IUC URANIUM (USA) CORPORATION

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August 28, 1998

Mr. Joseph Holonich
U.S. Nuclear Regulatory Commission
Uranium Recovery Branch
Office of Nuclear Materials Safety & Safeguards
Mail Stop T7J9
Two White Flint North
11545 Rockville Pike
Rockville, MD 20852-2738

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 throughly 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 contract 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

August 28, 1998

INTERNATIONAL URANIUM (USA) CORPORATION

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August 28, 1998

Mr. Joseph J. Holonich, Branch Chief
High Level Waste and Uranium Recovery Projects Branch
Division of Waste Management
Office of Nuclear Regulatory Commission
2 White Flint North, Mail Stop T7J9
11545 Rockville Pike
Rockville, Maryland 20852

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

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.

> *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 <u>only</u> 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^2/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 $pCi/m^2/s$ 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

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.

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 be 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 SC and CL. 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.

<u>REFERENCE</u>: 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 the 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.

7. *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 the 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

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.

7 *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.

<u>REFERENCES:</u> 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 (I. Arango, 1996, \Magnitude 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. t90) 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, slime) 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 analyzed 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

IUSA'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, IUSA 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 IUSA'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.

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.

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

<u>REFERENCES</u>: 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! 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.

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 downs: am 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.@ 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 SUA-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 <u>Impacts on Ecological Systems</u>

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 peregrinis 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 4.3.2. The staff is unable to locate a Section 4.3.2.1 in the reclamation plan. IUSA 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), IUSA 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

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)

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ATTACHMENT 1

CELL 4 DESIGN

TAILINGS MANAGEMENT SYSTEM

WHITE MESA PROJECT

BLANDING, UTAH

APPENDIX B

Umetco Minerals Corporation

AUGUST, 1988

APPENDIX B

TABLE OF CONTENTS

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 • 303/744-7105
1924 EAST FIRST STREET • CASPER, WYOMING 82601 • 307/234-2126

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

TABLE OF CONTENTS

Note: The attached logs have been redrawn

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

Test Hole	Depth (Ft.)	NATURAL Moisture Content (%)	NATURAL Dry Density (pcf)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	ATTERBERG LIMITS Liquid Limit (%)	ATTERBERG LIMITS Plasticity Index (%)	GRADATION ANALYSIS Maximum Size	GRADATION ANALYSIS Passing #200 (%)	GRADATION ANALYSIS Less than 2 µ (%)	REMOLDED Dry Density (pcf)	REMOLDED Moisture Content (%)	PERMEABILITY ft./yr.	PERMEABILITY cm./sec.	Specific Gravity	Soil Type
2	0-5			117.5	10.8	20	3	#16	58	19	111.6	16.4	0.57	5.5×10^{-7}		Sandy Silt
3	7-8	7.2				21	6	#16	62							
5	7½-10			104.1	18.5	33	8	3/4 In.	56	12	102.1	22.0	0.085	8.2×10^{-8}	2.65	Sandy Clayey Silt
6	1-2	10.3				25	7	#16	77							Calcareous Silty Clay
6	8½-9	6.1				27	8	#4	70							Sandy Clayey Silt
8	5-5½	13.1					NP	3/4 In.	62							Sandy Clay
9	0-1	8.1					NP	#16	53							Calcareous Sandy Silt
10	4-6½					24	10	#4	73							Sand - Silt
11	5½-6½	14.0				26	6	#16	65							Sandy Clay
12	2-5			101.0	20.6	53	35	#16	88	59	95.0	18.3	0.068	6.6×10^{-8}	2.67	Siltstone-Claystone
13	7-8	13.1				39	13	#8	84							Weathered Claystone
14	1-2	19.3				40	21	#4	89							Calcareous Silt Clay
15	1½-4½			106.8	19.0	26	8	3/8 In.	65	27	103.4	18.0	0.012	1.2×10^{-8}	2.64	Weathered Claystone
17	2-3	11.4				19	4	#8	59							Mod. Calcareous Sandy Clay
19	0-3			117.5	12.8	23	6	#16	70		109.9	12.4	0.035	3.4×10^{-8}		Sandy Silt
22	1-2	13.2				26	10	#4	73							Sandy Clayey Silt
23	1-3					48	24	#30	87							Sandy Clay
23	6-3					61	30	#30	96							Weathered Claystone
25	1-3½	13.3				26	9	#4	57							Claystone
26	4½-5	15.3				41	20	#4	91							Sandy Clay
28	0-2	12.7				28	10	3/8 In.	72							Weathered Claystone
29	2-3	8.5				19	2	#16	59							Sandy Clay
32	8-8½	5.6				23	6	#30	73							Sandy Silt
37	0-4			118.8	11.5	23	5	#8	72							Sandy Clayey Silt
38	5-7										110.5	11.5	0.63	6.1×10^{-7}		Sandy Clayey Silt
40	4-5½			111.0	16.7	29	14	3/8 In.	69		102.4	17.9	0.041	4.0×10^{-8}		Sandy Clay
				110.0	16.2	26	9	#8	64	27	106.4	16.4	0.012	1.6×10^{-8}	2.65	Sandy Clay

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

Test Hole	Depth (Ft.)	NATURAL Moisture Content (%)	NATURAL Dry Density (pcf)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	ATTERBERG LIMITS Liquid Limit (%)	ATTERBERG LIMITS Plasticity Index (%)	GRADATION ANALYSIS Maximum Size	GRADATION ANALYSIS Passing #200 (%)	GRADATION ANALYSIS Less than 2 μ (%)	REMOLDED Dry Density (pcf)	REMOLDED Moisture Content (%)	PERMEABILITY ft./yr.	PERMEABILITY cm./sec.	Specific Gravity	Soil Type
40	9-9½	6.8				22	8	3/8 In.	60							Sandy Clay
42	13½-14½	7.6				26	10	3/8 In.	73							Sandy Clay
43	11-12	12.1				41	22	#4	86							Claystone
43	13½-16½			110.0	16.9	40	24	3/8 In.	85	44	104.1	15.8	0.024	2.3×10^{-8}	2.62	Claystone
44	6½-7	7.5				30	11	3/8 In.	79							Calcareous Sandy Clay
46	0-2	12.3				22	6	#16	76							Sandy Clayey Silt
✓48	5-5½					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	3.2×10^{-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	5-5½	7.8				28	14	#30	71							Sandy Clay
55	9½-10½					28	13	#4	71							Sandy Clay
✓58	5½-6	12.5				35	11	#4	75							Sandy, Silty Clay
61	0-1	11.5				21	4	#16	75							Sandy Silt
62	11-11½	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	7½-8	8.6				28	13	#8	67							Sandy Clay
70	3½-4½	16.4				27	4	1½ 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

TABLE II

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	5.5×10^{-7}
TH 5 @ 7½'-10'	Calcareous Silty Clay	102.1	22.0	101	500	0.085	8.2×10^{-8}
TH 12 @ 2'-5'	Weathered Claystone	95.0	18.3	94	500	0.068	6.6×10^{-8}
TH 15 @ 1½'-4½'	Calcareous Sandy Clay	103.4	18.0	97	500	0.012	1.2×10^{-8}
TH 19 @ 0'-3'	Sandy, Clayey Silt	109.9	12.4	94	500	0.035	3.4×10^{-8}
TH 37 @ 0'-4'	Sandy, Clayey Silt	110.5	11.5	93	500	0.63	6.1×10^{-7}
TH 38 @ 5'-7'	Sandy Clay	102.4	17.9	92	500	0.041	4.0×10^{-8}
TH 40 @ 4'-5½'	Sandy Clay	106.4	16.4	97	500	0.017	1.6×10^{-8}
TH 43 @ 13½-16½'	Claystone	104.1	15.8	95	500	0.024	2.3×10^{-8}
TH 49 @ 5'-7'	Sandy Clay	105.2	13.9	95	500	0.33	3.2×10^{-7}

TABLE III

RESULTS OF ATTERBERG LIMITS

SAMPLE	SOIL TYPE	PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit (%)	SHRINKAGE RATIO
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

