the radon barrier materials is required.*

*In Appendix A of the Tailings Cover Design (Titan Environmental, 1996), data is provided for the tailings and materials that will be used to construct the tailings cover. However, insufficient information is given about the location and depth of the samples, which is necessary to determine whether these samples are representative of the long-term properties of all materials on site. The samples at question include:*

- (a) The samples labeled "Tailings," "Composite (21, 3 and 5), "Site #1," and "Site #4," in a March 4, 1998, Rogers and Associates Engineering Corporation report.*
- (b) The samples labeled "Random (2, 3 and 5), "Site 1," and "Site 4" in a May 9, 1988, Rogers and Associates report.*
- (c) The sample labeled "UT-1" in a 1996 Advanced Terra Testing report.*
- (d) The sample labeled "UT-1" in a September 3, 1996, Rogers and Associates report, and*
- (e) The samples labeled "Test Pit 1, 2 and 3" in a March 8, 1982, D'Appolonia report (depth of samples only is needed).*

*For the above listed samples, IUSA should provide a map showing the sampling locations and the depths at which these samples were acquired.*

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REFERENCE: Titan Environmental, "Tailings Cover Design White Mesa Mill, October 1996 for Reclamation of White Mesa Facilities, Blanding, Utah," prepared for Energy Fuels Nuclear, Inc., Denver, Colorado, October 1996.

#### Response #14

Please refer to the IUSA response to #3 above concerning the location and characterization of samples for details concerning a, b, c, and d.

Concerning the samples labeled "Test Pit 1, 2, and 3" in the March 8, 1982, D'Appolonia report, these sample were taken from the bottom of bulldozer cuts which were excavated 10 feet deep to expose the undisturbed clay layer. As stated in the response to Question 3, sample UT-1 came from the same test pits.

15. *Confidence limits are required for the guideline value for the correlation between gamma readings and the Ra-226 concentration.*

*On page 8 of its December 16, 1997 response, IUSA states that a correlation will be performed between gamma readings and the Ra-226 concentration. However, IUSA has not specified the confidence limit for the guideline value to be used in this correlation. In the past, the NRC staff has found this method of correlating gamma readings to soil radium concentration acceptable when the guideline value is set at the lower 95 percent confidence limit of the correlation.*

*IUSA should indicate the confidence limit it will use in making the correlation between gamma readings and radium concentration from which the guideline gamma reading value will be established.*

#### Response #15

IUSA agrees with NRC's comment and commits to the 95 percent confidence limit fee the guideline value for correlation between gamma readings and the Ra-226 concentration.

16. *SURFACE WATER HYDROLOGY AND EROSION PROTECTION*

*Additional information and analyses are needed to show that the breached area of Cell No. 4A is protected adequately from erosion.*

*Because there is a potential for gully headcutting to occur and potentially affect the reclaimed tailings area, erosion protection may be needed to mitigate the effects of local scouring and future gully headcutting. Therefore, IUSA should provide estimates of the peak*

*probable maximum flood (PMF) flow through the breach, water surface profiles, channel velocities, riprap needed, scour depths, and other design information including drawings, calculations, and analyses.*

#### **Response #16**

IUSA provided partial response to this comment in previous submittals to the NRC; in addition, the attached information (Attachments 7 through 11) have been prepared.

In the 1990 Hydrologic Design Report, IUSA showed that the six-hour PMP was 10.0 inches. This design storm is adjusted for elevation and distributed over time for both the six-hour and one-hour durations on Attachment 7 and plotted on attachments 8 and 9 in accordance with methods of HMR 49. The peak discharge of this storm (PMF) through the Cell 4A breach, 2057 cfs, is shown in the tabulation of the Rational Method calculation (Attachment 10), along with the peak depths and velocities for channel bed widths of 60 and 80 feet on both sand soil and bedrock. If the channel is in sandy soils (as expected), the scour from the PMF could be 3-4 feet *if the duration of the flood is not taken into account*. However, PMF is a once-ever, very brief event that would not be expected to last for enough time to cause 3-4 feet of scour, which is calculated on the assumption of long-term or repetitive flows of the design peak magnitude, which is *extremely* conservative.

In any case, no riprap would be required if the breach were cut into bedrock, because the allowable velocity over the rock is about the same as the peak PMF velocity. If the breach is cut into sandy soil, riprap with  $d_{50}$  of 9.5 to 11 inches would be applied to the channel and slopes, up to the PMF peak depth mark, in a layer at least 1.5 feet thick. The downstream end of the channel riprap would terminate in a key, cut into the subgrade, 4.0 feet wide by 4.0 feet deep and extending across the entire channel. The key will be terminated at the depth of bedrock if less than 4.0 feet below grade.

#### **17. Additional information is needed to assess the adequacy of the discharge channel**

*To evaluate the adequacy of the discharge channel, it is necessary to know the drainage area contributing flow to the channel and the bases for IUSA's estimates of peak PMF flow rates, channel velocities, flood routings, etc. Such information, including detailed drawings and calculations, are needed to determine if the channel and its associated erosion protection are adequate. Therefore, IUSA should provide this information or, if such information has been provided to NRC in the past, IUSA should provide appropriate references.*

#### **Response #17**

The PMF peak discharge through the Cell #1 discharge channel was calculated in the same way as described above for comment #16. The drainage area, 143 acres, was measured from the most recent 1"=200' scale topo map of the site. The longest flow path is 4800 feet. The peak PMF discharge

was calculated to be 1344 cfs (Attachment 11). With a channel base width of 100 feet, the peak velocity through the channel would be 7.96 fps and the peak flow depth would be 1.62 feet. If the channel base is constructed to be 120 feet wide, the peak velocity would be 7.45 fps and the depth would be 1.45 feet. In either case, the channel will be in bedrock, with allowable velocities of 8 -10 fps, so riprap will not be necessary; i.e., there should be no scour from the PMF.

18. *Additional rock durability tests are necessary to characterize adequately the sandstone rock IUSA proposes to use as riprap.*

*One rock durability test is not considered to be adequate to document that the sandstone rock riprap is sufficiently durable to meet longevity requirements. Given the marginal quality of the rock, IUSA should conduct several durability tests taken from samples at the proposed source. Results of these tests should be provided to NRC.*

#### **Response #18**

IUSA agrees that additional testing needs to be done to confirm the integrity of the rock proposed for the riprap installations. A program for obtaining representative samples of the existing stockpiles of rock material, as well as selecting appropriate tests to confirm the acceptability of the onsite rock is being formulated. The details of this program will be forwarded to the NRC under separate cover. IUSA commits to having a sampling program in place and obtain samples by September 30, 1998. Laboratory testing and summary of results is expected to be complete by October 16, 1998.

As an alternative to the onsite rock, IUSA has located a developed source of limestone rock approximately 15 miles south of the Mill site. This rock was approved and utilized on the U.S. Department of Energy, Monticello Tailings Remedial Action Project, and would be available in the event the onsite rock is unacceptable. This rock would be only incrementally more costly than the onsite material because of less handling costs (excluding haulage) than the onsite material, and less waste through the screening and sizing operation. Adjustments to the reclamation cost estimate will be made in the event the off site source is preferable.

19. *Additional information is needed to describe the proposed rock toes.*

*Detailed drawings and calculations should be provided to document the design bases and to show the design configurations of the rock toes, particularly in those locations where the toes transition into areas where sandstone bedrock is present. The competency of the bedrock layer also should be assessed.*



## Response #19

Attached is a drawing (Figure 4 – Attachment 12) showing a plan view and cross section of a typical rock apron at the toe of cell #3 outslope. The apron is at least 7.0 feet wide and 2.0 feet thick with a surface slope of 0.01 ft/ft, and the  $d_{50}$  of the rock in the apron is 8.0 inches. The depth of scour in the sand soil predicted under PMP is 1.3 feet, so the apron thickness is greater than the scour depth. If bedrock is encountered at less than this depth, the key will be taken only to the top of rock. The calculations of rock sizes needed to resist erosion and the scour depth at the edge of the rock apron are included in table form with this response (Attachment 12).

It is not clear what purpose will be served by trying to assess the competency of the bedrock, or what is meant by Acompetency. *a* Whatever the properties of the bedrock, they will not be changed by anything that IUSA might do to achieve reclamation. The durability of the bedrock will be similar to that of sandstone samples from the site.

## 19. CLARIFICATIONS AND EDITORIALS

*The discussion concerning potential impacts to endangered and threatened species should be brought up-to-date.*

*In Section 1.7 of the reclamation plan, IUSA discusses the effects of the site on endangered and threatened species as of 1978, when the Environmental Report for the site was written. Between that time and the present, new species may have been placed on or removed from the endangered species list, and new species may have been observed on site.*

*IUSA should update its analysis of potential effects from reclamation activities on listed, proposed, or candidate endangered or threatened species. If appropriate, IUSA should confirm, at a minimum, that the evaluations conducted for the 1978 endangered species analysis still are applicable.*

## Response #20

During the preparation of Energy Fuels Nuclear's (EFN), the predecessor to IUSA, license renewal application for Source Material License SU-1358, NRC staff prepared an Environmental assessment (EA) which was issued on February 27, 1997 with a final finding of no significant impact (FONSI) prepared and issued on March 5, 1997. In this EA NRC staff addressed the issue of endangered species on the site as follows:

### 4.5 Impacts on Ecological Systems

#### 4.5.1 Endangered Species

In the vicinity of the site, four animal species classified as either endangered or threatened (i.e., the bald eagle (*Haliaeetus leucocephalus*), the American peregrine falcon (*Falco peregrinus anatum*), the black-footed ferret (*Mustela nigripes*), and the Southwestern willow flycatcher (*Empidonax traillii extimus*)) could occur. While the ranges of the bald eagle, peregrine falcon and willow flycatcher encompass the project area, their likelihood of utilizing the site is extremely low. The black-footed ferret has not been seen in Utah since 1952 and is not expected to occur any longer in the area.

No populations of fish are present on the project site, nor are any known to exist in the immediate area of the site. Four species of fish designated as endangered or threatened occur in the San Juan River 29 km (18 miles) south of the site. There are no discharges of mill effluents to surface waters, and therefore, no impacts are expected for the San Juan River due to operations of the White Mesa mill.

Currently, no designated endangered plant species occur on or near the plant site."

21. *Two references require clarification*

*In Section 3.2.3.2 of the reclamation plan, two references are made to information contained in Section A.3.2. The staff is unable to locate a Section 4.3.2.1 in the reclamation plan. U.S.A should clarify these references*

**Response #21**

The reference in Section 3.2.3.2 of the reclamation plan to Section 4.3.2.1 is incorrect. The correct reference should have been Section 3.2 in Attachment A (Plans and Specifications for Reclamation).

22. *An apparent inconsistency between values used for the moisture content of the clay and random fill should be resolved*

*On page 5 of Appendix B of the Tailings Cover Design (Titan, 1996), U.S.A states that the moisture content of the clay and random fill used for the radon flux calculations are 14.1 percent and 9.8 percent, respectively. These values are inconsistent with the values used to calculate the freeze-thaw effects on the cover, which are 13.9 percent for the clay and 11.8 percent for the random fill (Titan, 1996, Appendix E, page 3)*

**Response #22**

The moisture content of the clay and random fill used in the radon flux calculations and the values for the clay and random fill used for the freeze-thaw effects on the cover are unrelated numbers as

Mr. Joseph J. Holonich  
U.S. Nuclear Regulatory Commission

-29-

August 28, 1998

they were sets of assumed values for two different unrelated calculations. The moisture content used for the Freeze/Thaw calculation was actually 12.5% which is the average between the random fill moisture of 11.8% and the 13.9% of the clay. The difference in moisture content of the materials used in these calculations is also within a range that would be expected during normal construction activity and therefore not unreasonable.

23. *Applicability of replacement pages should be clarified*

*In Attachment 3 to IUSA's December 16, 1997, response, six pages from the "Geotechnical Data Base for Monticello Millsite Characterization" are provided as replacement pages for several illegible pages in IUSA's initial submittal. It is not clear how this data from the Monticello site is applicable or relevant to IUSA's reclamation activities and analyses. Therefore, IUSA should clarify the applicability of this data base to the present licensing action.*

### **Response #23**

The original data included in the Titan Tailing Cover Design report was submitted at a time when EFN (IUSA's predecessor) was considering accepting tailings material from the Monticello Tailing site. IUSA has not pursued this action, therefore the data for the Monticello site can be deleted from the submittal.

If you have any questions or comments concerning this information, please call.

Very truly yours,



Harold R. Roberts  
Executive Vice President

HRR/pl  
Enclosures

CC: Center for Nuclear Waste Regulatory Analysis  
ATTN: Patrick MacKin  
6220 Culebra Road  
P.O. Drawer 28512  
San Antonio, Texas 78228-0510  
(2 copies)

## **ATTACHMENT 1**

CELL 4 DESIGN  
TAILINGS MANAGEMENT SYSTEM

WHITE MESA PROJECT  
BLANDING, UTAH

APPENDIX B

Umetco Minerals Corporation

AUGUST, 1988

APPENDIX B  
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- 6 Table C-1, Summary of Laboratory Test Results - Cell 4

SECTION 1

FIGURE C-1

Boring and Test Pit Locations



chen and associates, inc.  
CONSULTING ENGINEERS



SOIL & FOUNDATION  
ENGINEERING

96 S. ZUNI

DENVER, COLORADO 80223

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1924 EAST FIRST STREET • CASPER, WYOMING 82601

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SECTION 2

Extracted Data From

SOIL PROPERTY STUDY  
EARTH LINED TAILINGS RETENTION CELLS  
WHITE MESA URANIUM PROJECT  
BLANDING, UTAH

Prepared for:

ENERGY FUELS NUCLEAR, INC.

PARK CENTRAL  
1515 ARAPAHOE STREET  
DENVER, COLORADO 80202

Job No. 16,406

July 18, 1978



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Note: The attached logs have been redrawn

TABLE 1  
SUMMARY OF LABORATORY TEST RESULTS

Page 1 of 2

Test Hole	Depth (ft.)	NATURAL		Maximum Dry Density (pcf)	Optimum Moisture Content (%)	ATTERBERG LIMITS		GRADATION ANALYSIS				REMOLED		PERMEABILITY		Specific Gravity	Soil Type
		Moisture Content (%)	Dry Density (pcf)			Liquid Limit (%)	Plasticity Index (%)	Maximum Size	Passing #200 (%)	Less than 2.0mm (%)	Dry Density (pcf)	Moisture Content (%)	ft./yr.	cm./sec.			
2	0-5			117.5	10.8	20	3	#16	58	19	111.6	16.4	0.51	$5.5 \times 10^{-7}$		Sandy Silt	
3	7-8	7.2				21	6	#16	62							Sandy Clayey Silt	
5	7 1/2-10			104.1	18.5	33	8	3/4 in.	56	12	102.1	22.0	0.015	$8.2 \times 10^{-8}$	2.65	Calcareous Silty Clay	
6	1-2	10.3				25	7	#16	77							Sandy Clayey Silt	
6	8 1/2-9	6.1				27	8	#4	70							Sandy Clay	
8	5-5 1/2	13.1					NP	3/4 in.	62							Calcareous Sand/ Silt	
9	0-1	8.1					NP	#16	53							Sand - Silt	
10	4-6 1/2					24	10	#4	73							Sandy Clay	
11	5 1/2-6 1/2	14.0				26	6	#16	65							Siltstone-Claystone	
12	2-5			101.0	20.6	53	35	#16	88	59	95.0	18.3	0.008	$6.6 \times 10^{-8}$	2.67	Weathered Claystone	
13	7-8	13.1				39	13	#8	84							Calcareous Silty Clay	
14	1-2	19.3				40	21	#4	89							Weathered Claystone	
15	1 1/2-4 1/2			106.8	19.0	26	8	3/8 in.	65	27	103.4	18.0	0.012	$1.2 \times 10^{-8}$	2.64	Mod. Calcareous Sand/ Clay	
17	2-3	11.4				19	4	#8	59							Sandy Silt	
19	0-3			117.5	12.8	23	6	#16	70		109.9	12.4	0.035	$3.4 \times 10^{-8}$		Sandy Clayey Silt	
22	1-2	13.2				26	10	#4	73							Sand/ Clay	
23	1-3					48	24	#30	87							Weathered Claystone	
23	6-3					61	30	#30	96							Claystone	
25	1-3 1/2	13.3				26	9	#4	57							Sandy Clay	
26	4 1/2-5	15.3				41	20	#4	91							Weathered Claystone	
28	0-2	12.7				28	10	3/8 in.	72							Sand/ Clay	
29	2-3	8.5				19	2	#16	59							Sandy Silt	
32	8-8 1/2	5.6				23	6	#30	73							Sandy Clayey Silt	
37	0-4			118.8	11.5	23	5	#8	72		110.5	11.5	0.63	$6.1 \times 10^{-7}$		Sandy Clayey Silt	
38	5-7			111.0	16.7	29	14	3/8 in.	69	27	102.4	17.9	0.041	$4.0 \times 10^{-8}$		Sandy Clay	
40	4-5 1/2			110.0	16.2	20	9	#8	64		106.4	16.4	0.012	$1.6 \times 10^{-8}$	2.65	Sandy Clay	

TABLE 1  
SUMMARY OF LABORATORY TEST RESULTS

Page 2 of 2

Test Hole	Depth (ft.)	NATURAL		Maximum Dry Density (pcf)	Optimum Moisture Content (%)	ATTERBERG LIMITS		GRADATION ANALYSIS			REMOLDED		PERMEABILITY		Specific Gravity	Soil Type
		Moisture Content (%)	Dry Density (pcf)			Liquid Limit (%)	Plasticity Index (%)	Maximum Size	Passing #200 (%)	Less than 2 $\mu$ (%)	Dry Density (pcf)	Moisture Content (%)	ft./yr.	cm./sec.		
40	9-9}	6.8				22	8	1/8 in.	60							Sandy Clay
42	13}-14}	7.0				26	10	3/8 in.	73							Sandy Clay
43	11-12	12.1				41	22	#4	86							Claystone
43	13}-16}		110.0	16.9		40	24	3/8 in.	85	44	104.1	15.8	0.074	$2.3 \times 10^{-8}$	2.62	Claystone
44	6}-7	7.5				30	11	3/8 in.	79							Calcareous Sandy Clay
46	0-2	12.3				22	6	#16	76							Sandy Clayey Silt
48	5-5}					30	9	3/8 in.	65							Sandy Clay
49	5-7		110.7	15.6		25	9	#16	71		105.2	13.9	0.33	$3.2 \times 10^{-8}$		Sandy Clay
49	14-15					28	5	#8	55							Calcareous Sandy Silt
54	0-2	12.1				23	9	#8	64							Sandy Clay
55	5-5}	7.8				28	14	#30	71							Sandy Clay
55	9}-10}					28	13	#4	71							Sandy Clay
58	5}-6	12.5				35	11	#4	75							Sandy, Silty Clay
61	0-1	11.5				21	4	#16	75							Sandy Silt
62	11-11}	8.1					NP	1 in.	34							Calcareous Sand & Silt
63	4-6					30	14	#8	68							Sandy Clay
65	1-2	9.0					NP	#16	44							Silty Sand
68	7}-8	8.6				28	13	#8	67							Sandy Clay
70	3}-4}	16.4				27	4	1 1/2 in.	46							Calcareous Sand & Silt
72	0-2	12.2				22	8	#16	59							Sandy Clay
75	10-11	12.4				41	25	#4	75							Weathered Claystone
75	12-14					45	22	#16	93							Claystone

10,400

TABLE 11

## LABORATORY PERMEABILITY TEST RESULTS

Sample	Soil Type	Compaction			Surcharge Pressure (psf)	Permeability	
		Dry Density (pcf)	Moisture Content (%)	% of ASTM D698		(Ft/Yr)	(Cm/Sec)
TH 2 @ 0'-5'	Sandy Silt	111.6	16.4	95	500	0.57	$5.5 \times 10^{-7}$
TH 5 @ 7½'-10'	Calcareous Silty Clay	102.1	22.0	101	500	0.085	$8.2 \times 10^{-8}$
TH 12 @ 2'-5'	Weathered Claystone	95.0	18.3	94	500	0.068	$6.6 \times 10^{-8}$
TH 15 @ 1½'-4½'	Calcareous Sandy Clay	103.4	18.0	97	500	0.012	$1.2 \times 10^{-8}$
TH 19 @ 0'-3'	Sandy, Clayey Silt	109.9	12.4	94	500	0.035	$3.4 \times 10^{-8}$
TH 37 @ 0'-4'	Sandy, Clayey Silt	110.5	11.5	93	500	0.63	$6.1 \times 10^{-7}$
TH 38 @ 5'-7'	Sandy Clay	102.4	17.9	92	500	0.041	$4.0 \times 10^{-8}$
TH 40 @ 4'-5½'	Sandy Clay	106.4	16.4	97	500	0.017	$1.6 \times 10^{-8}$
TH 43 @ 13½'-16½'	Claystone	104.1	15.8	95	500	0.024	$2.3 \times 10^{-8}$
TH 49 @ 5'-7'	Sandy Clay	105.2	13.9	95	500	0.33	$3.2 \times 10^{-7}$

TABLE III  
RESULTS OF ATTERBERG LIMITS

SAMPLE	SOIL TYPE	PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS			SHRINKAGE RATIO
			Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	
2 @ 0 - 5'	Sandy Silt	58	20	17	17.	1.81
5 @ 7½ - 10'	Calcareous Silty Clay	56	33	25	25	1.62
15 @ 1½-4½'	Calcareous Sandy Clay	65	26	18	17.5	1.76
19 @ 0-3'	Sandy, Clayey Silt	70	23	17	18	1.80
26 @ 4½-5'	Weathered Claystone	91	41	21	12	1.90
38 @ 5 - 7'	Sandy Clay	69	29	15	14	1.89

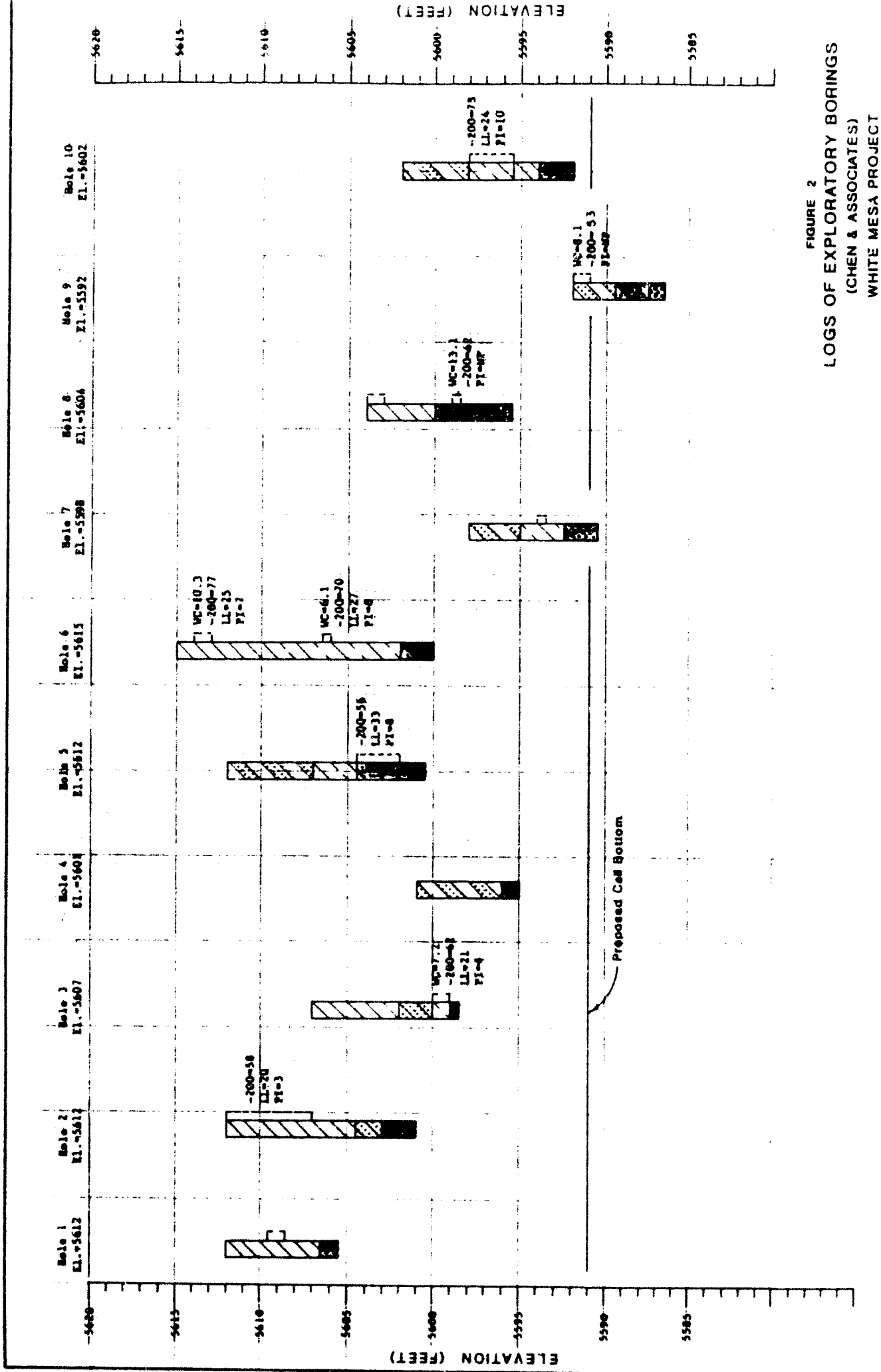
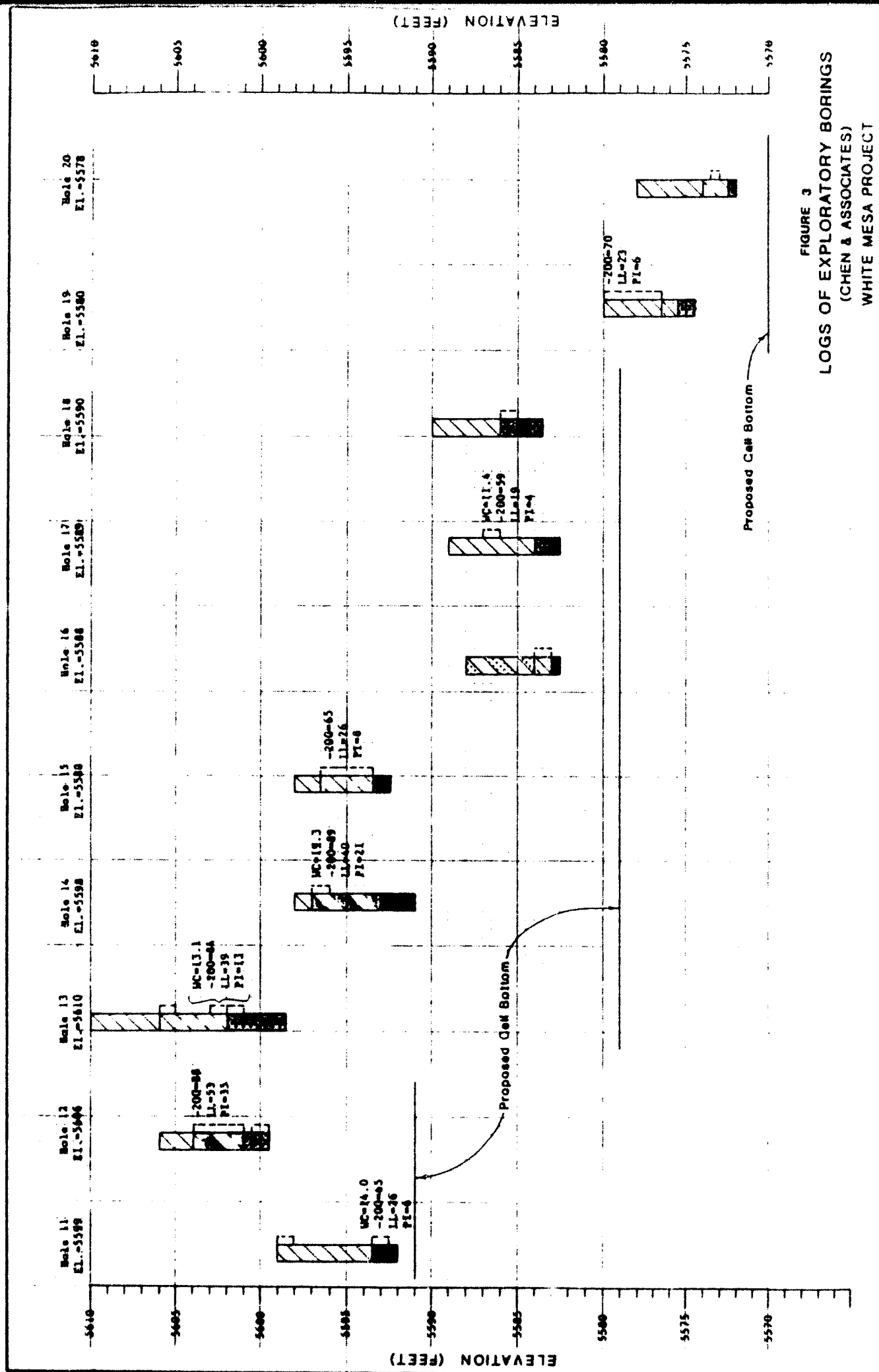