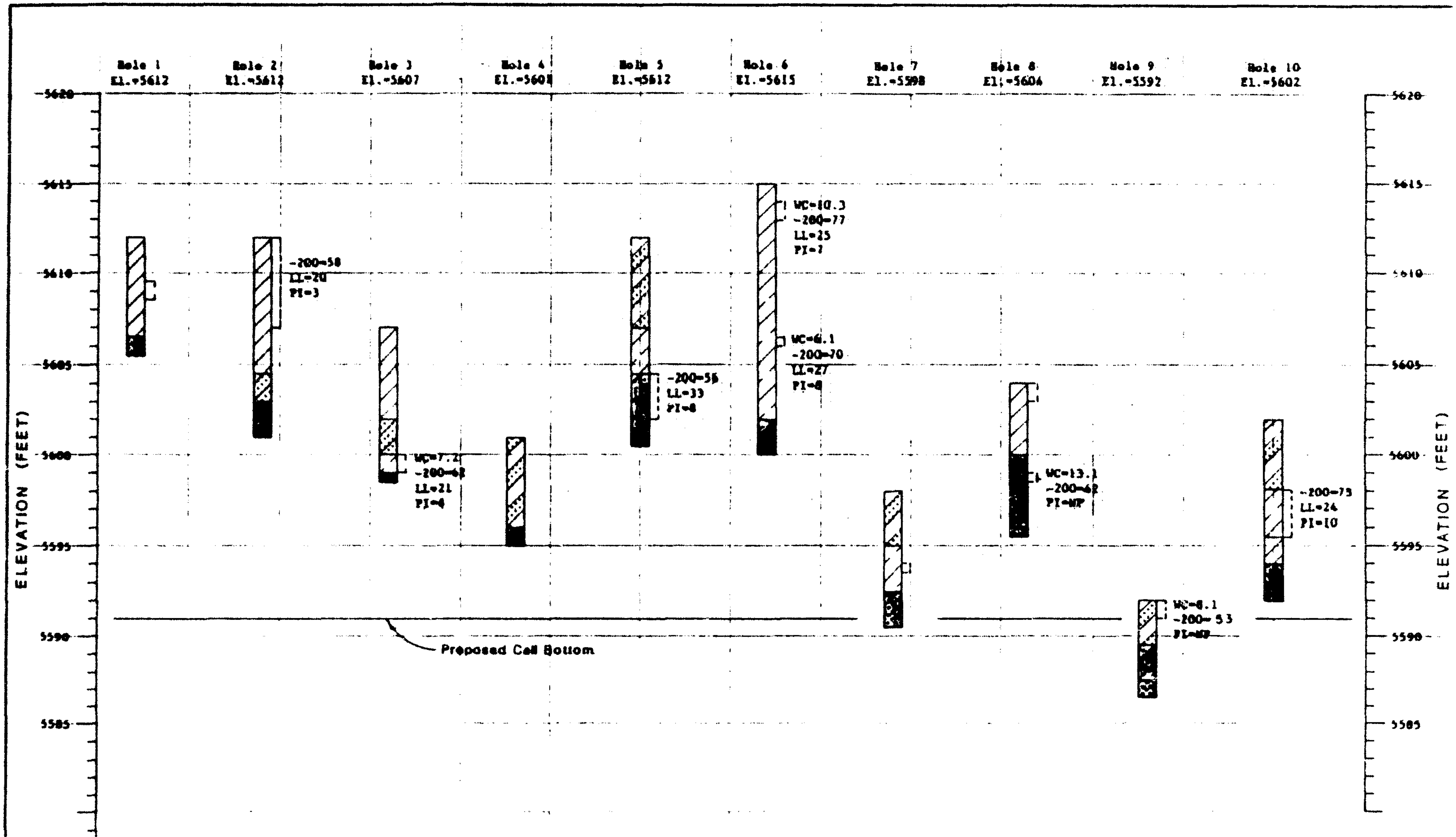


FIGURE 2
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

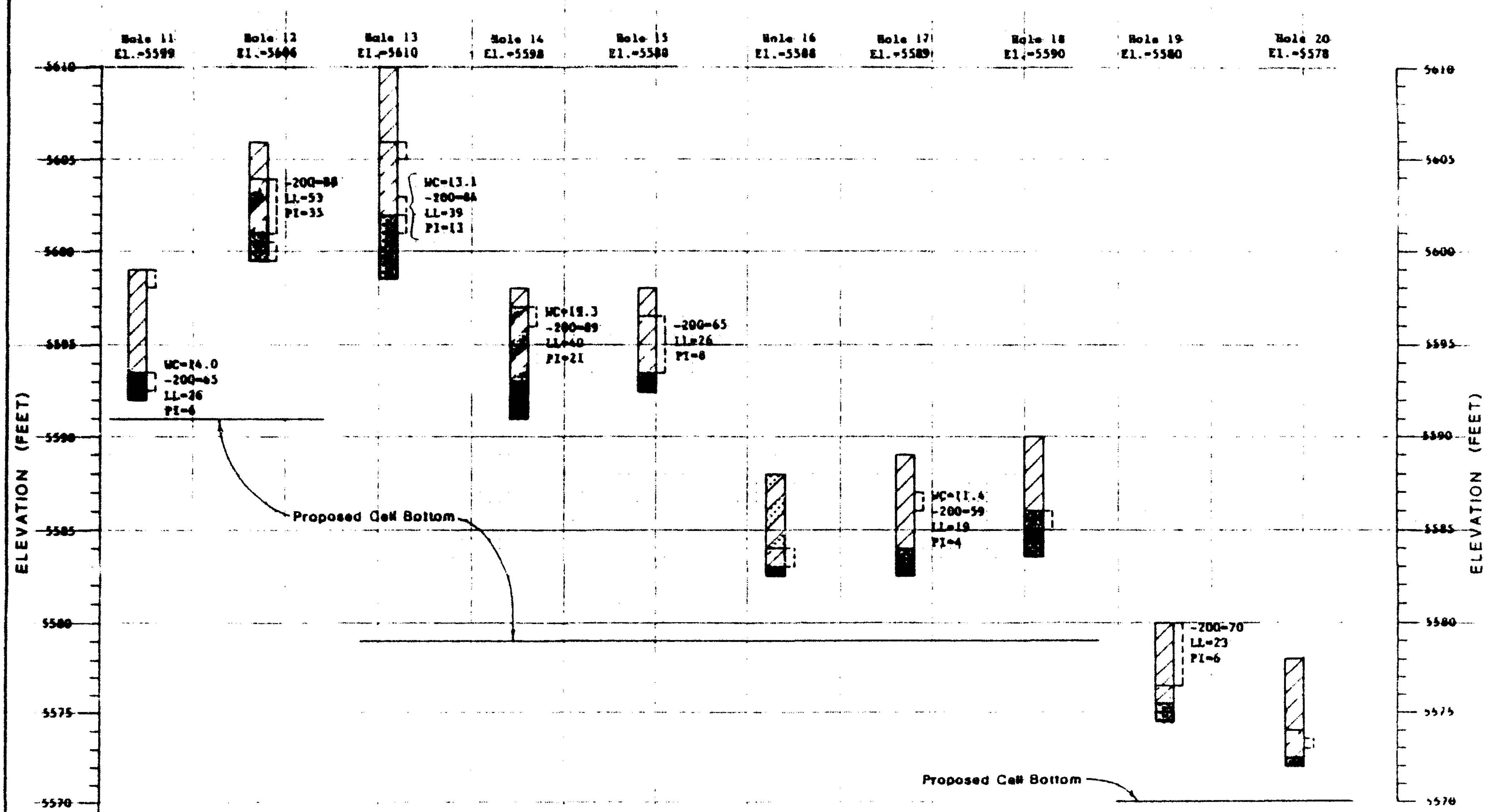


FIGURE 3
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

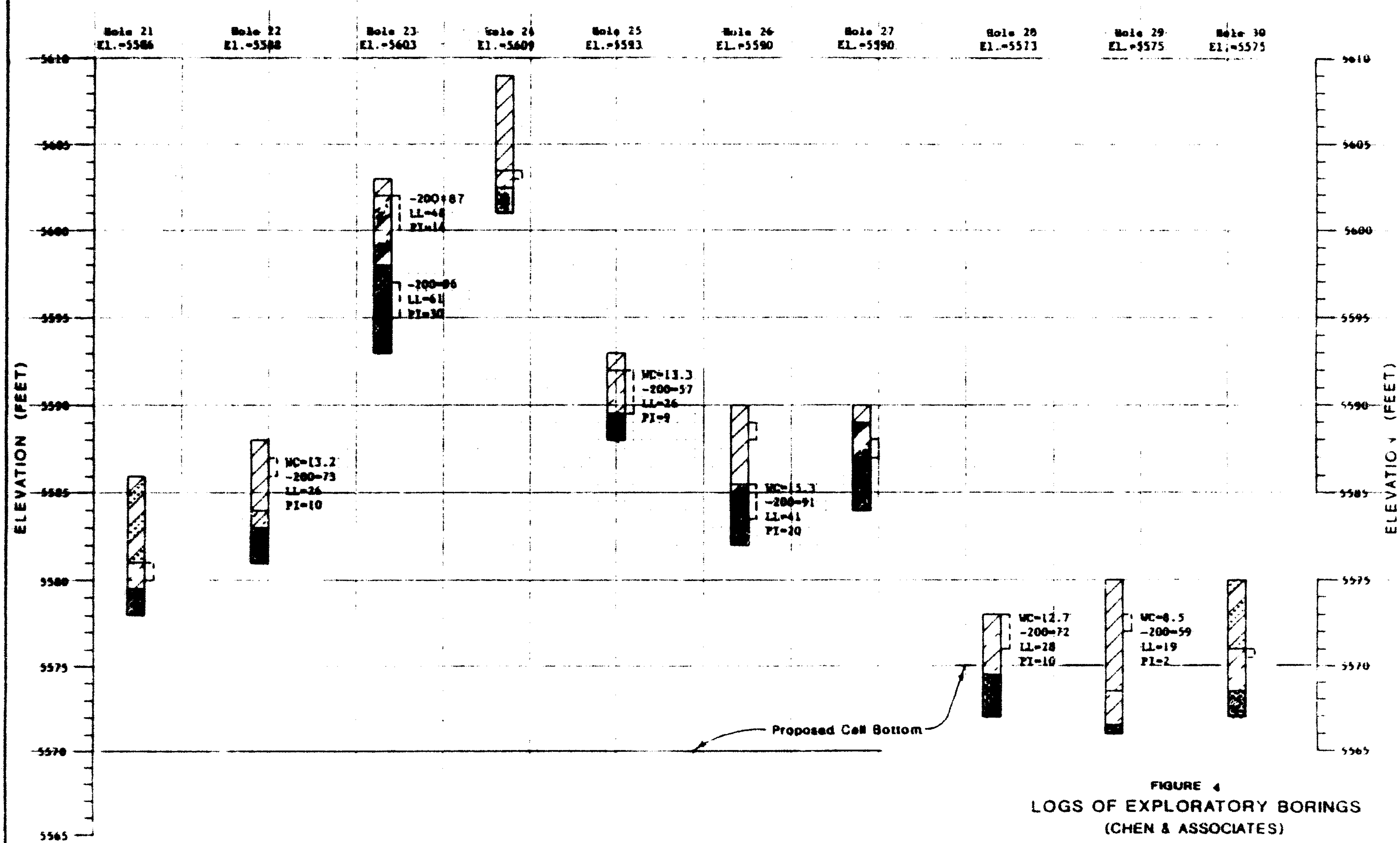


FIGURE 4
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

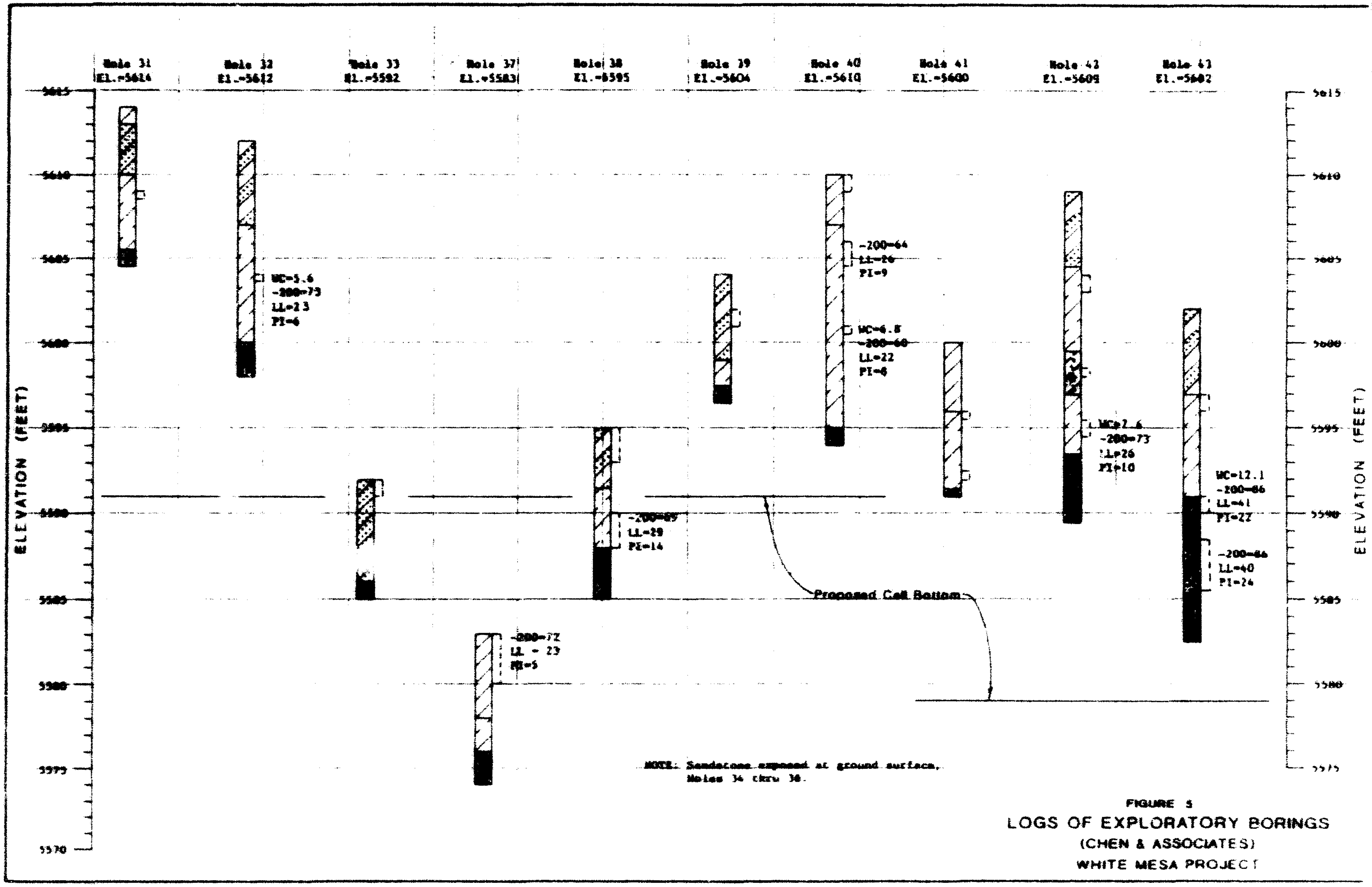


FIGURE 5
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

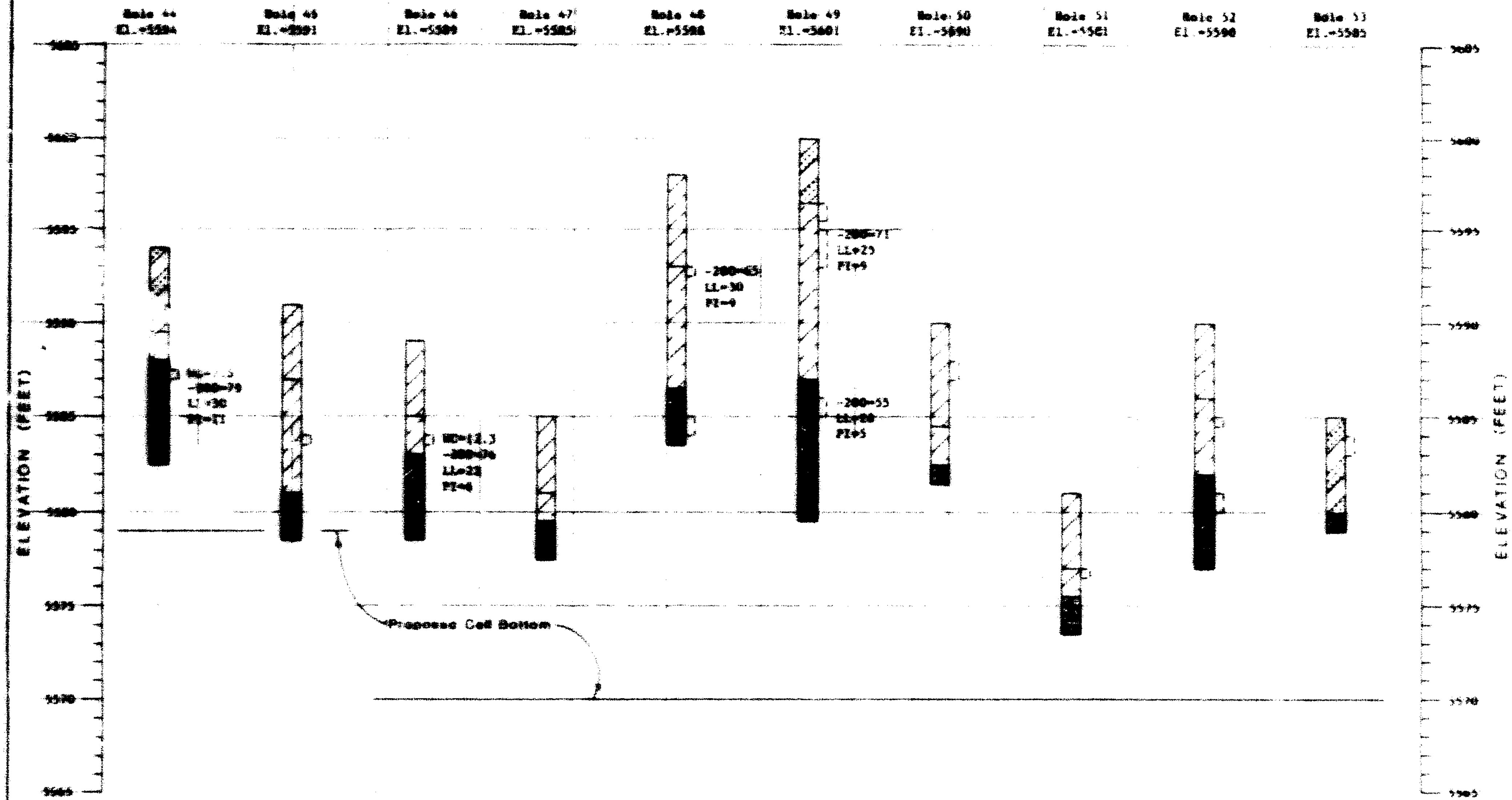


FIGURE 6

LOGS OF EXPLORATORY BORINGS

(CHEN & ASSOCIATES)

WHITE MESA PROJECT

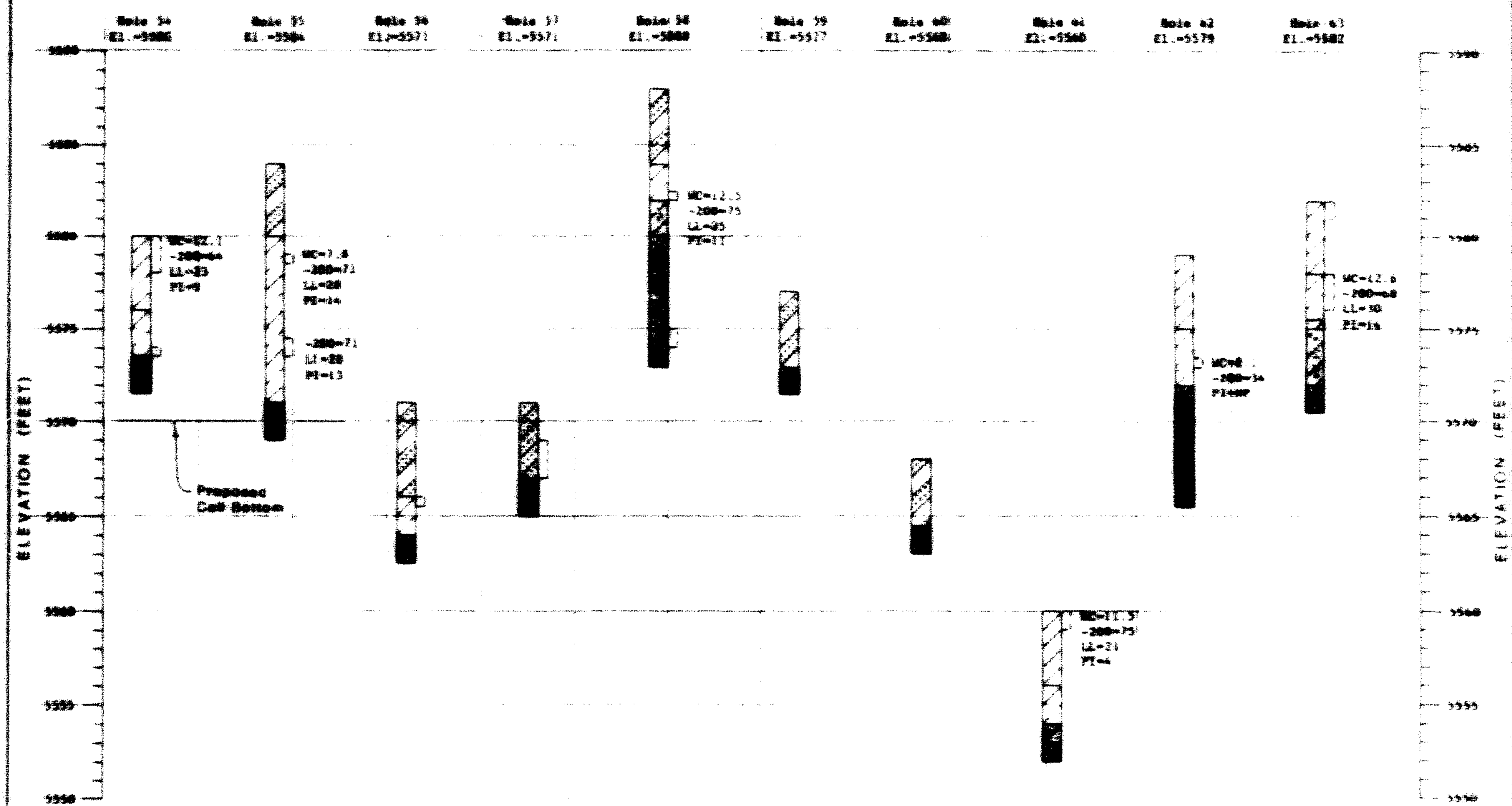


FIGURE 7
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

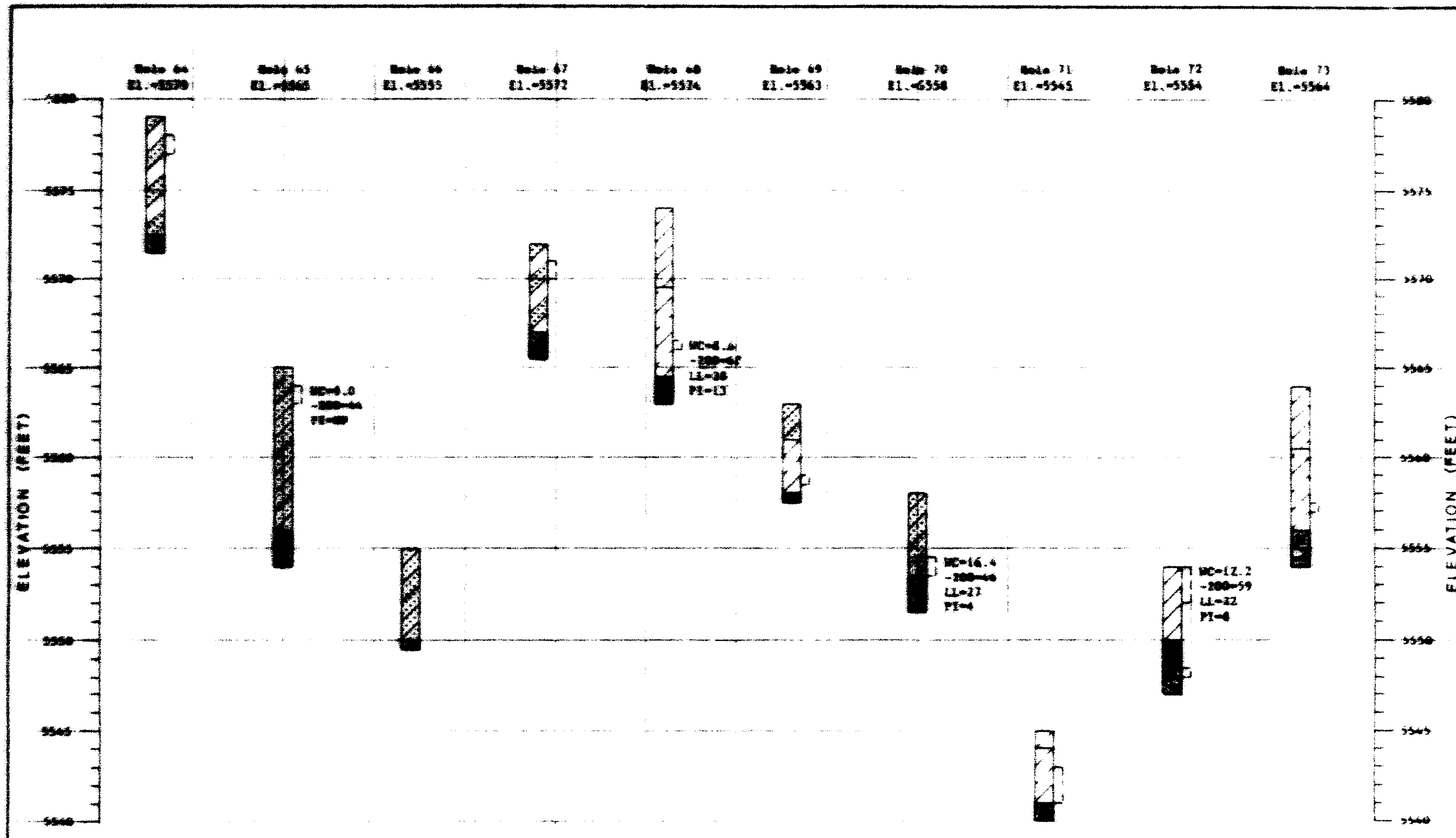
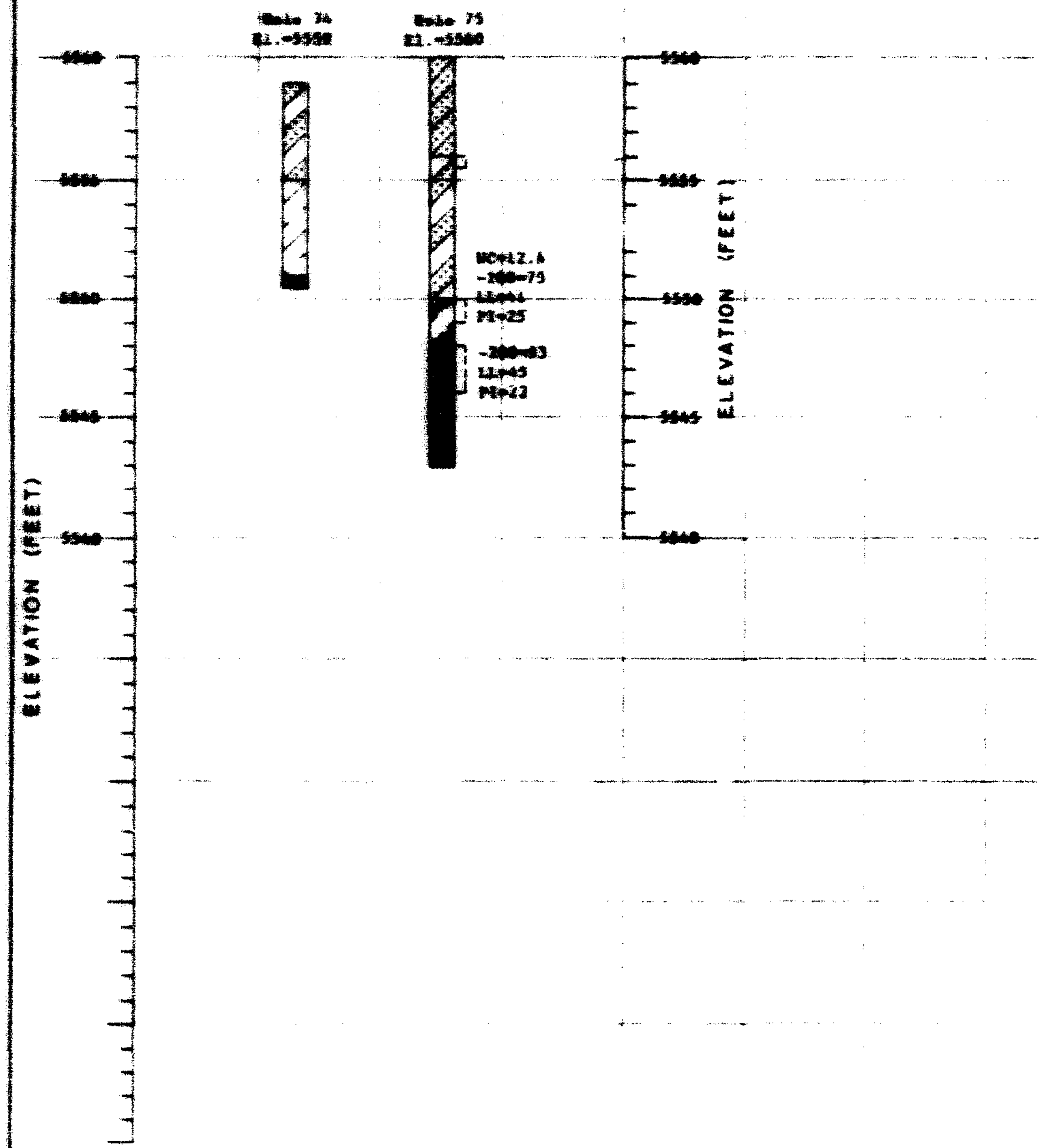


FIGURE 8
LOGS OF EXPLORATORY BORINGS
(CH[illegible]N & ASSOCIATES)
W[illegible] MESA PROJECT



Legend:

- Sand (SM), 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 (CL-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 drilling 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 (%);
-200 = Percent Passing No. 200 Sieve;
LL = Liquid Limit (%);
PI = Plasticity Index (%)
NP = Non-Plastic.

FIGURE 9

LOGS OF EXPLORATORY BORINGS

(CHEN & ASSOCIATES)

WHITE MESA PROJECT

chen and associates, inc.
CONSULTING ENGINEERS
SOIL & FOUNDATION ENGINEERING
96 S. ZUNI • DENVER, COLORADO 80223 • 303/744-7105

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

TABLE OF CONTENTS

Note: The attached logs have been redrawn.