FIGURE 2  
LOGS OF EXPLORATORY BORINGS  
(CHEN & ASSOCIATES)  
WHITE MESA PROJECT



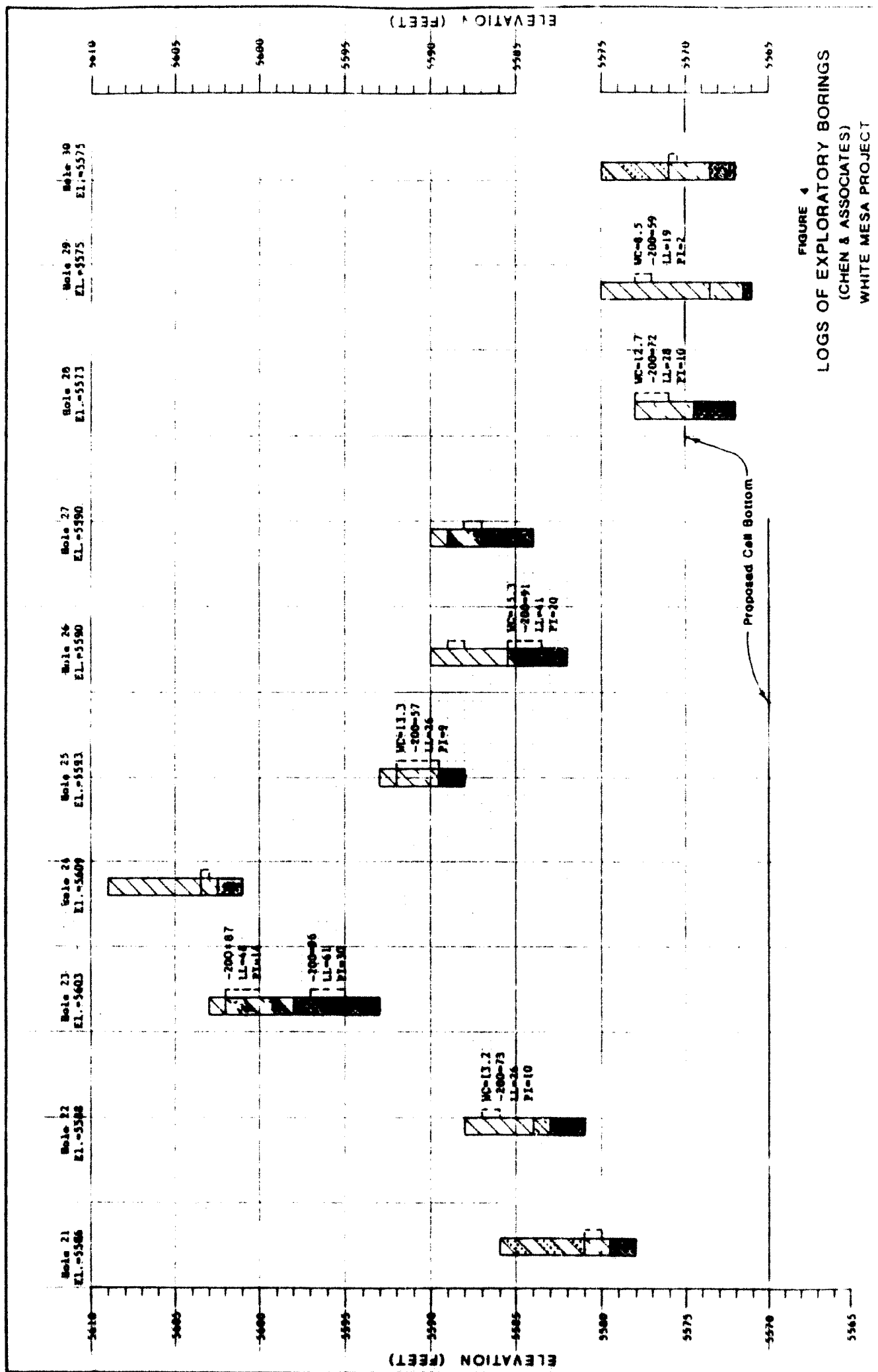


FIGURE 4  
LOGS OF EXPLORATORY BORINGS  
(CHEN & ASSOCIATES)  
WHITE MESA PROJECT



Bore 31 EL. -5614	Bore 32 EL. -5612	Bore 33 EL. -5592	Bore 37 EL. -5583	Bore 38 EL. -5595	Bore 39 EL. -5604	Bore 40 EL. -5610	Bore 41 EL. -5600	Bore 42 EL. -5609	Bore 43 EL. -5602
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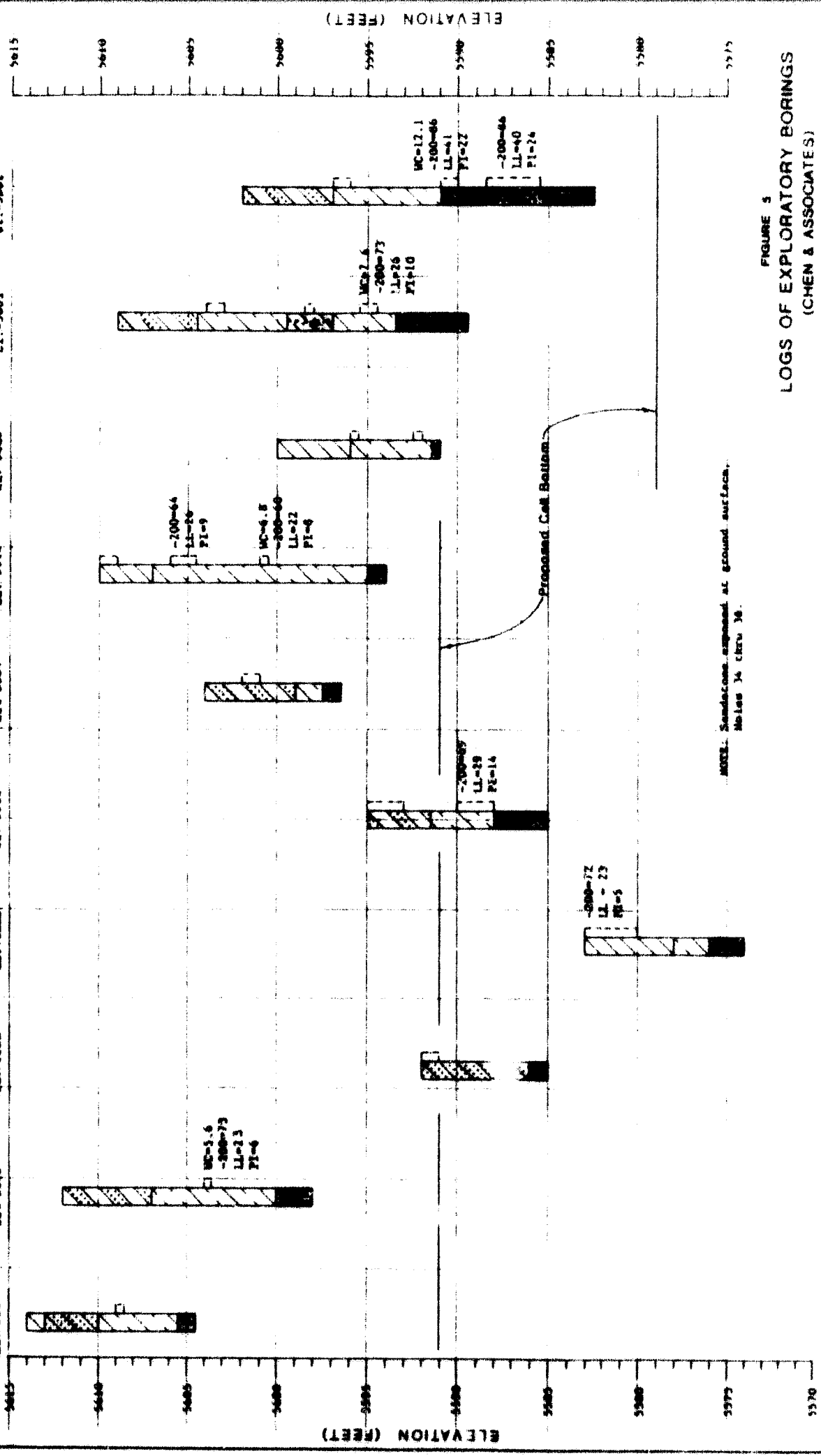
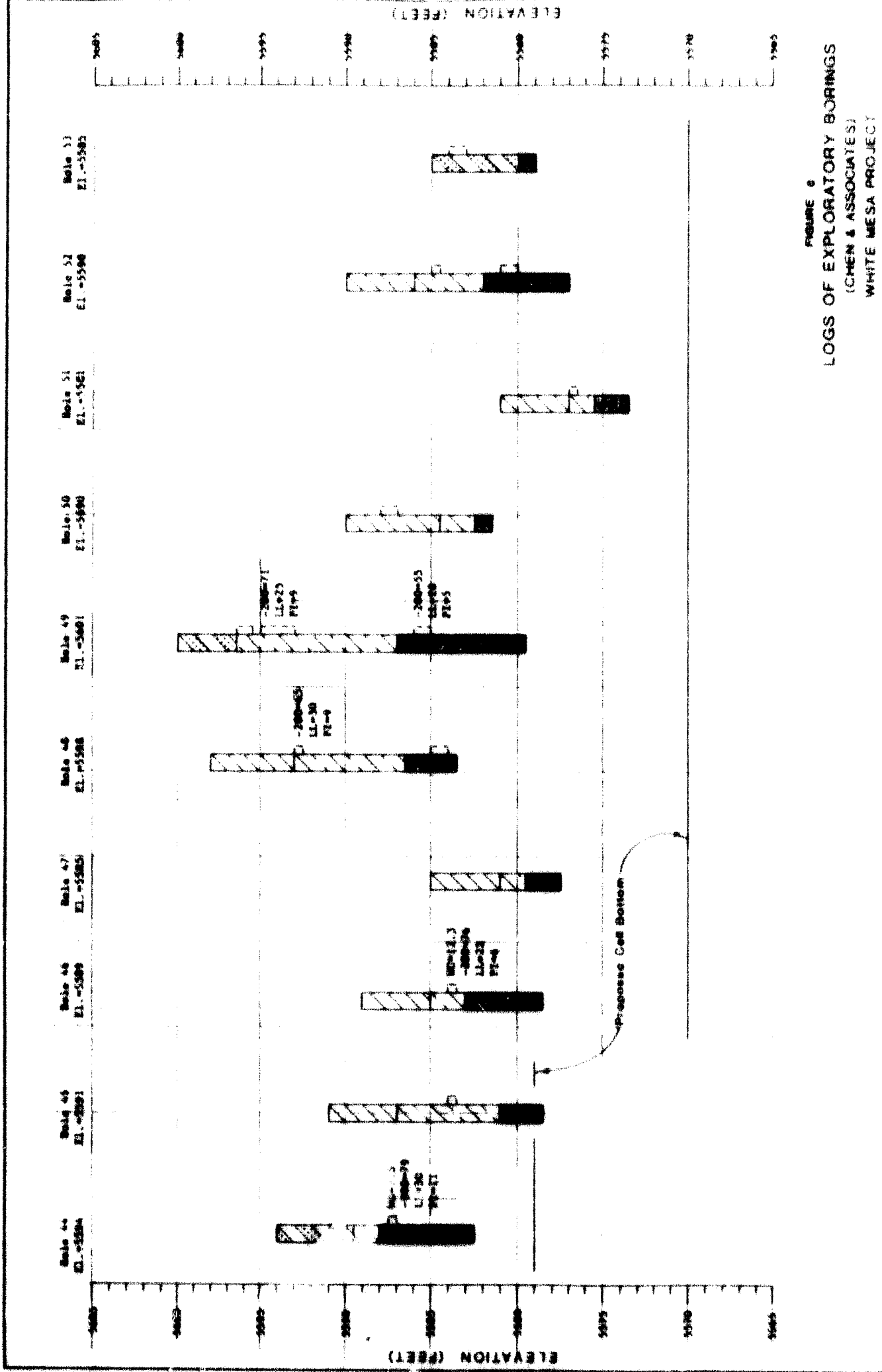


FIGURE 5  
LOGS OF EXPLORATORY BORINGS  
(CHEN & ASSOCIATES)  
WHITE MESA PROJECT



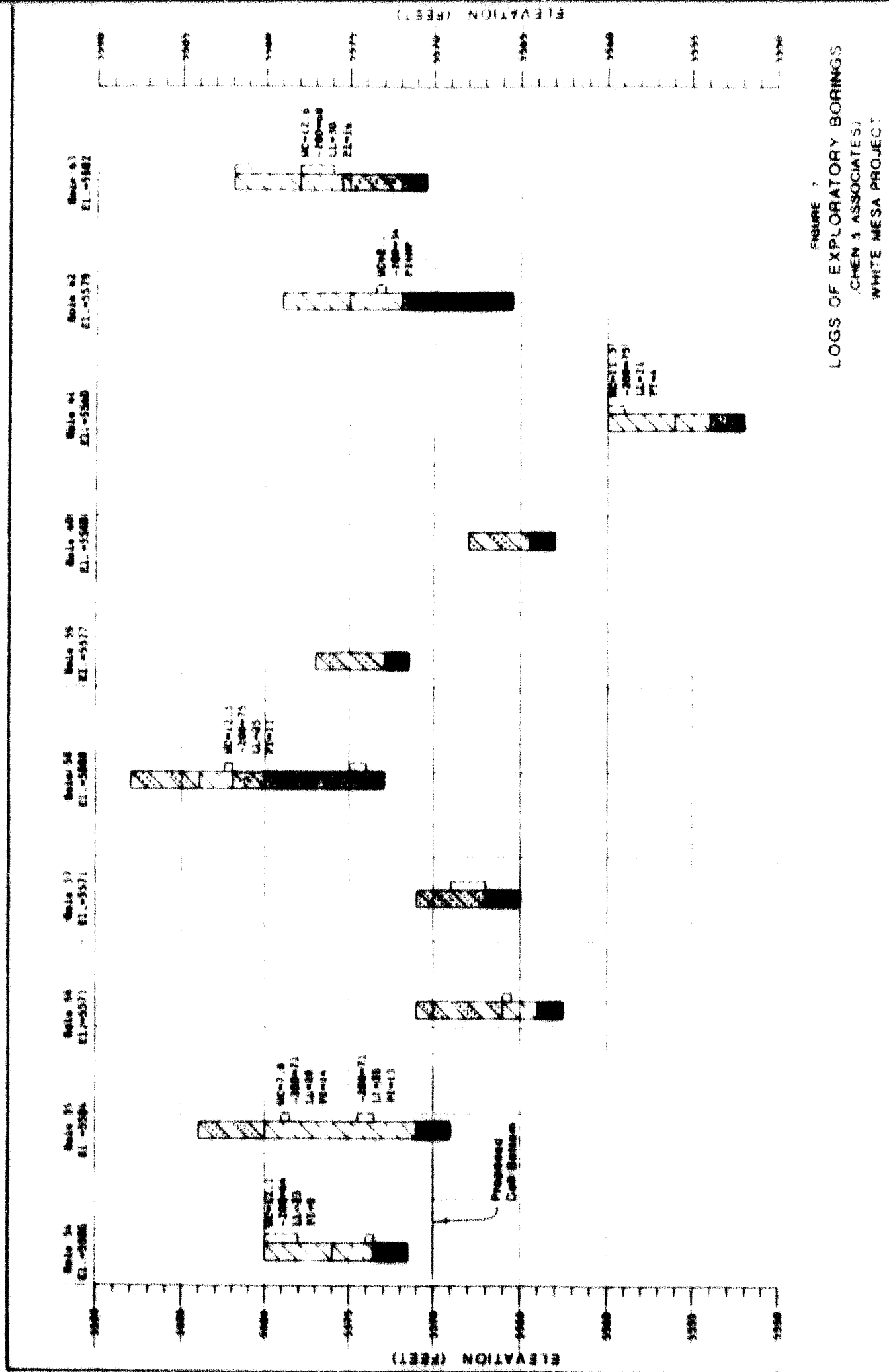


FIGURE 7  
LOGS OF EXPLORATORY BORINGS  
(CHEN & ASSOCIATES)  
WHITE MESA PROJECT

Bole 66  
EL. -5570

Bole 65  
EL. -5565

Bole 64  
EL. -5555

Bole 67  
EL. -5572

Bole 68  
EL. -5574

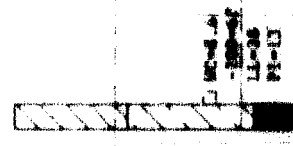
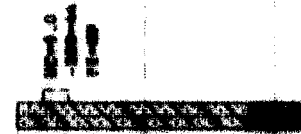
Bole 69  
EL. -5563

Bole 70  
EL. -5558

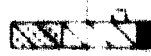
Bole 71  
EL. -5545

Bole 72  
EL. -5554

Bole 73  
EL. -5564



MC=1.4  
-200-55  
LI=22  
PI=4



MC=16.4  
-200-55  
LI=22  
PI=4



MC=12.2  
-200-55  
LI=22  
PI=4

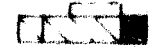
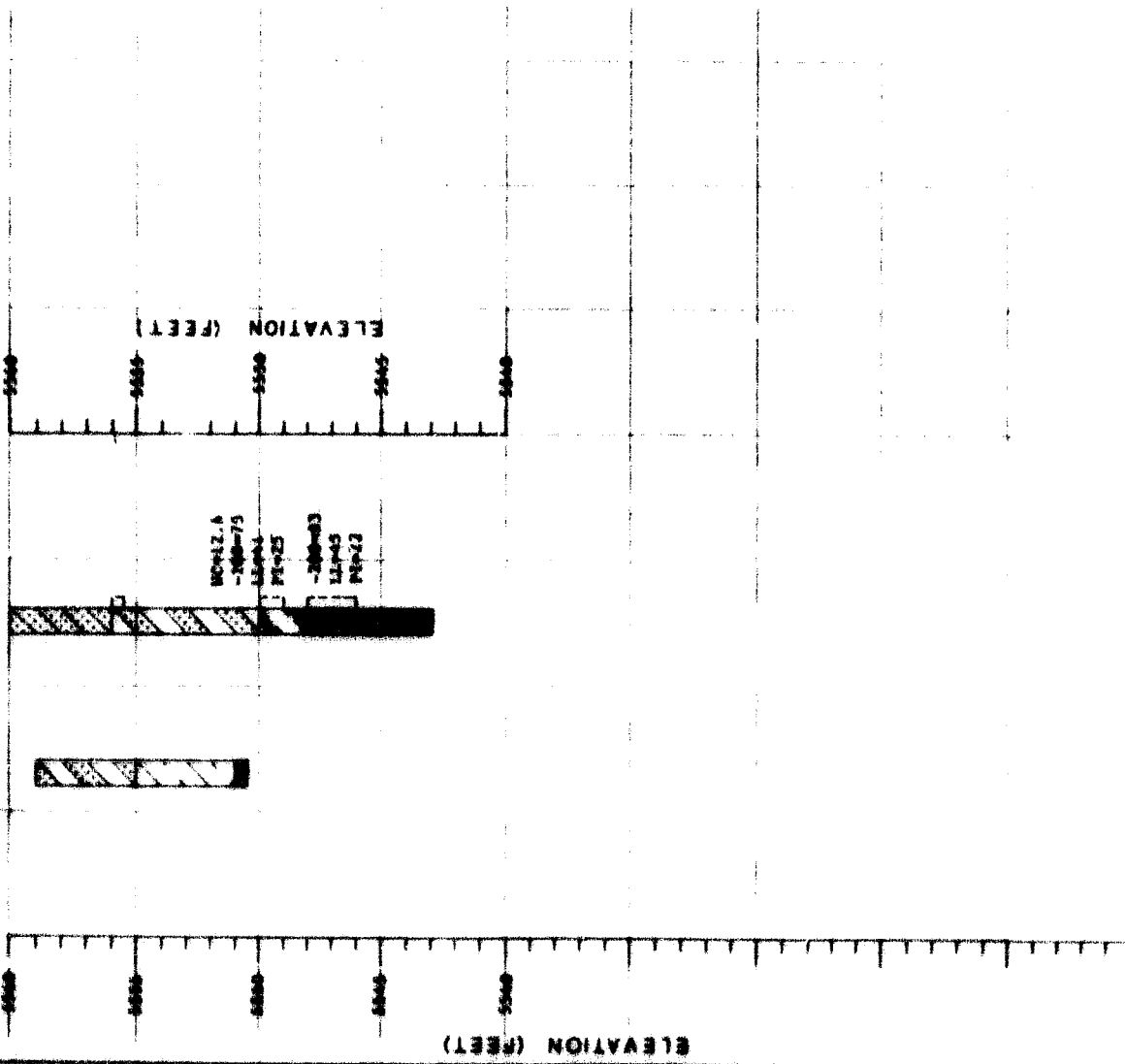


FIGURE 8  
LOGS OF EXPLORATORY BORINGS  
(CHEN & ASSOCIATES)  
WI - MESA PROJECT

Bole 74  
El. -5558

Bole 75  
El. -5560



Legend:

- Sand (SH), silty, fine to medium grained, approximately 40-50% silt, slightly moist to moist, reddish brown.
- Sand, silty to sandy silt (SM-ML), fine to medium grained, approximately 50-60% silt, slightly moist to moist, reddish brown.
- Silt (ML), sandy, approximately 60-70% silt, fine to medium sand size, slightly calcareous with depth, slightly moist to moist, reddish brown to light brown.
- Clay, silty to sandy silt (CL-ML), approximately 60-75% low to non-plastic fines, fine to medium sand size, slightly to moderately calcareous with depth, slightly moist, light brown.
- Clay (CL), highly calcareous, sandy to silty, approximately 50-75% low plasticity fines, scattered very hard lenses/layer, dry to slightly moist, light tan to white.
- Weathered claystone (CI-CH), approximately 90% medium to high plasticity fines, moist, gray-brown.
- Claystone, bedrock, slightly moist, greenish gray.
- Claystone-sandstone bedrock, lightly cemented, roughly stratified, fine to medium grained, greenish gray.
- Sandstone bedrock, well cemented with depth, fine to medium grained, tan to gray.
- Disturbed auger sample.

NOTES:

- (1) Test holes were drilled on May 17 and 18, 1978 with a 12-inch, single-flight, power auger.
- (2) Elevations are approximate and taken from contours shown on Fig. 1.
- (3) No free water was found in test holes at the time of drilling.
- (4) WC - Water Content (%);  
LL - Liquid Limit (%);  
PI - Plasticity Index (%);  
MP - Non-Plastic.

FIGURE 9  
LOGS OF EXPLORATORY BORINGS  
(CHEN & ASSOCIATES)  
WHITE MESA PROJECT



SOIL & FOUNDATION  
ENGINEERING

chen and associates, inc.  
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SECTION 3

Extracted Data From

SOIL PROPERTY STUDY  
PROPOSED TAILINGS RETENTION CELLS  
WHITE MESA URANIUM PROJECT  
BLANDING, UTAH

Prepared for:

ENERGY FUELS NUCLEAR, INC.  
1515 ARAPAHOE STREET  
DENVER, COLORADO 80202

Job No. 17,130

January 23, 1979

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| II | Laboratory Permeability Test Results |

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| 3 | Holes 86 through 94   |
| 4 | Holes 95 through 104  |
| 5 | Holes 105 through 114 |
| 6 | Holes 115 through 120 |
| 7 | Holes 121 through 128 |
| 8 | Legend                |

Note: The attached logs have been redrawn.

TABLE 1

## SUMMARY OF LABORATORY TEST RESULTS

Page 1 of 3

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY (PCF)	ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH (PSF)	TRIAXIAL SHEAR TESTS		PERCENT PASSING NO. 200 SIEVE	SOIL TYPE
				LIQUID LIMIT (%)	PLASTICITY INDEX (%)		DEVIATOR STRESS (PSF)	CONFINING PRESSURE (PSF)		
76	0 - 1	4.5		21	5				78	Sandy silt
	9.5 - 10	4.4			NP				26	Silty, gravelly sand
77	7.5 - 8	8.6		30	15				71	Sandy clay
79	0 - 1	4.1		20	5				83	Sandy silt
	5 - 5.5	5.5			NP				41	Calcareous sandy clay
80	4.5 - 7			39	20				78	Calcareous sandy clay
	8 - 8.5	10.1		40	20				86	Weathered claystone
81	3 - 4	6.3		26	8				64	Silty, sandy clay
83	4 - 6			24	7				64	Sandy, clayey silt
84	0 - 2			18	2				65	Sandy silt
	9 - 9.5	2.7			NP				27	Silty sand
86	8 - 8.5	2.6			NP				12	Sandstone
87	0 - 1	3.1		16	1				61	Sandy silt
89	0 - 3			21	5				66	Sandy silt
90	8 - 8.5	12.9		35	15				61	Weathered claystone
92	0 - 1	5.9		21	5				80	Sandy silt
94	5 - 5.5	13.7		27	10				68	Sandy clay
95	6 - 7			23	5				62	Sandy silt
96	0 - 2	5.2		21	4				79	Sandy silt
	8.5 - 9.5			32	6				66	Calcareous sandy clay
98	0 - 1	3.8		20	5				74	Sandy silt
	4 - 4.5	17.8		40	25				76	Weathered claystone
99	8 - 9.5			40	20				89	Weathered claystone



## CHIEN AND ASSOCIATES

TABLE 1

## SUMMARY OF LABORATORY TEST RESULTS

Page 2 of 3

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY (PCF)	ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH (PSF)	TRIAxIAL SHEAR TESTS		PERCENT PASSING NO. 200 SIEVE	SOIL TYPE
				LIQUID LIMIT (%)	PLASTICITY INDEX (%)		DEVIATOR STRESS (PSF)	CONFINING PRESSURE (PSF)		
99	11 - 12	13.5		26	10				73	Claystone
100	0 - 1			17	NP				44	Silty sand
	5.5 - 6	12.0			NP				61	Sandstone-siltstone
102	6.5 - 7	16.7		30	8				79	Calcareous sandy clay
	13.5 - 14	9.5		23	6				87	Claystone-siltstone
103	10 - 10.5	7.0		28	12				57	Sandy clay
104	8 - 8.5	9.2		33	9				70	Calcareous sandy clay
105	0 - 1	5.4		22	6				77	Sandy silt
	6.5 - 7	4.5			NP				86	Sandy silt
106	5 - 5.5	10.4		28	6				59	Claystone-sandstone
107	7.5 - 9				NP				23	Sandstone
108	0 - 1	4.0		18	3				69	Sandy silt
	9.5 - 10	2.2		38	16				93	Claystone
109	4 - 5			25	7				75	Sandy, clayey silt
111	9 - 9.5	5.8		25	10				53	Claystone
113	5 - 8			40	20				84	Weathered claystone
	10.5 - 11			24	10				54	Claystone-sandstone
114	0 - 2			22	6				58	Sandy, clayey silt
115	4.5 - 6				NP				58	Calcareous
116	0 - 3			22	5				72	Sandy silt
	7 - 8			24	10				42	Claystone-sandstone
117	1 - 2	10.6		25	5				77	Sandy silt
118	0 - 2			25	6				77	Sandy silt

# SUMMARY OF LABORATORY TEST RESULTS

[illegible]

## LABORATORY PERMEABILITY TEST RESULTS

Sample	Classification	Compaction			Surcharge Pressure (psf)	Ft./Yr.	Permeability cm/sec.
		Dry Density (pcf)	Moisture Content (%)	% of ASTM D698			
TH 80 @ 4 1/2-7'	Calcareous sandy clay -200=78; LL=39; PI=20	100.2	19.4	96	500	0.81	$7.8 \times 10^{-7}$
TH 84 @ 0-2'	Sandy silt -200=65; LL=18; PI=2	113.8	11.7	96	500	4.45	$4.3 \times 10^{-6}$
TH 96 @ 8 1/2-9 1/2'	Calcareous sandy clay -200=66; LL=32; PI=6	96.9	20.7	97	500	1.55	$1.5 \times 10^{-6}$
TH 96 @ 8 1/2-9 1/2'	Calcareous sandy clay	95.7	20.3	96	500	26.90*	$2.6 \times 10^{-5}$
TH 99 @ 8-9 1/2'	Weathered claystone -200=89; LL=40; PI=20	99.8	18.5	95	500	0.22	$2.1 \times 10^{-7}$
TH 100 @ 0-1'	Very silty sand -200=44; PI=NP	117.5	9.7	98	500	0.38	$3.7 \times 10^{-7}$
TH 114 @ 0-2'	Sandy, clayey silt -200=58; LL=22; PI=6	112.4	12.9	95	500	0.60	$5.8 \times 10^{-7}$
TH 120 @ 1-2'	Sandy, clayey silt -200=69; LL=24; PI=6	108.2	14.7	95	500	0.11	$1.1 \times 10^{-7}$
TH 122 @ 4-6'	Sandy, silty clay -200=66; LL=25; PI=8	108.8	15.5	96	500	0.43	$4.2 \times 10^{-7}$
TH 123 @ 1-3'	Sandy, clayey silt -200=71; LL=23; PI=7	110.9	12.6	95	500	0.56	$5.4 \times 10^{-7}$
TH 128 @ 6-7'	Claystone -200=89; LL=41; PI=24	92.4	23.9	93	500	0.12	$1.2 \times 10^{-7}$
TH 128 @ 6-7'	Claystone -200=89; LL=41; PI=4	93.1	22.1	94	500	0.52*	$5.0 \times 10^{-7}$

\* 1.5 pH sulfuric acid liquor used during percolation test interval.

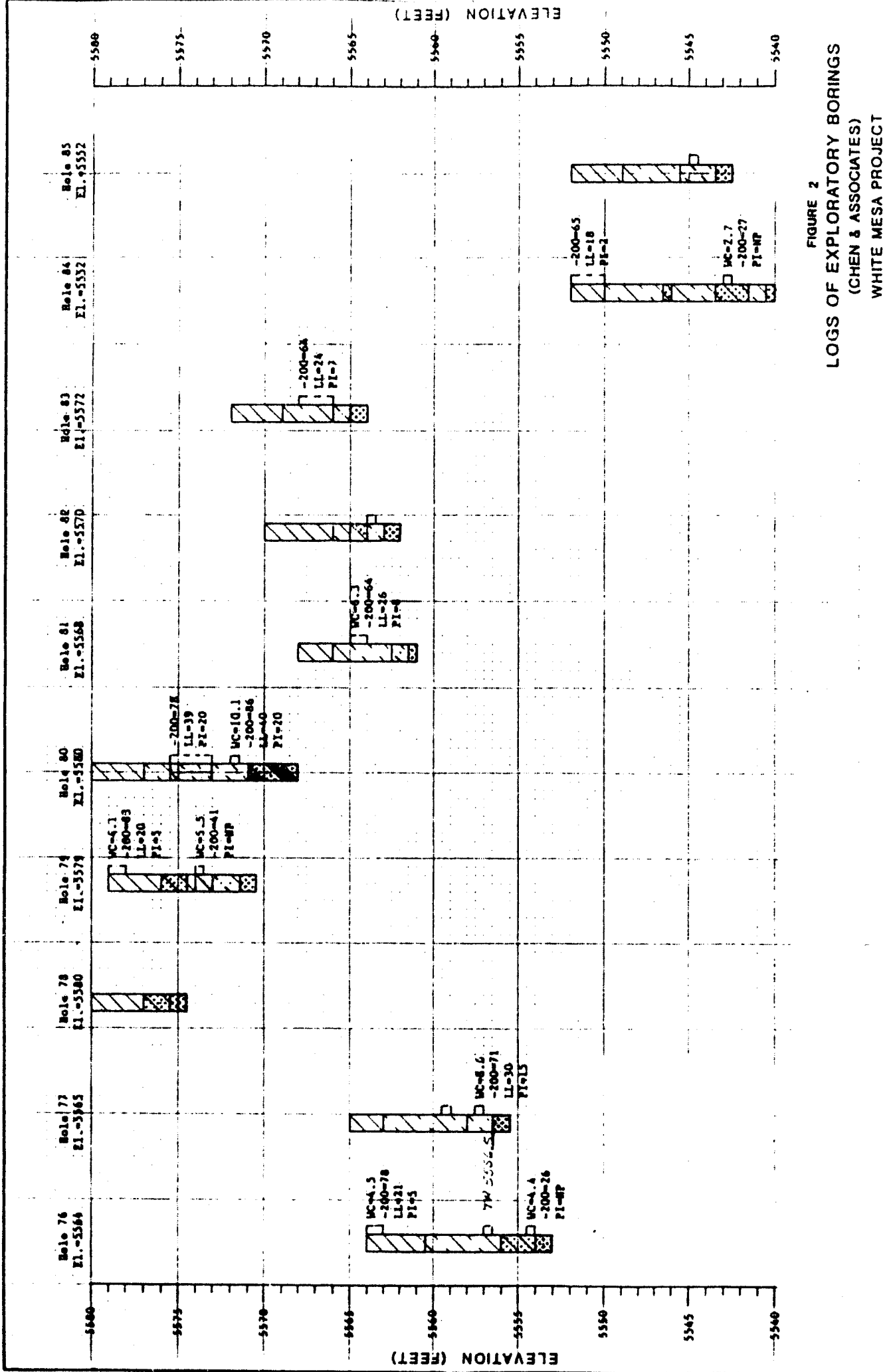


FIGURE 2  
LOGS OF EXPLORATORY BORINGS  
(CHEN & ASSOCIATES)  
WHITE MESA PROJECT

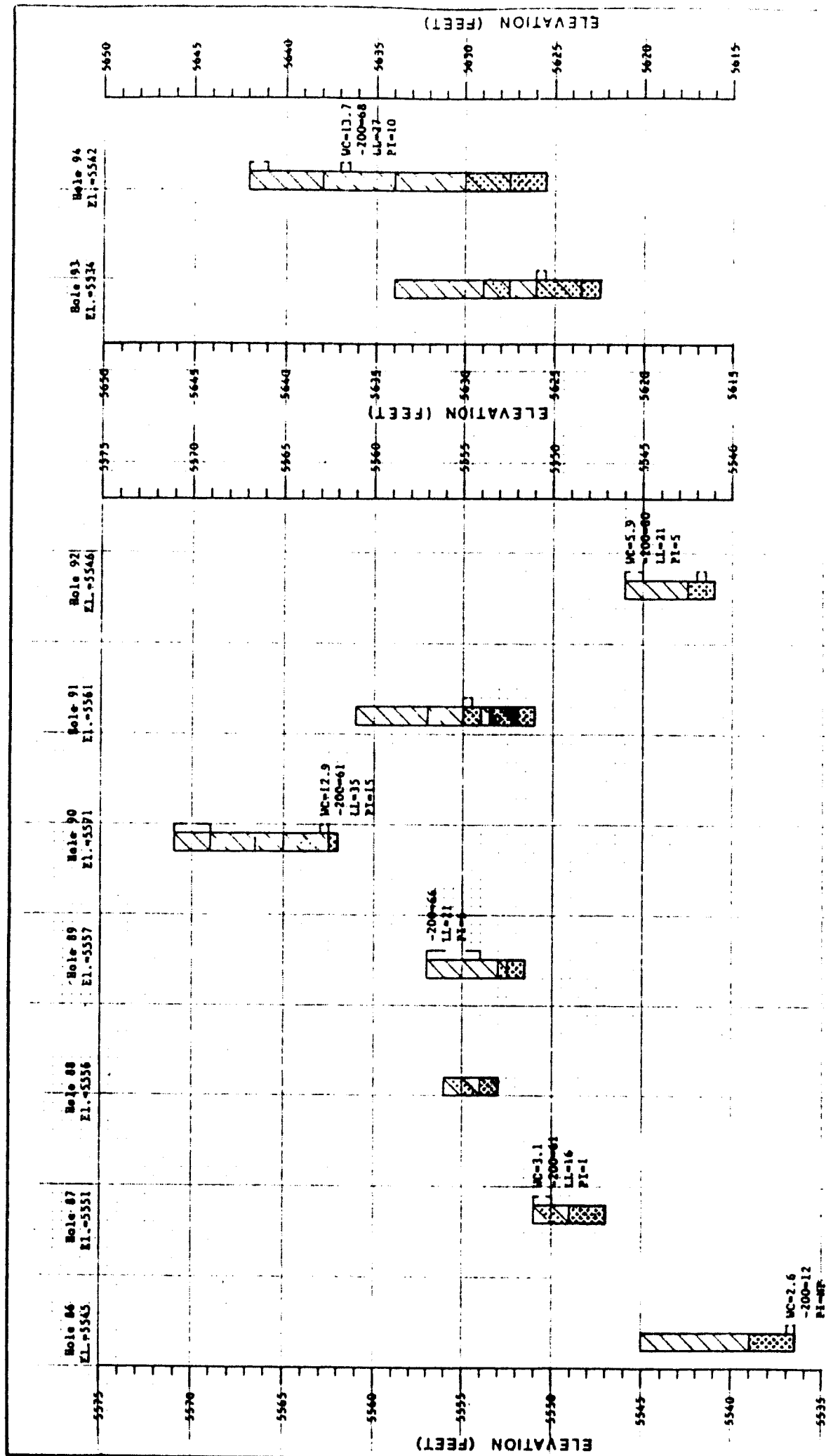


FIGURE 3  
LOGS OF EXPLORATORY BORINGS  
(CHEN & ASSOCIATES)  
WHITE MESA PROJECT

Bore 95	Bore 96	Bore 97	Bore 98	Bore 99	Bore 100	Bore 101	Bore 102	Bore 103	Bore 104
EL.-5624	EL.-5626	EL.-5610	EL.-5609	EL.-5610	EL.-5621	EL.-5616	EL.-5620	EL.-5632	EL.-5638