CHEN AND ASSOCIATES

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

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

CHEN AND ASSOCIATES

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

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	9.9		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

CHEN AND ASSOCIATES

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

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)		
118	6.5 - 8.5			40	20				89	Weathered claystone
119	4.5 - 5	10.0		26	12				68	Sandy clay
120	1 - 2			25	8				69	Sandy, clayey silt
	5 - 5.5	15.5		29	10				78	Sandy clay
	11 - 11.5	11.6		42	24				90	Claystone
122	4 - 6			25	8				66	Sandy, silty clay
	14.5 - 15	6.4		26	8				66	Sandy clay
123	1 - 3			23	7				71	Sandy, clayey silt
124	4.5 - 5	6.0		23	7				69	Sandy, clayey silt
125	0 - 1	3.8		22	6				67	Sandy silt
127	5 - 6			54	34				89	Claystone
128	6 - 8			41	24				90	Claystone

T. .E II

LABORATORY PERMEABILITY TEST RESULTS

Sample	Classification	Compaction Dry Density (pcf)	Compaction Moisture Content (%)	% of ASTM D698	Surcharge Pressure (psf)	Permeability Ft./Yr.	Permeability Cm/Sec.
TH 80 @ 4½-7'	Calcareous sandy clay -200=78; LL=39; PI=20	100.2	19.4	96	500	0.81	7.8×10^{-7}
TH 84 @ 0-2'	Sandy silt -200=65; LL=18; PI=2	113.8	11.7	96	500	4.45	4.3×10^{-6}
TH 96 @ 8½-9½'	Calcareous sandy clay -200=66; LL=32; PI=6	96.9	20.7	97	500	1.55	1.5×10^{-6}
TH 96 @ 8½-9½'	Calcareous sandy clay	95.7	20.3	96	500	26.90*	2.6×10^{-5}*
TH 99 @ 8-9½'	Weathered claystone -200=89; LL=40; PI=20	99.8	18.5	95	500	0.22	2.1×10^{-7}
TH 100 @ 0-1'	Very silty sand -200=44; PI=NP	117.5	9.7	98	500	0.38	3.7×10^{-7}
TH 114 @ 0-2'	Sandy, clayey silt -200=58; LL=22; PI=6	112.4	12.9	95	500	0.60	5.8×10^{-7}
TH 120 @ 1-2'	Sandy, clayey silt -200=69; LL=24; PI=6	108.2	14.7	95	500	0.11	1.1×10^{-7}
TH 122 @ 4-6'	Sandy, silty clay -200=66; LL=25; PI=8	108.8	15.5	96	500	0.43	4.2×10^{-7}
TH 123 @ 1-3'	Sandy, clayey silt -200=71; LL=23; PI=7	110.9	12.6	95	500	0.56	5.4×10^{-7}
TH 128 @ 6-7'	Claystone -200=89; LL=41; PI=24	92.4	23.9	93	500	0.12	1.2×10^{-7}
TH 128 @ 6-7'	Claystone -200=89; LL=41; PI=4	93.1	22.1	94	500	0.52*	5.0×10^{-7}*