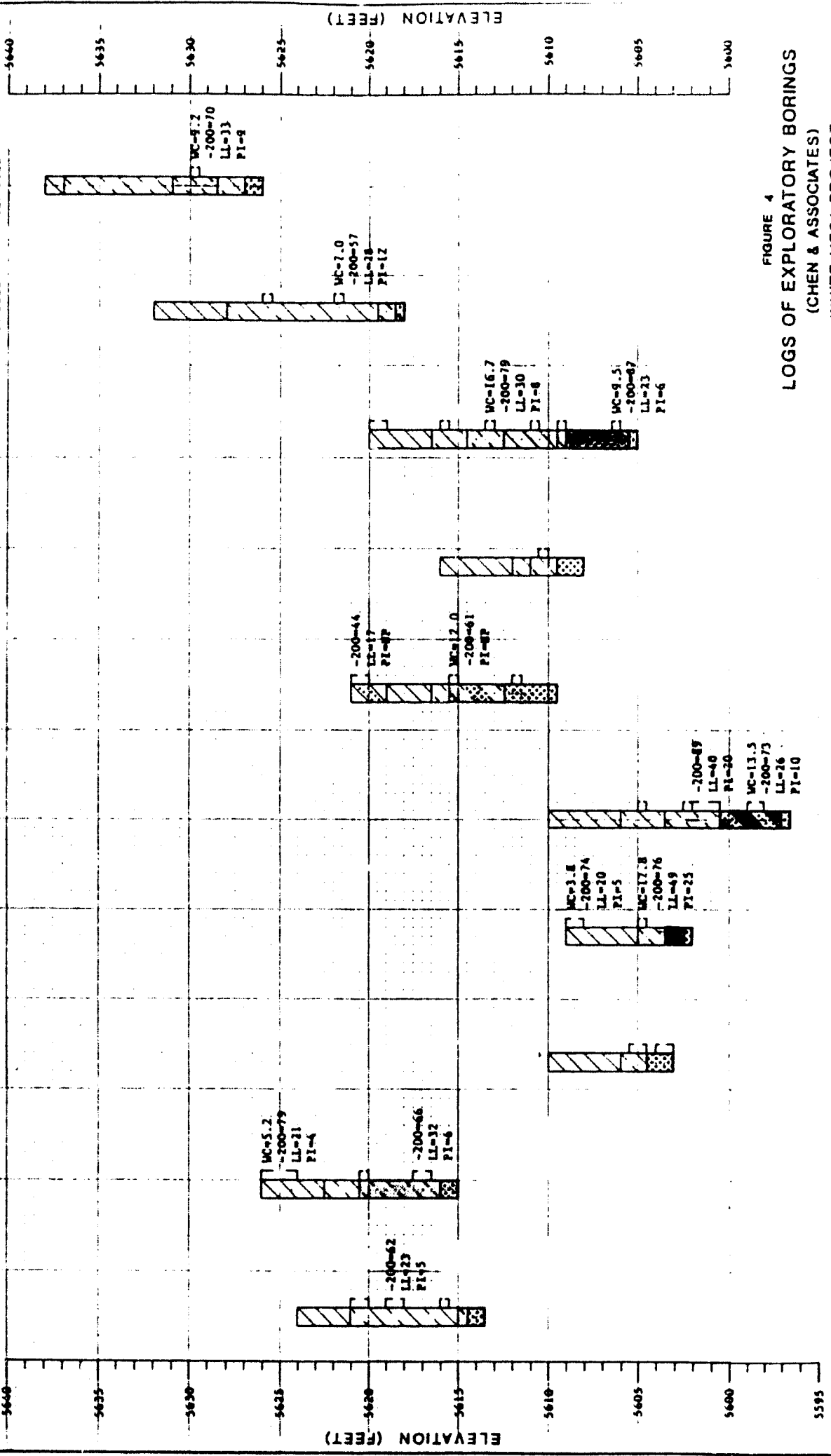
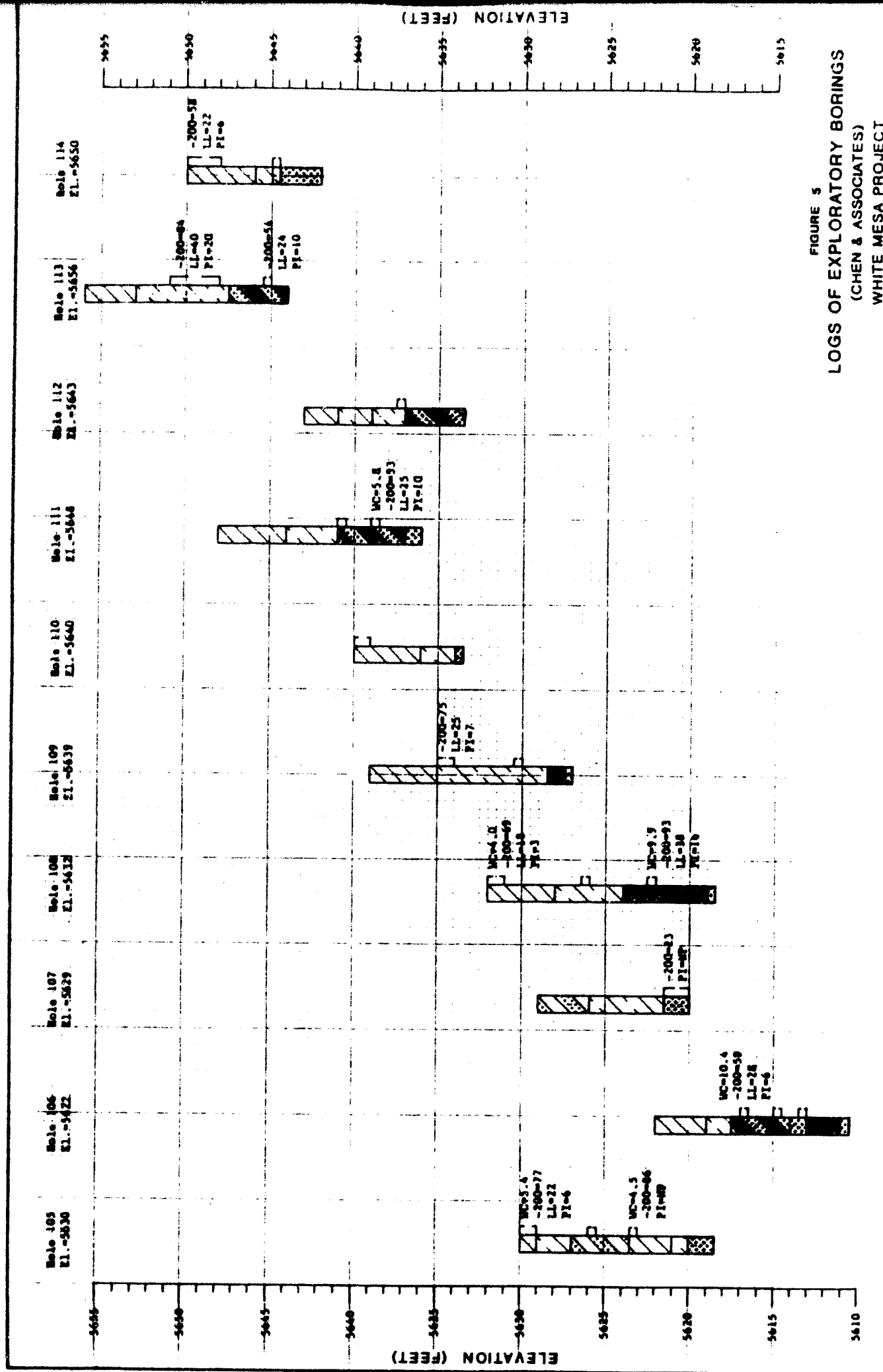
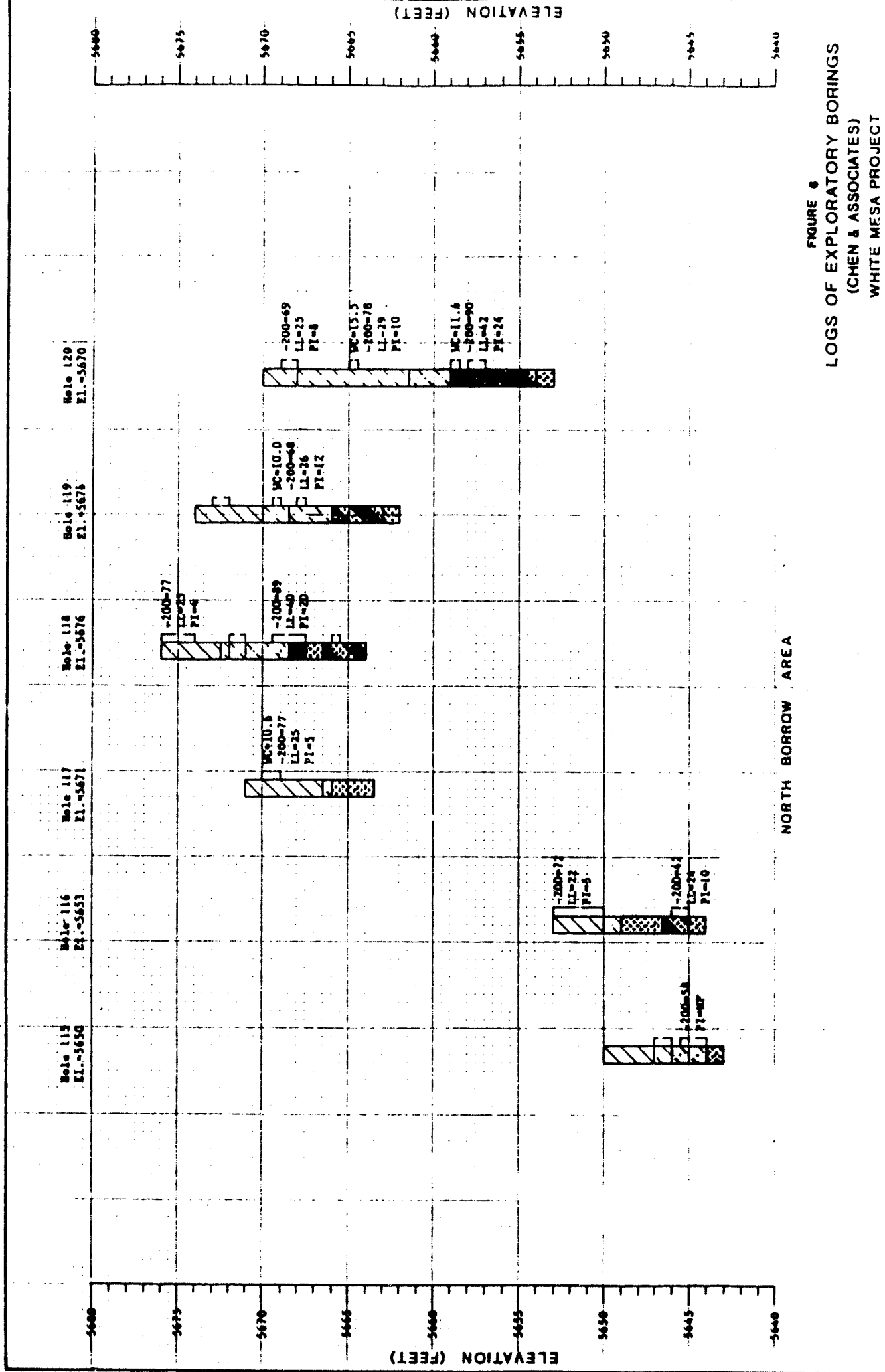


FIGURE 4  
 LOGS OF EXPLORATORY BORINGS  
 (CHEN & ASSOCIATES)  
 WHITE MESA PROJECT

FIGURE 5  
LOGS OF EXPLORATORY BORINGS  
(CHEN & ASSOCIATES)  
WHITE MESA PROJECT







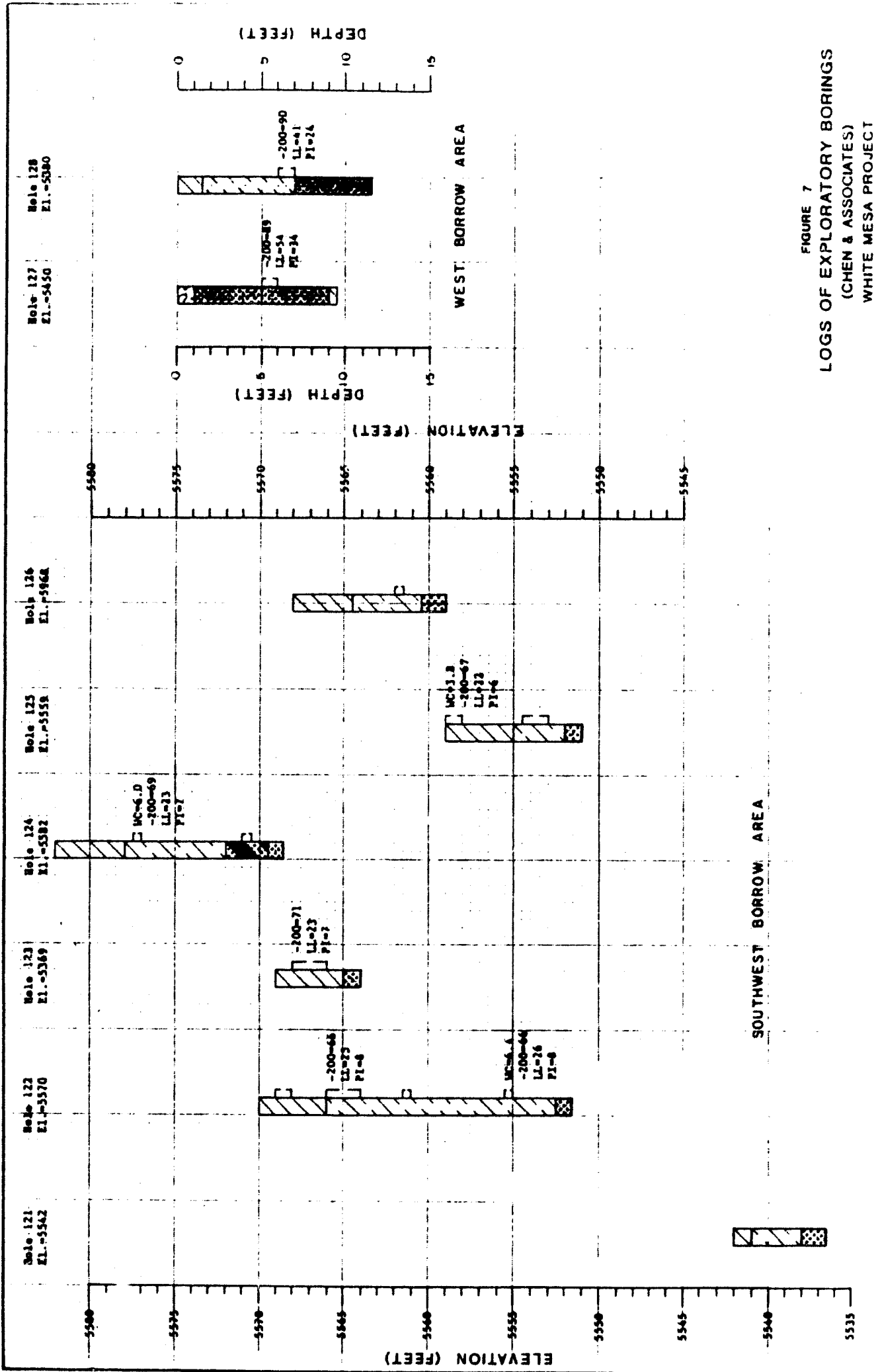















FIGURE 7  
LOGS OF EXPLORATORY BORINGS  
(CHEN & ASSOCIATES)  
WHITE MESA PROJECT

LEGEND:

-  Silt (ML), sandy, approximately 60-70% silt, fine to medium sand size, slightly calcareous with depth, slightly moist to moist, reddish brown to light brown.
-  Sand, silty to sandy silt (SM-ML), fine to medium grained, approximately 50-60% silt, slightly moist to moist, reddish brown.
-  Sand (SM), silty, fine to medium grained, approximately 30-50% silt, some scattered gravel, slightly moist, reddish brown.
-  Clay, silty to sandy silt (CL-ML), approximately 60-75% low to non-plastic fines, fine to medium sand size, slightly to moderately calcareous with depth, slightly moist, light brown.
-  Clay (CL), sandy, approximately 60-75% low to medium plastic fines, fine to medium sand size, slightly calcareous, slightly moist, reddish brown.
-  Clay (CL), highly calcareous, sandy to silty, approximately 50-75% low plasticity fines, scattered very hard lenses/layer, dry to slightly moist, light tan to white.
-  Weathered claystone (CL-CH), approximately 75-90% medium to high plasticity fines, slightly moist to moist, gray-brown to greenish.
-  Claystone bedrock, slightly moist, greenish gray to dark gray.
-  Siltstone bedrock, well-cemented, very hard, gray.
-  Claystone-sandstone bedrock, lightly cemented, generally grading coarser with depth, fine to medium grained, slightly moist, greenish gray.
-  Sandstone-siltstone bedrock, lightly cemented, slightly moist, gray-brown.
-  Sandstone bedrock, fairly clean to silty and clayey, well cemented with depth, fine to medium grained, scattered conglomerate lenses/layers, slightly moist to dry, tan to gray.
-  Disturbed auger sample.

NOTES:

- (1) Test holes were drilled on September 19 through 21, 1978, with a 12-inch, single-flight, power auger.
- (2) Elevations are approximate and taken from contours shown in Fig. 1.
- (3) No free water was found in the test holes at the time of drilling.
- (4) WC = Water Content (%);  
-200 = Percent Passing No. 200 Sieve;  
LL = Liquid Limit (%);  
PI = Plasticity Index (%);  
NP = Nonplastic.

FIGURE 8  
LOGS OF EXPLORATORY BORINGS  
(CHEN & ASSOCIATES)  
WHITE MESA PROJECT

SECTION 4

Extracted Data From

REPORT  
SITE SELECTION AND DESIGN STUDY  
TAILING RETENTION AND MILL FACILITIES  
WHITE MESA URANIUM PROJECT  
BLANDING, UTAH  
FOR ENERGY FUELS NUCLEAR, INC.

Dames and Moore

January 17, 1978

09973-015-14

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A-6	Boring No. 9
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A-8	Boring Nos. 16, 17, 18, 20, 21, 22
A-9	Boring No. 19
A-10	Boring Nos. 23, 24, 26, 27, 29
A-11	Boring 28
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B-12	Triaxial Compression Test Report Silt and Sand

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS LITTLE OR NO FINES		GW	WELL-GRADED GRAVELS, GRAVEL SAND MIXTURES LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL SAND MIXTURES LITTLE OR NO FINES
		GRAVELS WITH FINES APPRECIABLE AMOUNT OF FINES		GM	SILTY GRAVELS, GRAVEL SAND SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL SAND CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SAND LITTLE OR NO FINES		SW	WELL-GRADED SANDS, GRAVELLY SANDS LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SANDS LITTLE OR NO FINES
		SANDS WITH FINES APPRECIABLE AMOUNT OF FINES		SM	SILTY SANDS, SAND SILT MIXTURES
				SC	CLAYEY SANDS, SAND CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS UNCLAYEY OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

## SOIL CLASSIFICATION CHART

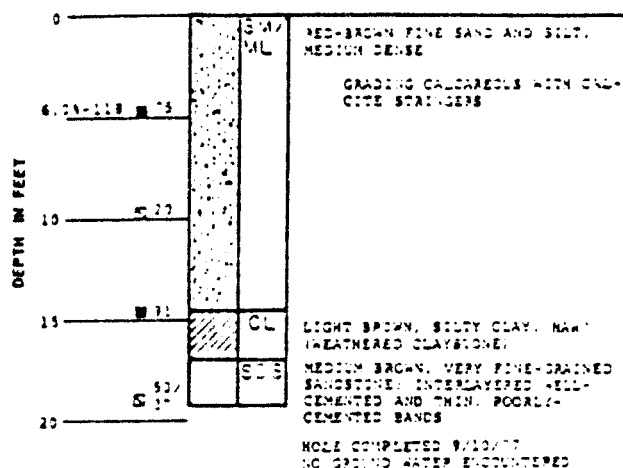
	SDS	SANDSTONE		SLN	SILTSTONE
	CLS	CLAYSTONE		CGL	CONGLOMERATE

## GRAPHIC LOG SYMBOLS FOR ROCK

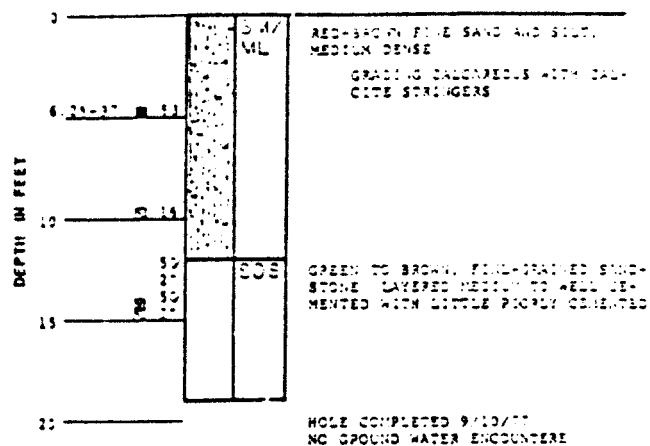
# UNIFIED SOIL CLASSIFICATION SYSTEM AND GRAPHIC LOG SYMBOLS

DANES & MOORE

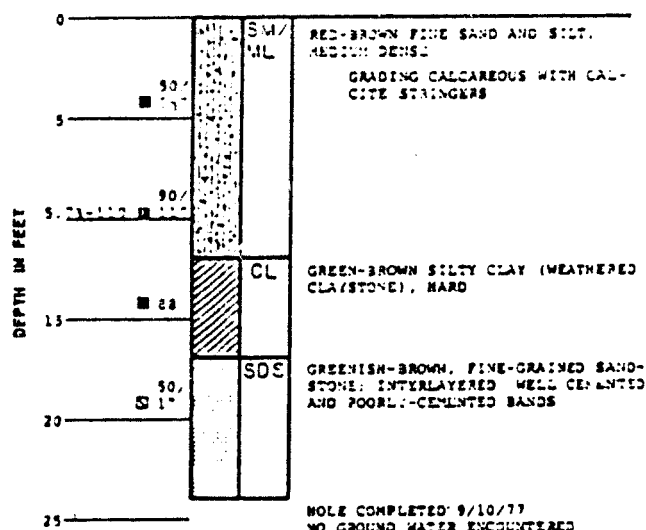
# **BORING NO. 1** EL. 5629.0 FT.



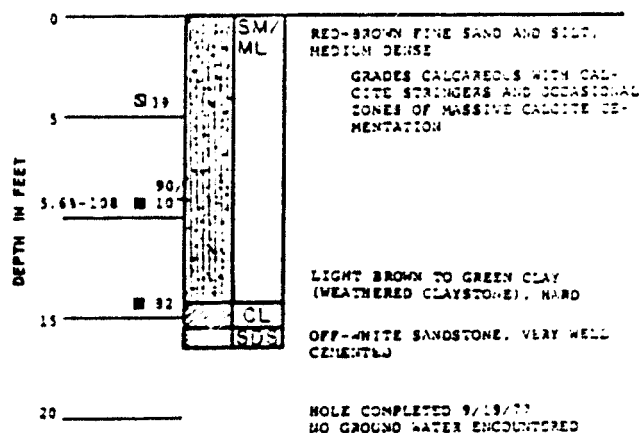
# **BORING NO. 5** EL. 5632.9 FT.



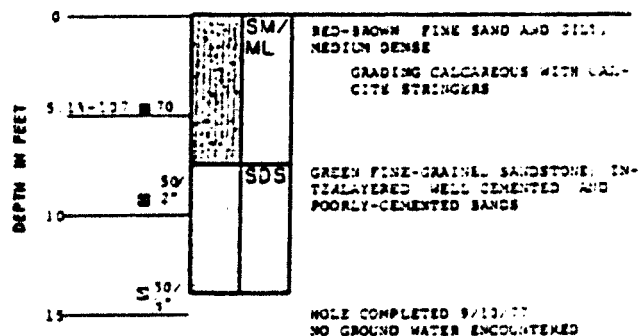
# **BORING NO. 2** EL. 5634.3 FT.



# **BORING NO. 6** EL. 5633.5 FT.



# **BORING NO. 4** EL. 5623.2 FT.



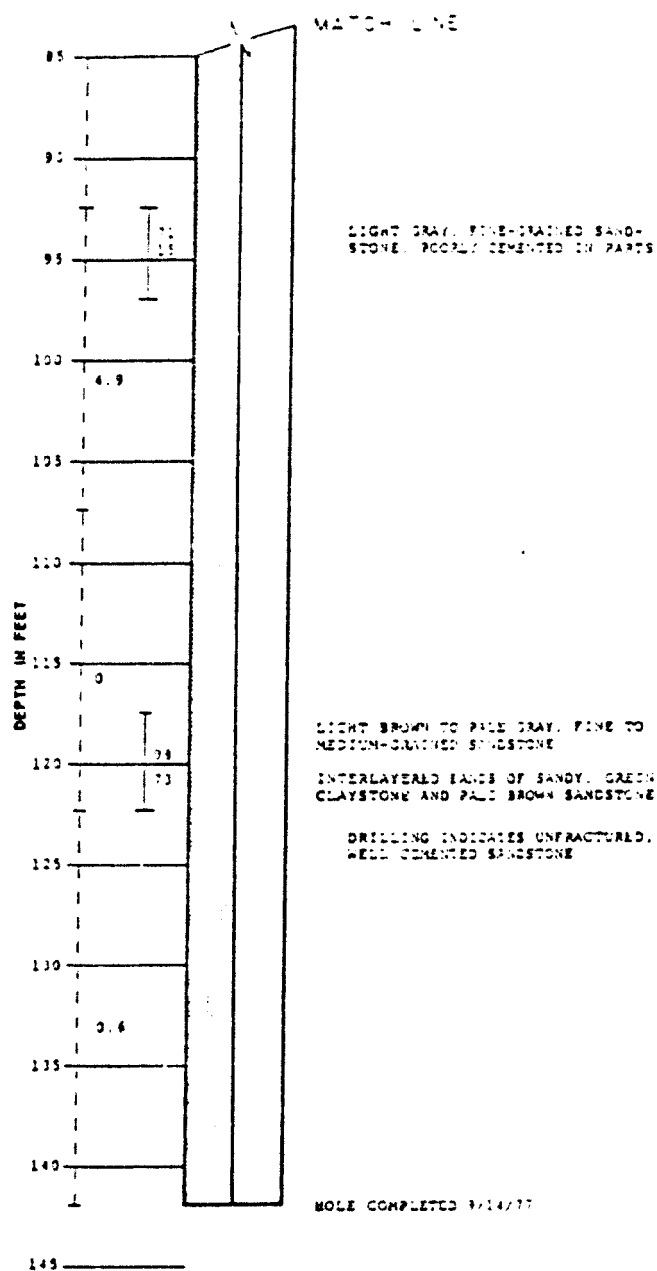
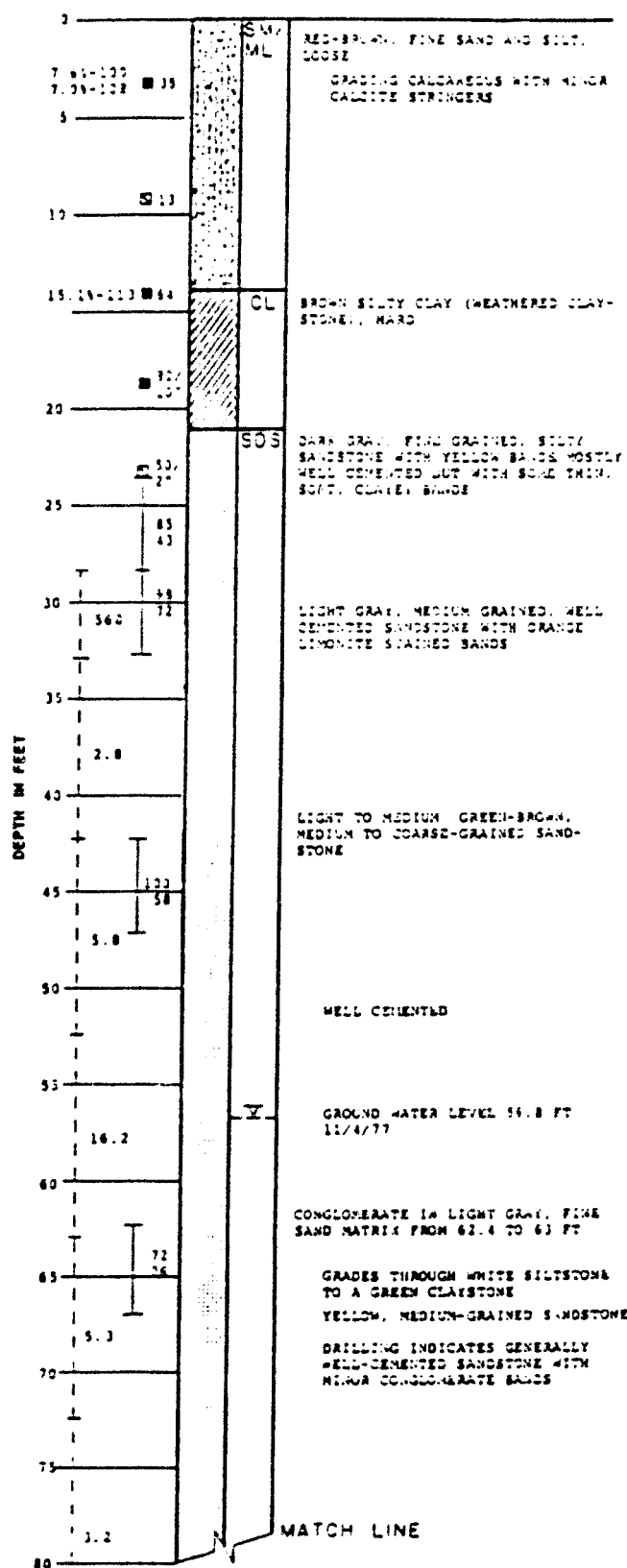
## **KEY**

- A-B ■ ■ ■ INDICATES DEPTH AT WHICH UNDISTURBED SAMPLE WAS EXTRACTED USING DAMES & MOORE SAMPLER
- C INDICATES DEPTH AT WHICH DISTURBED SAMPLE WAS EXTRACTED USING DAMES & MOORE SAMPLER
- C INDICATES SAMPLE ATTEMPT WITH NO RECOVERY
- C INDICATES DEPTH AT WHICH DISTURBED SAMPLE WAS EXTRACTED USING STANDARD PENETRATION TEST SAMPLER
- A FIELD MOISTURE EXPRESSED AS A PERCENTAGE OF THE DRY WEIGHT OF SOIL
- B DRY DENSITY EXPRESSED IN LBS/CU FT
- C BLOWS/FT OF PENETRATION USING A 140-LB HAMMER DROPPING 30 INCHES
- INDICATES NO CORE RUN
- D PERCENT OF CORE RECOVERY
- E RQD\*
- INDICATES PACKER TEST SECTION
- F PERMEABILITY MEASURED BY SINGLE PACKER TEST IN FLOW
- NA NOT APPLICABLE (USED FOR RQD IN CLAYS OR MECHANICALLY FRACTURED ZONES)
- NOTE: ELEVATIONS PROVIDED BY ENERGY FUELS NUCLEAR, INC.
- \* ROCK QUALITY DESIGNATION -- PERCENTAGE OF CORE RECOVERED IN LENGTHS GREATER THAN 4 INCHES

## **LOG OF BORINGS**

DAMES & MOORE

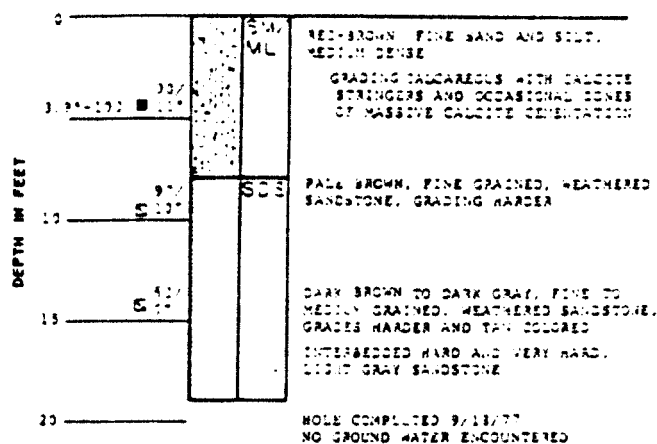
# BORING NO. 3 EL. 5634.4 FT.