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

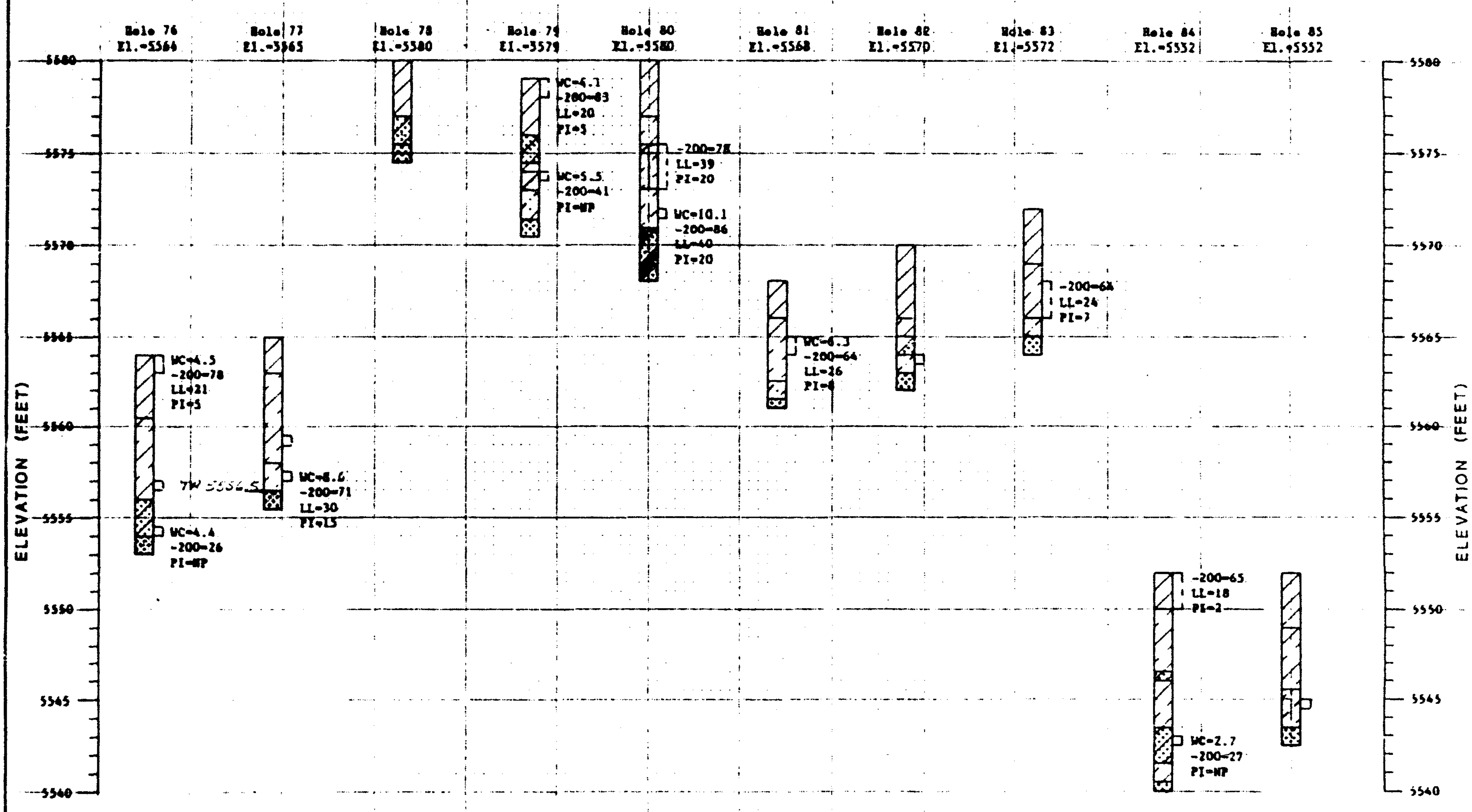


FIGURE 2
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

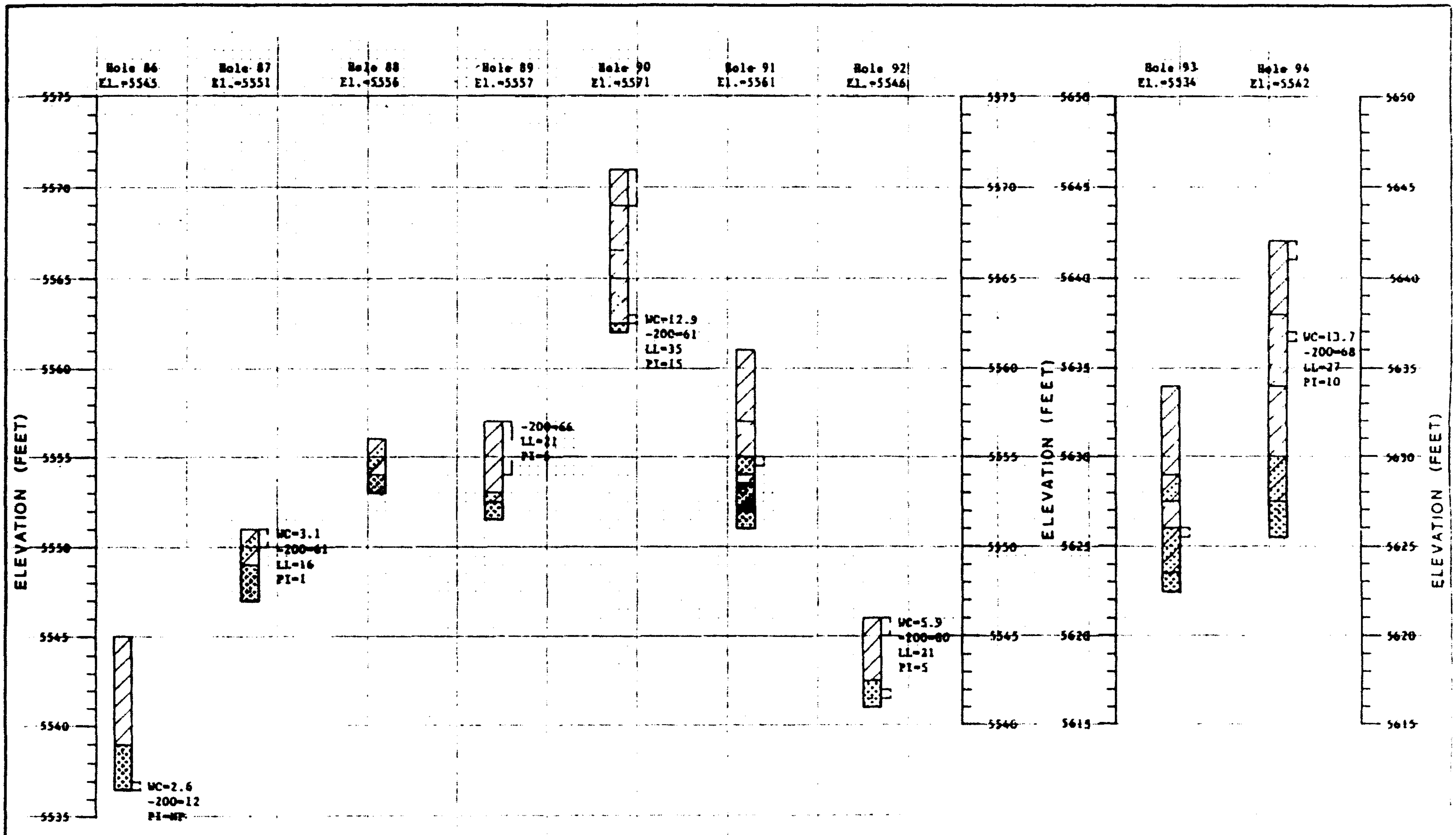


FIGURE 3
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

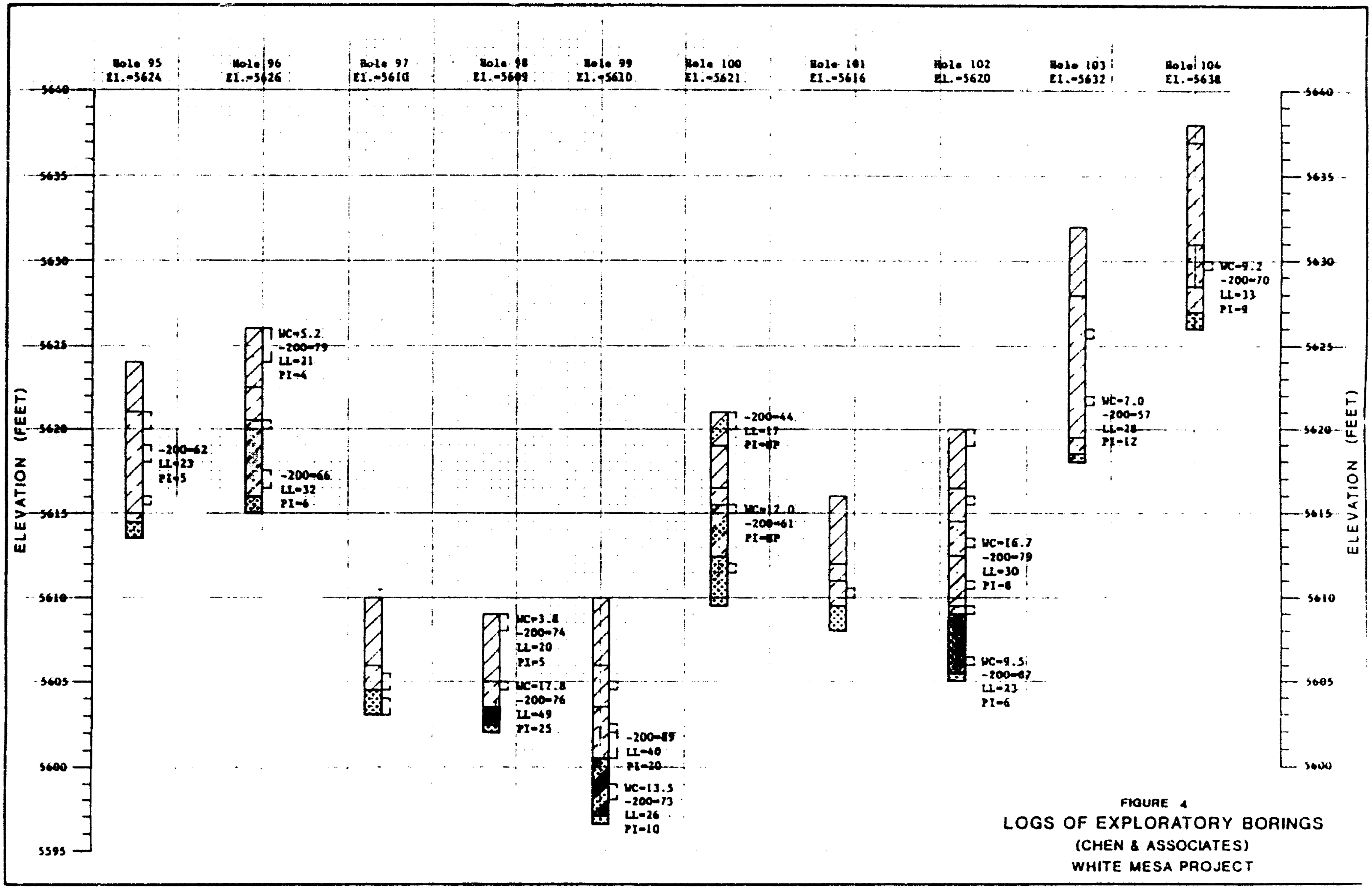


FIGURE 4
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

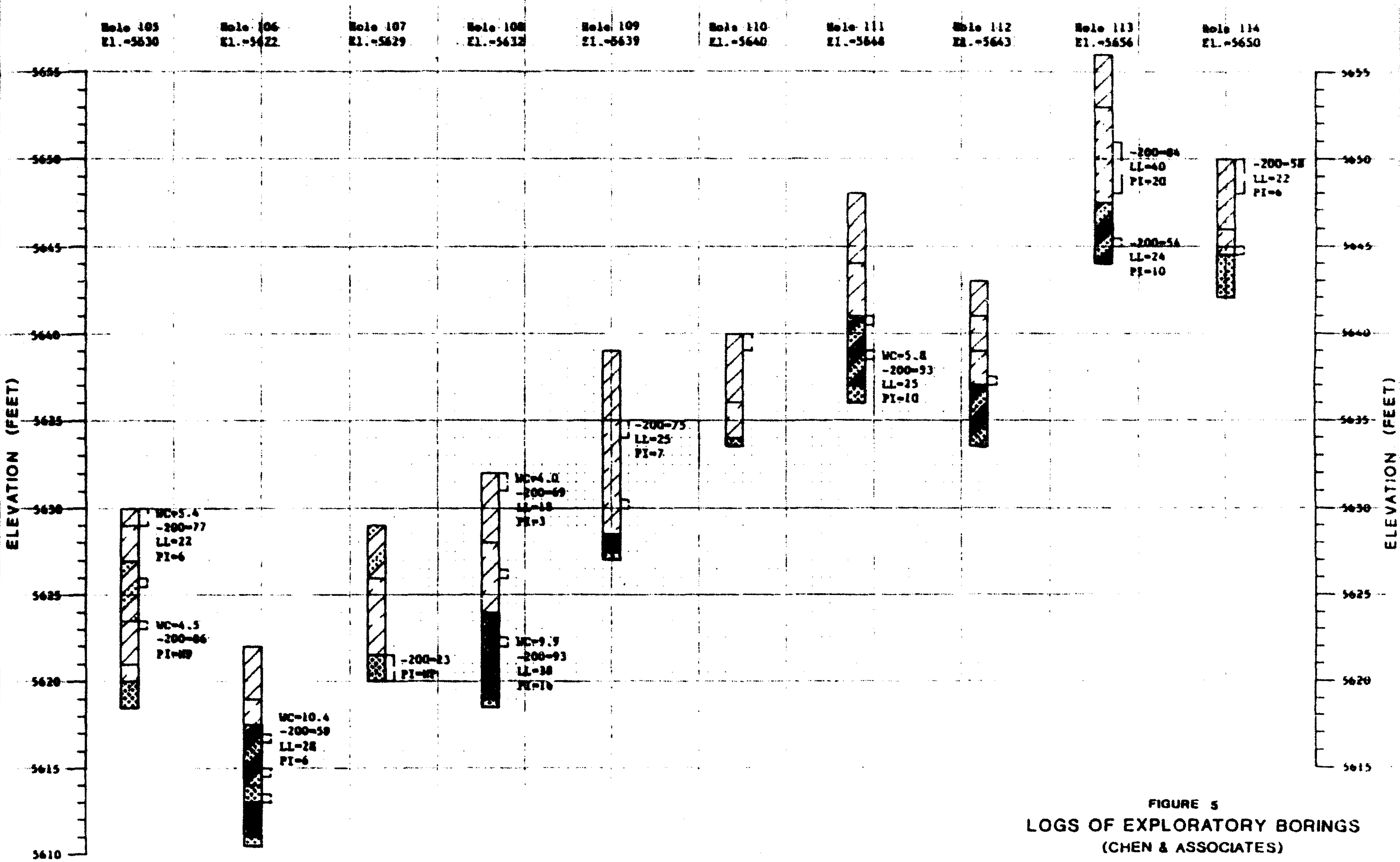


FIGURE 5
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

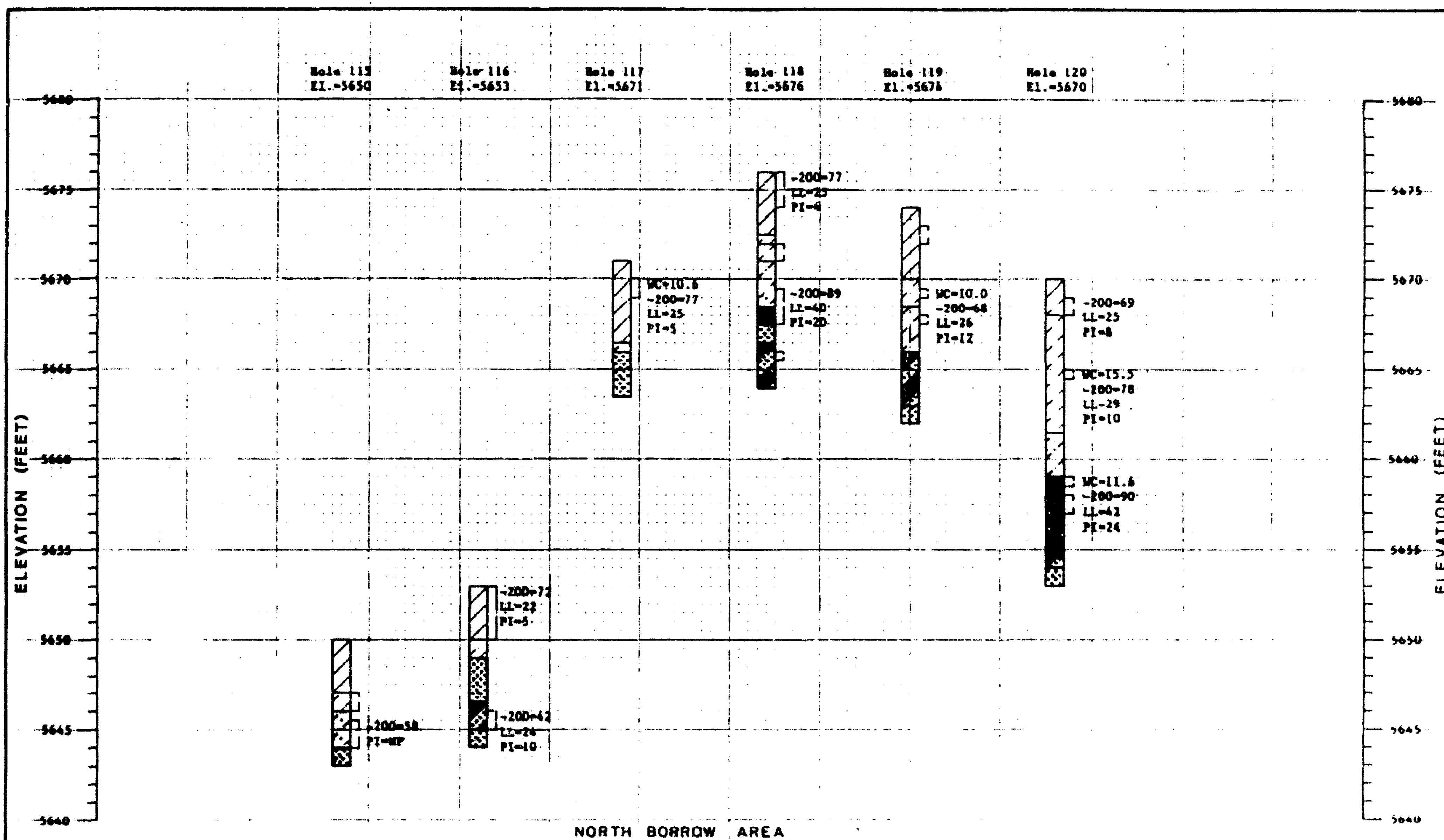


FIGURE 6
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

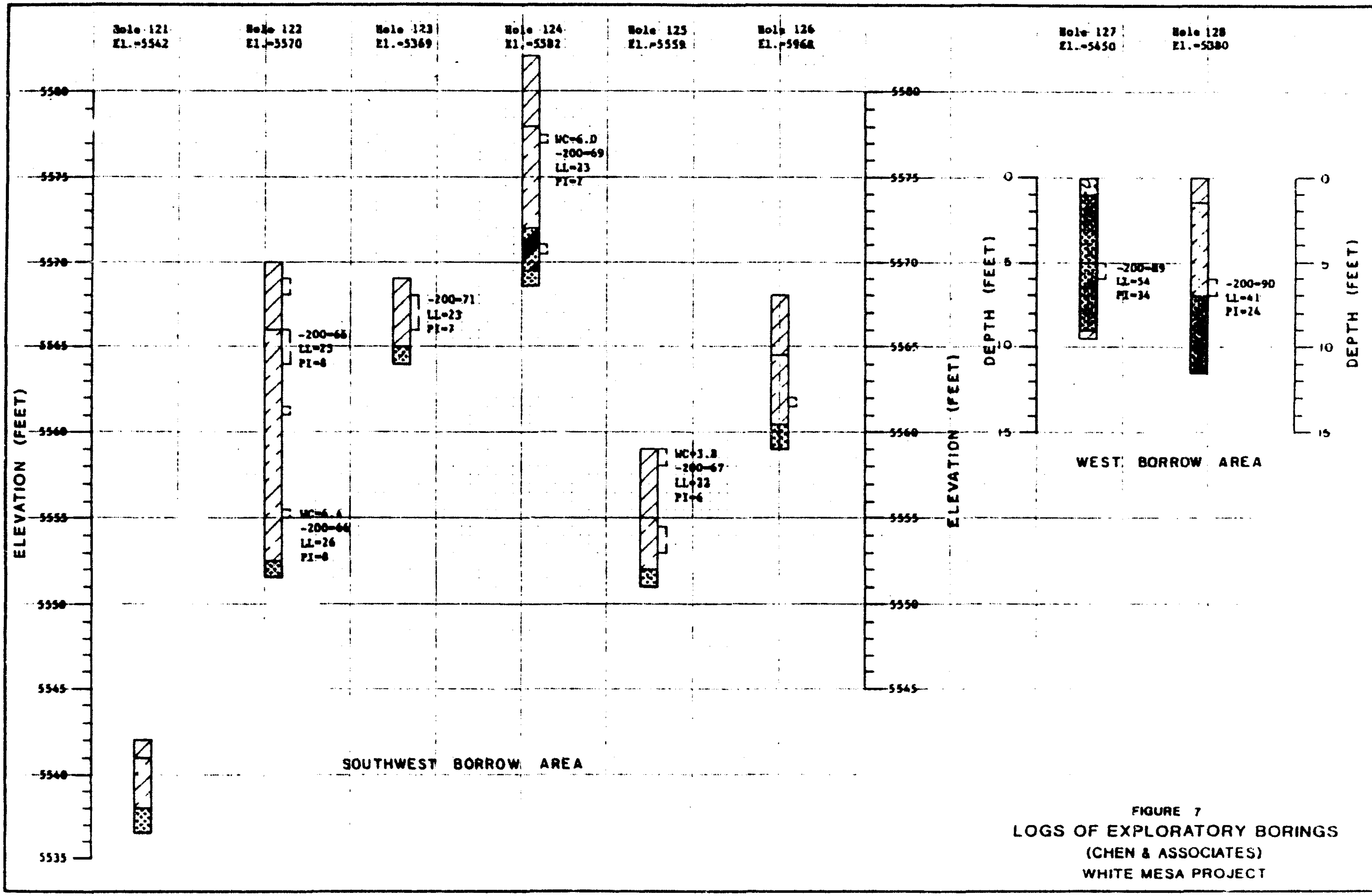


FIGURE 7
LOGS OF EXPLORATORY BORINGS
(CHEN & ASSOCIATES)
WHITE MESA PROJECT

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 n 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

TABLE OF CONTENTS

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
	MORE THAN 50 % OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	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
MORE THAN 50 % OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	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
MORE THAN 50 % OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS 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

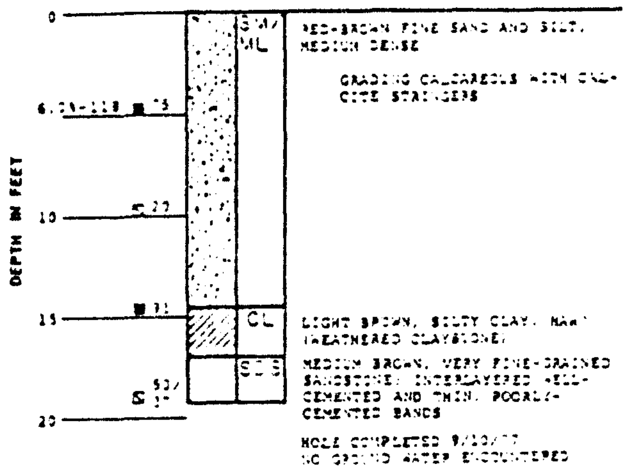
	SDS	SANDSTONE		SLN	SILTSTONE
	CLS	CLAYSTONE		CGL	CONGLOMERATE

GRAPHIC LOG SYMBOLS FOR ROCK

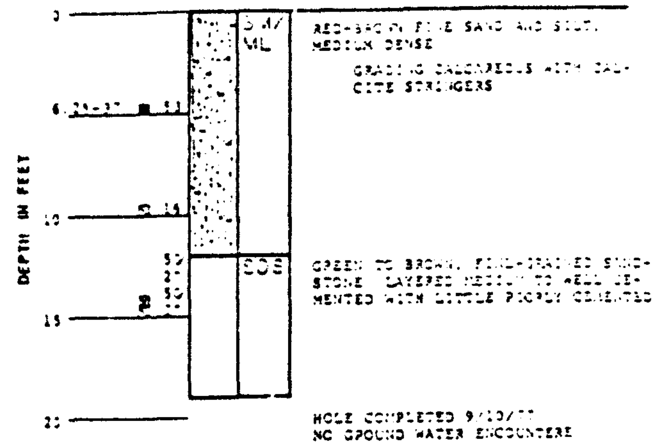
UNIFIED SOIL CLASSIFICATION SYSTEM AND GRAPHIC LOG SYMBOLS

DAMES & MOORE

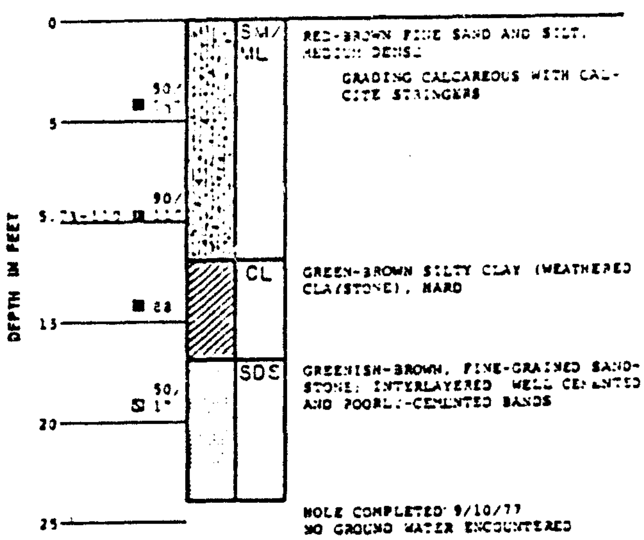
BORING NO. 1
EL. 5629.0 FT.



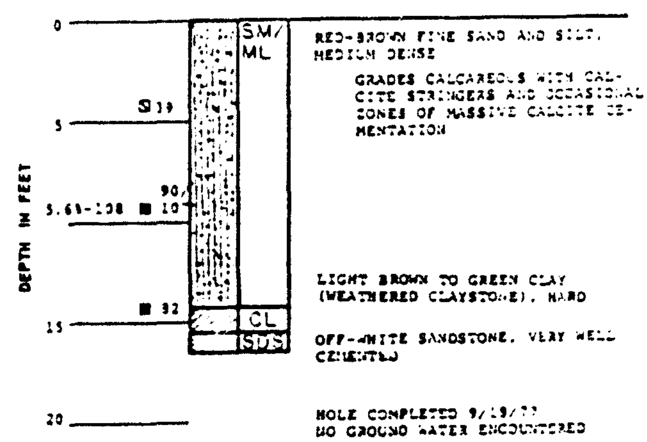
BORING NO. 5
EL. 5632.9 FT.



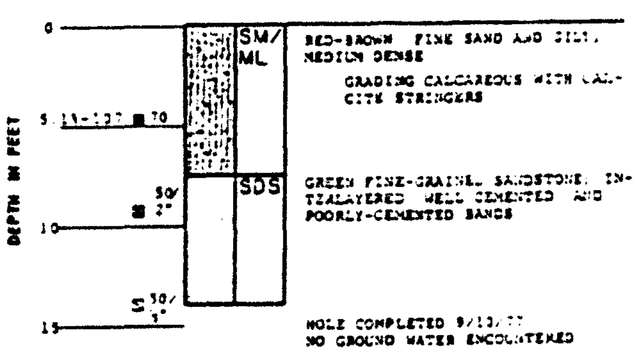
BORING NO. 2
EL. 5634.3 FT.



BORING NO. 6
EL. 5633.5 FT.



BORING NO. 4
EL. 5623.2 FT.



KEY

- A-B ■ C — 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
- D E — INDICATES NX CORE RUN
- D — PERCENT OF CORE RECOVERY
- E — RQD*
- F — INDICATES PACKER TEST SECTION
- F — PERMEABILITY MEASURED BY SINGLE PACKER TEST IN FT/YR
- NA — NOT APPLICABLE (USED FOR RQD IN CLAYS OR MECHANICALLY FRACTURED ZONES)

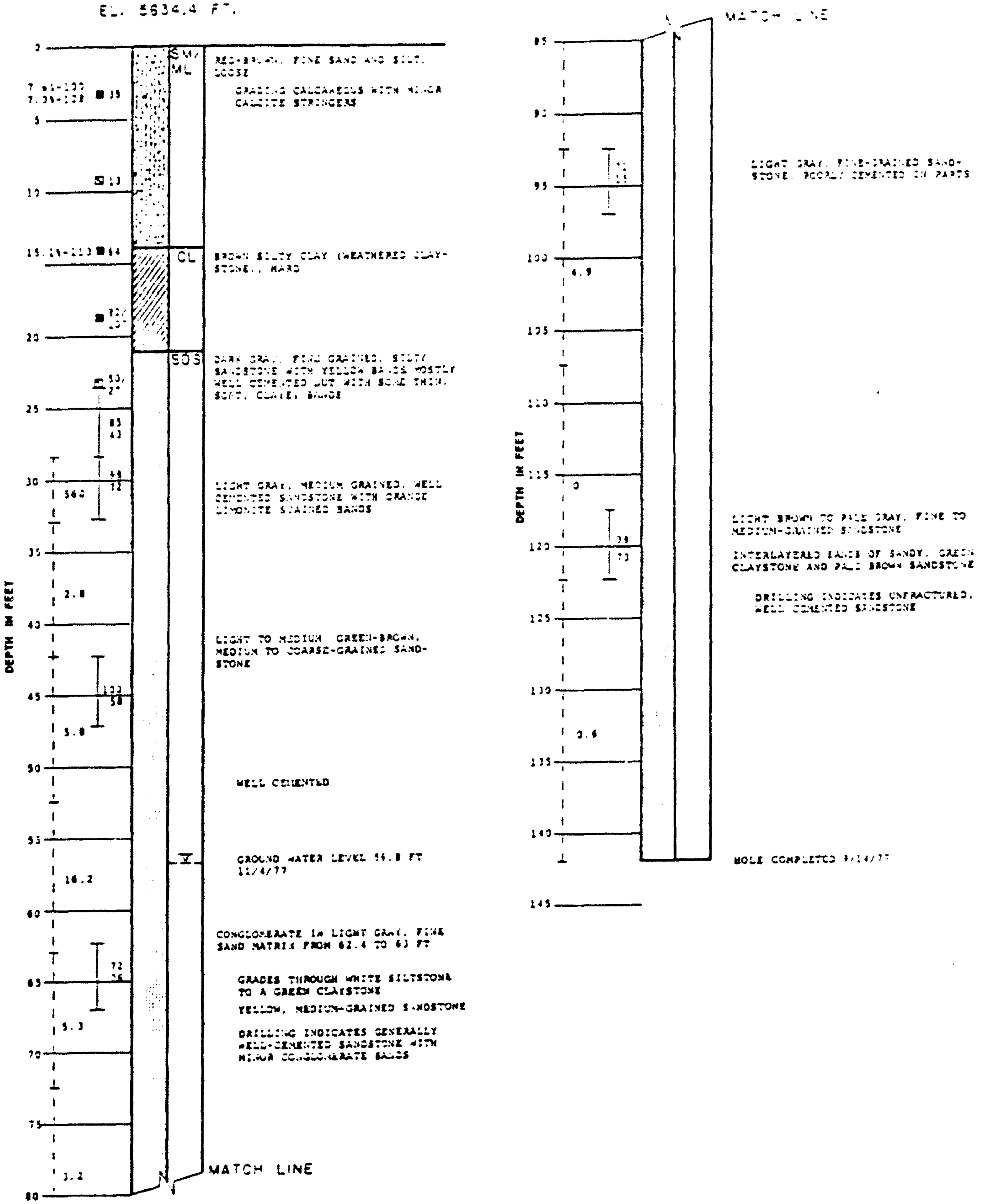
NOTE: ELEVATIONS PROVIDED BY ENERGY FUELS NUCLEAR, INC.

* ROCK QUALITY DESIGNATION -- PERCENTAGE OF CORE RECOVERED IN LENGTHS GREATER THAN 4 INCHES

LOG OF BORINGS

DAMES & MOORE

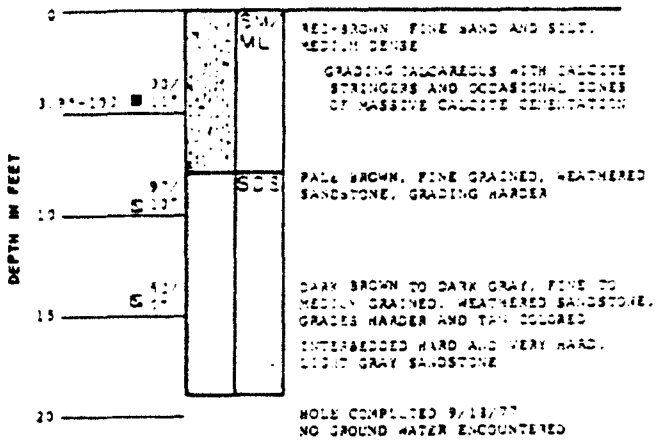
BORING NO. 3
EL. 5634.4 FT.



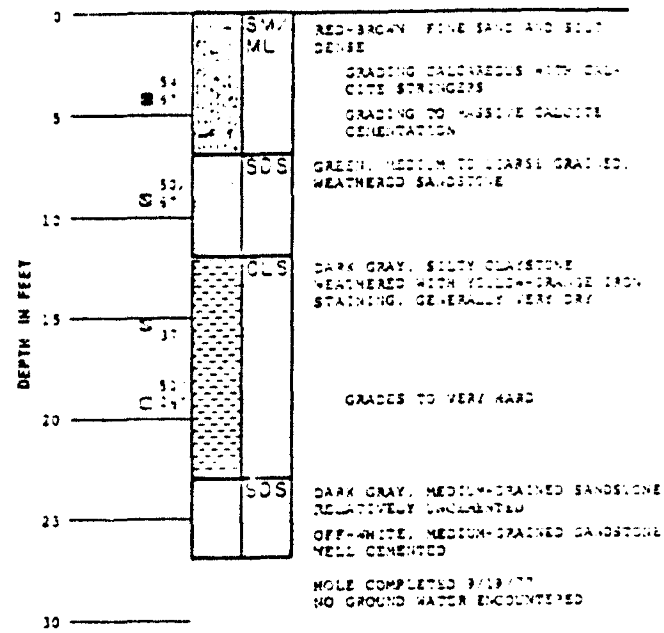
LOG OF BORINGS

DAMES & MOORE

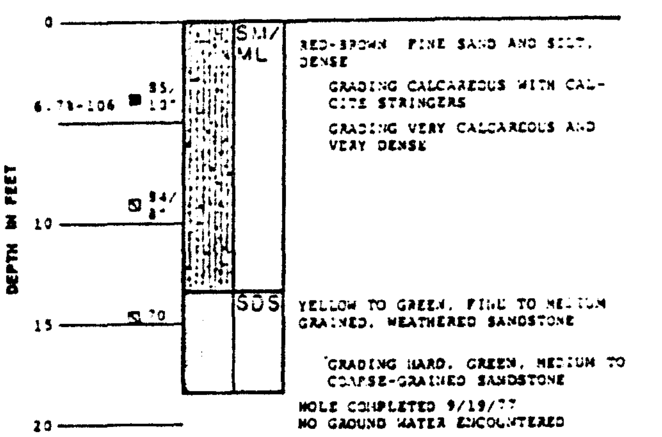
BORING NO. 7
EL. 5556.9 FT.



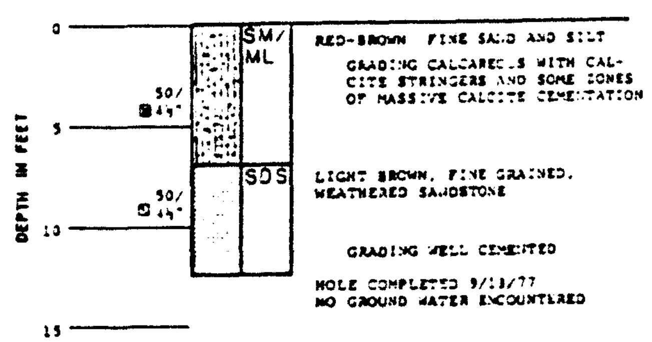
BORING NO. 8
EL. 5658.4 FT.



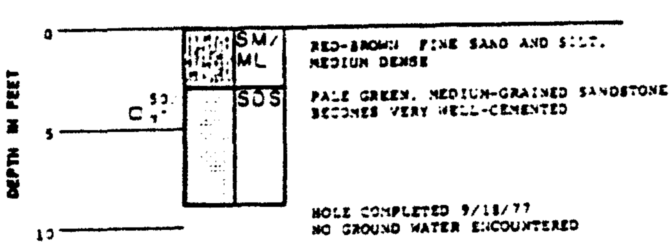
BORING NO. 10
EL. 5690.9 FT.



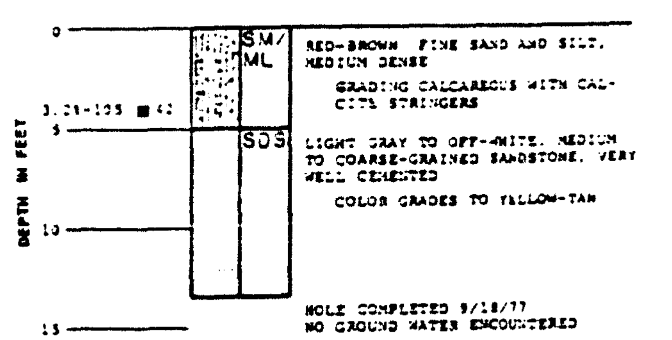
BORING NO. 11
EL. 5677.8 FT.



BORING NO. 13
EL. 5602.4 FT.



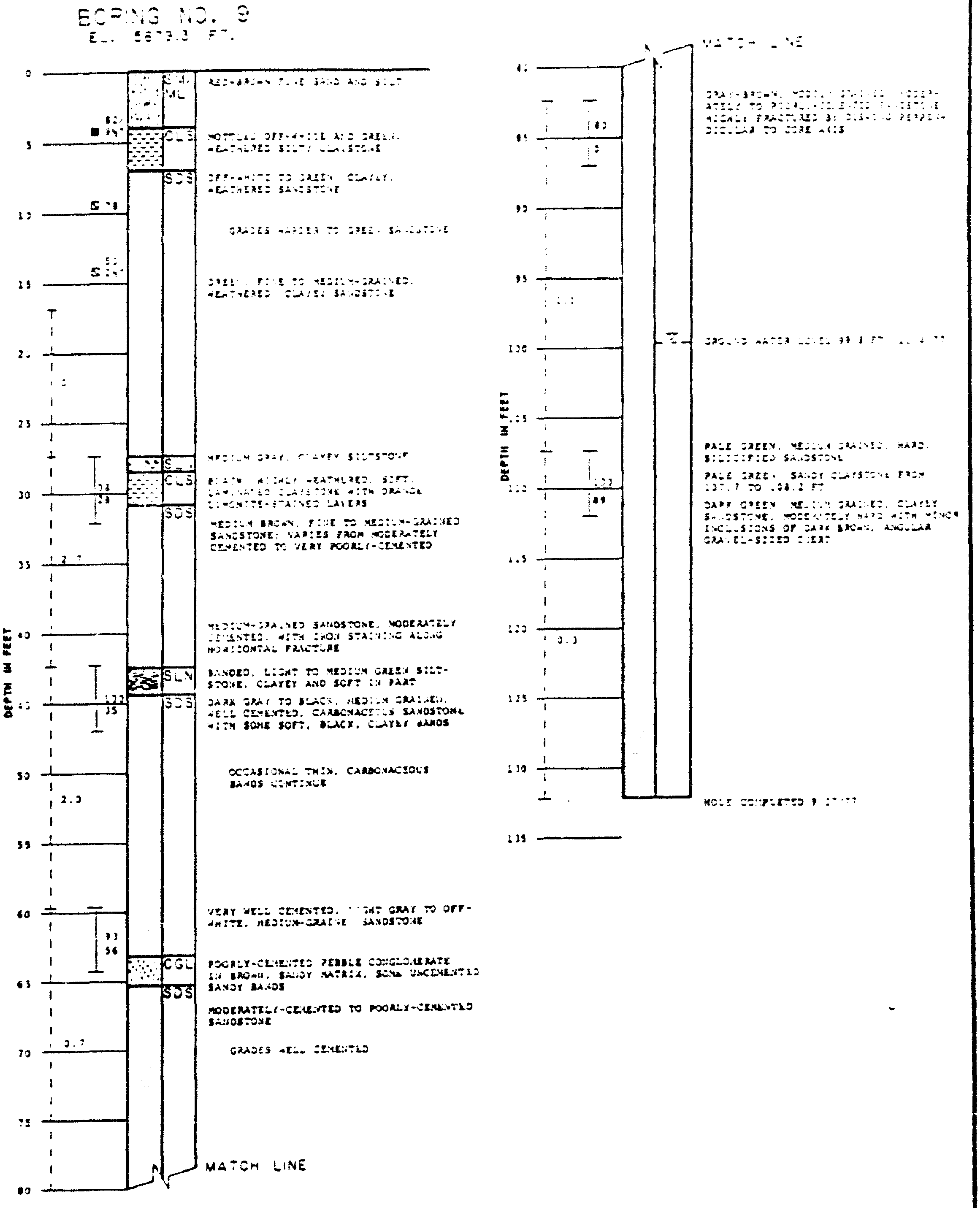
BORING NO. 14
EL. 5597.5 FT.



LOG OF BORINGS

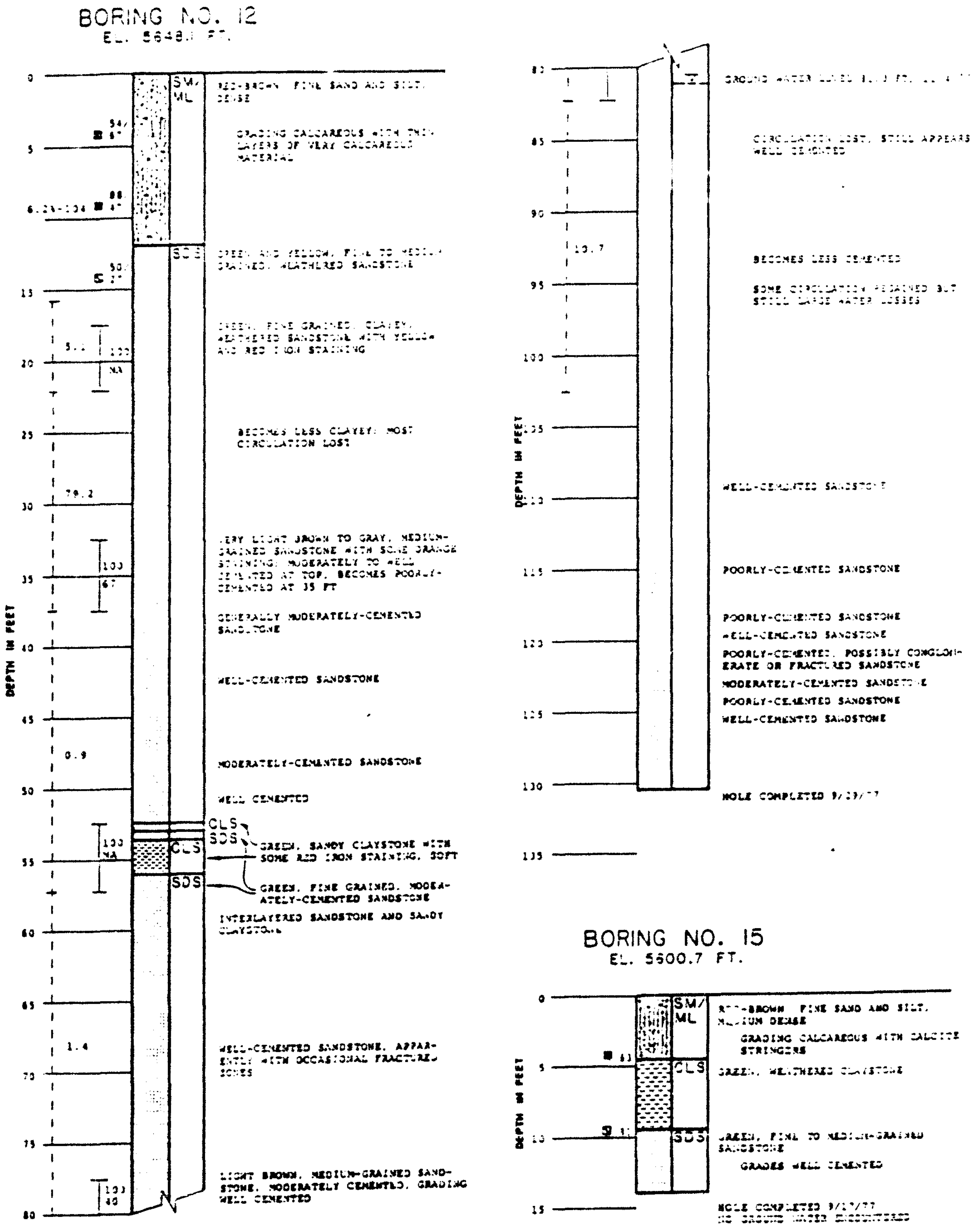
DAMES & MOORE

PLATE A-5



LOG OF BORINGS

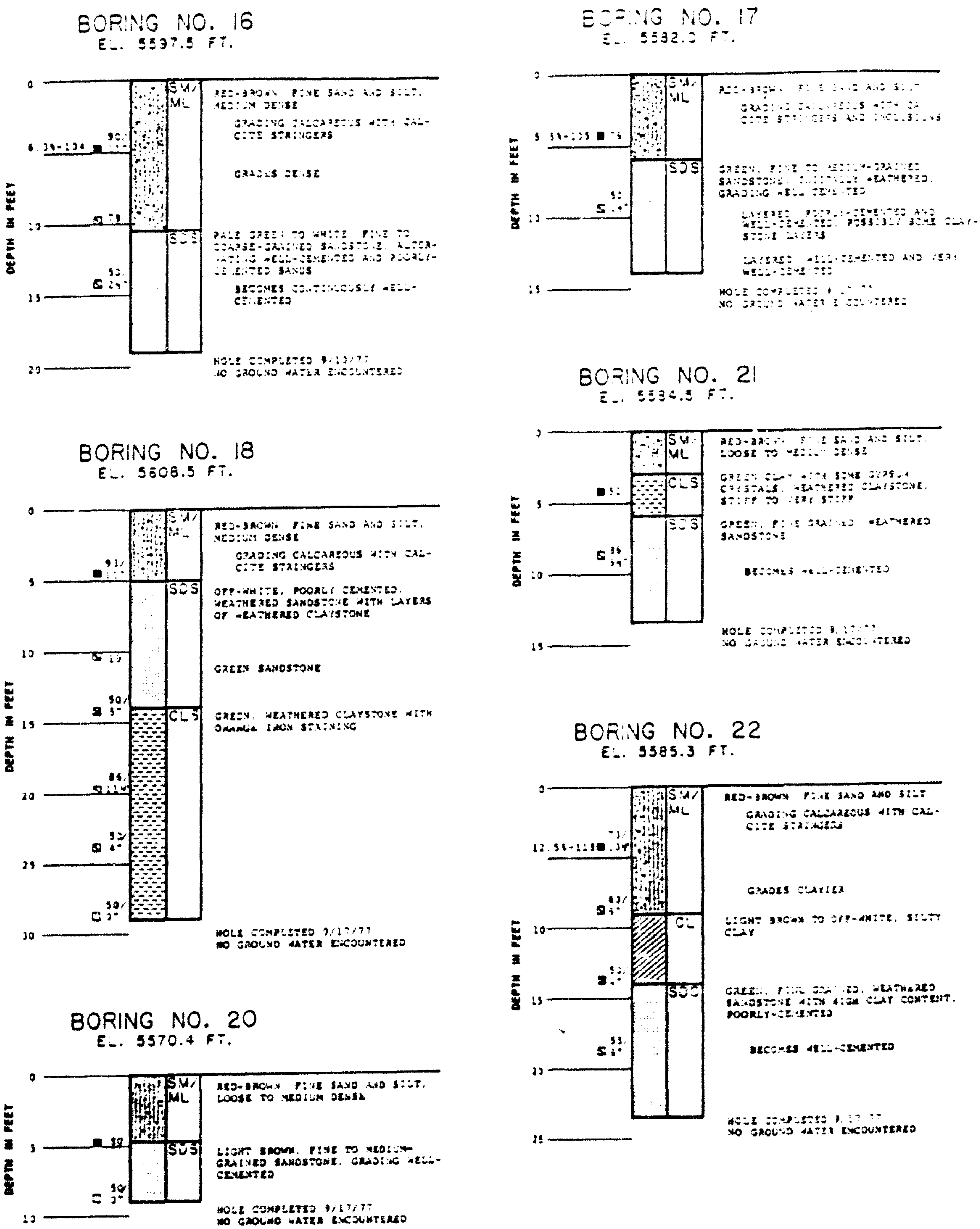
DAMES & MOORE



LOG OF BORINGS

DAMES & MOORE

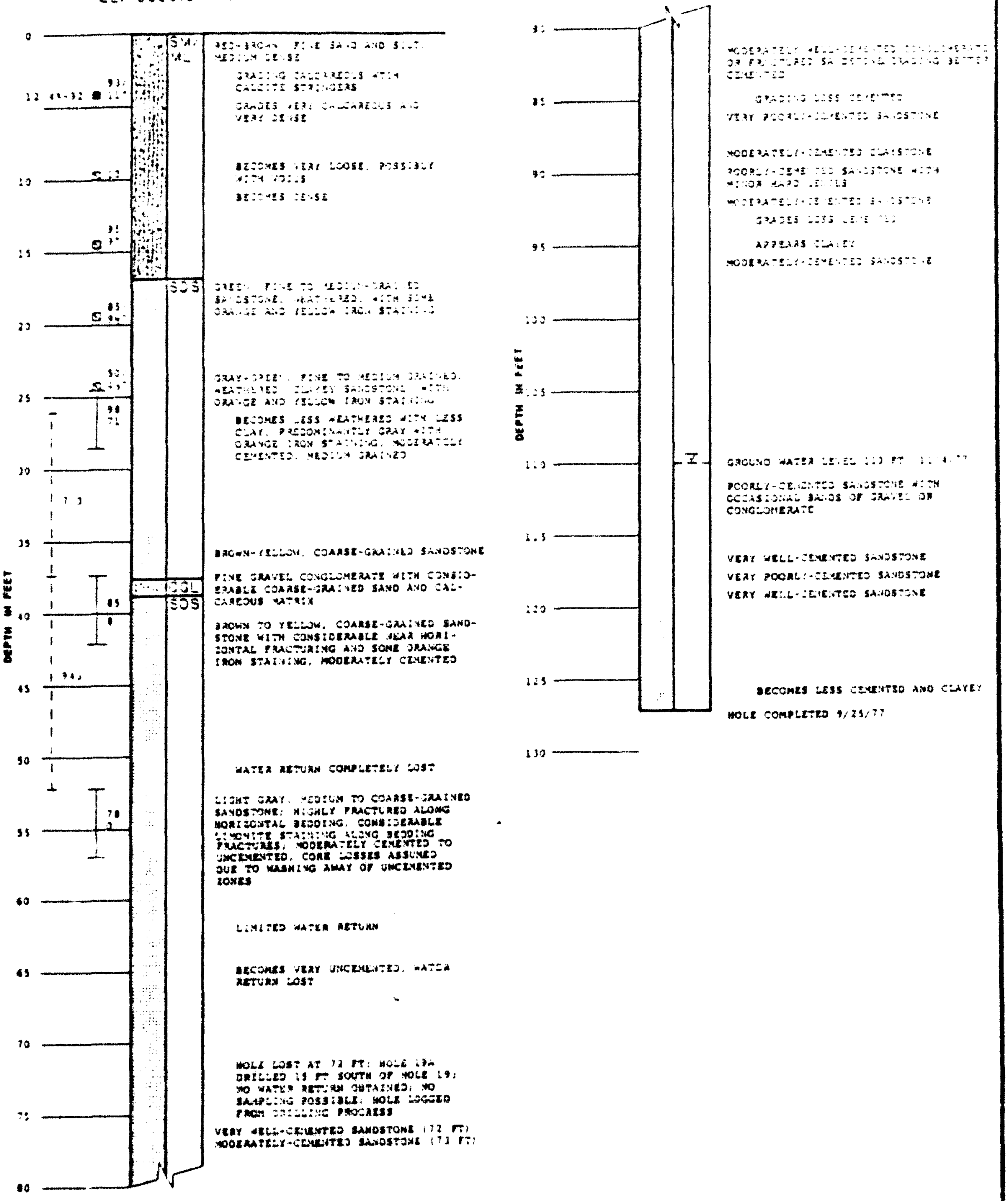
PLATE A-7



LOG OF BORINGS

BORING NO. 19

EL. 5600.3 FT.

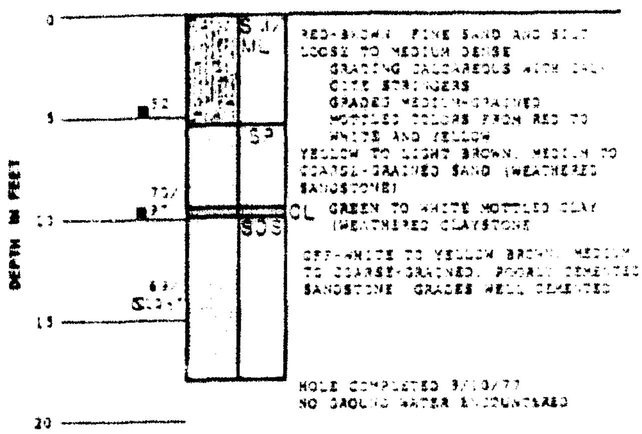


LOG OF BORINGS

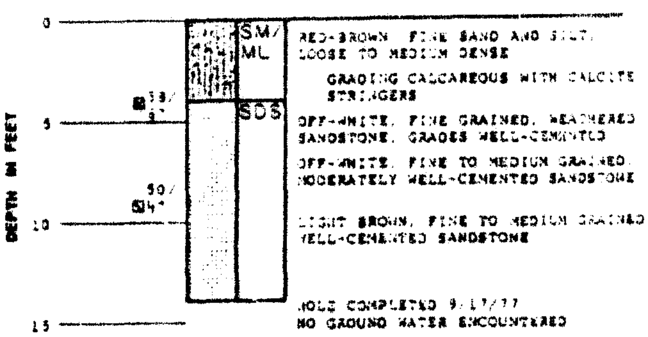
DAMES & MOORE

PLATE A-9

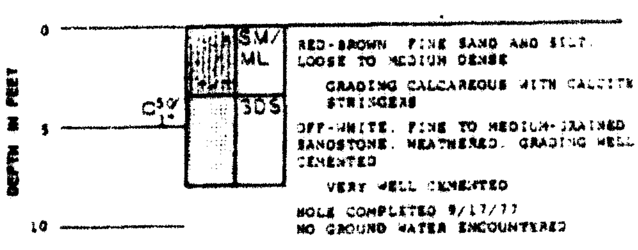
BORING NO. 23
EL. 5555.9 FT.



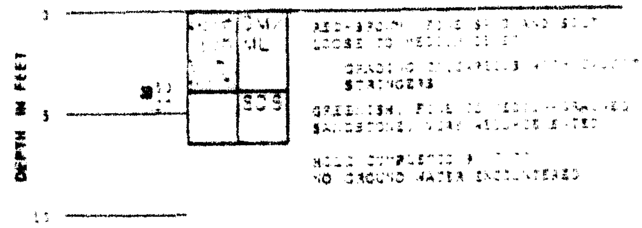
BORING NO. 24
EL. 5573.4 FT



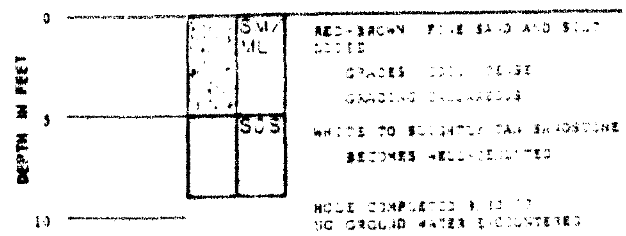
BORING NO. 26
EL. 5578.3 FT.