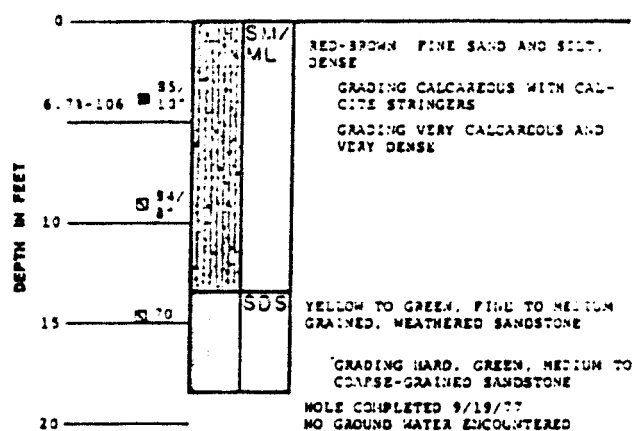
## LOG OF BORINGS

DAMES & MOORE

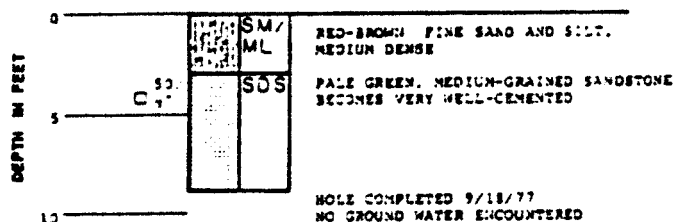
# **BORING NO. 7** EL. 5656.9 FT.



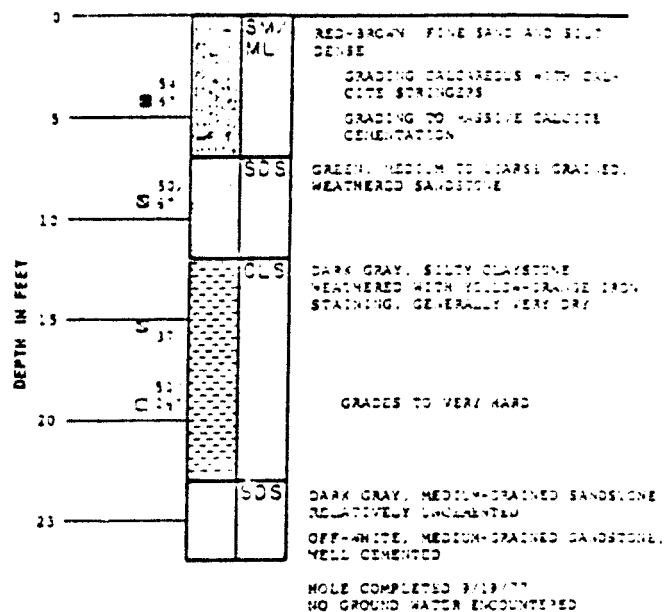
# **BORING NO. 10** EL. 5690.9 FT.



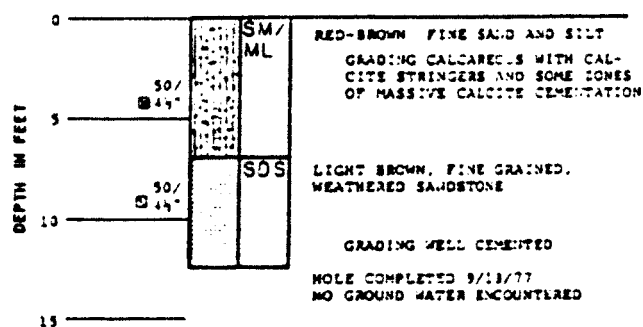
# **BORING NO. 13** EL. 5602.4 FT.



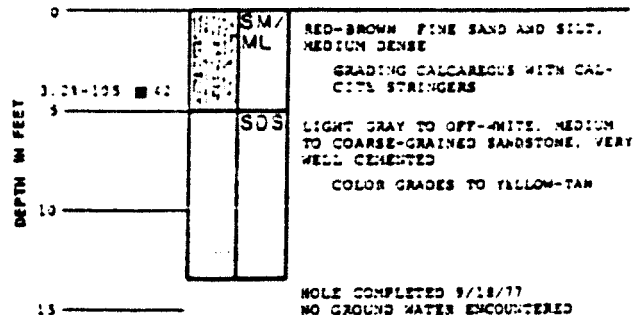
# **BORING NO. 8** EL. 5658.4 FT.



# **BORING NO. 11** EL. 5677.8 FT.



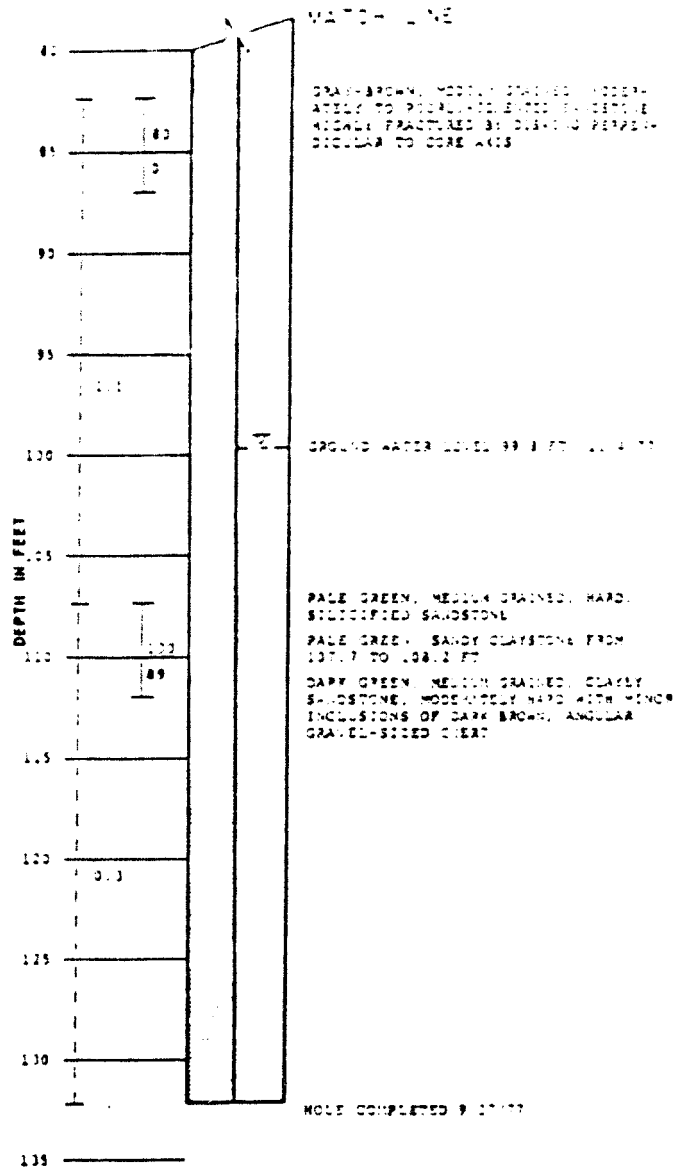
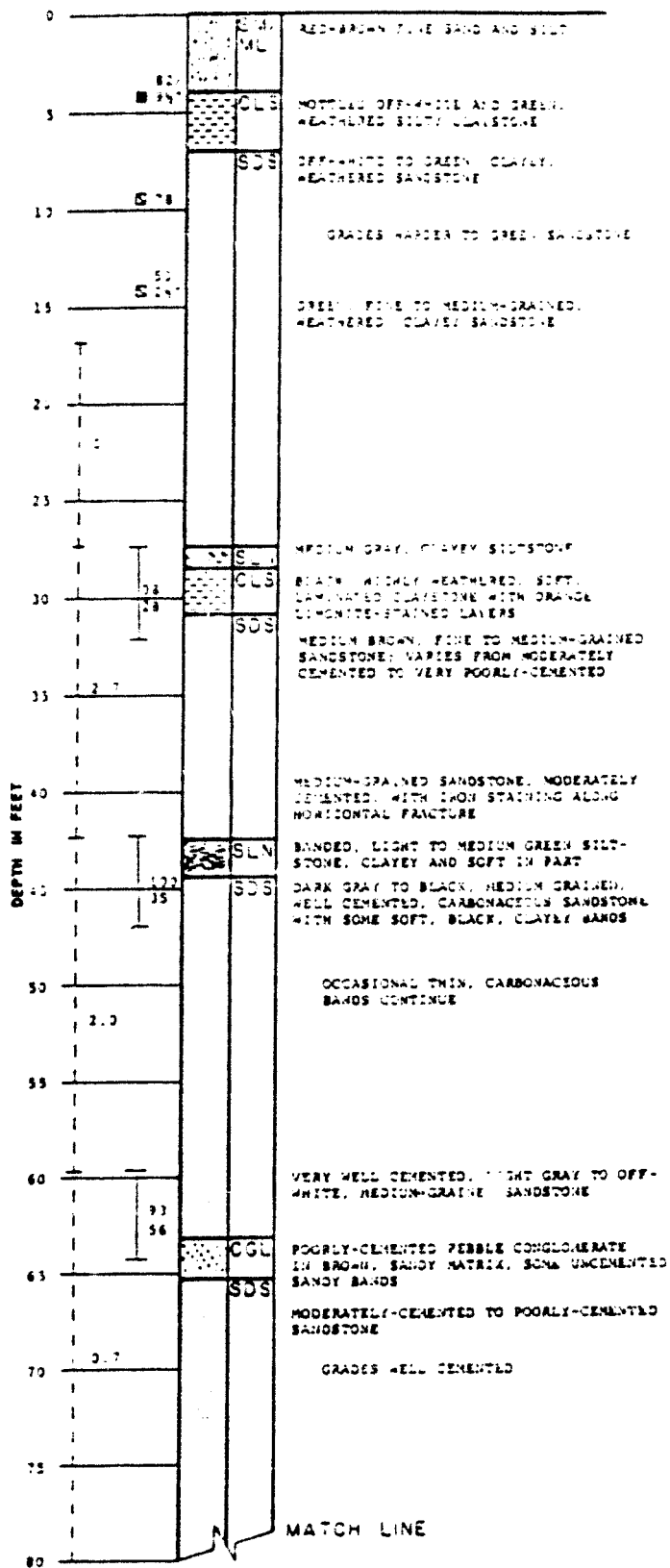
# **BORING NO. 14** EL. 5597.5 FT.



## **LOG OF BORINGS**



ECFING NO. 9  
E. 1023.4 P.

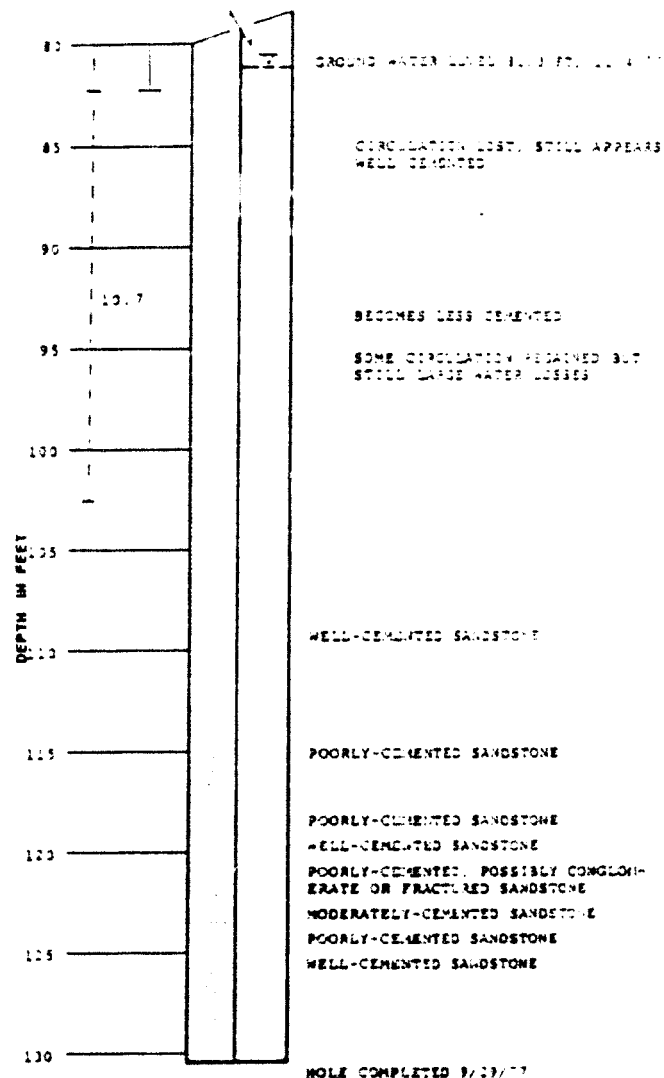
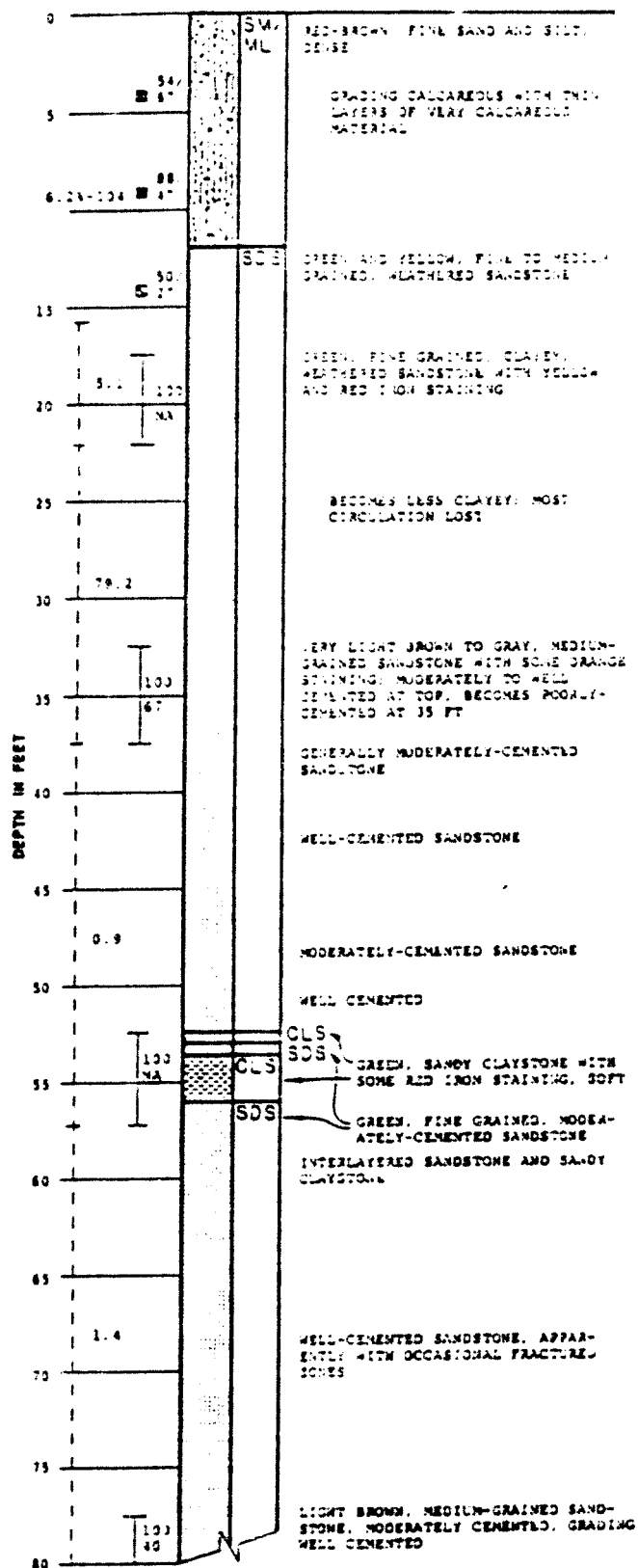


## LOG OF BORINGS

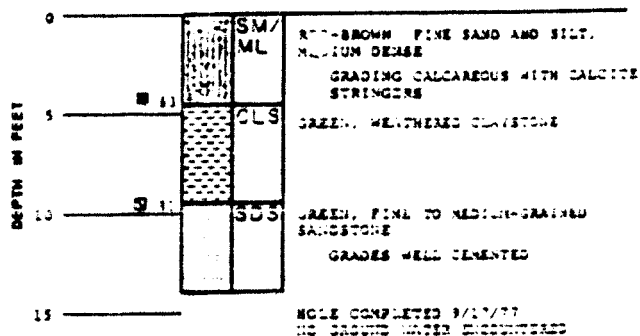
**DAMES & MOORE**

PLATE A-6

**BORING NO. 12**  
EL. 5648.1 FT.



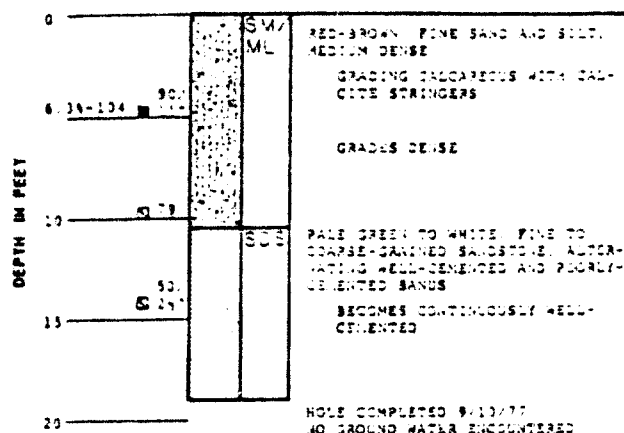
**BORING NO. 15**  
EL. 5600.7 FT.



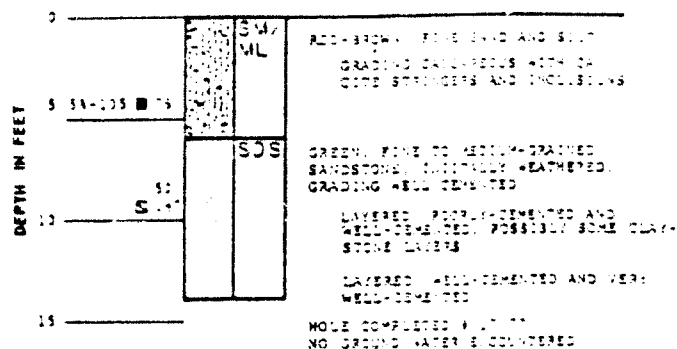
**LOG OF BORINGS**

DAMES & MOORE

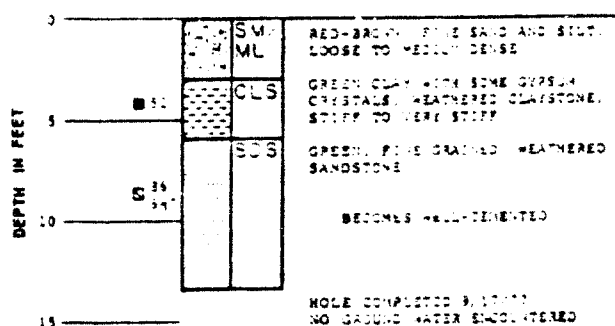
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EL. 5597.5 FT.



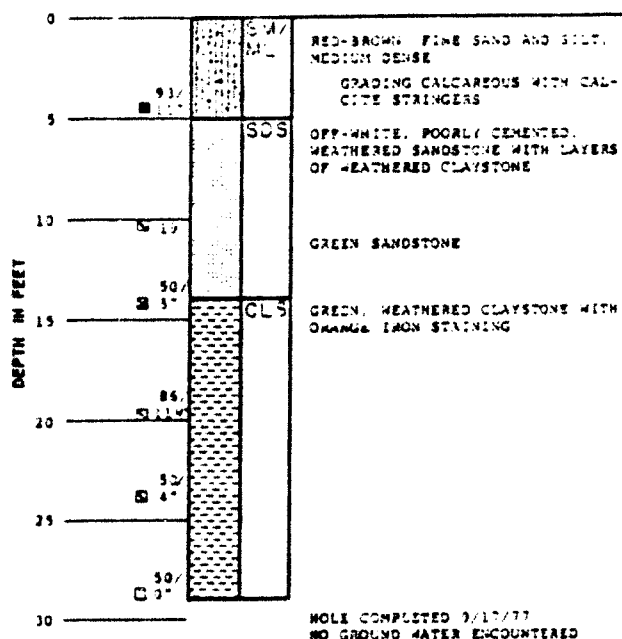
**BORING NO. 17**  
EL. 5592.0 FT.



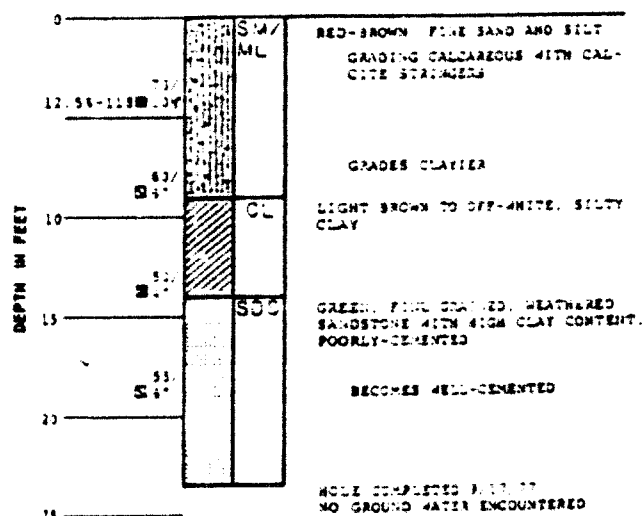
**BORING NO. 21**  
EL. 5594.5 FT.



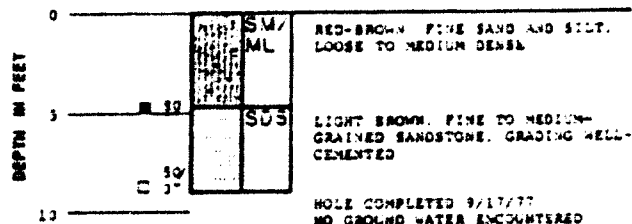
**BORING NO. 18**  
EL. 5608.5 FT.



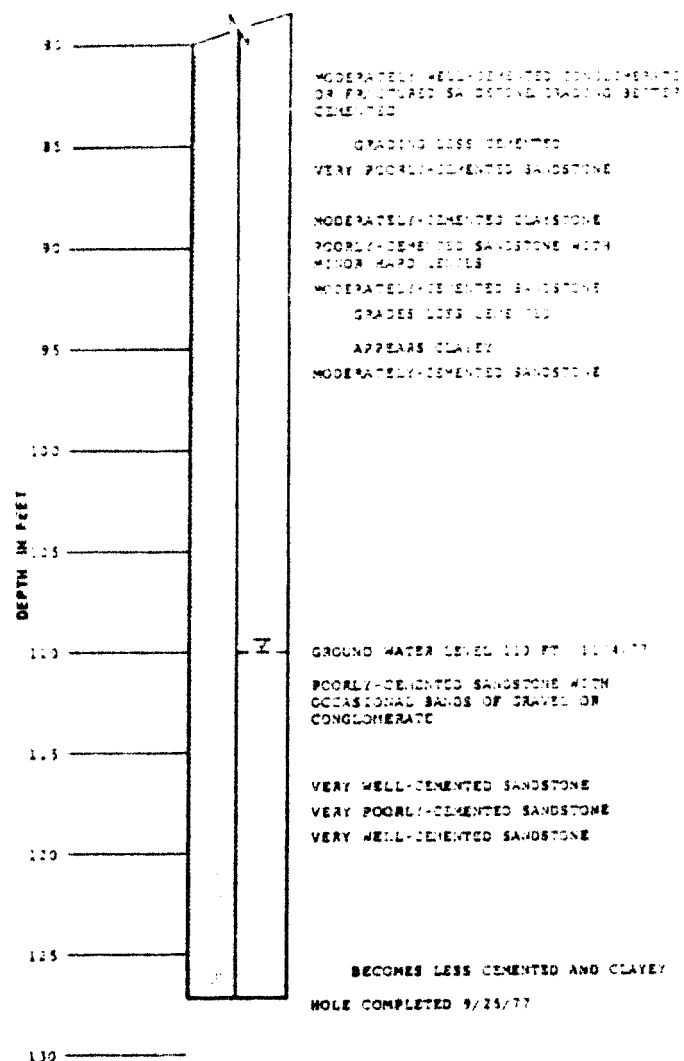
**BORING NO. 22**  
EL. 5585.3 FT.



**BORING NO. 20**  
EL. 5570.4 FT.



**LOG OF BORINGS**

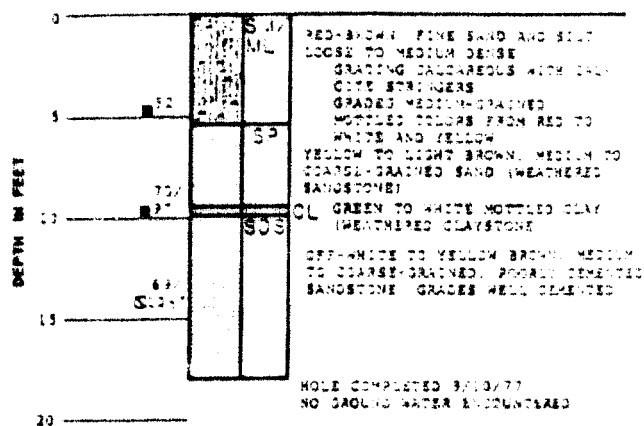


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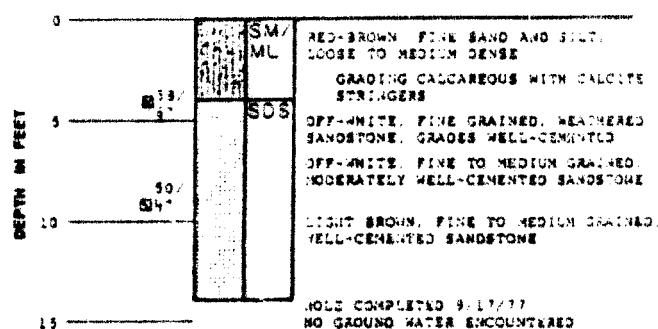
**DANES & MOORE**

PLATE A-9

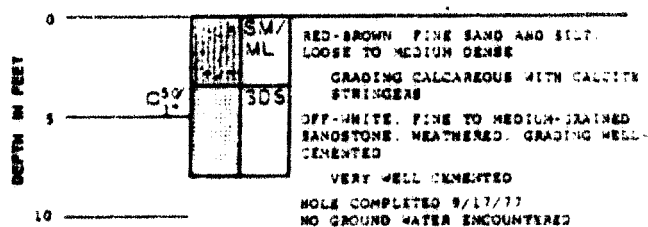
# BORING NO. 23 E.L. 5555.9 FT.



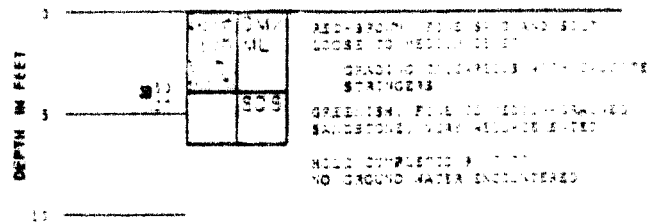
# BORING NO. 24 E.L. 5573.4 FT



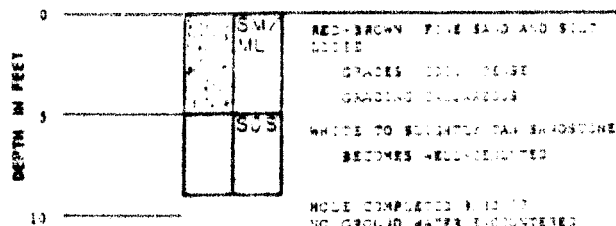
# BORING NO. 26 E.L. 5578.3 FT.



# BORING NO. 27 E.L. 5555.0 FT.



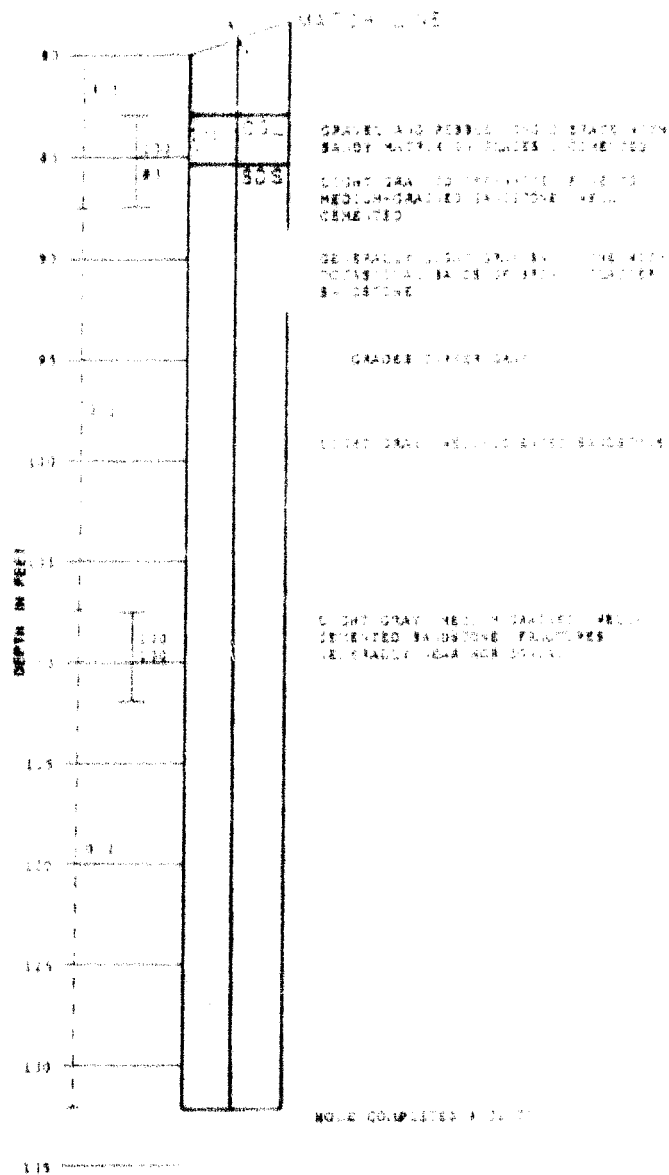
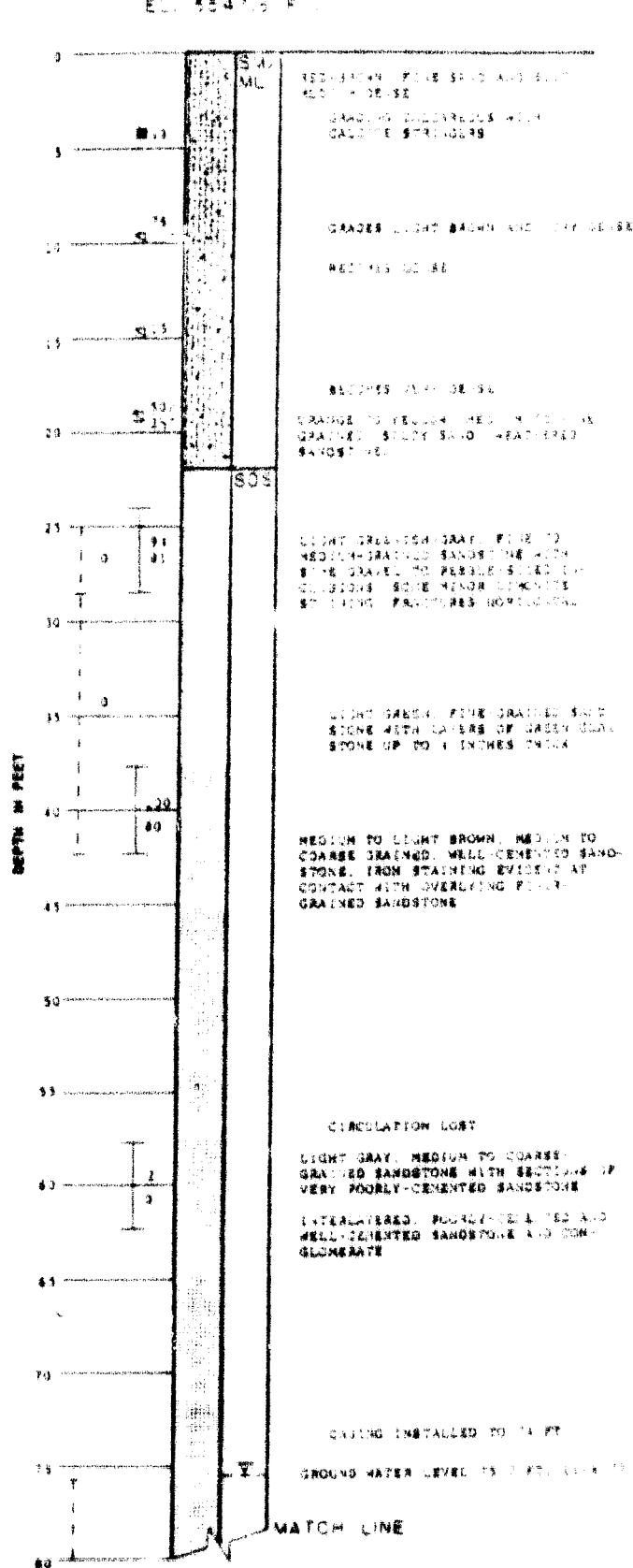
# BORING NO. 29 E.L. 5655.0 FT. (APPROX)



## LOG OF BORINGS

# BORING NO. 28

EL. 5547.6 FT.

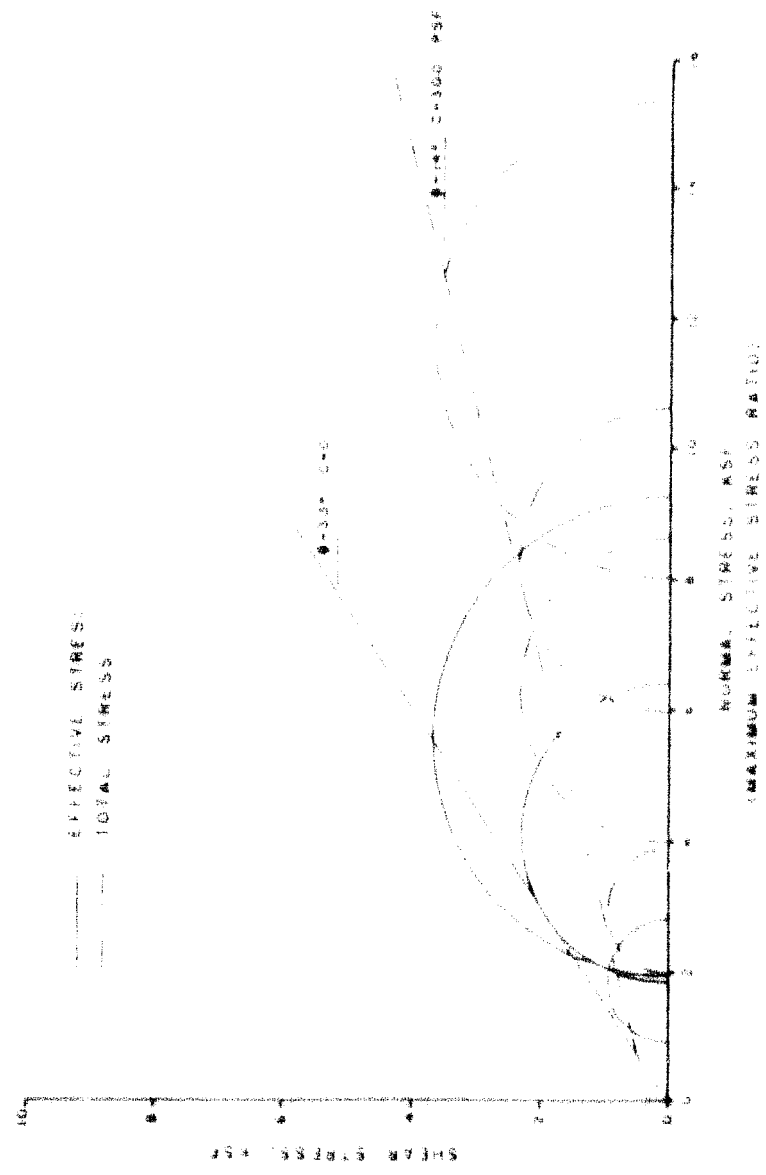


LOG COMPLETED 1/24/77

## LOG OF BORINGS

DAMES & MOORE

# TRIAXIAL COMPRESSION TESTS ON SILTY FINE SAND COMPACTED TO 95% OF AASHTO T-99 MAXIMUM DRY DENSITY



TEST NO.	DATE	TESTER	LABORATORY	TEST TYPE	STRESS STATE	FAILURE STATE	FAILURE MODE	FAILURE ORIENTATION	FAILURE ANGLE	FAILURE STRAIN	FAILURE STRESS	FAILURE STRAIN RATE	FAILURE TIME	FAILURE LOCATION	FAILURE COMMENTS
1	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
2	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
3	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
4	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
5	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
6	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
7	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
8	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
9	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
10	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
11	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
12	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
13	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
14	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
15	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
16	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
17	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
18	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
19	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...
20	10/10/68	J. L. ...	...	...	...	...	...	...	...	...	...	...	...	...	...

## TRIAXIAL COMPRESSION TEST REPORT TYPE OF TEST: ... TYPE MATERIAL: ...

CLASSIFICATION: ...  
 LABORATORY: ...  
 PROJECT: ...  
 LOCATION: ...  
 DATE: ...  
 PREPARED BY: ...  
 CHECKED BY: ...





SECTION 5

TEST PIT LOGS

From

Engineer's Report

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# **Second Phase Design - Cell 3 Tailings Management System**

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White Mesa Uranium Project  
Blanding, Utah

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Energy Fuels Nuclear, Inc.  
Denver, Colorado

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D4-1

D4-2

D4-3

DATE BEGAN: 1/1-81

TEST PIT NO. 4-1

FIELD ENGINEER R. Greenwood

DATE FINISHED: 1/1-81

CHECKED BY R. Greenwood

N ~319.690'

E ~2,573.206'

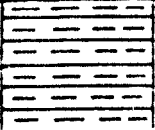
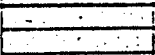
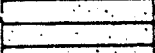
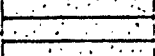
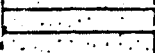
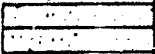
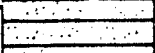
GROUND SURFACE EL: ~5594'

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS
				<div>SAMPLE NO.</div> <div>DEPTH</div> <div>TYPE</div>
	1.0	SS	Medium stiff red sandy silt. Dry. Roots to depth	1 10"-12" JAR
			1.5'	
	2.0	SS	Very stiff red-white sandy silt. Dry. Very calcareous	2 22"-24" JAR
			2.8'	
5590	3.0	SS	Soft-medium stiff red sandy silt. Moist, w/streaks of white. Slightly calcareous	
	4.0	SS		
				3 4'6" JAR
	5.0	SS		
				A 5'-5'6" BAG 3.8% CaCO <sub>3</sub>
	6.0	SS		
	7.0	SS		
5586	8.0	SS		4 8'4" JAR
	9.0	SS	9.0'	
			Very soft greenish gray claystone. Moist, weathered	
	10.0			5 10'4" JAR
				Consistency of soil but with intact layering
	11.0			B 10'-10'6" BAG 9.55% CaCO <sub>3</sub>
5582	12.0			

FIELD ENGINEER R. Greenfield

Checked by E. Greenwood

GROUND SURFACE EL: ~5594'

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS
	13.0		"Same as above"	
				
	14.0		Soft yellowish brown sandstone. Weathered	Intact, Easy rippable
				
	15.0			
				
5578	16.0			Ripping becomes difficult
			Bottom of test pit 16.0'	

DATE BEGAN: 1/22/81

TEST PIT NO. 4-2

FIELD ENGINEER R. Greenwood

DATE FINISHED: 1/24/81

CHECKED BY R. Greenwood

N ~ 319.950' E ~ 2,577.953'

GROUND SURFACE EL ~ 5583'

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS		
				SAMPLE NO.	DEPTH	TYPE
5580	1.0		Red sandy silt. Dry w/ roots to depth			
	2.0					
	2.0					
5580	3.0		Medium dense red silty sand. Moist.	A	2'-5'	BAG
				2.3% CaCO <sub>3</sub>		
				1	3'	JAR
	4.0					
	5.0					
5576	6.0		Soft yellowish brown sandstone	2	5'8"	JAR
	7.0					
			Bottom of test pit 7.0'			

ROUND SURFACE EL: ~ 552

ELEV (FEET)		DEPTH (FEET)		PROFILE	DESCRIPTION	REMARKS		
						SAMPLE NO.	DEPTH	TYPE
		1.0		SS SS	Medium dense red sandy silt. Dry. Roots to depth			
			1.3'	SS SS				
5580		2.0		SS	Medium dense red-white silty sand. Very calcereous	1	1'8"	JAR
				SS		A	2'-3'4"	BAG
		3.0		SS		21.6% CaCO <sub>3</sub>		
			3.3'	SS				
5578		4.0		SS	Soft greenish gray to yellowish brown claystone-sandstone. Weathered	2	3'9"	JAR
		5.0		SS				
			5.0'					
					Bottom of test pit			
					5.0'			

DATE BEGAN: 1/24/87

TEST PIT NO. 4-4

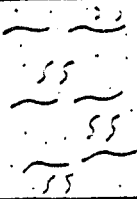
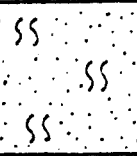

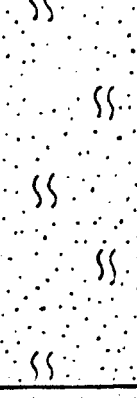
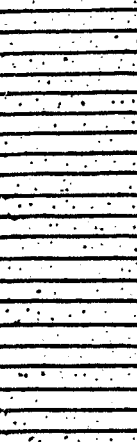
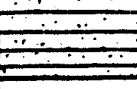

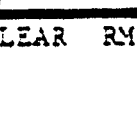

FIELD ENGINEER R. Greenwood

TE FINISHED: 1/24/87

CHECKED BY R. Greenwood

GROUND SURFACE EL: 5581'

N 210.740' E 2,577.530'


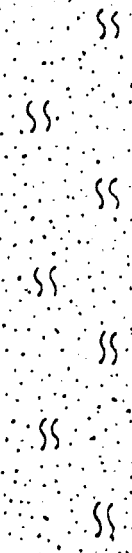
ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS		
5580	1.0		Medium dense red sandy silt. Dry. Roots to depth	SAMPLE NO.	DEPTH	TYPE
			1.5'			
	2.0		Dense red-white silty sand. Slightly calcareous. Dry	1	2'1"	JAR
			2.7'			
	3.0		Medium dense red silty sand. Moist	A	3'	BAG
				1.7% CaCO <sub>3</sub>		
5576	5.0					
	6.0			2	6'	JAR
	7.0					
	8.0					
			8.0'			
5572	9.0		Soft yellowish brown sand- stone. Weathered			
	10.0					
	11.0					
	12.0					
			12.0'			
			Bottom of test pit 12.0'			

GROUND SURFACE EL: 3573'

[illegible]



GROUND SURFACE EL: ~ 5570'

GROUND SURFACE			REMARKS					
ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	SAMPLE NO.	DEPTH	TYPE		
5578	1.0		Medium stiff red sandy silt					
	2.0						2.0'	
	3.0						Dense red-white silty sand. Dry. Very calcareous	1
4.0	3.7'	A 3% CaCO <sub>3</sub>	4'	BAG				
5574	5.0		Medium dense red silty sand. Moist		2	4.5'	JAR	
	6.0							
	7.0							
	8.0							8.0'
								8.5'
			Bottom of test pit 8.5'					
					</			

DATE BEGAN: 1/15/81

TEST PIT NO. 4-7


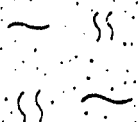
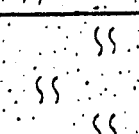
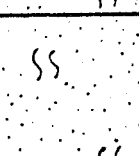
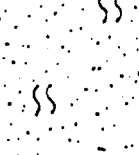
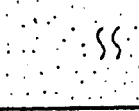
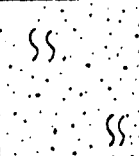
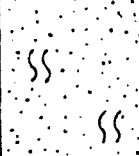

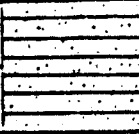
FIELD ENGINEER R. Greenwood

TE FINISHED: 1/15/81

CHECKED BY R. Greenwood

N 322.030' E 2576.346'

GROUND SURFACE EL: 5582'

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS		
			Medium stiff red sandy silt. Dry. Roots to depth	<u>SAMPLE NO.</u>	<u>DEPTH</u>	<u>TYPE</u>
	1.0					
	2.0		2.0'	1	2'	JAR
	3.0		Dense redish-white silty sand. Very dry. Chalky. Very calcareous 3.0'			
5580	4.0		Medium dense red-white silty sand. Moist. Very calcareous	2	4'	JAR
	5.0			A	3'6"-6'	BAG
	6.0		6.0'	3	6'	JAR
	7.0		Dense redish white silty sand. Very dry. Chalky. Very calcareous			
5576	8.0					
	9.0		9.0'			
	10.0		Soft yellowish brown sandstone 10.0'			
			Bottom of test pit 10.0'			

DATE BEGAN: 1/15/61  
DATE FINISHED: 1/15/61  
GROUND SURFACE ELEV: ~5520'

TEST PIT NO. 4-8  
N ~319.916' E ~2,576,056'

FIELD ENGINEER: R. Greenwood  
CHECKED BY: R. Greenwood

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS		
				<u>SAMPLE NO.</u>	<u>DEPTH</u>	<u>TYPE</u>
5522	1.0	SS	Medium stiff red sandy silt. Dry. Roots to depth			
	2.0	SS		1	2'	JAR
	2.7	SS	Dense red silty sand. Dry. Some with white streaks			
	3.0	SS		A	3'-6'	BAG
	4.0	SS	Loose red silty sand. Moist. Some white streaks	7.2% CaCO <sub>3</sub>		
	5.0	SS		2	4'5"	JAR
5584	5.0	SS				
	6.0	SS	Dense whitish red silty sand. Moderately calcareous	3	6'	JAR
	7.0	SS				
	8.0	SS				
5580	9.0	SS				
	10.0		Soft yellowish brown sandstone			
					10.0'	
			Bottom of test pit 10.0'			

DATE BEGAN: 1/15/81

TEST PIT NO. 4-9

FIELD ENGINEER R. Greenwood

FINISHED: 1/15/81

CHECKED BY R. Greenwood

GROUND SURFACE EL: 5594'

N 210 305' E 30 575 660'

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS
				<div>SAMPLE NO.</div> <div>DEPTH</div> <div>TYPE</div>
	1.0	SS —	Medium stiff red sandy silt. Dry. Roots to depth	
	2.0	SS —		1 2' JAR
	3.0	SS —	Dense red silty sand. Dry. White streaks. Slightly calcareous	
	4.0	SS —	Stiff redish white sandy silt. Dry. Very calcareous	2 4' JAR
5590	5.0	SS —	Medium dense red silty sand. Moist. Some white streaks. Slightly calcareous	A 4'-8' BAG
	6.0	SS —		24% CaCO <sub>3</sub>
	7.0	SS —	Dense redish white silty sand. Chalky. Very calcareous. Dry	3 7' JAR
5586	8.0	SS —		
	9.0		Very soft yellowish brown sandstone. Weathered	
	10.0		Medium hard yellowish brown sandstone	
	11.0			11.0'
			Bottom of test pit 11.0'	

DATE BEGAN: 1/12/81

TEST PIT NO D-1

FIELD ENGINEER R. Greenwood

DATE FINISHED: 1/14/81

N 110 000' E 2 577.830'

CHECKED BY R. Greenwood

GROUND SURFACE EL: 5577'

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS		
5574	1.0	SS	Stiff red sandy silt. Dry. Roots to depth	SAMPLE NO.	DEPTH	TYPE
		SS	1.3'			
	2.0	SS	Dense red silty sand. Dry. 1.0'			
	3.0	SS	Medium dense red silty sand. Moist	1	2'4"	JAR
5570	4.0	SS		ST-1	3'-5'	3" Shelby
		SS	4.3'	2	4'2"	JAR
	5.0		Very soft greenish gray claystone. Moist			
	6.0					
5570	7.0		7.0'	3	7'2"	JAR
	8.0		Soft yellowish brown sandstone 8.0'			
			Bottom of test pit 8.0'			

DATE BEGAN: 1/12/81

TEST PIT NO. D-2

FIELD ENGINEER R. Greenwood

TEST FINISHED: 1/14/81

N ~ 310 225' E ~ 570 155'

CHECKED BY R. Greenwood

GROUND SURFACE EL: ~ 5570'

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS		
				<u>SAMPLE NO.</u>	<u>DEPTH</u>	<u>TYPE</u>
5570	1.0	SS	Medium stiff red sandy silt. Dry. Roots to depth			
	2.0	SS	2.0'			
5566	3.0	SS	Dense red silty sand. Dry. Streaks of white	1	3'	JAR
	4.0	SS	Medium dense red silty sand. Moist. Streaks of white			
	5.0	SS				
	6.0	SS		ST-1	5'-7'	3" Shelby
	7.0	SS	7.0'	2	6'	JAR
5562	8.0	SS	Dense red silty sand. Dry	ST-2	6'-8'	3" Shelby
	9.0	SS				
	10.0	SS	9.75'	3	8'	JAR
	11.0		Soft yellowish brown sandstone			
			11.5'			
			Bottom of test pit 11.5'			

DATE BEGAN: 1/15/81

DATE FINISHED: 1/15/81

GROUND SURFACE EL.: ~5579'

TEST PIT NO. D4-3

N ~319.430' E ~2.576.315'

FIELD ENGINEER R. Greenwood

CHECKED BY R. Greenwood

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS		
				SAMPLE NO.	DEPTH	TYPE
5574	1.0	SS	Medium stiff red sandy silt. Dry. Roots to depth 1.0'	1	2'	JAR
	2.0	SS	Dense red silty sand. Dry. Some roots 1.7'			
	3.0	SS	Dense red silty sand. Dry. Streaks of white. Slightly calcareous 3.3'			
	4.0	SS	Medium dense red silty sand. Moist	ST-1	3'6"-5'6"	3"Shelby
	5.0	SS		2	5'	JAR
	6.0	SS	6.0'			
	7.0		Soft yellowish brown sandstone 7.0'			
			Bottom of test pit 7.0'			

SECTION 6

TABLE C-1  
SUMMARY OF LABORATORY TEST RESULTS  
CELL 4



Cell 4

**PROJECT NO.**

DATE \_\_\_\_\_

[illegible]



0 BH-11

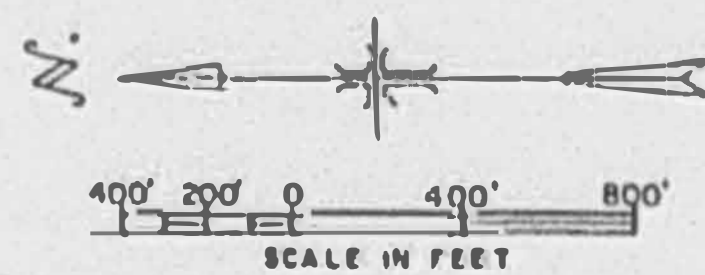


STOCKPILE TABLE

MATERIAL	STOCKPILE NO	QUANTITY C.Y.
VEGETATIVE SUPPORT MATERIAL	VSM-2	142,790
	VSM-1	102,370
	VSM-3	21,060
	VSM-4	55,110
	TOTAL	324,770 c.y.
RANDOM FILL	RF-1	140,410
	RF-2	850,000
	RF-3	1,372,870
	RF-4	441,060
	RF-5	312,180
	TOTAL	3,122,320 c.y.
CLAY	C-1	382,140
	C-2	83,820
	TOTAL	435,740 c.y.
TOTAL QUANTITY OF ALL STOCKPILES		3,882,800 c.y.

LEGEND

- ENVIRONMENTAL SAMPLING STATION
- △ MONITORING WELL
- CHEN AND ASSOCIATES DRILL HOLES
- DAMES AND MOORE DRILL HOLES
- TEST PITS D'APPOLONIA



WESTERN CONSULTING ENGINEERS AND SURVEYORS	
Umetco Minerals Corporation	
WHITE MESA URANIUM MILL SITE	
BORING & TEST PIT LOCATIONS	
DATE	REVISION - DESCRIPTION
06/21/88	01
06/21/88	02
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8905220394



**ATTACHMENT 2**

DRAWN BY	CHECKED BY	DATE	SHEET	DRAWING	NUMBER

DRAWING RM 78-682-89



2-16

T 36 S

N 305,000

E 2,579,000

E 2,579,000

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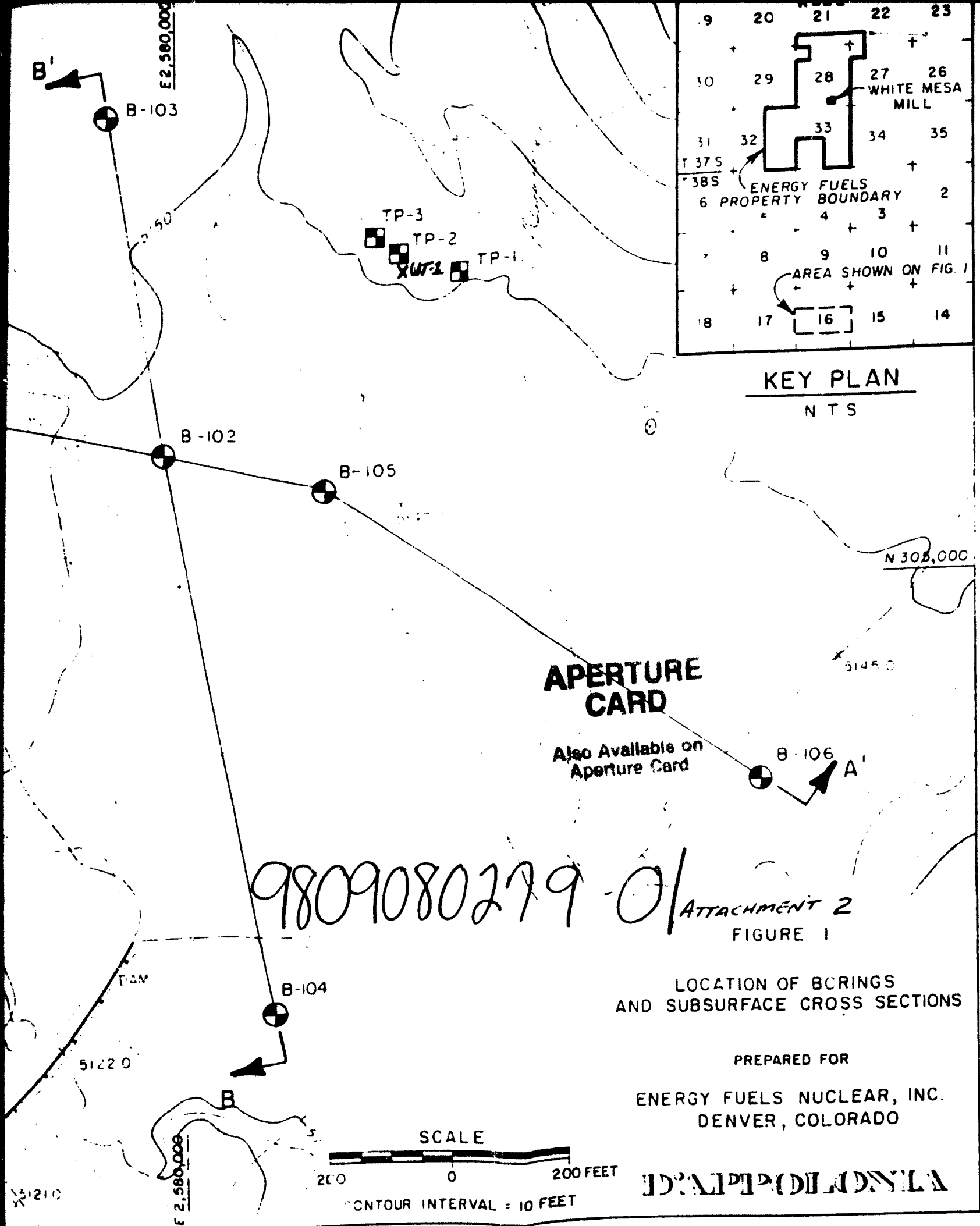
B-100

3-101

REFERENCE

TOPOGRAPHIC MAP OF BLANDING MILL  
SITE, SHEET 4, BY DELTA AERIAL  
SURVEYS, INC., 12-8-76

SECTION 16  
R 22 E



**ATTACHMENT 3**

# MAGNITUDE SCALING FACTORS FOR SOIL LIQUEFACTION EVALUATIONS

By Ignacio Arango,<sup>1</sup> Member, ASCE

**ABSTRACT:** Energy concepts are applied to the conditions that are likely to have existed at distant liquefaction sites in past earthquakes. From this, magnitude scaling factors are derived that reflect field cyclic strength conditions. It is shown in the paper that the factors are independent of the field acceleration assumed to have existed at the sites, and are only dependent on the magnitude-equivalent number of cycles relationship. These factors are compared with others based on laboratory cyclic strengths from Seed and Idriss and on statistical regression of data from field case histories of liquefaction. The factors derived based on energy concepts are similar to those derived by Ambraseys based on a statistical analyses of extensive liquefaction data. It is concluded that the factors derived in the paper based on energy concepts, or by Ambraseys, appropriately represent field conditions and avoid the limitations and extrapolations of the laboratory-based derivation by Seed and Idriss, and they are recommended for use in the analysis of liquefaction potential.

## INTRODUCTION

The first comprehensive listing of site conditions at various locations where seismic liquefaction did or did not take place was presented by Seed and Peacock (1970). Their chart, summarizing the results, correlates the relative density of the soil deposit to the calculated cyclic stress ratio. In the chart, the cyclic stress ratio inducing liquefaction in the field is independent of the magnitude of the earthquake.

Noting the scarcity of reliable field data concerning the liquefaction potential of sands with high densities (or corresponding penetration resistance), Seed et al. (1975) used the results of the laboratory shake table tests carried out by DeAlba et al. (1976) to extend the data to earthquake magnitudes in two ranges: 5–6 and 7–7.5. The laboratory tests were performed on specimens prepared at relative densities of 54, 68, 82, and 90%. Based on the density-penetration resistance relationships by Gibbs and Holtz (1957), these densities correspond to penetration resistances (normalized to an effective confining pressure of 1 ksc) equal to 11, 19, 27, and 32, respectively. The chart developed by Seed, Arango, and Chan is the first published liquefaction chart that reflects the influence of earthquake magnitude on the liquefaction susceptibility of a sand deposit.

Subsequently, Seed (1979) incorporated penetration data obtained at the U.S. Army Engineer Waterways Experiment Station, and the Chinese liquefaction criteria (unpublished data), to extend the 1975 chart to earthquakes of magnitude 8.3.

Davis and Berri (1983) compared Seed's (1979) results with field liquefaction data derived mainly from Japanese earthquakes. The comparison led them to conclude that Seed's results were generally overconservative for small earthquakes and possibly unconservative for very large earthquakes. The discussers found an unacceptable lack of agreement between the field data utilized in their study and the combined field/large-scale laboratory test data presented by Seed (1979).

Seed and Idriss (1982) used the field data from Seed's 1979 study to prepare a chart showing the relationship between the SPT-*N* blowcount normalized to an effective confining pres-

sure equal to 1 ksc,  $N_1$ , and the cyclic stress ratios inducing or failing to induce liquefaction under a magnitude 7.5 event. The authors pointed out that "one of the limitations of the [previous] chart was the limited number of reliable data points available to define the boundary separating liquefiable from non-liquefiable soils." A second limitation was its "inability to differentiate between appropriate boundaries for different magnitude earthquakes." The first difficulty was alleviated by inclusion of supplementary data obtained from the Haicheng (1974) and the Tangshan (1976) earthquakes in China; the earthquakes in Guatemala (1976), and Argentina (1977); and the Miyagiken-Oki earthquakes (1978) in Japan. The combination of the field data gathered by Seed in 1979 with those from the preceding earthquakes provided in the words of the authors "a realistic basis for developing correlations between standard penetration tests and the liquefaction characteristics of sands and silty sands for magnitude 7-1/2 earthquakes." The authors also reasoned that the results could be extended to other magnitude events by noting that from a liquefaction point of view, the main difference between different magnitude events is the equivalent uniform number of stress cycles that they induce. Based on previous statistical studies carried out by Seed et al. (1975), and on a representative shape of the laboratory relationship between cyclic triaxial test stress ratio and the number of cycles required to cause liquefaction, the authors obtained the scaling factors shown in Table 1.

Thus, by multiplying the boundary liquefaction/no-liquefaction curve obtained for magnitude 7.5 data by the scaling factors shown in Table 1, boundary curves for other earthquake magnitudes were obtained.

Seed et al. added additional field data to the 1982 chart in 1984. In its present form (Seed et al. 1984), the relationship between penetration resistance and cyclic stress ratio causing liquefaction in a magnitude 7.5 earthquake is as shown in Fig. 1. In this figure, the property selected to characterize the soil deposit is the SPT-*N* blow count normalized to an effective

**TABLE 1. Magnitude Scaling Factors Derived by Seed and Idriss in 1982 Based on Laboratory Simple Shear Test Data**

Earthquake magnitude <i>M</i>	Number of equivalent uniform cycles	Magnitude scaling factor
		cyclic strength $M = M$ cyclic strength $M = 7.1/2$
(1)	(2)	(3)
8.5	26	0.89
7.5	15	1.00
6.75	10	1.13
6	5–6	1.32
5.25	2–3	1.50

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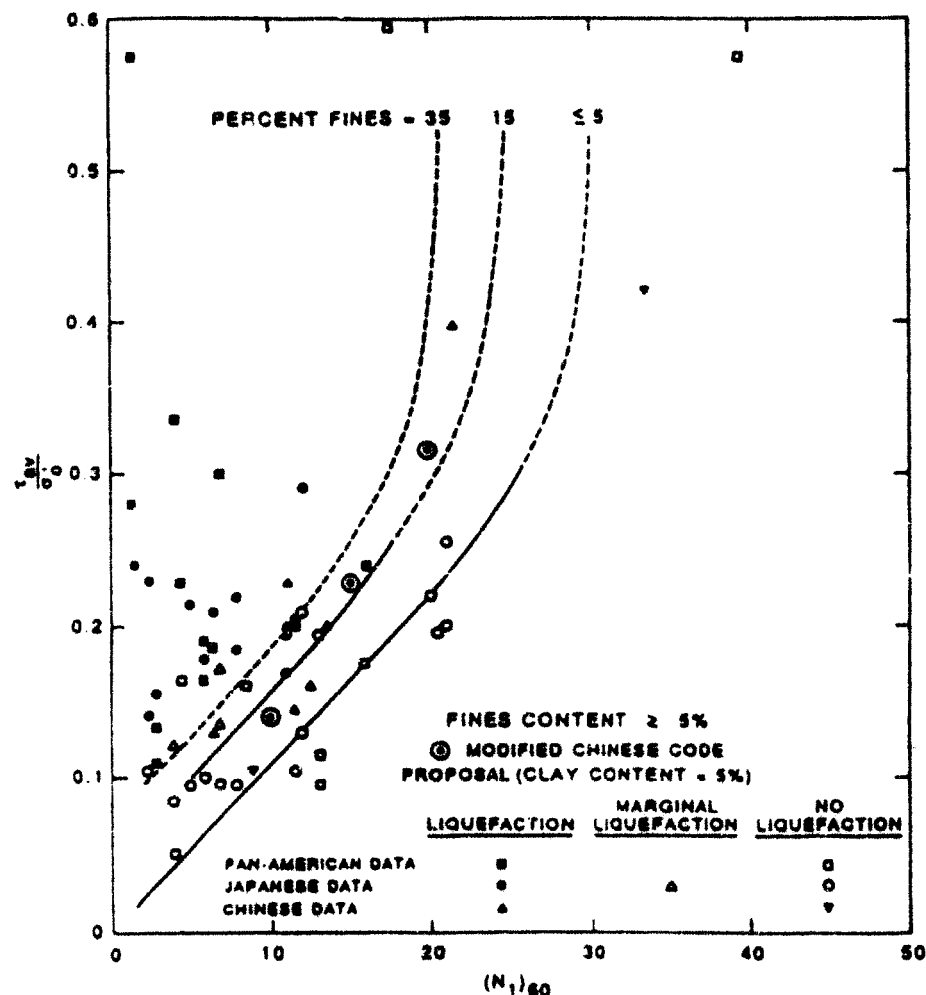


FIG. 1. Relationship between Stress Ratio Causing Liquefaction and  $(N_1)_{60}$  Values for Silty Sands for  $M = 7.5$  Earthquakes

overburden pressure equal to 1 ksc and to a hammer energy equivalent to 60% of that of a free fall,  $(N_1)_{60}$ . In today's practice for other magnitude events, the scaling factors listed in Table 1 are generally used.

A critical review of the findings presented by Seed and Idriss (1982) and by Seed et al. (1984) was prepared by Ambraseys (1988). In the reviewer's opinion, "the conversion of ground acceleration records to an equivalent uniform cycle format neglects any consideration of the nature of the earthquake ground motions and their behavior. It is not considered reasonable to allow the level of shear stress for a given magnitude earthquake to vary in a deposit only with a fixed number of equivalent cycles, without also including some consideration of the distance of the site from the seismic source, allowing for attenuation." Ambraseys then proceeded to develop relationships between average stress ratio causing liquefaction and standard penetration resistance for different magnitude earthquakes without recourse to a scaling factor. To this end, he separated the available field liquefaction data in groups corresponding to four earthquake magnitude ranges (6.0–6.6; 6.7–7.2; 7.3–7.5; and 7.6–8.2) and statistically analyzed the stress ratio blow count relationship for each group. From the study, Ambraseys derived the earthquake magnitude scaling factors (MSFs) in Table 2.