BORING NO. 27



BORING NO. 29
EL. 5655.0 FT. (APPROX)

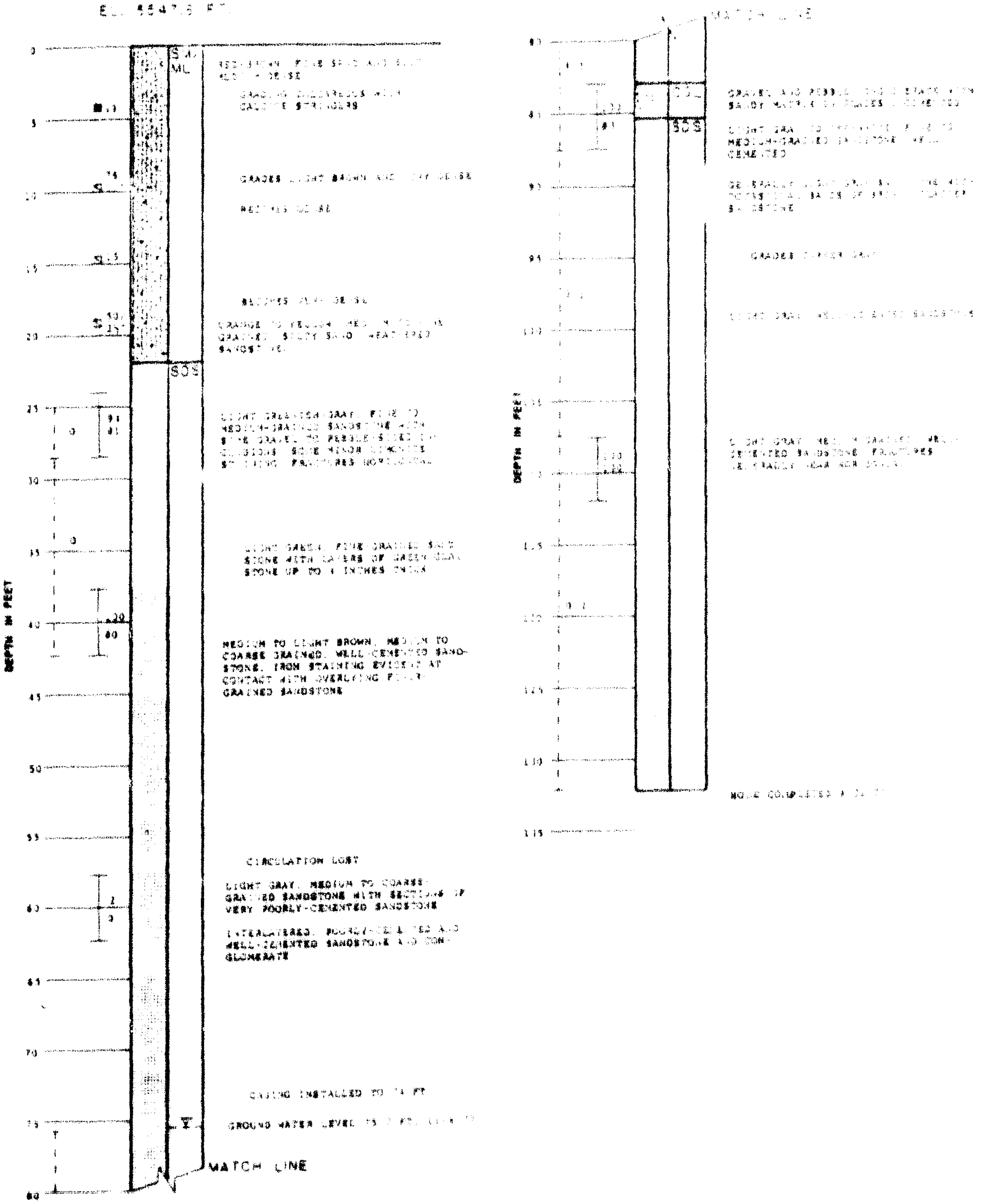


LOG OF BORINGS

DAMES & MOORE

PLATE A-10

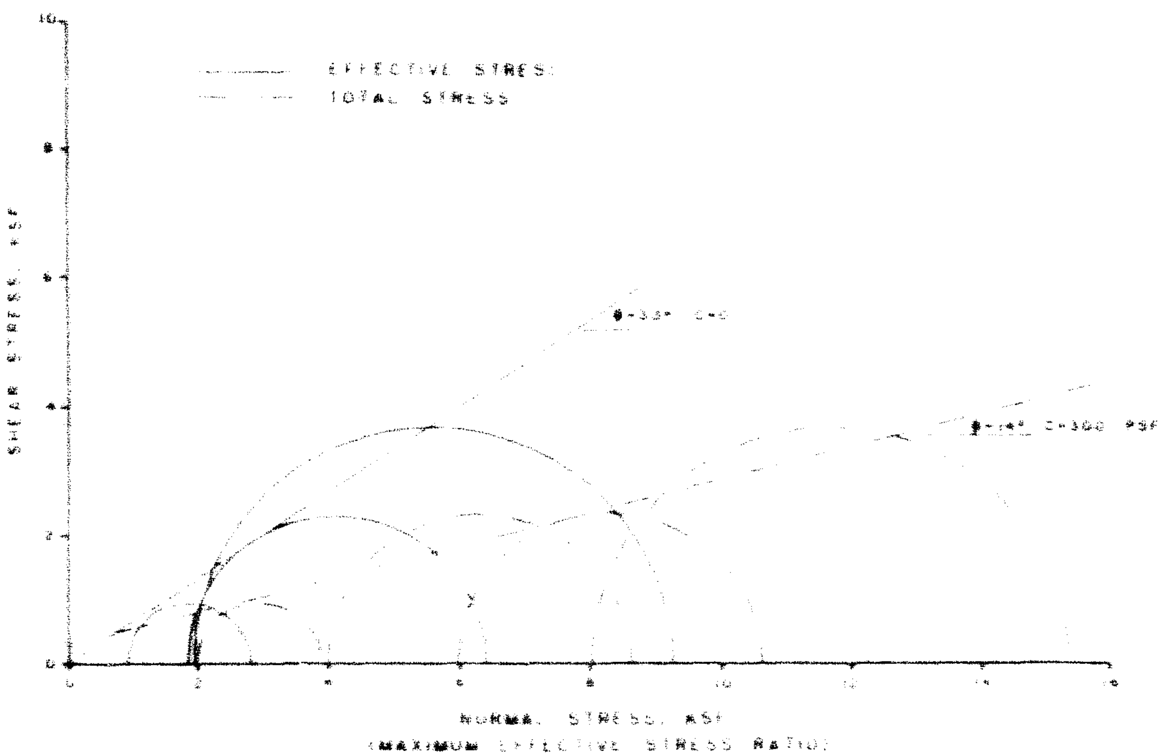
BORING NO. 28



LOG OF BORINGS

DAMES & MOORE

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



TRIAXIAL COMPRESSION TEST REPORT

TYPE OF TEST

TYPE MATERIAL

SAMPLE DESCRIPTION

CLASSIFICATION

LIQUID LIMIT PLASTIC LIMIT SPECIFIC GRAVITY

PROJECT

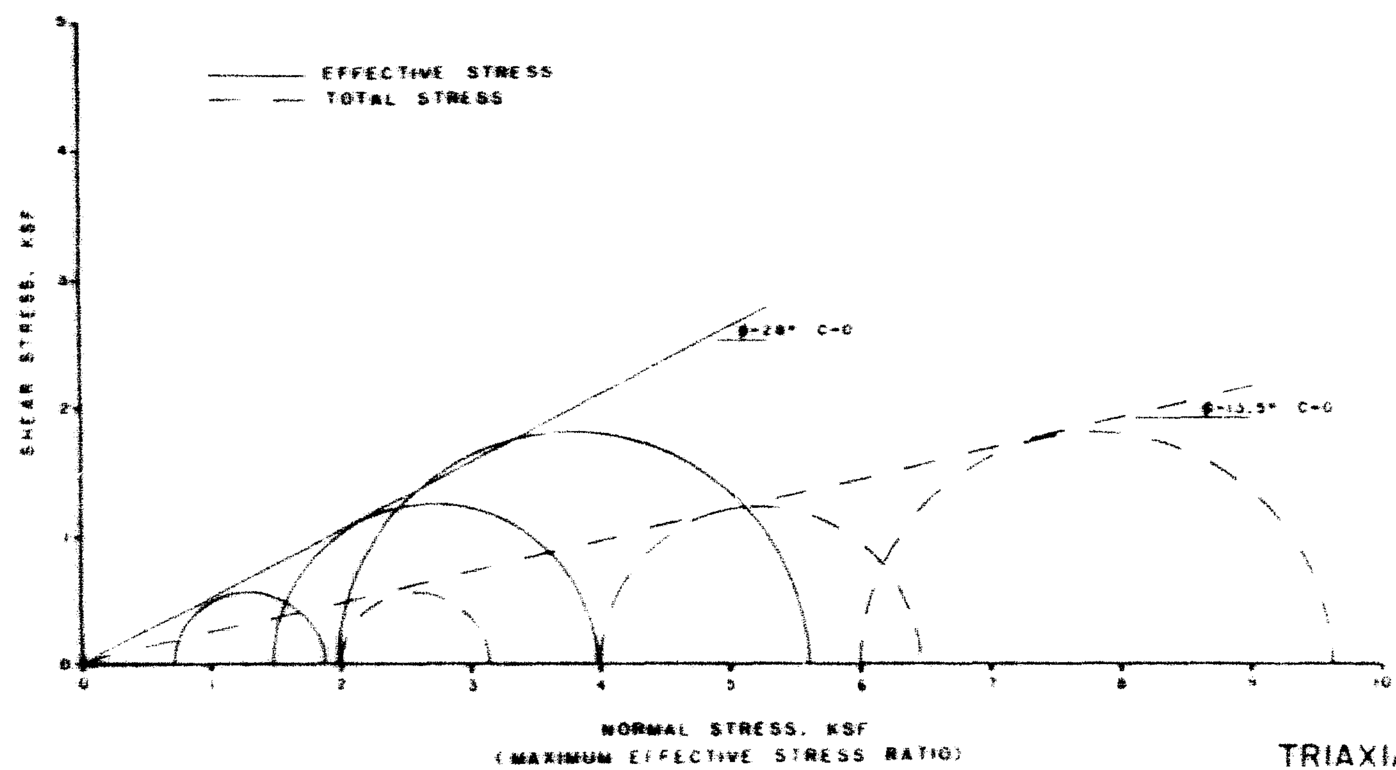
LOCATION

JOB NO. PREPARED BY

CHECKED BY

PLATE

MULTI PHASE TRIAXIAL COMPRESSION TESTS ON SILTY FINE SAND AT NATURAL DENSITY



		①		②		③	
	BORING	14		14		14	
	SAMPLE	1		1		1	
	DEPTH (FEET)	4'		4'		4'	
INITIAL	w, %	3.2		14.7		17.7	
	γ_d, PCF	104.7		108.6		112.6	
	e_0	.5803		.5317		.4693	
	S	.15		1.0		1.0	
FINAL	w, %	19.7		17.7		17.6	
	γ_d, PCF	108.6		112.6		113.7	
	e_f	.5327		.4693		.4542	
	S	1.0		1.0		1.0	
BACK PRESSURE (PSF)							
STRAIN RATE (INCHES/MINUTE)							
STRESS CONDITION							
TOTAL STRESS	ϵ, %	1.5	2.5	1.0	1.5	1.0	2.0
	TIME TO FAIL (MIN)	15	25	10	15	10	19
	σ_3, PSF	2000	2000	4000	4000	6000	6000
	$\sigma_1-\sigma_3$	1171	1160	2484	2467	3744	3620
	σ_1, PSF	3171	3160	6484	6467	9744	9620
	$\frac{1}{2}(\sigma_1-\sigma_3)$	585	580	1242	1234	1884	1814
	$\frac{1}{2}(\sigma_1+\sigma_3)$	2585	2580	5242	5234	7882	7814
	u, PSF	1166	1282	2518	2506	3443	4016
	A, $u/(\sigma_1-\sigma_3)$	.996	1.11	.953	1.02	.919	[illegible]
EFFECTIVE STRESS	ϵ, %	1.5	2.5	1.0	1.5	1.0	2.0
	TIME TO FAIL (MIN)	15	25	10	1500	10	19
	$\bar{\sigma}_3$, PSF	834	718	1482	1494	2532	1982
	$\bar{\sigma}_1-\bar{\sigma}_3$	1171	1160	2484	2467	3744	3620
	$\bar{\sigma}_1$, PSF	2005	1878	3966	3961	6322	5640
	$\frac{1}{2}(\bar{\sigma}_1-\bar{\sigma}_3)$	585	580	1242	1234	1882	1814
	$\frac{1}{2}(\bar{\sigma}_1+\bar{\sigma}_3)$	1420	1298	2724	2728	4410	3796
	u, PSF	1166	1282	2518	2506	3443	4018
	A, $u/(\sigma_1-\sigma_3)$	.956	1.11	.943	1.02	.919	1.11
	$\bar{\sigma}_1/\bar{\sigma}_3$	2.40	2.62	2.48	2.65	2.47	2.85

TRIAXIAL COMPRESSION TEST REPORT

TYPE OF TEST Tx-CU-PP

TYPE MATERIAL Brn Silt & F. Sand

SAMPLE DESCRIPTION

CLASSIFICATION SM/ML

LIQUID LIMIT N/A PLASTIC LIMIT N/A SPECIFIC GRAVITY G_s 2.65 assumed

PROJECT Energy Fuels

LOCATION Blanding, UT

JOB NO. [illegible] PREPARED BY [illegible]

CHECKED BY

PLATE B-12

SECTION 5

TEST PIT LOGS

From

Engineer's Report

Second Phase Design – Cell 3 Tailings Management System

White Mesa Uranium Project
Blanding, Utah

Energy Fuels Nuclear, Inc.
Denver, Colorado

TABLE OF CONTENTS

DATE BEGAN: [illegible]

DATE FINISHED: 2/1-/81

GROUND SURFACE EL: ~5594'

TEST PIT NO. [illegible]

N ~319,690' E~2,578,206'

FIELD ENGINEER R. Greenwood

CHECKED BY R. Greenwood

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS
	1.0		Medium stiff red sandy silt. Dry. Roots to depth — 1.5'	SAMPLE NO. 1, DEPTH 10"-12", TYPE JAR
	2.0		Very stiff red-white sandy silt. Dry. Very calcareous — 2.8'	SAMPLE NO. 2, DEPTH 22"-24", TYPE JAR
	3.0		Soft-medium stiff red sandy silt. Moist, w/streaks of white. Slightly calcereous	
5590	4.0			SAMPLE NO. 3, DEPTH 4'6", TYPE JAR
	5.0			SAMPLE NO. A, DEPTH 5'-5'6", TYPE BAG; 3.8% $CaCO_3$
	6.0			
	7.0			
5586	8.0			SAMPLE NO. 4, DEPTH 8'4", TYPE JAR
	9.0		— 9.0'	
	10.0		Very soft greenish gray claystone. Moist, weathered	SAMPLE NO. 5, DEPTH 10'4", TYPE JAR
	11.0			Consistency of soil but with intact layering
5582	12.0			SAMPLE NO. B, DEPTH 10'-10'6", TYPE BAG; 9.55% $CaCO_3$

DATE BEGAN: [illegible]

TE FINISHED: [illegible]

GROUND SURFACE EL: ~5594'

TEST PIT NO [illegible]

N ~319,690' E~2,578,206 '

FIELD ENGINEER R. Greenwood

CHECKED BY R. Greenwood

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS
	13.0		"Same as above" 13.5'	
	14.0		Soft yellowish brown sandstone. Weathered	Intact, Easy rippable
	15.0			Ripping becomes difficult
5578	16.0		16.0'	
			Bottom of test pit 16.0'	

DATE BEGAN: 1/24/81 | TEST PIT NO. 4-2 | FIELD ENGINEER R. Greenwood
DATE FINISHED: 1/24/81 | | CHECKED BY R. Greenwood
GROUND SURFACE EL. ~5583' | N ~319,950' E ~2,577,955'

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS: SAMPLE NO.	DEPTH	TYPE
	1.0		Red sandy silt. Dry w/ roots to depth			
	2.0		2.0'			
5580	3.0		Medium dense red silty sand. Moist.	A 2.3% $CaCO_3$	2'-5'	BAG
				1	3'	JAR
	4.0					
	5.0		5.7'			
	6.0		Soft yellowish brown sandstone	2	5'8"	JAR
5576	7.0		7.0'			
			Bottom of test pit 7.0'			

DATE BEGAN: 1/14/81

DATE FINISHED: 1/14/81

GROUND SURFACE EL: ~5582'

TEST PIT NO. 4-3

N~319,552' E~2,577,950'

FIELD ENGINEER R. Greenwood

CHECKED BY R. Greenwood

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS: SAMPLE NO.	DEPTH	TYPE
	1.0		Medium dense red sandy silt. Dry. Roots to depth 1.3'			
5580	2.0		Medium dense red-white silty sand. Very calcereous 3.3'	1	1'8"	JAR
	3.0			A 21.6% $CaCO_3$	2'-3'½"	BAG
5578	4.0		Soft greenish gray to yellowish brown claystone-sandstone. Weathered 5.0'	2	3'9"	JAR
	5.0					
			Bottom of test pit 5.0'			

DATE BEGAN: 1/14/8[illegible]

TE FINISHED: 1/14/81

GROUND SURFACE EL: ~5581'

TEST PIT NO. 4-4

N~310,740' E~2,577,530'

FIELD ENGINEER R. Greenwood

CHECKED BY R. Greenwood

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS		
				SAMPLE NO.	DEPTH	TYPE
5580	1.0		Medium dense red sandy silt. Dry. Roots to depth 1.5'			
	2.0		Dense red-white silty sand. Slightly calcareous. Dry 2.7'	1	2'1"	JAR
	3.0 4.0		Medium dense red silty sand. Moist	A 1.7% $CaCO_3$	3'	BAG
5576	5.0 6.0 7.0 8.0		8.0'	2	6'	JAR
5572	9.0 10.0 11.0 12.0		Soft yellowish brown sandstone. Weathered 12.0'			
			Bottom of test pit 12.0'			

ENERGY FUELS NUCLEAR RM78-682-C

TEST PIT NO. 4-4 SHEET 1 OF 1

DATE BEGAN: [illegible]

DATE FINISHED: 1/14/81

GROUND SURFACE EL: ~5578'

TEST PIT NO. 4-5

N ~319,760' E ~2,576,743'

FIELD ENGINEER: R. Greenwood

CHECKED BY: R. Greenwood

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS		
				SAMPLE NO.	DEPTH	TYPE
	1.0		Medium stiff red sandy silt. Dry. Roots to depth 1.5'			
5576	2.0		Dense red silty sand. Dry. Slightly calcareous	1	2'	JAR
	3.0					
	4.0		3.8'	A 2.9% $CaCO_3$	3'-5'	BAG
	5.0		Medium dense red silty sand. Moist 5.0'	2	4'	JAR
5572	6.0		Soft yellowish brown sandstone			
	7.0		7.0'			
			Bottom of test pit 7.0'			

DATE BEGAN: [illegible]

DATE FINISHED: 1/14/'81

GROUND SURFACE EL: ~[illegible]'

TEST PIT NO. 4-6

N ~[illegible]' E ~2,5[illegible],079'

FIELD ENGINEER R. Greenwood

CHECKED BY R. Greenwood

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS: 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	3'	JAR
			3.7'			
	4.0		Medium dense red silty sand. Moist	A 3% $CaCO_3$	4'	BAG
5574	5.0			2	4.5'	JAR
	6.0					
	7.0					
	8.0		8.0'			
			Soft yellowish brown sandstone 8.5'			
			Bottom of test pit 8.5'			

DATE BEGAN: [illegible]

TE FINISHED: 1/15/81

GROUND SURFACE EL: ~5584'

TEST PIT NO. 4-7

N ~322,030' E~2,576,346'

FIELD ENGINEER [illegible]

CHECKED BY R. Greenwood

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS: SAMPLE NO.	DEPTH	TYPE
	1.0		Medium stiff red sandy silt. Dry. Roots to depth			
	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
				A 29.7% $CaCO_3$	3'6"-6'	BAG
	5.0					
	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: 1/25/81 TEST PIT NO 4-8 FIELD ENGINEER R. Greenwood

DATE FINISHED: 1/25/81 CHECKED BY R. Greenwood

GROUND SURFACE EL.: ~5589' N ~319,916' E ~2,576,056'

ELEV (FEET)	DEPTH (FEET)	DESCRIPTION	SAMPLE NO.	DEPTH	TYPE
5588	1.0	Medium stiff red sandy silt. Dry. Roots to depth 1.7'			
	2.0	Dense red silty sand. Dry. Some with white streaks 2.7'	1	2'	JAR
	3.0	Loose red silty sand. Moist. Some white streaks	A 7.2% $CaCO_3$	3'-6'	BAG
	4.0		2	4'5"	JAR
5584	5.0	5.0'			
	6.0	Dense whitish red silty sand. Moderately calcareous	3	6'	JAR
	7.0				
	8.0				
5380	9.0	9.0'			
	10.0	Soft yellowish brown sandstone 10.0'			
		Bottom of test pit 10.0'			

DATE BEGAN: [illegible]	TEST PIT NO. 4-9	FIELD ENGINEER: [illegible]
DATE FINISHED: 1/15/81		CHECKED BY: R. [illegible]
GROUND SURFACE EL: ~5594'	N ~[illegible]' E ~[illegible]'	