In a separate effort, Williams (1994) applied a logit regression approach to the field liquefaction database by also separating them in cases belonging to earthquakes around magnitudes 5.5, 6.0, 6.5, 7.0, 7.5, 8.0, and 8.5. MSFs that have approximately a 32% probability of misclassification were derived. This probability of misclassification is, according to Williams, about the same as that of the boundary line for

TABLE 2. Magnitude Scaling Factors Obtained by Ambraseys in 1988 Based on Statistical Analysis of Field Liquefaction Data

Earthquake magnitude (1)	Magnitude scaling factor (2)
8.5	0.44
8	0.67
7.5	1.00
7	1.30
6.75	1.48
6.5	1.69
6	2.20
5.5	2.86

TABLE 3. Magnitude Scaling Factors Obtained by Williams in 1994 Based on Logit Analysis of Field Liquefaction Data

Earthquake magnitude (1)	Magnitude scaling factor (2)
8.5	0.62
8	0.72
7.5	1.00
7	1.34
6.75	1.58
6.5	1.88
6	2.79
5	4.46

clean sands shown in Fig. 1. The MSFs obtained by the author are given in Table 3.

A different approach to derive MSFs was carried out in Arango (1994). The writer, utilizing the field liquefaction data observed at sites with the largest documented epicentral dis-



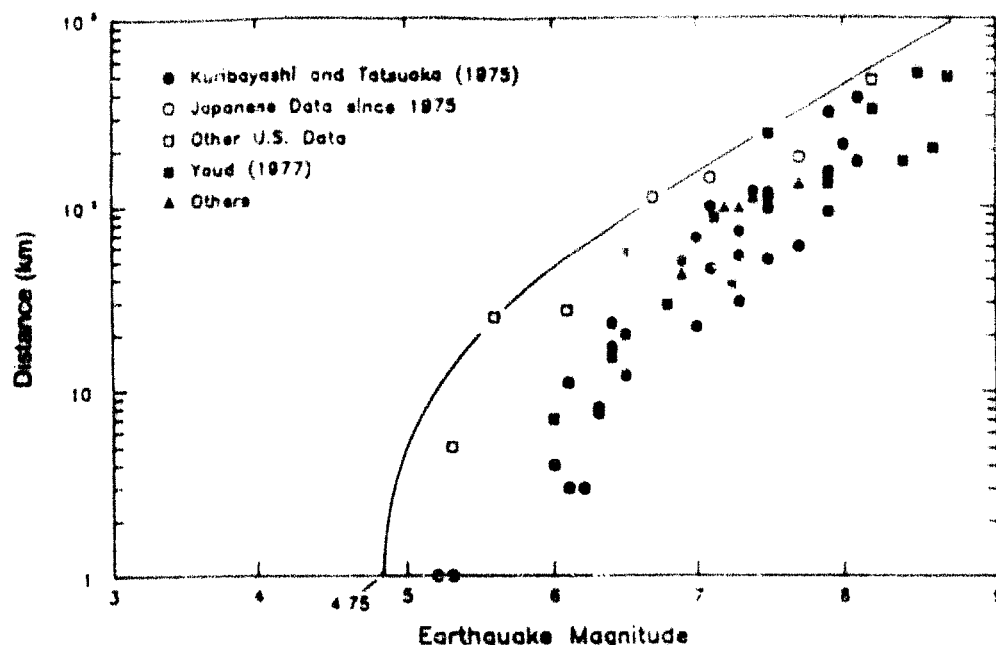


FIG. 2. Maximum Distance of Liquefaction from Zone of Faulting

tances, applied energy concepts to derive MSFs that are independent of laboratory test results. This paper summarizes the methodology used and the results obtained following this energy-based approach.

#### LIQUEFACTION AT LARGE EPICENTRAL DISTANCES

Compilation of data regarding the distance from a given epicenter to the most distant occurrence of liquefaction has been done by several investigators (Kuribayashi and Tatsuoka 1975; Youd 1977; Davis and Berril 1983; Seed et al. 1984; and Carter and Seed 1988). Fig. 2 (Carter and Seed 1988) includes data on 125 cases of liquefaction throughout the world. Maximum distances to sites of observed liquefaction can be read on this figure, and are presented in Table 4.

Carter and Seed (1988) estimated the minimum ground surface accelerations for which liquefaction has been induced by earthquake shaking as shown in Table 5.

The values of the minimum accelerations were obtained by averaging the results of several attenuation relationships (Orphal and Lahoud 1974; McGuire 1977; Cornell et al. 1979; Donovan and Bornstein 1974; Iwasaki et al. 1978; McGuire 1978; Battis 1981; and Hasegawa et al. 1981).

The characteristics of the liquefied soil deposits at these distant places are not well documented. Davis and Berril (1983) reasoned that these sites must represent "soft, highly liquefiable deposits near the ground surface with low  $N_1$  values and a high water table." In their study, the discussers considered the ground water to be at the ground surface, and  $N_1$  values equal to 1 and 5. Seed et al. (1984) considered that these deposits consist of "very loose sands, say with an  $(N_1)_{60}$  value of about 4." Ambraseys (1988) concluded from his studies that "the farthestmost sites at which liquefaction was observed should have been associated with  $N_1$ -values of less than 6 for clean sands, and about 3 or less for sands with 15% fines." Such loose deposits would probably have a relative density somewhere around 30–40%, which appears to be the loosest state encountered in young alluvial plain and lake deposits in nature (Arango 1994).

#### EARTHQUAKE ENERGY

The energy of an expanding earthquake wave front can be estimated from accelerogram recordings from which ampli-

TABLE 4. Maximum Distance of Liquefaction from Zone of Faulting [after Carter and Seed (1988)]

Earthquake magnitude (1)	Maximum distance, km (2)
8.25	500
8	400
7.5	230
6.5	75
5.5	20
5	5

TABLE 5. Minimum Ground Accelerations for which Liquefaction Has Been Induced by Ground Shaking [after Carter and Seed (1988)]

Earthquake magnitude (1)	Minimum acceleration (g) (2)
8.25	0.025
8	0.03
7.5	0.04
7	0.05
6.5	0.06
6	0.08
5.5	0.12

tudes and frequencies of the ground motions can be obtained. The equation of motion of an undamped simple oscillator of mass  $m$  and stiffness  $k$  is

$$m\ddot{y} + ky = m\ddot{z} \quad (1)$$

where  $y(t)$  = motion relative to the ground, and  $z(t)$  = absolute motion with respect to a fixed reference. For that oscillator, the total energy (TE) at any time  $t$  is given by the expression

$$TE = \frac{1}{2} m \dot{y}^2 + \frac{1}{2} k y^2 \quad (2a)$$

$$\sqrt{\frac{2TE(t)}{m}} = \{[\dot{y}(t)]^2 + [\omega_n y(t)]^2\}^{1/2} \quad (2b)$$

where the first term represents the kinetic energy (KE) and the second term represents the strain energy (SE) in the system.

It can be shown (Hudson 1979) that the previous expressions are equivalent to the equation

$$\sqrt{\frac{2TE(t)}{m}} = \left\{ \left[ \int_0^t a(t) \cos \omega_d t dt \right]^2 + \left[ \int_0^t a(t) \sin \omega_d t dt \right]^2 \right\}^{1/2} \quad (3)$$

that at the end of the earthquake is the same as the Fourier amplitude spectrum of the ground acceleration. Note that both the kinetic and the strain energies are proportional to the square of the absolute ground acceleration.

In an undamped oscillator, the kinetic energy is zero at the maximum displacement, and it is at a maximum at the static equilibrium point. For the strain energy of the system, the reverse is true. Therefore,  $(KE)_{MAX} = (SE)_{MAX} = \text{total energy of the system} = (TE)$ .

### Maximum Kinetic Energy

If, for locations distant from the source, earthquake waves are considered to become harmonic, the expression for the maximum KE is given by

$$KE = \frac{1}{2} m v^2 = \frac{W}{2g} v^2 = \frac{W}{2g} \times \frac{a^2 T^2}{4\pi^2} \quad (4)$$

where  $W$  = weight of the oscillating mass,  $v$  and  $a$  = its maximum velocity and maximum acceleration, respectively; and  $T$  = fundamental period of the system.

Relationships between earthquake magnitude, distance from the center of energy release, and kinetic energy can be estimated for rock motions if values of acceleration and predominant wave periods are available. The acceleration attenuation relationships proposed by Idriss (1993), and the magnitude, distance, and predominant period relationships for rocks proposed by Seed et al. (1968), or more recently by Idriss (1991), can be used for this purpose.

A similar calculation for soil sites is difficult to perform. This is because the fundamental period of vibration  $T$  depends not only on the stiffness of the soil layers, but also on the geometry, and both of these may vary. As shown in Fig. 3, however, loose sand deposits with an  $(N_1)_{60}$  value of 4 and thicknesses of between 5 m and 10 m overlying about 50 m of stiffer soils underlain by rock would typically have fundamental periods in excess of 0.7 s.

Since in (4)  $W = V\rho$  where  $V$  represents volume and  $\rho$  represents weight per unit volume, it follows that the maximum kinetic energy of a given soil deposit can be expressed as

$$\frac{KE}{V} \sim \text{constant} \times A^2 \quad (5)$$

### Maximum Strain Energy

With reference to Fig. 4, the differential of elastic SE associated with shearing stresses is

$$d(SE) = \frac{\tau^2}{2G} dV \quad (6)$$

where  $\tau/2$  = average shear stress,  $G$  = average shear modulus, and  $V$  represents the volume of the mass under strain. Since, as shown in Fig. 3, the cyclic stress induced in the ground by the earthquake waves is proportional to the ground surface acceleration, it follows that the strain energy per unit volume is

$$\frac{\text{Strain Energy}}{\text{Unit Volume}} = \frac{SE}{V} = \frac{\tau^2}{2G} \sim \text{constant} \times A^2 \quad (7)$$

### Total Energy

The TE, whether instantaneous or maximum, developed by earthquake shaking in a soil deposit per unit volume is seen to be proportional to the square of the ground acceleration.

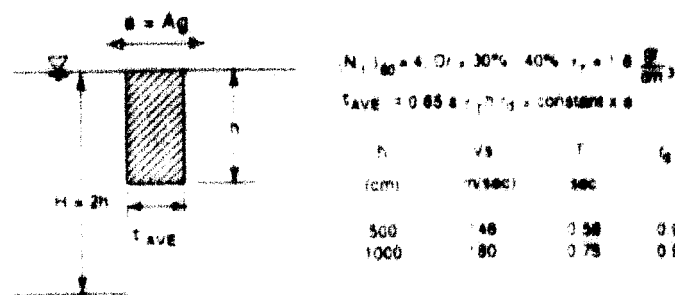


FIG. 3. Assumed Geotechnical Characteristics of Distant Liquefaction Sites

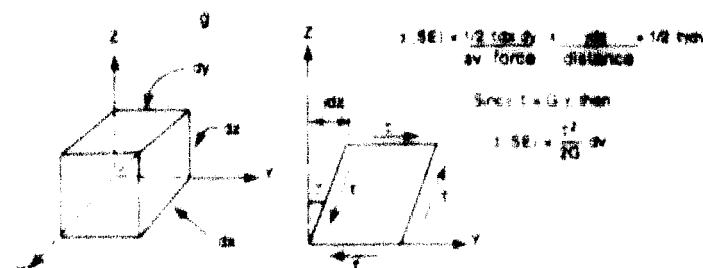


FIG. 4. Strain Energy

Without an external source, no real systems like soil deposits maintain an undiminished amplitude of vibration. Material damping is a name for the complex physical effects that convert kinetic and strain energies in a vibrating system into heat, dislocations, slips, etc.

The time history of acceleration developed at any point in the ground during an earthquake consists of a relatively erratic series of cycles of varying amplitudes. Therefore, both the energies input and absorbed by the profile vary from one cycle to the next during the seismic episode. For analytical as well as for practical purposes, it has been found useful in geotechnical earthquake engineering practice to convert the irregular series of earthquake-induced accelerations to an equivalent series of uniform cycles (Seed et al. 1975; Seed and Idriss 1982).

This approach is adopted in the following derivation. Accordingly, it will be assumed that the time dependent energy that a soil deposit absorbs can be taken as the energy corresponding to that of an "effective acceleration" multiplied by the equivalent uniform number of cycles corresponding to the size of the given earthquake.

### ENERGY REQUIRED FOR LIQUEFACTION

Ambraseys (1988) associated the relationship between earthquake magnitude and maximum epicentral distance to known liquefaction sites to a relationship between attenuated earthquake magnitude and energy. From these the author derived "the least possible values for the energy density required to induce liquefaction ground failures."

### Very Loose Sands

Other investigators have postulated minimum energy values required to reach liquefaction in several types of laboratory tests. For example, Arango (1994) presents cumulative values of the energy absorbed by undrained triaxial specimens in reaching initial liquefaction in 36 stress-controlled cyclic triaxial tests and in 21 strain-controlled cyclic triaxial tests. The writer found that the energy absorbed was a function of the relative density of the test specimen but was relatively independent of the type of test and of the frequency of loading. For specimens at a relative density of 50%, the energy absorbed ranged between  $1.4 \times 10^{-1}$  and  $2.8 \times 10^{-1}$  J/cm<sup>3</sup>.

Figueras et al. (1994) reported the results of 27 torsional

shear tests on specimens at relative densities varying between 50% and 70%. The dissipated energy per unit volume up to liquefaction, varied with the testing conditions (confining pressure and density). For specimens at a relative density near 50%, the unit energy ranged between about  $0.5 \times 10^{-3}$  and  $1.5 \times 10^{-3}$  J/cm<sup>3</sup>, which is in relatively good agreement with the triaxial test results reported by Arango in 1994.

Thus, it is reasonable to assume, based on seismologic and laboratory considerations, that the onset of liquefaction of a soil deposit with given characteristics is associated with a minimum level of energy. If all distant liquefaction sites included in Fig. 2 had the same characteristics (i.e., high ground-water table, low  $(N_1)_{50}$  values near 2-4, similar predominant period), it is reasonable to assume that to liquefy all of these sites absorbed the same energy. Therefore,

$$[(TE)_{M+1}]N_{M+1} = [(TE)_{M+1}] \times N_{M+1} \quad (8)$$

$$N_{M+1} = N_M \times \frac{(TE)_M}{(TE)_{M+1}} = N_M \times \left( \frac{A_M}{A_{M+1}} \right)^2 \quad (9)$$

where  $A$  = acceleration in  $g$ ;  $M$ ,  $M$ , and  $M+1$  = earthquake magnitudes; and  $N_M$ ,  $N_{M+1}$  represent the equivalent number of cycles for magnitudes  $M$  and  $M+1$ .

Adopting 15 as the number of significant stress cycles generated by earthquakes of magnitude  $M = 7.5$  (Seed et al. 1975), (9) provides the equivalent number of cycles  $N_M$  for various earthquake magnitudes that at the acceleration levels shown in Table 5 would have induced equal amounts of energy to the ground. Equivalent uniform number of cycles based on this energy approach are shown in Table 6.

TABLE 6. Duration of Shaking in Earthquakes of Various Magnitudes Obtained in this Investigation by Energy Approach

Earthquake magnitude $M$ (1)	Equivalent uniform number of cycles $N_M$ (2)
8.25	38.4
8	26.7
7.5	15
7	9.6
6	3.8
5.5	1.7

The equivalent uniform number of cycles  $N_M$  from this table, together with the cyclic stress ratios corresponding to the accelerations in Table 5 can be combined to provide the field-derived stress ratios versus the number of cycles relationship required to induce liquefaction in very loose sand deposits with  $(N_1)_{50} \approx 4$ . This relationship is shown in Fig. 5 and in Table 7.

## Denser Sands

Field cyclic strength curves for denser soil deposits can be obtained using the empirical data for clean sands proposed by Seed et al. (1984) (Fig. 1). Take for example, clean sand deposits with  $(N_1)_{50}$  values equal to 4 and 11, respectively. The critical cyclic stress ratios for liquefaction under a magnitude 7.5 earthquake are 0.05 and 0.12, respectively (see Fig. 1). For deposits with an  $(N_1)_{50}$  equal to 11, the field stress ratios required for liquefaction in different numbers of cycles would be those shown by the lower curve in Fig. 5 multiplied by the factor 0.12/0.05. Field cyclic stress ratios required for liquefaction of denser sands at an  $(N_1)_{50}$  value of 11 are shown as the upper curve in Fig. 5.

## MAGNITUDE SCALING FACTORS

Cyclic stresses required to cause liquefaction in the field under various magnitude earthquakes can be obtained from Fig. 5. The ratio between the cyclic stress at a given earthquake magnitude and that corresponding to a magnitude 7.5, that is, the MSF, was calculated from the data for sands with an  $(N_1)_{50}$  value equal to 4. The results are shown in Table 8. It should be noted that MSFs obtained using the curve corresponding to a higher  $(N_1)_{50}$  would be identical.

The "field" liquefaction relationship and the MSFs derived previously assume that the distances to the remote liquefaction sites were accurately documented. They also assume that the liquefaction phenomena occurred at sites with level ground and static ground water such that the conditions shown in Fig. 3 are truly representative. Neither of these assumptions can be documented to everybody's satisfaction. However, the following discussion shows that, in fact, the MSFs are independent of acceleration, and are only dependent on the equivalent uniform number of stress cycles selected to represent different earthquake magnitudes.

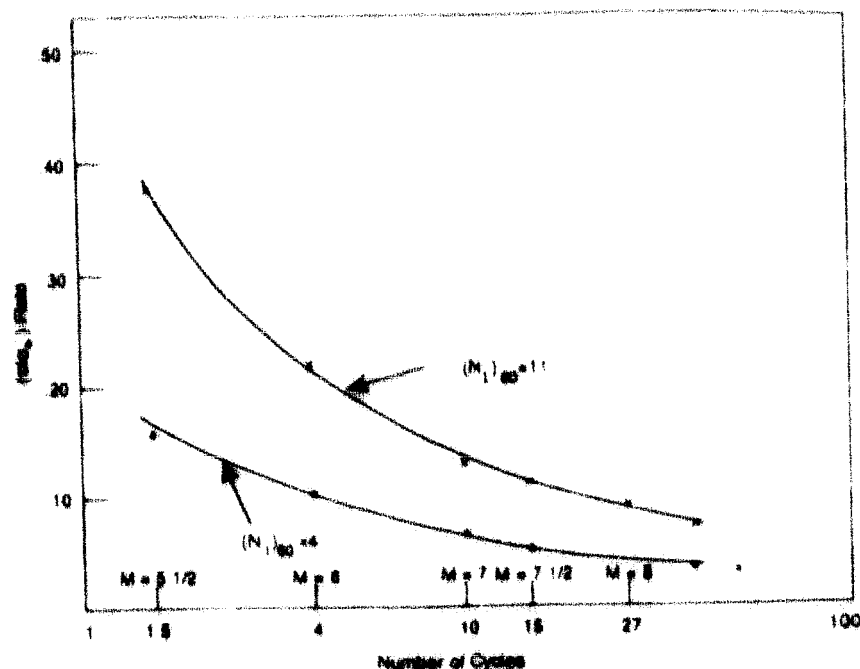


FIG. 5. Magnitude Scaling Factors Derived from Energy Concepts

**TABLE 7. Ground Accelerations and Corresponding Induced Cyclic Stress Ratios at Distant Liquefaction Sites**

Earthquake magnitude $M$ (1)	Equivalent uniform number of cycles $N_u$ (2)	Acceleration $A$ (g) (3)	Induced cyclic stress ratio $\frac{\tau}{\sigma'_v}$ (4)
8.25	38.4	0.023	0.033
8	26.7	0.03	0.039
7.5	15	0.04	0.052
7	9.6	0.05	0.065
6.5	—	0.06	0.078
6	3.8	0.08	0.104
5.5	1.7	0.12	0.156

**TABLE 8. Magnitude Scaling Factors Derived in this Study Based on Consideration of Distant Liquefaction Sites**

Earthquake magnitude (1)	Equivalent uniform number of cycles (2)	Magnitude scaling factor (3)
8.25	38.4	0.63
8	26.7	0.75
7.5	15.0	1.00
7	9.6	1.25
6	3.8	2.00
5.5	1.7	3.00

Consider a well characterized site for which the  $(N_1)_{50}$  value is well documented, and which liquefied under a magnitude 7.5 earthquake inducing an also well-documented ground surface acceleration equal to  $A_g = a$ . Other sites with the same geotechnical characteristics would liquefy nearby or at other locations during different earthquake magnitudes,  $M$ , if the energy absorbed from the new events equals or exceeds that corresponding to that during the magnitude 7.5 event. Based on the energy approach, the energy causing liquefaction in the magnitude 7.5 event is

$$(TE)_{M=7.5} \cdot N_{M=7.5} = (A_{M=7.5})^2 \cdot N_{M=7.5} \quad (10)$$

where  $N_{M=7.5} = 15$ . The energy required under other events is

$$(TE)_{M=M} \cdot N_{M=M} = (A_{M=M})^2 \cdot N_{M=M} \quad (11)$$

From (10) and (11), one obtains

$$A_{M=M} = (A_{M=7.5}) \times \left( \frac{15}{N_M} \right)^{1/2} \quad (12)$$

The cyclic stress ratios inducing liquefaction in the field are therefore

$$\left( \frac{\tau}{\sigma'_v} \right)_{M=M} = A_{M=M} = (A_{M=7.5}) \times \left( \frac{15}{N_M} \right)^{1/2} \quad (13)$$

and the MSF would be expressed as

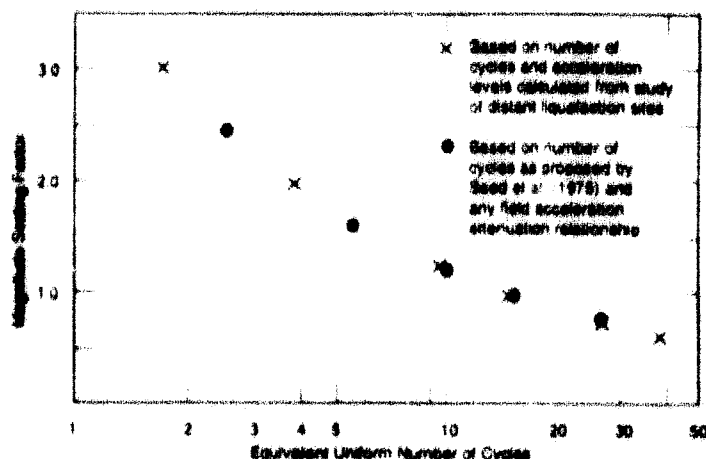
$$MSF = \frac{(\tau/\sigma)_{M=M}}{(\tau/\sigma)_{M=7.5}} = \left( \frac{15}{N_M} \right)^{1/2} \quad (14)$$

It may be seen that the MSFs are independent of the acceleration level and only depend on the relationship between magnitude and equivalent uniform number of cycles.

It is then possible, for example, to use the magnitude and number of significant cycles derived by Seed et al. (1975), adopted by Seed and Idriss in 1982 (Table 1), and based on energy concepts [(14)], to derive the MSFs shown in Table 9. MSFs based on energy concepts derived from the field strength relationship (Table 6) and from Seed et al.'s 1975 equivalent uniform number of cycles (Table 9) are compared in Fig. 6.

**TABLE 9. Magnitude Scaling Factors Derived in this Study Based on Equivalent Uniform Number of Cycles Proposed by Seed et al. (1975)**

Earthquake magnitude (1)	Equivalent uniform number of cycles (2)	Magnitude scaling factors (3)
8.5	26	0.76
7.5	15	1.0
6.75	10	1.22
6	5-6	1.65
5.25	2-3	2.45



**FIG. 6. Cyclic Stress Ratio Inducing Liquefaction in Field**

This figure shows that the scaling factors are identical. That is, the scaling factors derived from the consideration of the equivalent uniform number of cycles versus minimum acceleration levels at distant liquefaction sites duplicate the answers calculated with any other set of acceleration attenuation relationships provided that those relationships are associated with the magnitude versus equivalent uniform number of cycles (Table 1) proposed by Seed et al. (1975).

## MAGNITUDE SCALING FACTORS COMPARED

Fig. 7 compares MSFs developed by Seed and Idriss (1982), Ambraseys (1988), Williams (1994), and the present study. While Seed and Idriss used the magnitude versus significant number of cycles proposed earlier by Seed et al. (1975), the authors then proceeded to obtain cyclic strengths based entirely on laboratory simple shear test results with data extrapolated to the region of lower number of cycles, representative of earthquake magnitudes less than about 6.25. The difference between the results based on laboratory data (Seed and Idriss) and field liquefaction data is evident. The very close agreement between the results obtained by Ambraseys and by this investigation is also evident. Ambraseys' results were obtained by regression of liquefaction field data from many earthquakes. The results of this investigation are based on the application of energy concepts to the field liquefaction data from many magnitudes at very remote sites.

Because the approaches used by Ambraseys and Williams are both statistical regressions, a closer agreement between their conclusions would have been expected. Williams' scaling factors for earthquake magnitudes less than 6 diverge considerably from the values derived by the other two field-based studies. For earthquakes below magnitude 6, Williams utilized the database summarized in Table 10. The scarcity of liquefaction data from earthquakes of magnitude 5.9 or less is clearly seen in the tabulation. In contrast, the lowest magnitude range studied by Ambraseys was 6.0-6.6 for which data are available for 10 cases of liquefaction, and 20 cases of no liquefaction.

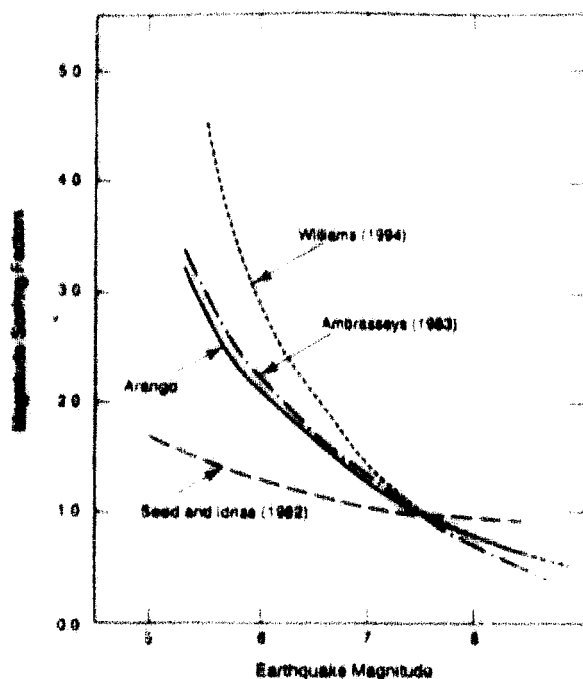


FIG. 7. Comparison of Earthquake Magnitude Sealing Factors from Various Sources

TABLE 10. Liquefaction Database for Magnitudes Less Than 6 in Williams' Study

Year (1)	Magnitude (2)	Location (3)	Sites with	
			Liquefaction (4)	No liquefaction (5)
1937	5.3	Daly City	1	29
1965	4.9	San Francisco	0	1
1969	5.7	Santa Rosa	0	2
1980	5.5	Chibachenchu	0	1
1987	5.9	Whittier Narrows	0	23

MSFs derived on this basis are likely to be more reliable than those derived based on the data summarized in Table 10. The scarcity of data may be responsible for the divergence of Williams' results relative to Ambraseys' curve shown in Fig. 7.

## CONCLUSIONS AND RECOMMENDATION

The following conclusions are derived from the studies summarized in this paper.

MSFs to be used in liquefaction potential evaluations have been derived in the past based on laboratory test results (Seed and Idriss 1982) or on the regression of field liquefaction data (Ambraseys 1988; Williams 1994). MSFs derived based on laboratory results are lower than those derived from field data for earthquake magnitudes less than 7.5 and higher for magnitudes beyond 7.5.

Seed's and Idriss' factors were based on laboratory simple shear test data extrapolated to the number of cycles to liquefaction representative of lower magnitude earthquakes, less than about magnitude 6.25.

Ambraseys' and Williams' factors differ considerably from each other for magnitudes less than about 6.0. A review of the data used by both investigators shows that for the lower magnitude earthquakes, the data in Ambraseys' regression analysis represent a more balanced statistical base (10 cases of liquefaction and 20 cases of no liquefaction), while Williams' data consist of only one case of liquefaction and 54 cases of no liquefaction. Thus, Ambraseys' results are supported by a more credible backlog of field performance, and are likely to be more reliable.

The total energy approach applied in this study to the case histories of liquefaction at the farthest distances from the earthquake centers of energy release gives MSFs that are very close to the factors derived by Ambraseys.

In the energy approach, a field liquefaction resistance curve was developed based on the average of the acceleration levels estimated by eight attenuation relationships. From this curve, MSFs were derived. This paper shows that the factors are actually independent of the field acceleration, and are only dependent on the earthquake magnitude-equivalent uniform number of cycles relationship. The paper also shows that in the energy approach, the use of the data from the farthest liquefaction sites, or of the magnitude-equivalent uniform number of cycles relationship proposed by Seed et al. (1975) and any field acceleration attenuation model, results in the same MSFs.

It is recommended that for the purposes of evaluating liquefaction potential, the commonly used earthquake MSFs introduced by Seed and Idriss (1982) be replaced by those developed in this study based on energy principles. The proposed factors are shown in Fig. 6.

## ACKNOWLEDGMENTS

This study was supported by a technical grant by Bechtel Corporation. The writer thanks the reviewers and Dr. Farhang Ostadan, Bechtel National, for suggestions to improve the paper. Ms. Lynda Preston and Ms. Marjorie Quero edited and typed the manuscript, respectively.

## APPENDIX I. REFERENCES

- Ambraseys, N. N. (1988). "Engineering seismology." *Earthquake Engrg. and Struct. Dynamics*, 17(1), 1-105.
- Arango, I. (1994). "Methodology for liquefaction potential evaluation of sites east of the Rockies." *Bechtel-Sponsored Tech. Grant*, Bechtel Technical Publications, San Francisco, Calif.
- Babus, J. (1981). "Regional modification of acceleration functions." *Bull., Seismological Soc. of Am.*, 71(4), 1309-1321.
- Carter, D. P., and Seed, H. B. (1988). "Liquefaction potential of sand deposits under low levels of excitation." *Rep. No. UCB/EERC-88/11*, Coll. of Engrg., Univ. of California, Berkeley, 119.
- Cornell, C. A., and Shakal, A. F. (1979). "Seismic motion and response prediction alternatives." *J. Earthquake Engrg. and Struct. Dynamics*, 7(4), 295-315.
- Davis, R. O., and Berni, J. B. (1983). "Comparison of a liquefaction theory with field observations." *Geotechnique*, London, U.K., 33(4), 455-460.
- DeAlba, P., Seed, H. B., and Chan, C. K. (1976). "Sand liquefaction in large-scale simple shear tests." *J. Geotech. Engrg. Div., ASCE*, 102(9), 909-927.
- Donovan, N. C., and Bornstein, A. E. (1974). "Uncertainties in seismic risk procedures." *J. Geotech. Engrg. Div., ASCE*, 104(7), 869-887.
- Figueras, J. L., Sada, A. S., Liang, L., and Dahisara, N. M. (1994). "Evaluation of soil liquefaction by energy principles." *J. Geotech. Engrg., ASCE*, 120(9), 1554-1569.
- Gibbs, H. J., and Holtz, W. G. (1957). "Research on determining the density of sands by spoon penetration testing." *Proc., Fourth Int. Conf. on Soil Mech. and Found. Engrg.*, Butterworths, London, England.
- Hasegawa, H. S., Basham, P. W., and Berry, M. J. (1981). "Attenuation relations for strong seismic ground motion." *Bull., Seismological Soc. of Am.*, 71(6), 2071-2095.
- Hudson, D. E. (1979). "Reading and interpreting strong motion accelerograms." *Engineering Monographs on Earthquake Criteria, Structural Design, and Strong Motion Records*, Vol. 1, Earthquake Engrg. Res. Inst., Univ. of California, Berkeley.
- Idriss, I. M. (1991). "Earthquake ground motions at soft soil sites." *Sec. and Int. Conf. on Recent Adv. in Geotech. Earthquake Engrg. and Soil Dynamics*, S. Prakash, ed., Rolla, Mo.
- Idriss, I. M. (1993). "Procedures for selecting earthquake ground motions at rock sites." *Rep. to the Nat. Inst. of Standards and Technol.*, U.S. Dept. of Commerce, Washington, D.C.
- Iwasaki, T., Katayama, T., Kawashima, K., and Saka, M. (1978). "Statistical analysis of strong motion acceleration records obtained in Japan." *Proc., Second Int. Conf. on Microzonation*, Mehmet A. Sherif, San Francisco, Calif., Vol. 2, 705-716.
- Kurihbayashi, E., and Tanioka, F. (1975). "Brief review of liquefaction during earthquakes in Japan." *Soils and Found.*, 15(4), 81-92.