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	SAMPLE NO.	DEPTH	TYPE
	1.0		Medium stiff red sandy silt. Dry. Roots to depth — 1.7'			
	2.0, 3.0		Dense red silty sand. Dry. White streaks. Slightly calcareous — 3.0'	1	2'	JAR
5590	4.0		Stiff redish white sandy silt. Dry. Very calcareous — 4.3'	2	4'	JAR
	5.0, 6.0		Medium dense red silty sand. Moist. Some white streaks. Slightly calcareous — 6.0'	A 24% $CaCO_3$	4'-8'	BAG
5586	7.0, 8.0		Dense redish white silty sand. Chalky. Very calcareous. Dry — 8.0'	3	7'	JAR
	9.0		Very soft yellowish brown sandstone. Weathered			
	10.0, 11.0		Medium hard yellowish brown sandstone — 11.0'			
			Bottom of test pit 11.0'			

DATE BEGAN: 1/14/81
ATE FINISHED: 1/14/81
GROUND SURFACE EL: ~ 5577'

TEST PIT NO D--1

N ~[illegible]' E~2,577,830'

FIELD ENGINEER R. Greenwood
CHECKED BY R. Greenwood

ELEV (FEET)	DEPTH (FEET)	PROFILE	DESCRIPTION	REMARKS		
				SAMPLE NO.	DEPTH	TYPE
	1.0		Stiff red sandy silt. Dry. Roots to depth 1.3'			
	2.0		Dense red silty sand. Dry. 1.9'			
5574	3.0		Medium dense red silty sand. Moist	1	2'4"	JAR
	4.0		4.3'	ST-1	3'-5'	3" Shelby
				2	4'2"	JAR
	5.0		Very soft greenish gray claystone. Moist			
	6.0					
5570	7.0		7.0'			
	8.0		Soft yellowish brown sandstone 8.0'	3	7'2"	JAR
			Bottom of test pit 8.0'			

DATE BEGAN: [illegible]

[DA]TE FINISHED: [illegible]

GROUND SURFACE EL: ~ 55[illegible]'

TEST PIT NO. [illegible]

N ~ [illegible]' E ~ [illegible]'

FIELD ENGINEER R. Greenwood

CHECKED BY R. Greenwood

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

DATE BEGAN: 1/15/81
DATE FINISHED: 1/15/81
GROUND SURFACE EL.: ~5578'

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
	1.0		Medium stiff red sandy silt. Dry. Roots to depth 1.0'			
			Dense red silty sand. Dry. Some roots 1.7'			
	2.0		Dense red silty sand. Dry. Streaks of white. Slightly calcareous	1	2'	JAR
	3.0		3.3'			
5574	4.0		Medium dense red silty sand. Moist	ST-1	3'6"-5'6"	3"Shelby
	5.0			2	5'	JAR
	6.0		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

Table C-1

SUMMARY OF LABORATORY TEST RESULTS

Cell 4

TECHNICIAN ______________________ PROJECT NO. ______________________

APPROVED BY ______________________ DATE ______________________

SAMPLE NO.	NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (pcf)	GRADATION		PERCENT PASSING NO 200 SIEVE	ATTERBERG LIMITS		SOIL OR BEDROCK TYPE	COMMENTS
			GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTICITY INDEX (%)		
TH 19 @ 0-3					70	23	6	Sandy clayey silt	
TH 23 @ 1-3					87	48	24	Weathered claystone	
TH 23 @ 6-8					96	61	30	Claystone	
TH 25 @ 1-3½	13.13				57	26	9	Sandy clay	
TH 26 @ 4½-5	15.3				91	41	20	Weathered claystone	
TH 28 @ 0-2	12.7				72	28	10	Sandy clay	
TH 29 @ 2-3	8.5				59	19	2	Sandy clayey silt	
TH 48 @ 5-5½					65	30	7	Sandy clay	
TH 49 @ 5-7					71	25	9	Sandy clay	
TH 49 @ 14-15					55	[illegible]	5	Calc. sandy silt	
TH 58 @ 5½-6	12.5				75	35	11	Sandy clay	
TH 76 @ 0-1	4.5				78	21	5	Sandy silt	
TH 76 @ 9½-10	4.4				26	NP	--	Silty, gravelly sand	
TH 77 @ 7½-8	8.6				71	30	15	Sandy clay	
TH 79 @ 0-1	4.1				83	20	5	Sandy silt	
TH 79 @ 5-5½	5.5				41	NP	---	Calc. silty sand	

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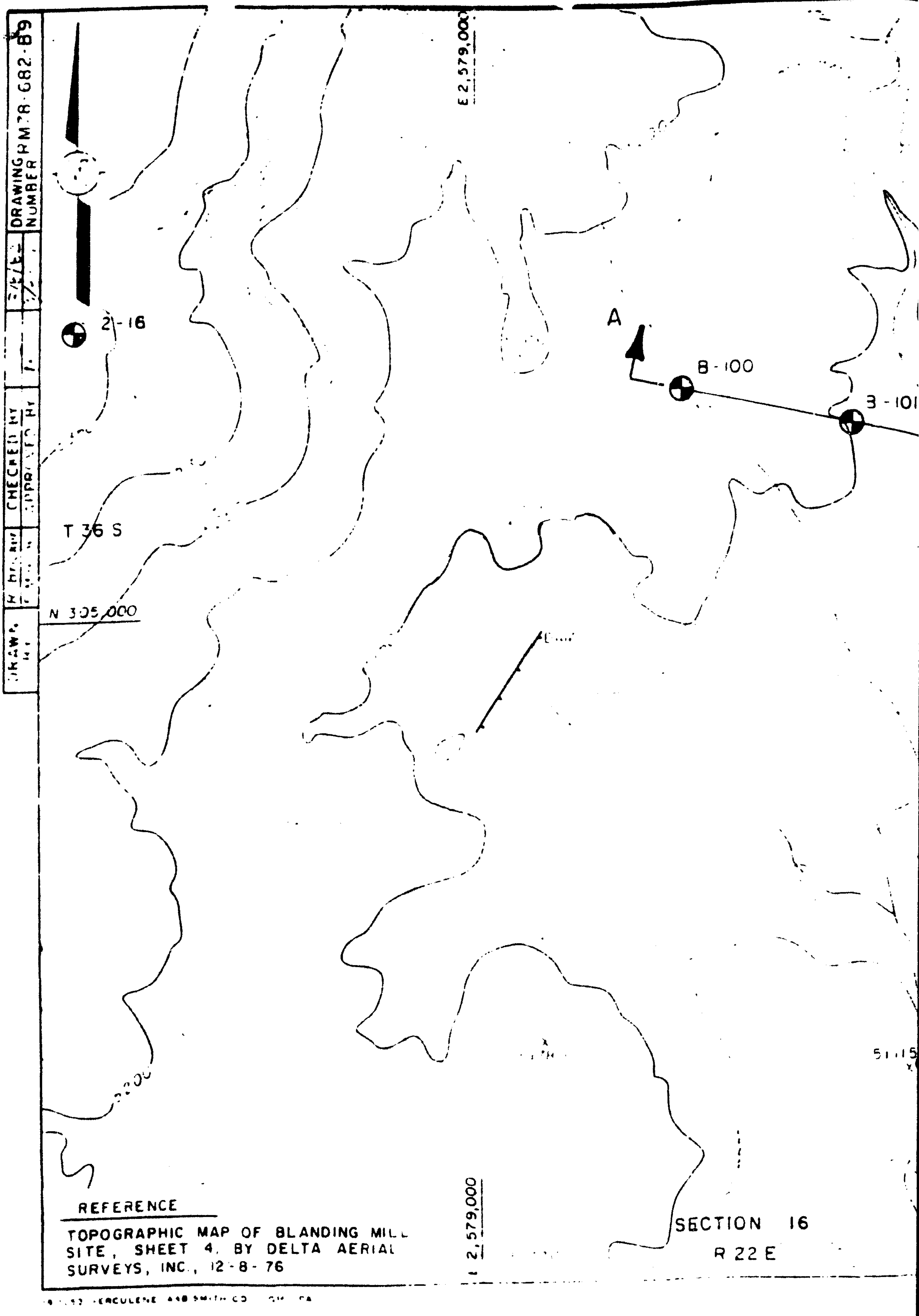
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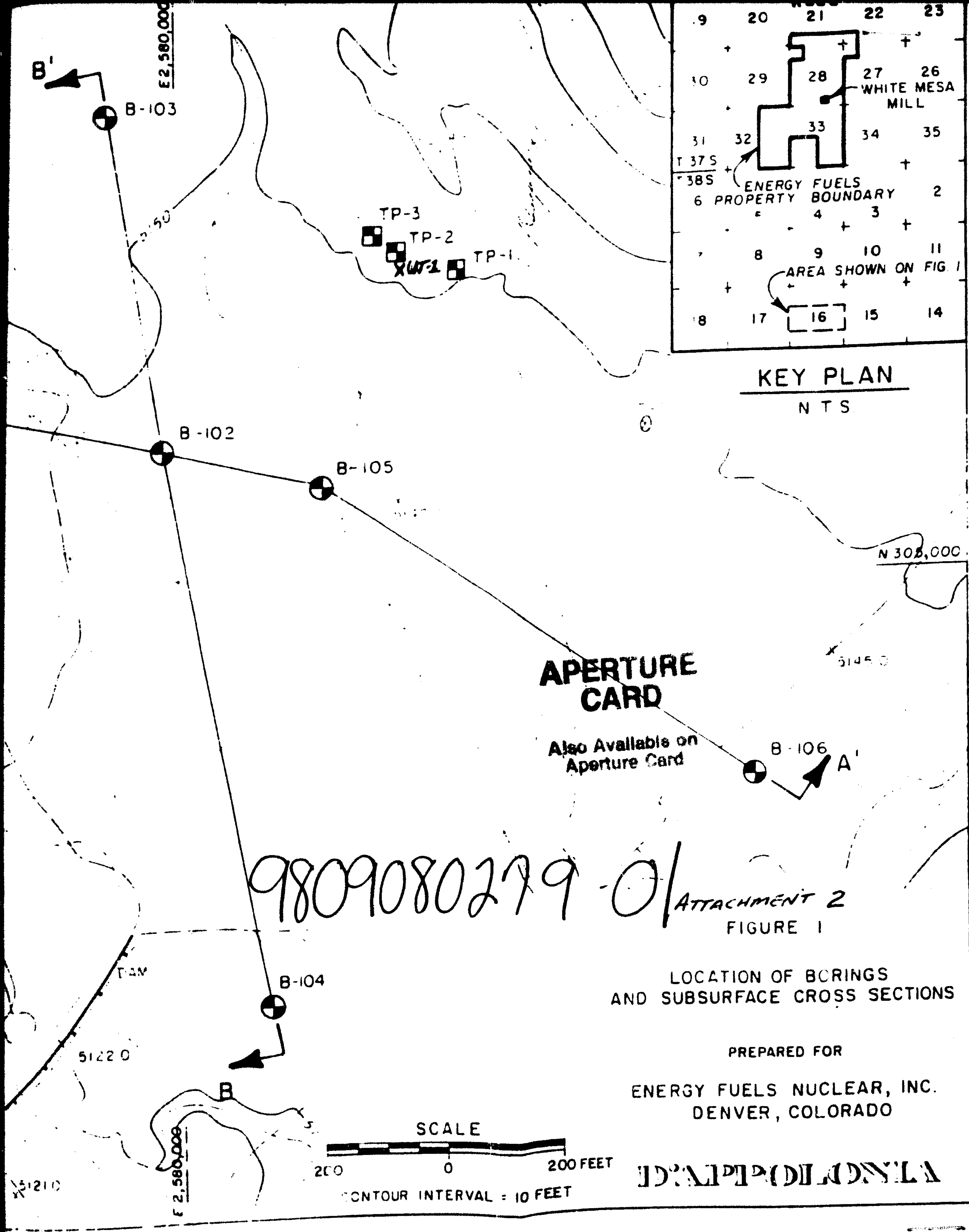

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ATTACHMENT 2

FIGURE 1

LOCATION OF BORINGS AND SUBSURFACE CROSS SECTIONS

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ATTACHMENT 3

ATTACHMENT 3

Magnitude Scaling Factors for Soil Liquefaction Evaluations

By Ignacio Arango,[1] 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 Berril (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 pressure 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 M (1)	Number of equivalent uniform cycles (2)	Magnitude scaling factor $\frac{\text{cyclic strength } M = M}{\text{cyclic strength } M = 7\text{-}1/2}$ (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

[1] Mgr., Geotech. Engrg., Bechtel Corp., 50 Beale St., San Francisco, CA 94119-3965.

Note. Discussion open until April 1, 1997. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on August 7, 1995. This paper is part of the *Journal of Geotechnical Engineering*, Vol. 122, No. 11, November, 1996. ©ASCE, ISSN 0733-9410/96/0011-0929–0936/$4.00 + $.50 per page. Paper No. 11350.

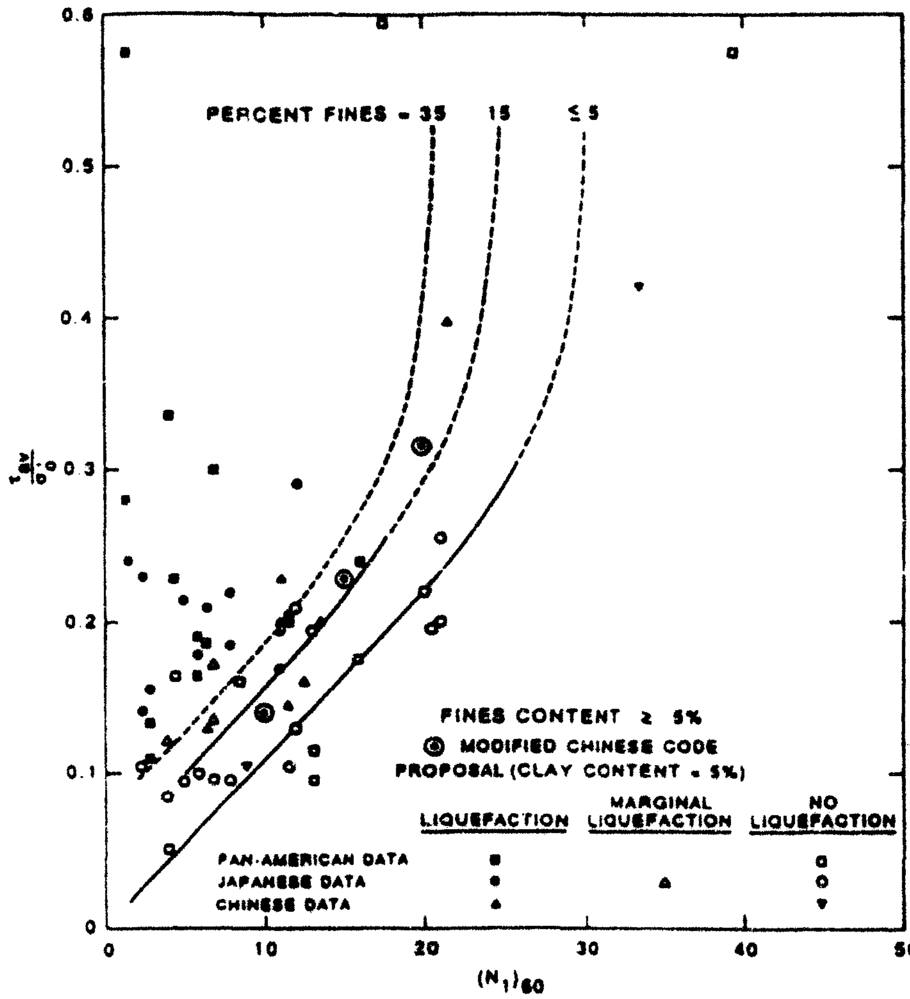

FIG. 1. Relationship between Stress Ratio Causing Liquefaction and $(N_1)_{60}$ Values for Silty Sands for M = 7.5 Earthquakes

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 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-

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

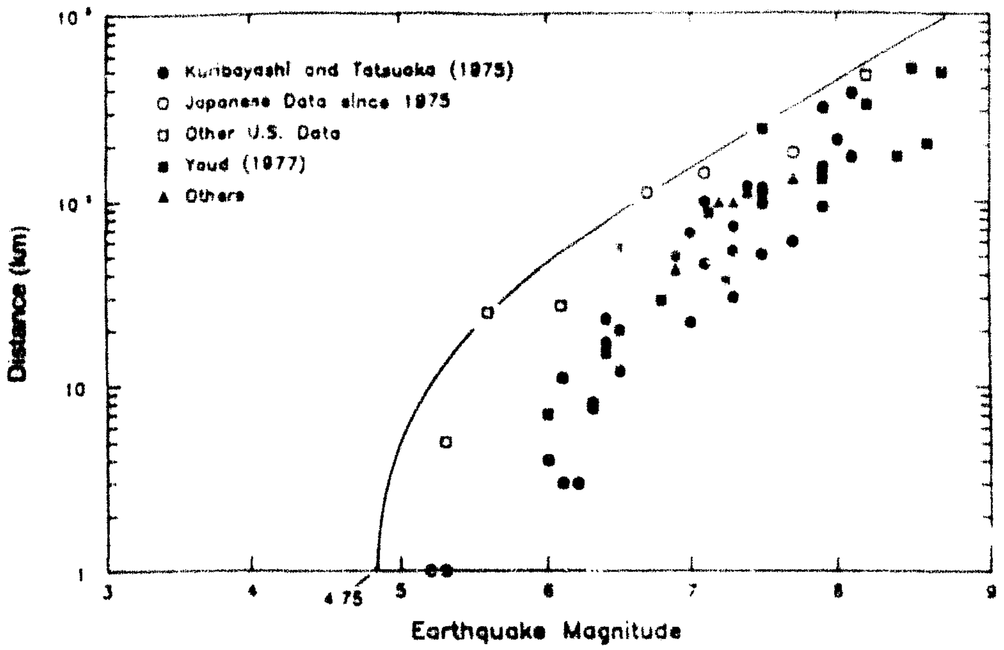

FIG. 2. Maximum Distance of Liquefaction from Zone of Faulting

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 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 farthermost 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 amplitudes 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

$$m\ddot{y} + ky = m\ddot{z} \tag{1}$$

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

$$\mathrm{TE} = \frac{1}{2} m\dot{y}^2 + \frac{1}{2} ky^2 \tag{2a}$$

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

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

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 (gs) (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

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

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, $(KE)_{MAX} = (SE)_{MAX}$ = 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

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

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 ρ 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.

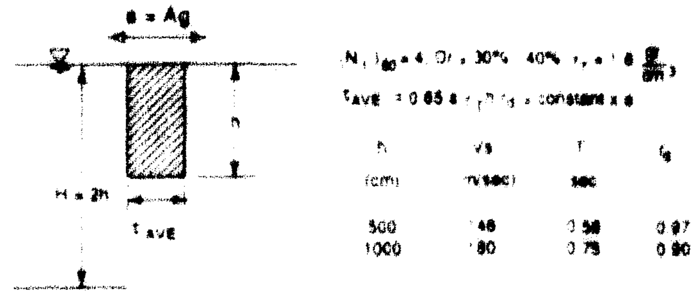

FIG. 3. Assumed Geotechnical Characteristics of Distant Liquefaction Sites

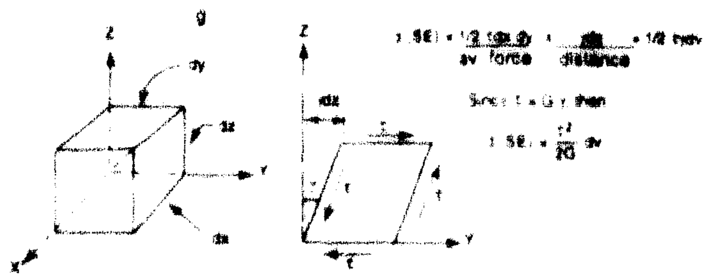

FIG. 4. Strain Energy

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 1.4×10^{-3} and 2.8×10^{-3} J/cm^3.

Figueroa 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 0.5×10^{-3} and 1.5×10^{-3} J/cm^3, 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 $(N_1)_{60}$ values near 2–4, similar predominant period), it is reasonable to assume that to liquefy all of these sites absorbed the same energy. Therefore,

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

$$N_{M_i+1} = N_{M_i} \times \frac{(TE)_{M_i}}{(TE)_{M_i+1}} = N_{M_i} \times \left(\frac{A_{M_i}}{A_{M_i+1}}\right)^2 \tag{9}$$

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

Adopting 15 as the number of significant stress cycles generated by earthquakes of magnitude $M = 7.5$ (Seed et al. 1975), (9) provides the equiv' 'nt number of cycles N_{M_i} 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 $(N_1)_{60} \approx 4$. 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 $(N_1)_{60}$ 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 $(N_1)_{60}$ 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 $(N_1)_{60}$ 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 $(N_1)_{60}$ 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 $(N_1)_{60}$ 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.