- McGuire, R. K. (1977). "Seismic design spectra and mapping procedures using hazard analyses based directly on oscillator response." *J. Earthquake Engrg. and Struct. Dynamics*, 5(3), 211-234.
- McGuire, R. K. (1978). "Seismic ground motion parameter relations." *J. Geotech. Engrg. Div., ASCE*, 104(4), 481-490.
- Orphal, D. L., and Lahoud, J. A. (1974). "Prediction of peak ground motion from earthquakes." *Bull., Seismological Soc. of Am.*, 64, 1563-1574.
- Seed, H. B. (1979). "Soil liquefaction and cyclic mobility evaluation for level ground during earthquakes." *J. Geotech. Engrg. Div., ASCE*, 105(2), 201-255.
- Seed, H. B., Arango, I., and Chan, C. K. (1975). "Evaluation of soil liquefaction potential during earthquakes." *Rep. EERC 75-28, Earthquake Engrg. Res. Ctr., Univ. of California, Berkeley*.
- Seed, H. B., and Idriss, I. M. (1982). "Ground motions and soil liquefaction during earthquakes." *Monograph Ser., Earthquake Engrg. Res. Inst., Univ. of California, Berkeley*.
- Seed, H. B., Idriss, I. M., and Arango, I. (1983). "Evaluation of liquefaction potential using field performance data." *J. Geotech. Engrg. Div., ASCE*, 109(3), 458-482.
- Seed, H. B., Idriss, I. M., and Kiefer, F. W. (1968). "Characteristics of rock motions during earthquakes." *Rep. No. EERC 68-5, Coll. of Engrg., Univ. of California, Berkeley*.
- Seed, H. B., Idriss, I. M., Makdisi, F., and Banerjee, N. (1975). "Representation of irregular time histories by equivalent uniform stress series in liquefaction analyses." *Rep. No. EERC 75-29, Earthquake Engrg. Res. Ctr., Univ. of California, Berkeley*.
- Seed, H. B., and Peacock, W. H. (1970). "Applicability of laboratory test procedures for measuring soil liquefaction characteristics under cyclic loading." *Rep. No. EERC 70-8, Coll. of Engrg., Univ. of California, Berkeley*.
- Seed, H. B., Tokimatsu, K., Harder, L. F., and Chung, R. M. (1984). "The influence of SPT procedures in soil liquefaction resistance evaluations." *Rep. No. UCB/EERC-84/15, Coll. of Engrg., Univ. of California, Berkeley*.
- Williams, T. (1994). "Magnitude scaling factors for analysis of liquefaction hazard." PhD presentation, Dept. of Civ. Engrg., Brigham Young Univ., Provo, Utah.
- Youd, T. L. (1977). "Discussion of 'Brief review of liquefaction during

earthquakes in Japan,' by E. Kuribayashi and F. Tatsuoka." *Soils and Found.*, 17(1), 82-85.

## APPENDIX II. NOTATION

The following symbols are used in this paper:

- $A$  = acceleration in  $g$ s;  
 $A_w$  = acceleration under earthquake  $M_w$ ;  
 $a, a(t)$  = acceleration in  $\text{cm/s}^2$ ;  
 $G, G_{\text{MAX}}$  = shear modulus; maximum shear modulus;  
 $g$  = acceleration of gravity;  
 $k$  = spring constant in simple oscillator;  
 $M, M_i$  = earthquake magnitudes;  
 $m$  = mass in simple oscillator;  
 $N$  = SPT-blowcount;  
 $N_1$  = blow count normalized to 1 ksc;  
 $(N_1)_{60}$  = normalized blow count corrected for 60% energy ratio;  
 $N_w, N_w$  = equivalent number of uniform cycles of loading in earthquakes of various magnitudes;  
 $r_d$  = parameter reflecting response of nonrigid soil columns;  
 $T$  = fundamental period of vibration;  
 $V$  = volume;  
 $v, V_s$  = velocity, shear wave velocity;  
 $W$  = weight;  
 $y, \dot{y}, \ddot{y}$  = displacement, velocity, and acceleration of simple oscillator, respectively;  
 $z$  = base acceleration in simple oscillator;  
 $\gamma$  = shear strain;  
 $\gamma_r$  = total unit weight of soil;  
 $\gamma_w$  = unit weight of water;  
 $\rho$  = weight per unit volume;  
 $\sigma_v$  = mean effective stress; and  
 $\omega_n$  = natural frequency of harmonic motion =  $(k/m)^{1/2}$ .

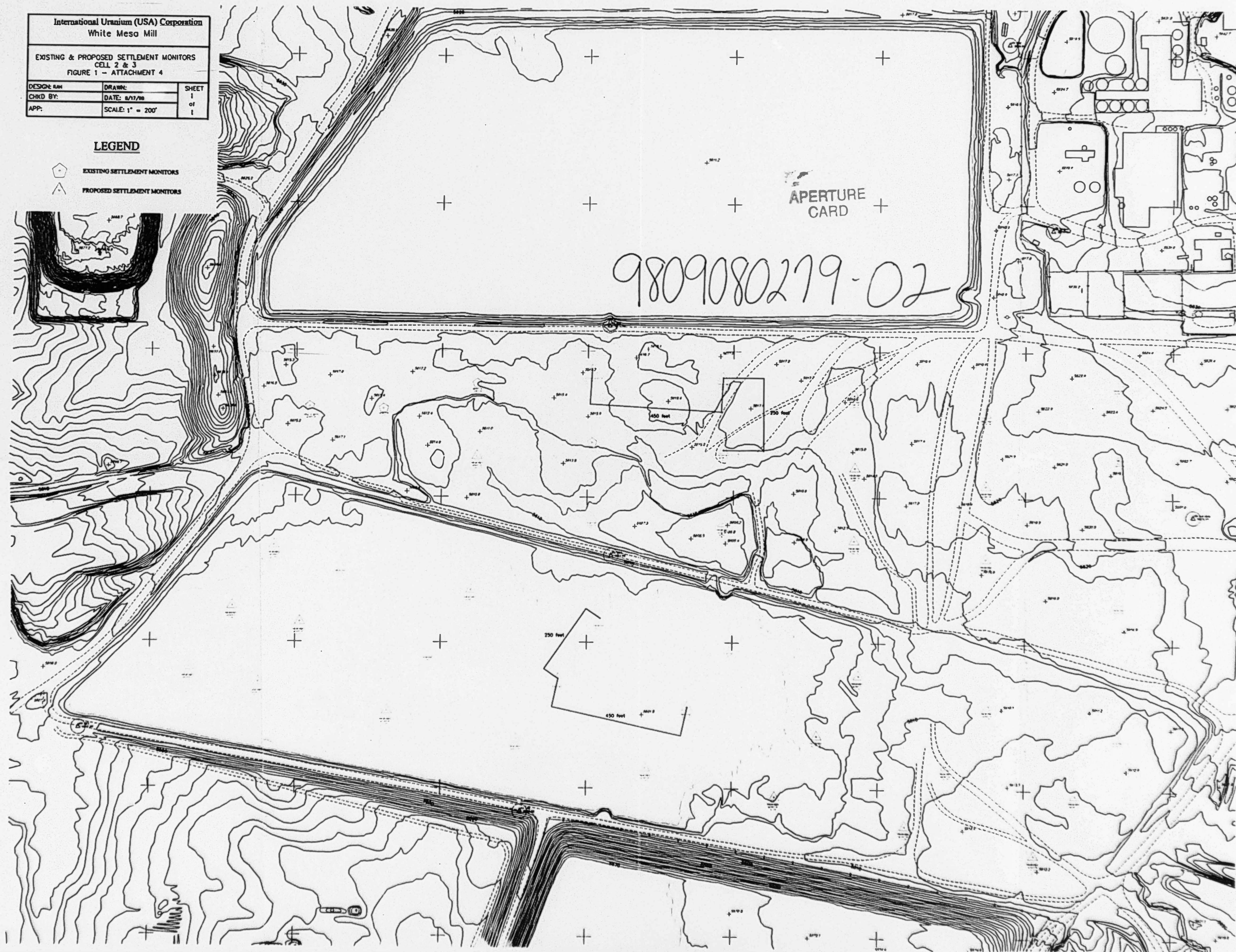
**ATTACHMENT 4**



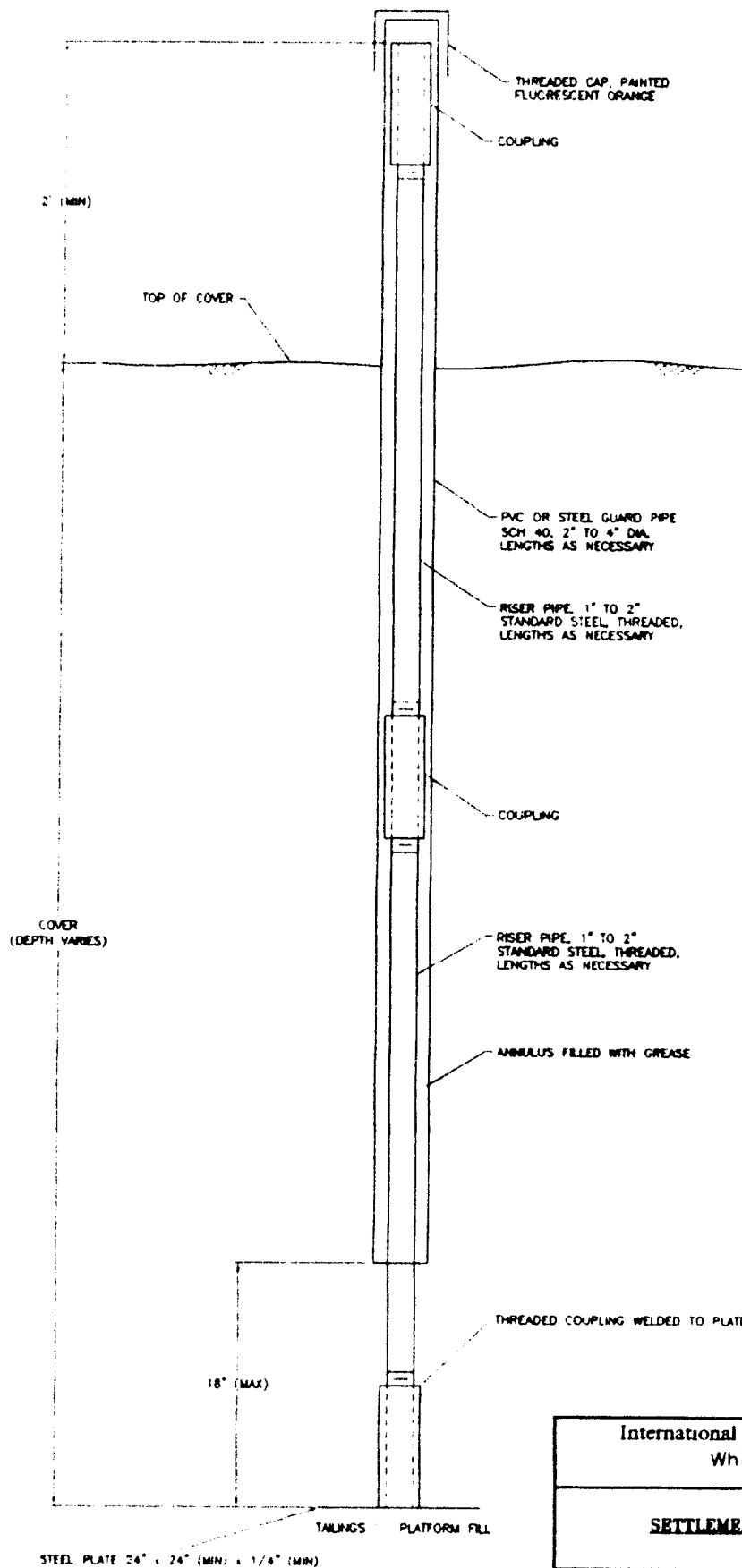
International Uranium (USA) Corporation White Meso Mill		
EXISTING & PROPOSED SETTLEMENT MONITORS CELL 2 & 3 FIGURE 1 - ATTACHMENT 4		
DESIGN: RWH	DRAWN:	SHEET 1 of 1
CHKD BY:	DATE: 8/17/88	
APP:	SCALE: 1" = 200'	

**LEGEND**

- EXISTING SETTLEMENT MONITORS  
PROPOSED SETTLEMENT MONITORS







International Uranium (USA) Corporation  
White Mesa Mill

**SETTLEMENT MONITORING POINT**

FIGURE 2  
ATTACHMENT 4

DESIGN: A. KUMH	DRAWN: K.P.	SHEET 1 of 1
CHKD BY:	DATE: 8/17/98	
APP: RAH	SCALE: NTS	

# ATTACHMENT 4 SETTLEMENT DATA

## ATTACHMENT 4 SETTLEMENT DATA Cell 2 East

DATE	ELEVATION
08/18/89	5624.61
08/25/89	5624.60
09/08/89	5624.59
09/29/89	5624.56
10/20/89	5624.93
11/21/89	5624.49
11/30/89	5624.49
01/11/90	5624.47
01/25/90	5624.48
02/22/90	5624.44
03/18/90	5624.43
04/05/90	5624.82
05/03/90	5624.41
06/07/90	5624.40
07/28/90	5624.38
08/10/90	5624.40
09/14/90	5624.38
10/12/90	5624.36
11/15/90	5624.36
12/14/90	5624.35
01/24/91	5624.35
02/22/91	5624.33
03/21/91	5624.33
04/25/91	5624.33
05/23/91	5624.33
06/21/91	5624.30
07/26/91	5624.30
08/22/91	5624.30
09/27/91	5624.31
10/31/91	5624.31
11/21/91	5624.31
01/02/92	5624.32
01/31/92	5624.3
02/28/92	5624.29
03/25/92	5624.28
04/24/92	5624.29
05/04/92	5624.30
05/07/92	5624.29
05/13/92	5624.29
05/20/92	5624.29
05/28/92	5624.29
06/05/92	5624.29
06/11/92	5624.29
06/18/92	5624.28
06/26/92	5624.29
07/02/92	5624.31
07/09/92	5624.26
07/16/92	5624.28
07/23/92	5624.27
07/28/92	5624.27
08/06/92	5624.29
08/12/92	5624.29
08/20/92	5624.27
08/27/92	5624.3
09/02/92	5624.29
09/09/92	5623.89
09/17/92	5624.27
09/24/92	5624.06
10/03/92	5624.24
10/10/92	5624.27
10/18/92	5624.27
10/22/92	5624.49

## Cell 2 West 1

DATE	ELEVATION
09/27/91	5619.96
10/31/91	5619.83
11/21/91	5619.79
01/02/92	5619.76
01/31/92	5619.73
02/28/92	5619.72
03/25/92	5619.68
04/24/92	5619.69
05/04/92	5619.66
05/07/92	5619.68
05/15/92	5619.68
05/20/92	5619.66
05/28/92	5619.67
06/05/92	5619.68
06/11/92	5619.67
06/18/92	5619.67
06/26/92	5619.67
07/02/92	5619.69
07/09/92	5619.65
07/16/92	5619.67
07/23/92	5619.66
07/28/92	5619.70
08/06/92	5619.67
08/12/92	5619.68
08/20/92	5619.68
08/27/92	5619.70
09/02/92	5619.69
09/09/92	5619.68
09/16/92	5619.47
09/24/92	5619.65
10/01/92	5619.46
10/08/92	5619.61
10/15/92	5619.47
10/21/92	5619.59
10/28/92	5619.59
11/04/92	5619.59
11/11/92	
11/18/92	5619.57

## Cell 2 West 2

DATE	ELEVATION
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## Cell 2 West 3

DATE	ELEVATION
------	-----------

## Cell 2 West 4

DATE	ELEVATION
------	-----------

# ATTACHMENT 4 SETTLEMENT DATA

ATTACHMENT 4 SETTLEMENT DATA  
Cell 2 East

DATE	ELEVATION	DATE	ELEVATION
10/25/92	5624.29	11/25/92	5619.80
11/26/92	5624.30	12/03/92	5619.56
12/03/92	5624.25	12/10/92	5619.55
12/10/92	5624.27	12/16/92	5619.54
12/25/92	5624.30	12/21/92	5619.71
01/01/93	5624.27	12/31/92	5619.52
01/06/93	5624.27	01/07/93	5619.45
01/11/93	5624.33	01/14/93	
01/21/93	5624.30	01/21/93	
01/28/93	5624.26	01/28/93	
02/04/93	5624.27	02/04/93	5619.45
02/11/93	5624.28	02/11/93	5619.46
02/18/93	5624.27	02/18/93	
02/25/93	5624.27	02/25/93	
03/04/93	5624.27	03/04/93	5619.47
03/11/93	5624.27	03/11/93	5619.15
03/18/93	5624.29	03/18/93	5619.47
03/24/93	5624.26	03/25/93	5618.75
03/31/93	5624.27	04/01/93	5619.27
04/08/93	5624.27	04/08/93	5619.46
04/15/93	5624.28	04/15/93	5619.45
04/22/93	5624.29	04/22/93	
05/30/93	5624.28	04/29/93	5619.47
06/30/93	5624.28	05/30/93	5619.47
07/29/93	5624.28	07/29/93	5619.44
08/26/93	5624.27	08/26/93	5619.45
09/30/93	5624.27	09/30/93	5619.42
10/27/93	5624.26	10/27/93	5619.4
11/22/93	5624.27	11/22/93	5619.41
12/28/93	5624.23	12/28/93	5619.43
01/27/94	5624.23	01/27/94	5619.4
02/24/94	5624.27	02/24/94	5619.35
03/31/94	5624.22	03/31/94	5619.36
05/05/94	5624.25	05/05/94	5619.33
06/06/94	5624.28	06/06/94	5619.32
07/29/94	5624.29	07/29/94	5619.31
08/29/94	5624.29	08/29/94	5619.31
10/28/94	5624.28	10/28/94	5619.3
12/22/94	5624.28	12/22/94	5619.28
01/27/95	5624.27	01/27/95	5619.29
02/23/95	5624.07	02/23/95	5619.27
03/31/95	5624.05	03/31/95	5619.26
04/28/95	5624.05	04/28/95	5619.26
06/02/95	5624.04	06/02/95	5619.29
07/31/95	5624.06	07/31/95	5619.29
09/01/95	5624.08	09/01/95	5619.25
09/29/95	5624.07	09/29/95	5619.23
10/19/95	5624.15	10/19/95	5619.27
11/30/95	5624.80	11/30/95	5619.27
12/21/95	5624.80	12/21/95	5618.99
01/26/96	5624.05	01/26/96	5619.27
03/01/96	5624.03	03/01/96	5619.27
04/25/96	5624.05	04/25/96	5618.89
05/30/96	5624.05	05/30/96	5619.23
06/27/96	5624.05	06/27/96	5619.26
07/27/96	5624.05	07/27/96	5619.27
08/30/96	5624.05	08/30/96	5619.26
09/27/96	5624.02	09/27/96	5619.29
10/26/96	5624.03	10/26/96	5619.24
11/27/96	5624.04	11/27/96	5619.25
12/20/96	5624.04	12/20/96	5619.28
01/31/97	5624.04	01/31/97	5619.31
02/27/97	5624.06	02/27/97	5619.3
03/27/97	5623.98	03/27/97	5619.23
04/25/97	5624.02	04/25/97	5619.2
05/30/97	5624.01	05/30/97	5619.22
06/26/97	5624.00	06/26/97	5619.22
07/31/97	5624.03	07/31/97	5619.26
08/28/97	5624.01	08/28/97	5619.2
09/25/97	5624.01	09/25/97	5619.18
10/25/97	5624.01	10/25/97	5619.21
11/18/97	5624.02	11/18/97	5619.23
12/18/97	5624.00	12/18/97	5619.23
01/16/98	5624.00	01/16/98	5619.23
02/06/98	5624.03	02/06/98	5619.23
03/06/98	5624.05	03/06/98	5619.22
04/03/98	5623.98	04/03/98	5619.25
05/06/98	5624.00	05/06/98	5619.25
06/01/98	5624.00	06/01/98	5619.21

Cell 2 West 1

Cell 2 West 2

Cell 2 West 3

Cell 2 West 4

Initial  
extension added

Initial

Initial

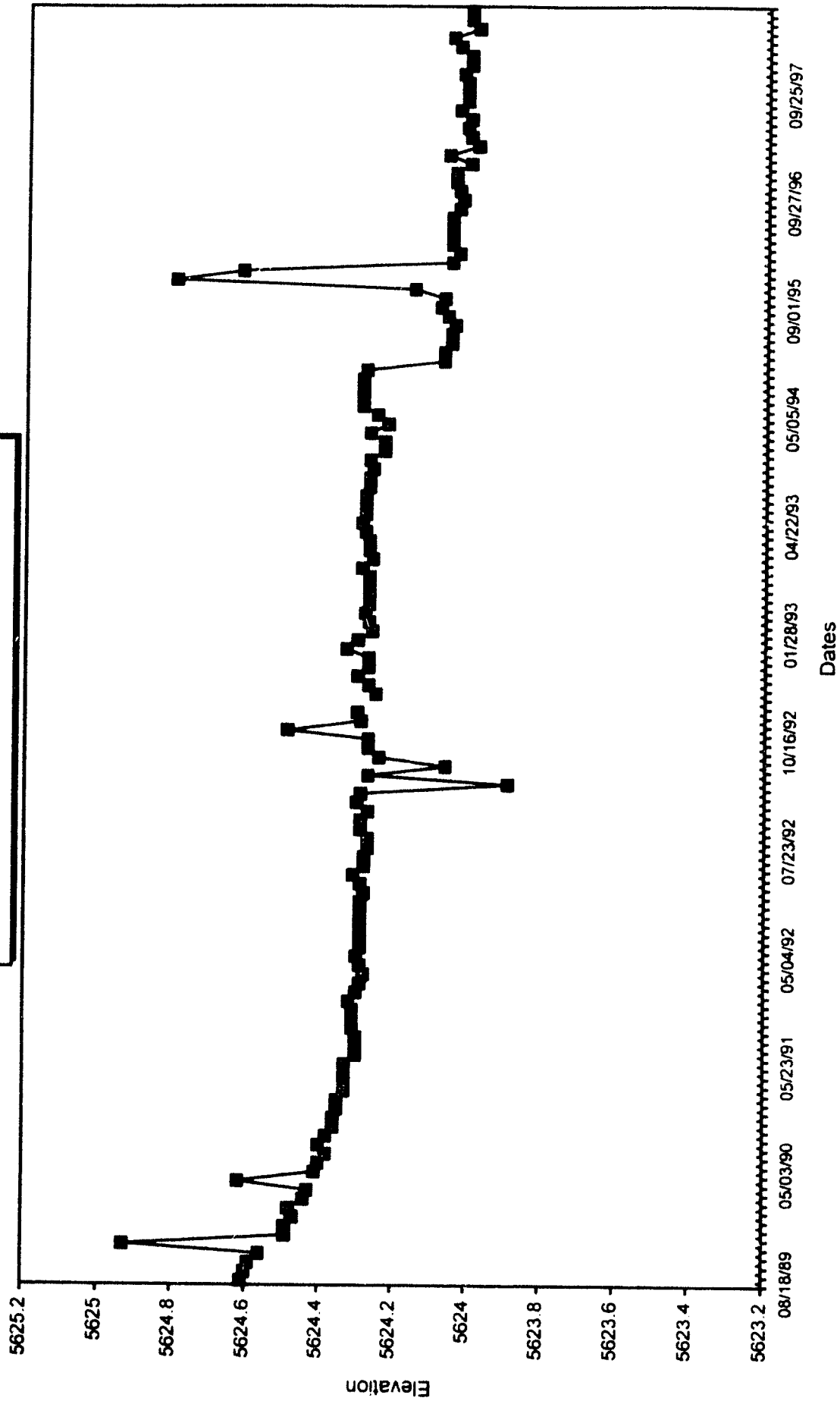
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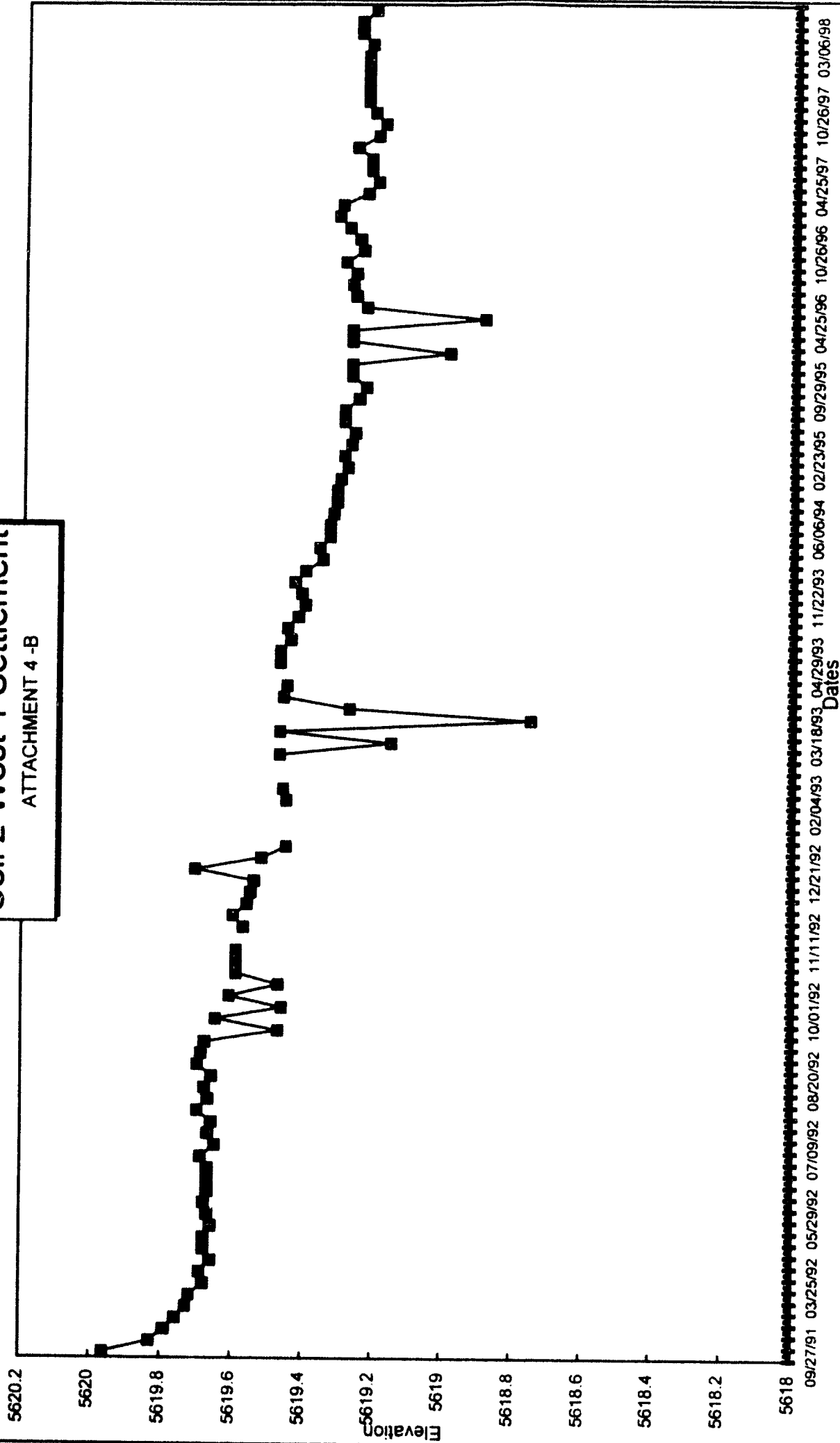
# Cell 2 East Settlement Monitor

ATTACHMENT 4-A



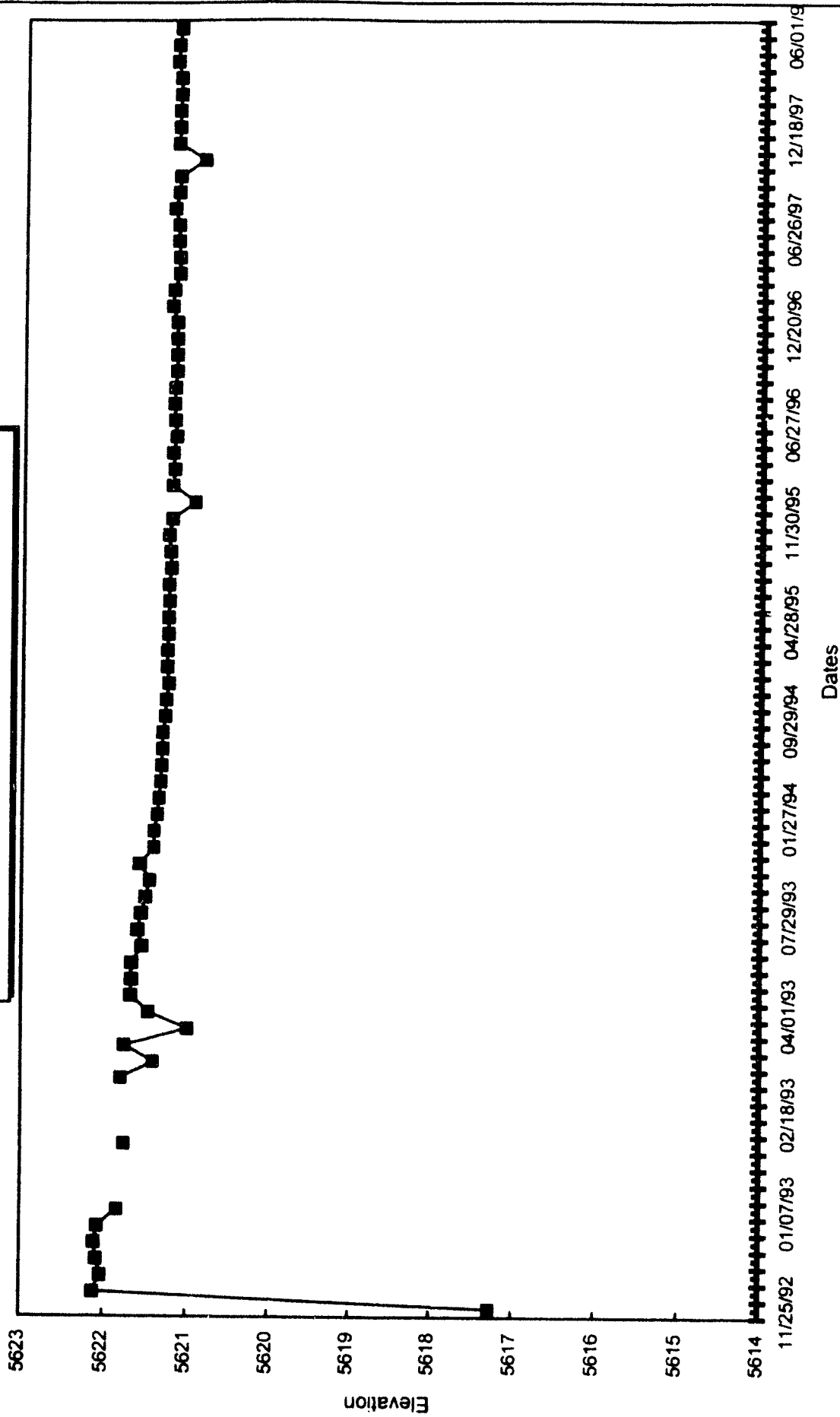
# Cell 2 West 1 Settlement

ATTACHMENT 4 - B



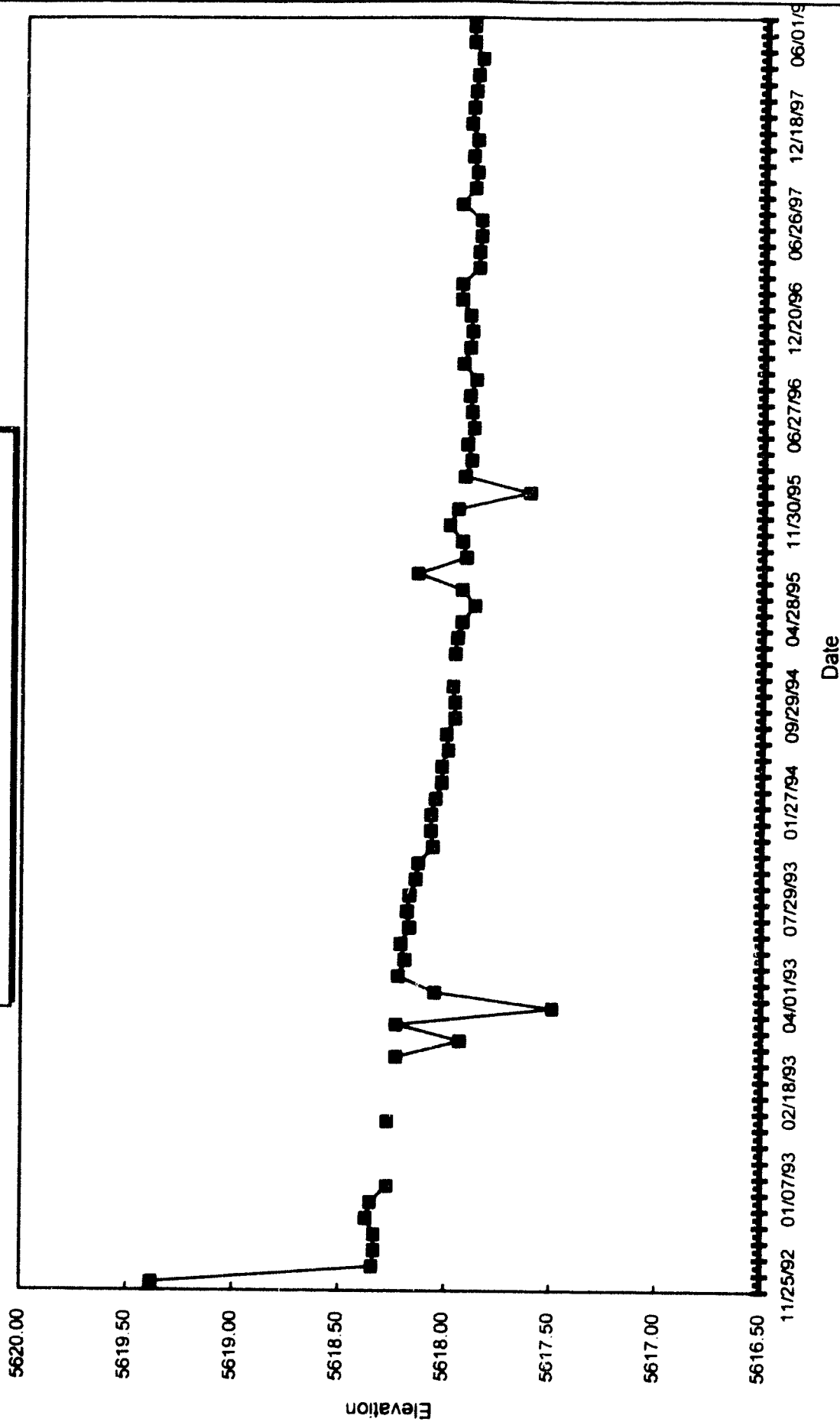
# Cell 2 West 2 Settlement Monitor

ATTACHMENT 4-C

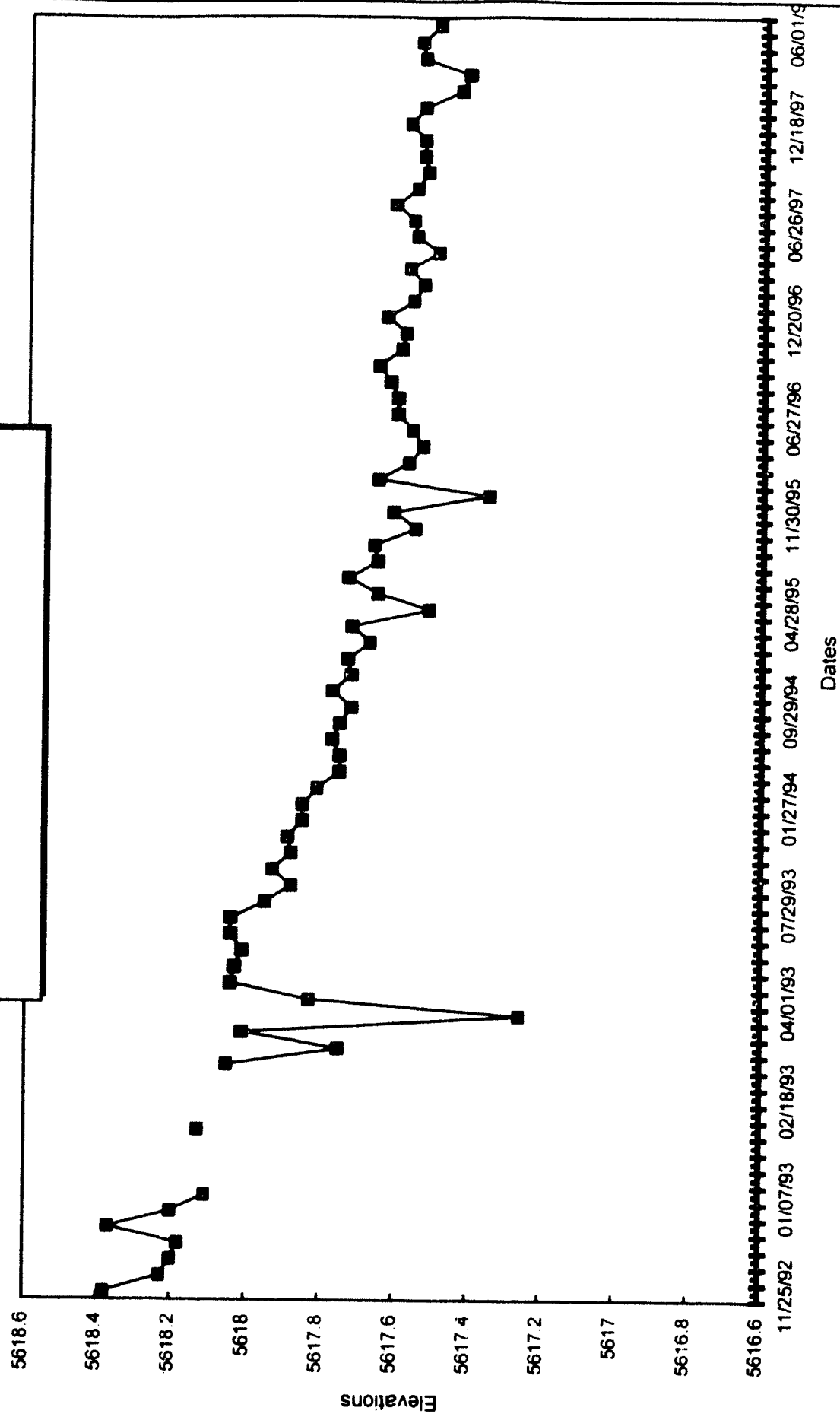


# Cell 2 West 3 Settlement Monitor

ATTACHMENT 4-D



**ATTACHMENT 4-E**





## **ATTACHMENT 5**

TABLE 22

ENERGY FUELS NUCLEAR, INC.  
WHITE MESA MILL  
Soil Sampling Results

(VALUES) x 10E-3  $\mu$ Ci/Kg

Date	BHV-1		BHV-2		BHV-3		BHV-4		BHV-5	
	Ra-226	U-Nat	Ra-226	U-Nat	Ra-226	U-Nat	Ra-226	U-Nat	Ra-226	U-Nat
Sep-80	0.650	0.420	0.340	0.420	0.420	0.420	0.410	12.194	0.230	14.891
Sep-81	0.400	1.800	0.300	0.600	0.300	0.600	0.200	3.000	0.300	0.800
Dec-81	0.780	0.770	0.440	0.560	0.690	0.420	0.750	0.630	0.550	0.420
Jun-82	0.423	0.384	0.412	0.180	0.265	0.207	0.478	0.260	0.449	0.216
May-83	0.471	0.410	0.569	0.550	0.461	0.340	0.643	0.340	0.147	0.140
Jan-84	0.713	0.866	0.616	0.683	0.489	0.471	0.124	0.324	0.132	0.310
Oct-84	2.960	0.886	2.330	0.069	2.880	0.721	3.490	0.804	2.550	0.817
Aug-85	1.630	0.800	2.190	0.424	2.270	0.424	4.330	0.294	1.280	0.577
Aug-86	0.369	0.654	0.466	0.666	0.382	0.694	0.396	0.826	0.728	0.836
Aug-87	0.600	0.800	1.500	0.900	0.800	0.800	1.200	0.700	1.500	1.300
Aug-88	1.500	1.600	1.300	0.700	0.800	0.900	1.000	1.300	3.800	5.000
Aug-89	1.200	1.600	1.100	3.000	0.800	1.500	1.100	1.400	2.900	5.700
Aug-90	2.900	5.800	1.000	1.400	0.900	1.400	1.800	1.300	3.700	3.200
Aug-91	3.900	8.800	1.700	2.600	2.600	5.700	1.800	2.600	2.500	4.400
Aug-92	1.200	2.200	0.900	1.400	0.800	1.200	0.900	0.900	1.100	1.800
Aug-93	2.000	1.700	1.400	1.700	1.100	1.900	0.800	1.600	4.800	3.500
Aug-94	1.000	1.600	0.700	0.800	0.700	0.900	0.700	1.100	3.000	3.800
Aug-95	2.810	4.700	0.680	0.200	0.880	0.650	0.580	0.240	2.800	1.600
Aug-96	1.700	2.150	0.600	0.460	0.300	0.210	0.500	0.520	1.900	2.010
Mean	1.43	2.00	0.98	0.92	0.93	0.98	1.12	1.60	1.81	2.69
Std. Dev.	1.02	2.12	0.60	0.77	0.76	1.19	1.06	2.60	1.39	3.34

## **ATTACHMENT 6**

IUC Cost Estimate  
MILL DECOMMISSIONING  
WIND BLOWN CONTAMINATION

1) Scoping Survey (REUSED CALCULATIONS 8/98)

- Initial Survey will be conducted on an area to be determined But approximated by a perimeter approximately 1000' out side of RESTRICTED AREA BOUNDARIES.

Area By CAD = 38,728 M sq ft  
= INCLUDES, TAILS, mill yard, and Pad.

Area that requires wind blown Survey is:

	Mft <sup>2</sup>
Area of Wind blown Survey	38,728
Less	
Cell 4A	1,909
Cell 3	3,234
Cell 2	2,907
Cell 1	2,576
MILL Yard	1,43
Occ Storage	977
	<hr/>
	25,402 Mft <sup>2</sup>

ESTABLISH GRID -

Assume 10x10 meter grid (Standard NAC/EPN) = 1076 ft<sup>2</sup>  
 Survey crew of 2 Capable of setting 500 grid points/day  
 $25,402,000 \text{ ft}^2 \div 1076 \text{ ft}^2 = 23,600 \text{ grid points}$   
 $23,600 \text{ grid points} \div 500 \text{ grid points/day} = 47.2 \text{ Days}$   
 $\approx 47 \text{ Days Survey}$   
 2 men crew  $2 \times 8 \text{ hrs} \times 47 = \boxed{752 \text{ hrs}}$

Total Grid Survey = 752 man hrs

Scanning Crew Consists of 2 = ~~one~~ one Scanner, one Board Keeper.

### Coverage

Assume: Seafloor Survey Completed by Scanning with MR Neton  
Held Close to ground while traversing at  $\pm 0.5 \text{ m/sec}$  as  
per Guidance in NURAG 5849

$$0.5 \text{ m/sec} \times 60 \text{ sec/min} \times 60 \text{ min/hr} \times 8 \text{ hr/day} = 14,400 \text{ m/day}$$

Assume eff of .8

$$.8 \times 14,400 \text{ m/day} = 11,520 \text{ m/day}$$

Assume 30 m swath for each  $10^\circ \times 10^\circ$  grid to cover 10% of  
surface area (NURAG 5849)

$$\text{Scanning crew surveys (swath)} \quad 11,520 \text{ m/day} \div 30 = 384 \text{ grid/day}$$

Scanning therefore takes:

$$23600 \text{ grids} \div 384 \text{ grids/day} = 61.5 \text{ days} \\ = \boxed{62 \text{ days}}$$

$$62 \text{ days} \times 2 \text{ men} \times 8 \text{ hrs/day} = \boxed{992 \text{ man hrs}}$$

Assume Map Preparation and Data Reduction will take Scanning crew another

20 days to complete

$$20 \text{ days} \times 2 \text{ men} \times 8 \text{ hrs/day} = 320 \text{ man hrs}$$

$$\text{Total Scanning Man hrs} = \boxed{\boxed{1312 \text{ man hrs}}}$$

$$\text{Sample crew cost} = \$13.00/\text{hr}$$

$$\$13.00/\text{hr} \times 1312 \text{ man hrs} = \$17,056^{02}$$

## **ATTACHMENT 7**

ATTACHMENT 7 - RESPONSE TO NRC COMMENTS 7/17/98  
TABLE OF SIX-HOUR LOCAL PMP RAINFALL DEPTH VS DURATION FOR WHITE MESA MIL

6-Hour Storm Rainfall is 10 inches (ref Hydrologic Design Report for White Mesa Mill, 1990)

6/1 Hr Ratio for WHITE MESA is 1.22 (Figure 4.7 and Table 4.4 HMR 49)

ONE-HOUR PMP IS                      8.20 inches at 5000 ft elevation  
   7.95 inches at 5600 ft elevation (1)

DURATION HOURS	% OF 1-HR PMP	RAINFALL DEPTH, IN INCHES, AT AVERAGE ELEVATION OF (based on Table 6.3A HMR 49)			
			5000 ft		5600 ft(1)
0	0		0.00		0.00
0.25	74		6.07		5.88
0.5	89		7.30		7.08
0.75	95		7.79		7.55
1	100		8.20		7.95
2	111		9.10		8.83
3	116		9.51		9.22
4	119		9.75		9.46
5	121		9.92		9.62
6	122		10.00		9.70

Plot of data is adaptation of Figure 12.10, HMR 55A, to site rainfall

(1) Average elevation of site in vicinity of base of cell 4A each tanks

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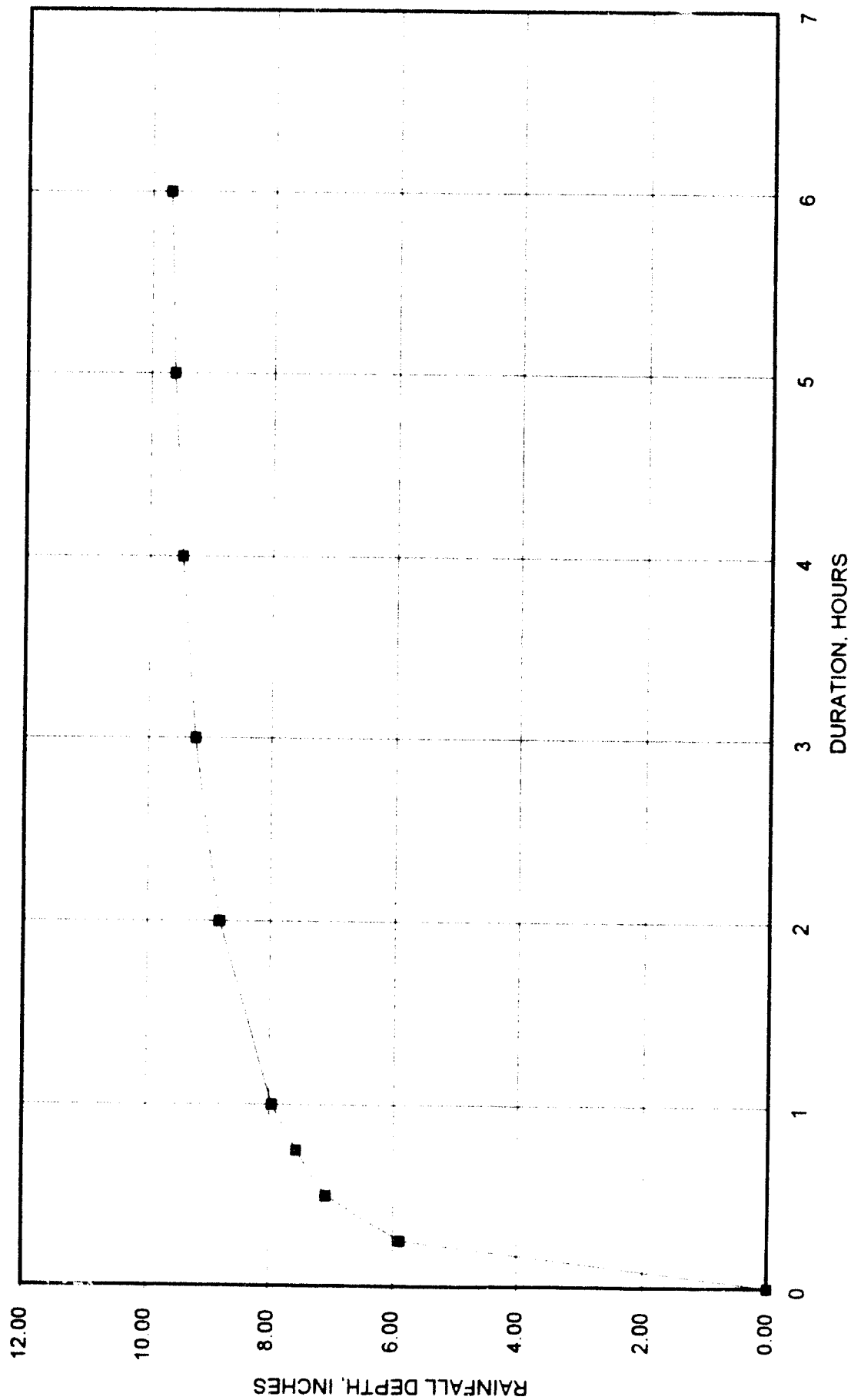
TIME DISTRIBUTION OF FIRST ONE HOUR, OR THE ONE-HOUR PMP  
(after Table 2.1, NUREG CR 4620)

RAINFALL DURATION MINUTES	RAINFALL DURATION HOURS	% OF ONE-HOUR PMP	RAINFALL DEPTH IN INCHES AT ELEVATION	
			5000 ft	5600 ft(1)
0	0	0	0	0
2.5	0.04	27.5	2.25	2.19
5	0.08	45	3.69	3.58
10	0.17	62	5.08	4.93
15	0.25	74	6.07	5.88
20	0.33	82	6.72	6.52
30	0.50	89	7.30	7.08
45	0.75	95	7.79	7.55
60	1.00	100	8.20	7.95

## **ATTACHMENT 8**

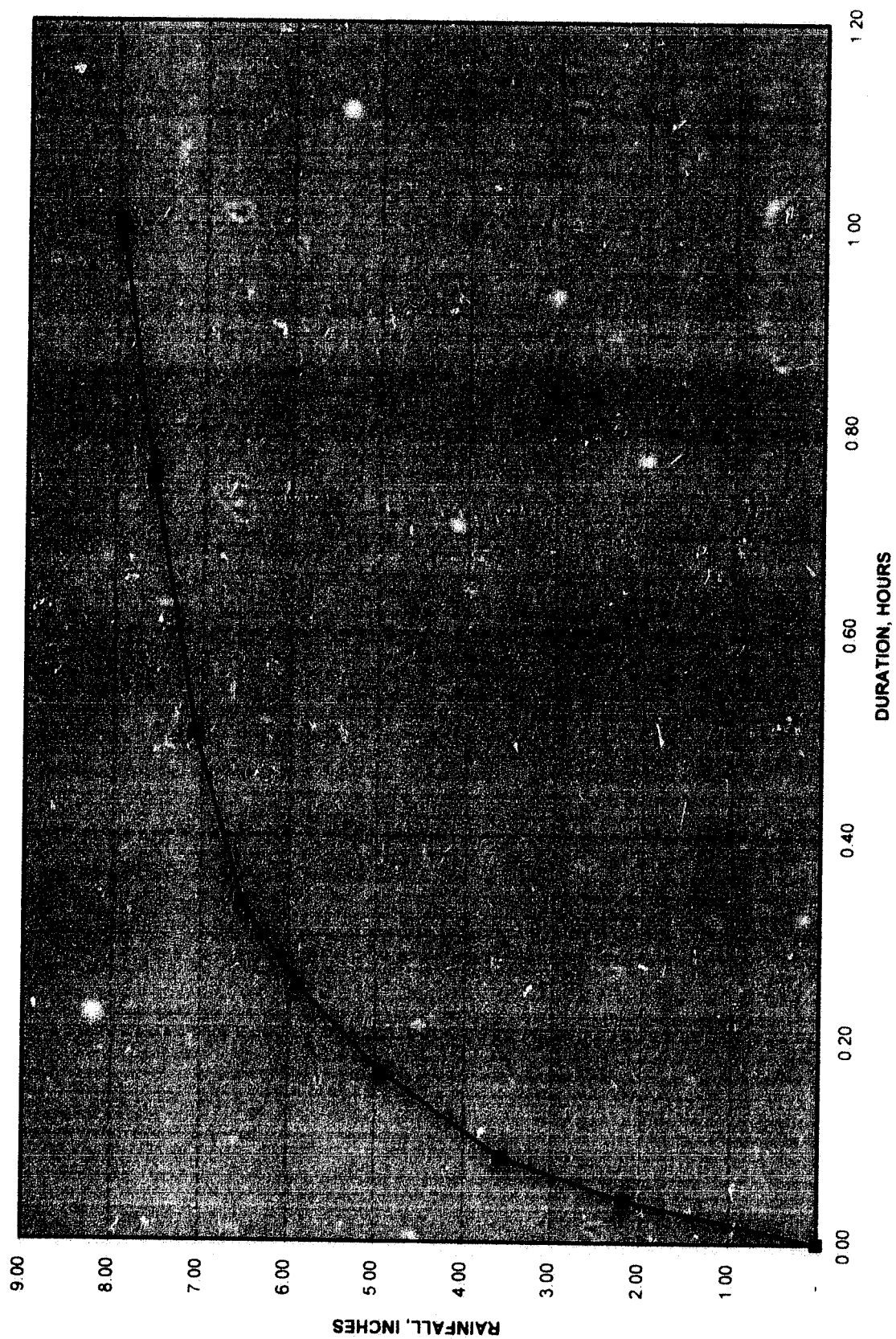


DEPTH VS DURATION FOR 6-HR PMP  
WHITE MESA MILL, UTAH  
ATTACHMENT 8 RESPONSE TO NRC COMMENTS 7/17/98



**ATTACHMENT 9**

RAINFALL-DURATION CURVE FOR ONE-HOUR PMP AT WHITE MESA MILL  
ATTACHMENT 9 - RESPONSE TO NRC COMMENTS 7/17/98



## **ATTACHMENT 10**

# ATTACHMENT 10 RESPONSES TO NRC COMMENTS 7/17/98

## RATIONAL METHOD CALCULATION OF PMF PEAK DISCHARGE, VELOCITY, DEPTH AND SCOUR THROUGH CELL 4A BREACH

FLOW PATH ELEMENT	ELEMENT LENGTH L	MAX ELEV	MIN ELEV	GRADIENT S	SLOPE ANGLE degrees	t <sub>c</sub> hours	RAINFALL WITHIN t <sub>c</sub> (1)	t <sub>r</sub> hr/hr	SURFACE AREA acres	PEAK DISCHARGE Q cfs
CELL 2 COVER	1230	5619.5	5617	0.0020	0.12	0.34	6.53	19.29	74.00	1142
CELL 2/3 BERM	10	5617	5615	0.2000	11.31	0.34	6.54	19.24	1.10	1159
CELL 3 COVER	900	5615	5613.2	0.0020	0.11	0.61	7.30	12.01	39.94	1543
CELL 3/4A BERM	180	5613.2	5577.2	0.2000	11.31	0.62	7.40	11.92	8.17	1621
CELL 4A	1400	5577.2	5562	0.0109	0.62	0.82	7.70	9.42	27.70	1829
CELL 4A INSLOPES	80	5569	5560	0.4875	25.99	0.04	2.00	47.62	5.90	25
CELL 4A BREACH	275	5562	5560	0.0073	0.42	0.92	7.80	8.44	0.38	2057

### FLOW PARAMETERS IN CELL 4A BREACH AT PEAK PMF DISCHARGE

	Breach Bottom Width b ft	Breach Side Slopes	Breach Channel Gradient S ft/ft	Manning Coeff n	Q (cfs) 49' x 5' S	Flow Depth y ft	Cross Section Area of Flow a ft <sup>2</sup>	Hydraulic Radius R ft	ACR 1167	Velocity v fps	Allowable Peak Velocity fps (ACR 1970)	Riprap Size (50 inches max)
Soil (SM) Channel	60	3:1	0.0073	0.03	486	3.38	237.1	2.91	485.31	8.67	2.4	11
Rock Channel	60	3:1	0.0073	0.025	405	3.05	210.9	2.66	406.22	9.79	8-10	
Soil (SM) Channel	80	3:1	0.0073	0.03	486	2.90	257.2	2.62	489.90	8.07	2.4	9.5
Rock Channel	80	3:1	0.0073	0.025	405	2.60	228.3	2.37	406.61	9.05	8-10	

Reference 1 Fig 4.11 NUREG CR 4620

### DEPTH OF SCOUR OF CELL 4A BREACH CHANNEL

All methods used are from Pemberton, E.L. and J.M. Lara, 1984 "Computing Degradation and Local Scour" Technical Guideline for Bureau of Reclamation

ds = depth of scour ft

q = unit discharge cfs/ft

		Soil Channel 60' wide	Soil Channel 80' wide
Method 1	ds = K * q <sup>0.24</sup> K = constant 2.45		
	q =	8.7	8.0
	ds =	4.1	4.0
Method 2	ds = 0.25 dm dm = mean water depth at design discharge =	3.0	2.6
	ds =	0.7	0.7
Method 3	ds = 0.6 * dfo dfo = q <sup>0.686</sup> / Fbo <sup>0.333</sup> = Fbo = zero bed factor = 1.0 ft/s <sup>2</sup> for fine sand	4.22	3.99
	ds =	2.53	2.40
Method 4	ds = 0.25 * dma dma = unit cross section of flow =	3.38	2.9
	ds =	0.845	0.725
Method 5	ds = dm * ((Vm/Vc) - 1) Vm = mean velocity = Vc =	8.68 2	8.00 2
	ds =	9.86	7.92

AVERAGE SCOUR DEPTH ft =

3.62

3.15

**ATTACHMENT 11**

# ATTACHMENT 11 RESPONSES TO NRC COMMENTS 7/17/98

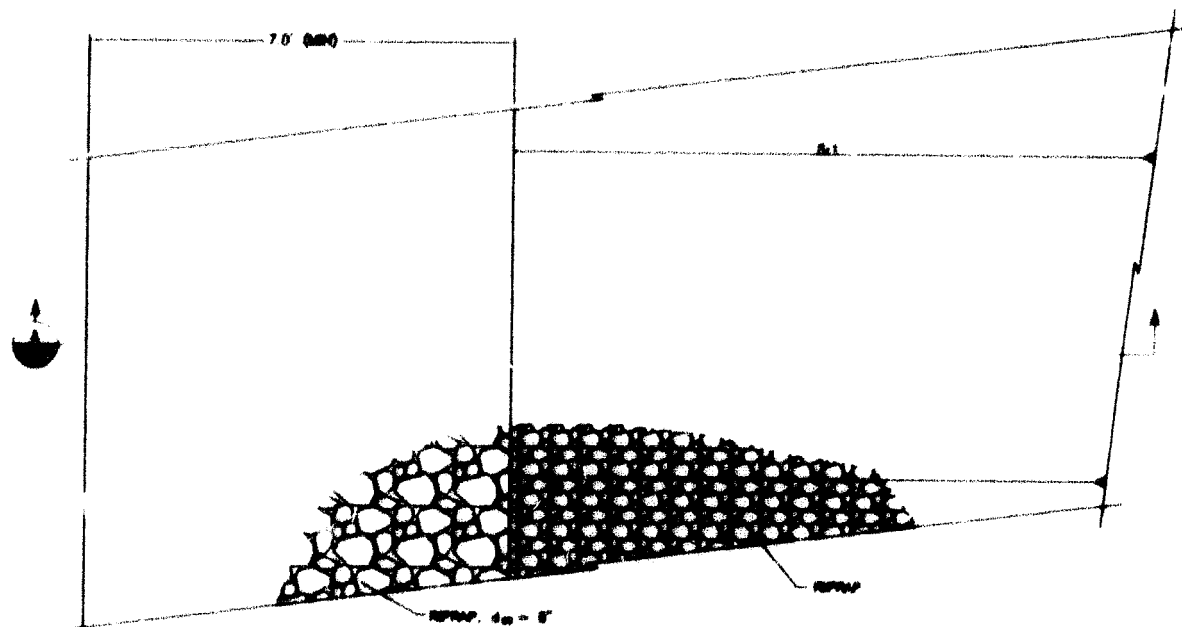
## RATIONAL METHOD CALCULATION OF PMF PEAK DISCHARGE, VELOCITY AND DEPTH THROUGH CELL #1 DISCHARGE CHANNEL

FLOWPATH ELEMENT	ELEMENT LENGTH ft	MAX ELEV	MIN ELEV	GRADIENT %	SLOPE ANGLE degrees	n	RAINFALL WITHIN 60 min in/hr	60 min in/hr	SURFACE AREA acres	PEAK DISCHARGE cfs
LONGEST	4800	5655	5610	0.0094	0.54	0.54	7.20	13.43	143	1344

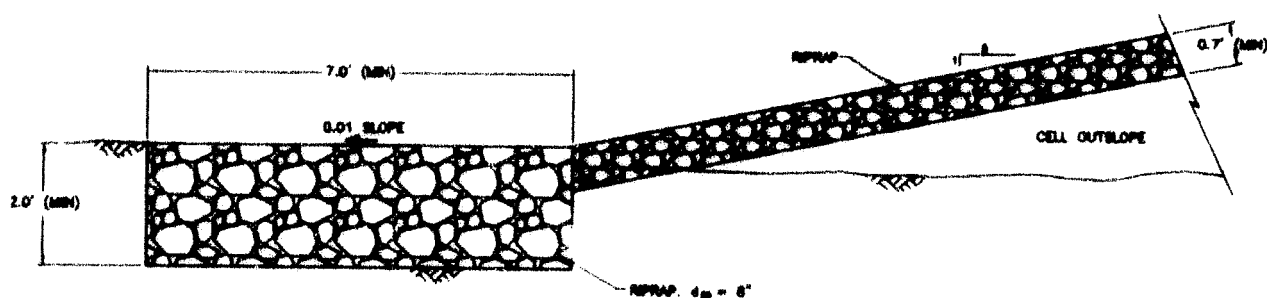
FLOW PARAMETERS IN CELL #1 DISCHARGE CHANNEL AT PEAK PMF DISCHARGE											
	Channel Bottom Width ft	Channel Side Slopes	Channel Gradient %	Manning Coefficient	Unit Discharge cfs/ft	Flow Depth ft	Cross Section Area of Flow sq ft	Hydraulic Radius ft	Flow Velocity ft/sec	Discharge cfs	Allowable Peak Velocity ft/sec
Bedrock Channel	100	3:1	0.0100	0.025	226	1.62	169.9	1.54	226.95	196	8.10
Bedrock Channel	120	3:1	0.0100	0.025	226	1.45	180.3	1.40	225.46	245	8.10

## **ATTACHMENT 12**





PLAN  
NTS



International Uranium (USA) Corporation  
White Mesa Mill



ROCK APRON AT TOE OF CELL OUTSLOPE

FIGURE 1  
ATTACHMENT 12

DESIGN: A. KUHN	DRAWN: A. KUHN	SHEET 1 of 1
CHKD BY:	DATE: 8/17/98	
APP:	SCALE: NTS	

ATTACHMENT 12 TABLE - RESPONSES TO NRC COMMENTS 7/17/98

ROCK APRON DESIGN TABLE - TAILING CELL EROSION PROTECTION  
WHITE MESA MILL

FLOW PATH ELEMENT	ELEMENT LENGTH L ft	ELEMENT WIDTH W ft	GRADIENT S ft/ft	SLOPE ANGLE degrees	Tc (minutes) (8.0-0.42)	RAINFALL WITHIN ft	INTENSITY in/hr	Peak Unit Discharge q cfs/ft	d50 inches
APRON	240	1	0.01	0.57	0.60	7.25	12.67	1.80	7.3

Notes

The top cover element length is 240 ft. This was used in the calculations for time of concentration and peak unit discharge.

The outslope element length is 240 ft. This was used in the calculations for time of concentration and peak unit discharge.

The d50 for the outslope was calculated per Apts. K. and Johnson, "Regrap Design for Overtopping Flow," ASCE Journal of Hydraulic Engineering, 1991.

The d50 for the apron was calculated per Apts. K. and Johnson, "Regrap Design for Overtopping Flow," ASCE Journal of Hydraulic Engineering, 1991.

DEPTH OF SCOUR AT DOWNSTREAM EDGE OF THE APRON

All methods used are from Pemberton, E.L. and J.M. Lara, 1984 "Computing Degradation and Local Scour" Technical Guideline for Bureau of Reclamation

ds = depth of scour, ft

q = unit discharge, cfs/ft

Method 1  $ds = K \cdot q^{0.24}$

K = constant 2.45

q = 1.81 cfs/ft

ds = 2.82 ft

Method 2  $ds = 0.25 \cdot dm$

dm = mean water depth at design discharge

ds = 0.22 ft

Method 3  $ds = 0.6 \cdot dfo$

dfo =  $q^{0.666} / Fbo^{0.33}$

Fbo = zero bed factor = 1.0 ft/s<sup>2</sup> for fine sand

ds = 0.09 ft

Method 4  $ds = 0.25 \cdot dma$

dma = unit cross section of flow = 0.87 ft

ds = 0.22 ft

Method 5  $ds = dma \cdot ((Vm/Vc) - 1)$

Vm = mean velocity = 1.81/0.78 fps

Vc = 0.5 fps

ds = 3.17 ft

AVERAGE SCOUR DEPTH = 1.30 ft

minimum depth of downstream edge scour barrier