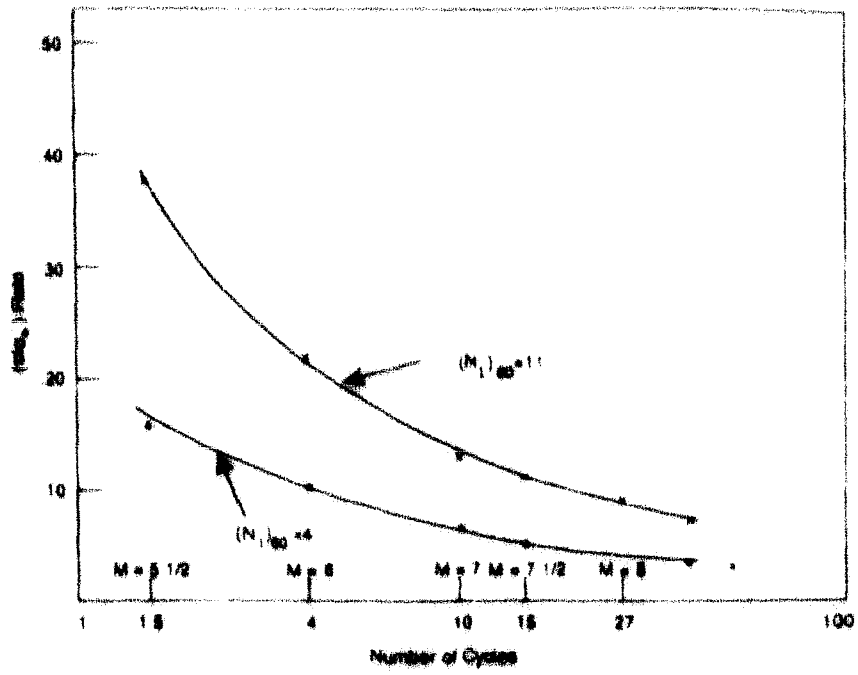

FIG. 5. Magnitude Scaling Factors Derived from Energy Concepts

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

Earthquake magnitude M (1)	Equivalent uniform number of cycles N_M (2)	Acceleration A (ge) (3)	Induced cyclic stress ratio $\frac{\tau}{\sigma'_0}$ (4)
8.25	38.4	0.025	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 $(N_1)_{60}$ value is well documented, and which liquefied under a magnitude 7.5 earthquake inducing an also well-documented ground surface acceleration equal to $Ag = a$. Other sites with the same geotechnical characteristics would liquefy nearby or at other locations during different earthquake magnitudes, M_i, 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

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

From (10) and (11), one obtains

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

The cyclic stress ratios inducing liquefaction in the field are therefore

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

and the MSF would be expressed as

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

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 8) 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

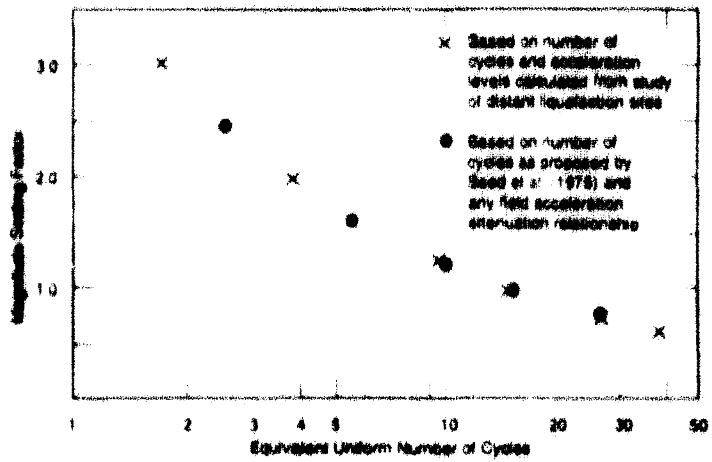

FIG. 6. Cyclic Stress Ratio Inducing Liquefaction in Field

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 aproaches 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.

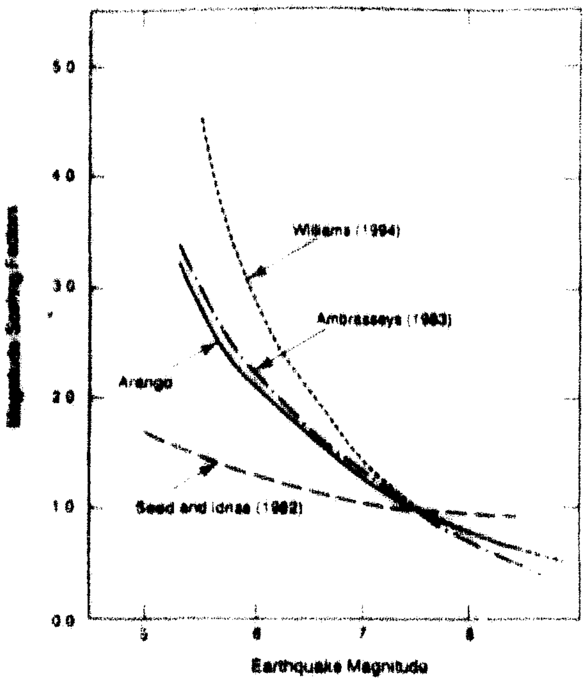

FIG. 7. Comparison of Earthquake Magnitude Scaling Factors from Various Sources

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

Earthquake			Sites with	
Year (1)	Magnitude (2)	Location (3)	Liquefaction (4)	No liquefaction (5)
1957	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 William's 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

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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.

Battis, 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 Berrill, J. B. (1983). "Comparison of a liquefaction theory with field observations." *Géotechnique*, 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.

Figueroa, J. L., Saada, A. S., Liang, L., and Dahisaria, 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." *Second Int. Conf. on Recent Adv. in Geotech. Earthquake Engr. 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 Saeki, 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.

Kuribayashi, E., and Tatsuoka, 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 gs;
A_{M_i} = acceleration under earthquake M_i;
a, $a(t)$ = acceleration in cm/s^2;
G, G_{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_M, N_{M_i} = 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;
$\ddot{z}$ = base acceleration in simple oscillator;
γ = shear strain,
γ_T = total unit weight of soil;
γ_w = unit weight of water;
γ = weight per unit volume;
σ_m = mean effective stress; and
ω_n = natural frequency of harmonic motion = $(k/m)^{1/2}$.

ATTACHMENT 4

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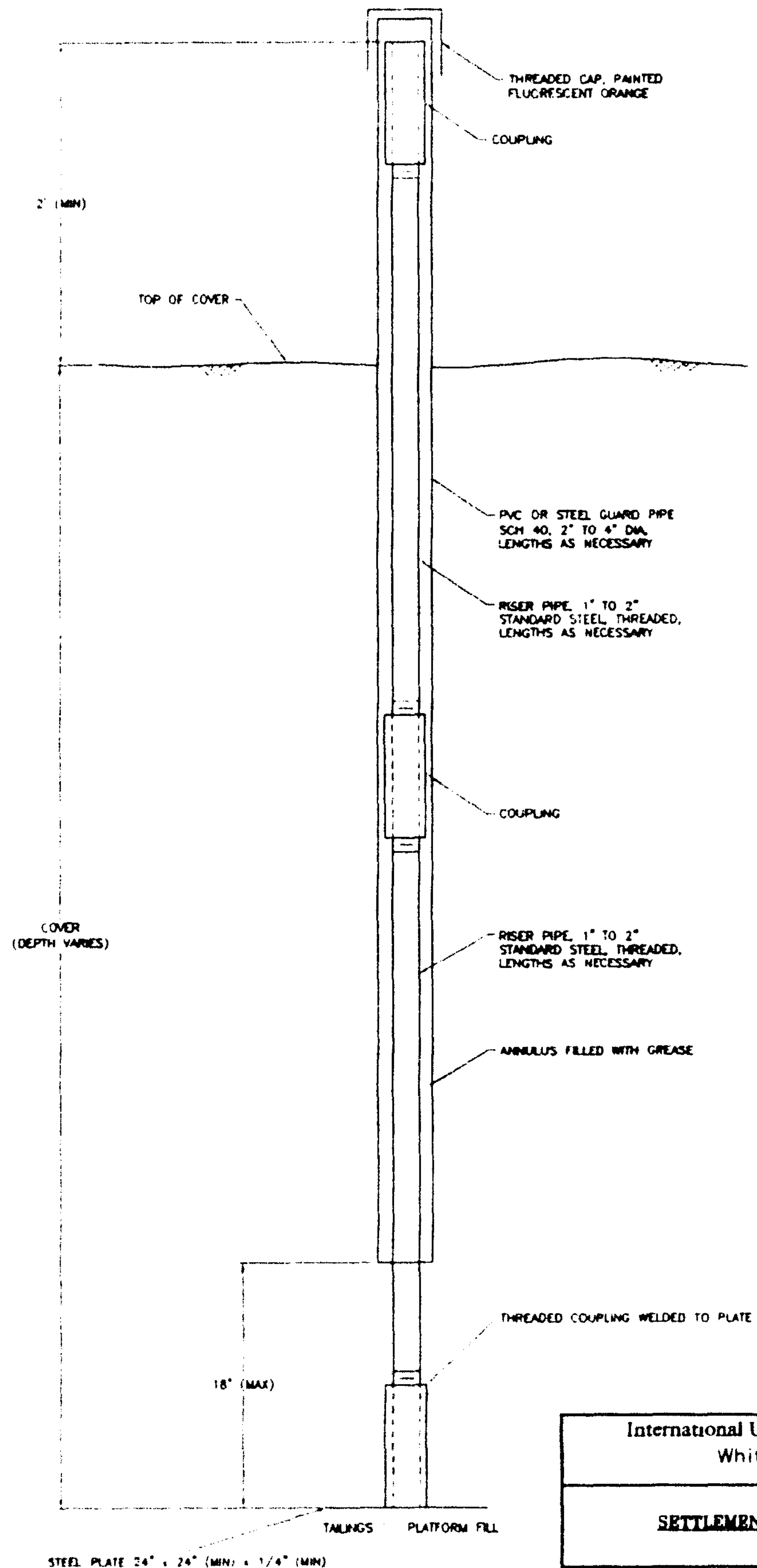

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International Uranium (USA) Corporation
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SETTLEMENT MONITORING POINT

FIGURE 2
ATTACHMENT 4

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CHKD BY:	DATE: 8/17/98	
APP: RAH	SCALE: NTS	

ATTACHMENT 4
SETTLEMENT DATA

ATTACHMENT 4 · SETTLEMENT DATA

Cell 2 East		Cell 2 West 1		Cell 2 West 2		Cell 2 West 3		Cell 2 West 4	
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08/25/89	562· 60								
09/08/89	5624 59								
09/29/89	5624 56								
10/20/89	5624 93								
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ATTACHMENT 4
SETTLEMENT DATA

ATTACHMENT 4 SETTLEMENT DATA

Cell 2 East

DATE	ELEVATION
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09/29/95	5624 07
10/19/95	5624 15
11/30/95	5624 80
12/21/95	5624 62
01/26/96	5624 05
03/01/96	5624 03
04/25/96	5624 05
05/30/96	5624 05
06/27/96	5624 05
07/26/96	5624 05
08/30/96	5624 03
09/27/96	5624 02
10/26/96	5624 03
11/27/96	5624 04
12/20/96	5624 04
01/31/97	5624
02/27/97	5624 06
03/27/97	5623 98
04/25/97	5624 02
05/30/97	5624 01
06/26/97	5624 00
07/31/97	5624 03
08/28/97	5624 01
09/25/97	5624 01
10/25/97	5624 01
11/18/97	5624 02
12/18/97	5624 00
01/16/98	5624 00
02/06/98	5624 03
03/06/98	5624 05
04/03/98	5623 98
05/06/98	5624 00
06/01/98	5624 00

Cell 2 West 1

DATE	ELEVATION
11/25/92	5619 60
12/03/92	5619 56
12/10/92	5619 55
12/16/92	5619 54
12/21/92	5619 71
12/31/92	5619 52
01/07/93	5619 45
01/14/93	
01/21/93	
01/28/93	
02/04/93	5619 45
02/11/93	5619 46
02/18/93	
02/25/93	
03/03/93	5619 47
03/10/93	5619 15
03/18/93	5619 47
03/25/93	5618 75
04/01/93	5619 27
04/08/93	5619 46
04/15/93	5619 45
04/22/93	
04/29/93	5619 47
06/30/93	5619 47
07/29/93	5619 44
08/26/93	5619 45
09/30/93	5619 42
10/27/93	5619 4
11/22/93	5619 41
12/28/93	5619 43
01/27/94	5619 4
02/24/94	5619 35
03/31/94	5619 38
05/05/94	5619 33
06/06/94	5619 33
07/29/94	5619 32
09/29/94	5619 31
10/28/94	5619 31
12/22/94	5619 3
01/27/95	5619 28
02/23/95	5619 29
03/31/95	5619 27
04/28/95	5619 26
06/02/95	5619 29
07/31/95	5619 29
09/01/95	5619 25
09/29/95	5619 23
10/19/95	5619 27
11/30/95	5619 27
12/21/95	5618 99
01/26/96	5619 27
03/01/96	5619 27
04/25/96	5618 89
05/30/96	5619 23
06/27/96	5619 26
07/27/96	5619 27
08/30/96	5619 26
09/27/96	5619 29
10/26/96	5619 24
11/27/96	5619 25
12/20/96	5619 28
01/31/97	5619 31
02/27/97	5619 3
03/27/97	5619 23
04/25/97	5619 2
05/25/97	5619 22
06/26/97	5619 22
07/31/97	5619 26
04/28/97	5619 2
09/25/97	5619 18
10/26/97	5619 21
10/26/97	5619 23
11/18/97	5619 23
12/18/97	5619 23
01/16/98	5619 23
02/06/98	5619 23
03/06/98	5619 22
04/03/98	5619 25
05/08/98	5619 25
06/01/98	5619 21

Cell 2 West 2

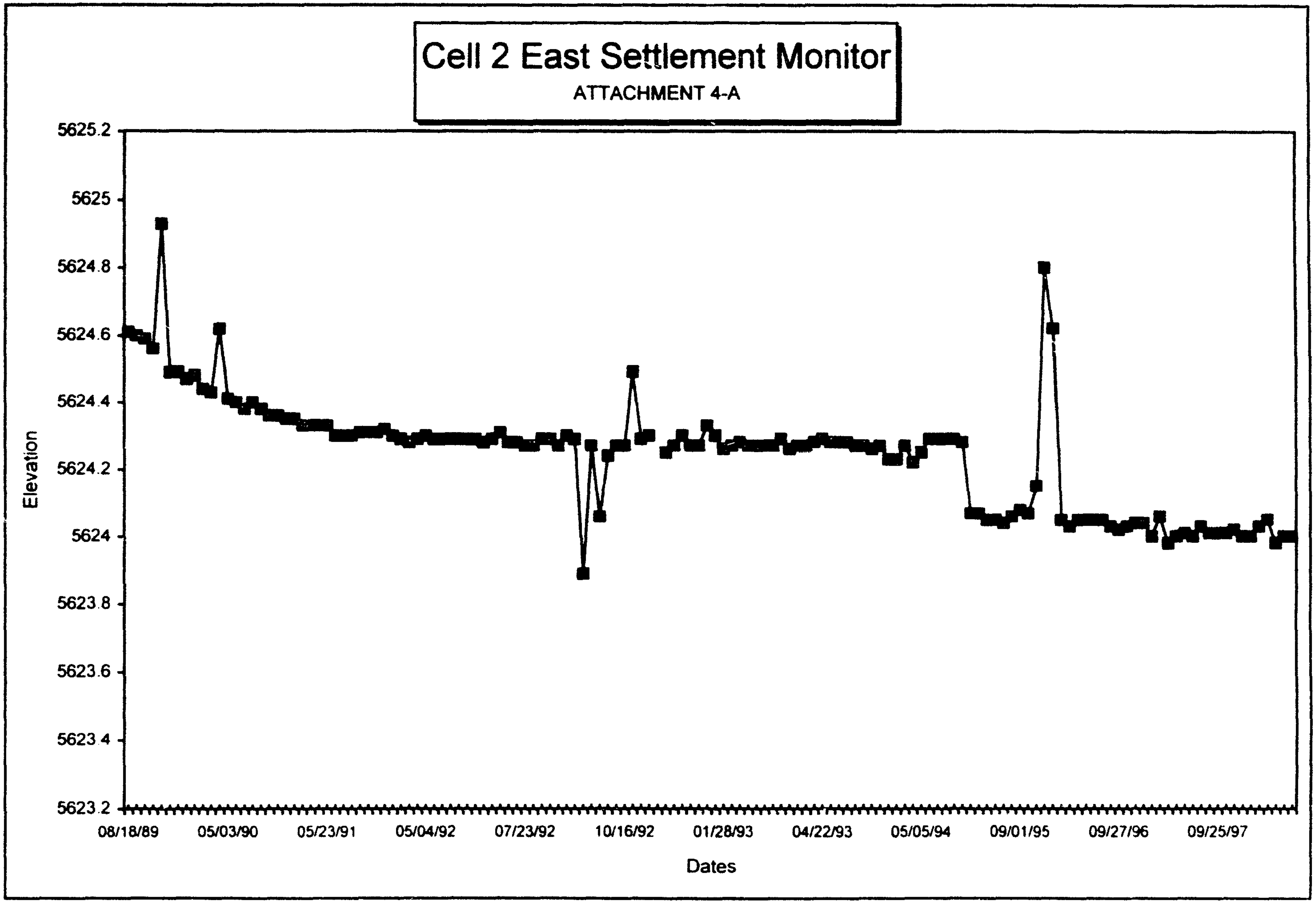
DATE	ELEVATION	
11/25/92	5617 28	Initial
12/03/92	5622 12	extension added
12/10/92	5622 03	
12/16/92	5622 08	
12/21/92	5622 11	
12/31/92	5622 07	
01/07/93	5621 83	
01/14/93		
01/21/93		
28/93		
02/04/93	5621 75	
02/11/93		
02/18/93		
02/25/93		
03/03/93	5621 79	
03/10/93	5621 41	
03/18/93	5621 75	
03/25/93	5620 99	
04/01/93	5621 47	
04/08/93	5621 68	
04/15/93	5621 67	
04/22/93	5621 67	
04/29/93	5621 55	
06/30/93	5621 6	
07/29/93	5621 56	
08/26/93	5621 51	
09/30/93	5621 46	
10/27/93	5621 58	
11/22/93	5621 41	
12/28/93	5621 41	
01/27/94	5621 37	
02/24/94	5621 35	
03/31/94	5621 33	
05/05/94	5621 32	
06/06/94	5621 31	
07/29/94	5621 31	
09/29/94	5621 28	
10/28/94	5621 27	
12/22/94	5621 24	
01/27/95	5621 26	
02/23/95	5621 26	
03/31/95	5621 25	
04/28/95	5621 25	
06/02/95	5621 24	
07/31/95	5621 25	
09/01/95	5621 22	
09/29/95	5621 23	
10/19/95	5621 25	
11/30/95	5621 21	
12/21/95	5620 93	
01/26/96	5621 21	
03/01/96	5621 19	
04/25/96	5621 21	
05/30/96	5621 17	
06/27/96	5621 19	
07/26/96	5621 2	
08/30/96	5621 19	
09/27/96	5621 17	
10/26/96	5621 17	
11/27/96	5621 17	
12/20/96	5621 17	
01/31/97	5621 23	
02/27/97	5621 21	
03/27/97	5621 15	
04/25/97	5621 15	
05/30/97	5621 16	
06/26/97	5621 16	
07/31/97	5621 21	
08/28/97	5621 16	
09/25/97	5621 15	
10/26/97	5620 84	
11/18/97	5621 17	
12/18/97	5621 16	
01/16/98	5621 16	
02/06/98	5621 15	
03/06/98	5621 15	
04/03/98	5621 19	
05/06/98	5621 18	
06/01/98	5621 15	

Cell 2 West 3

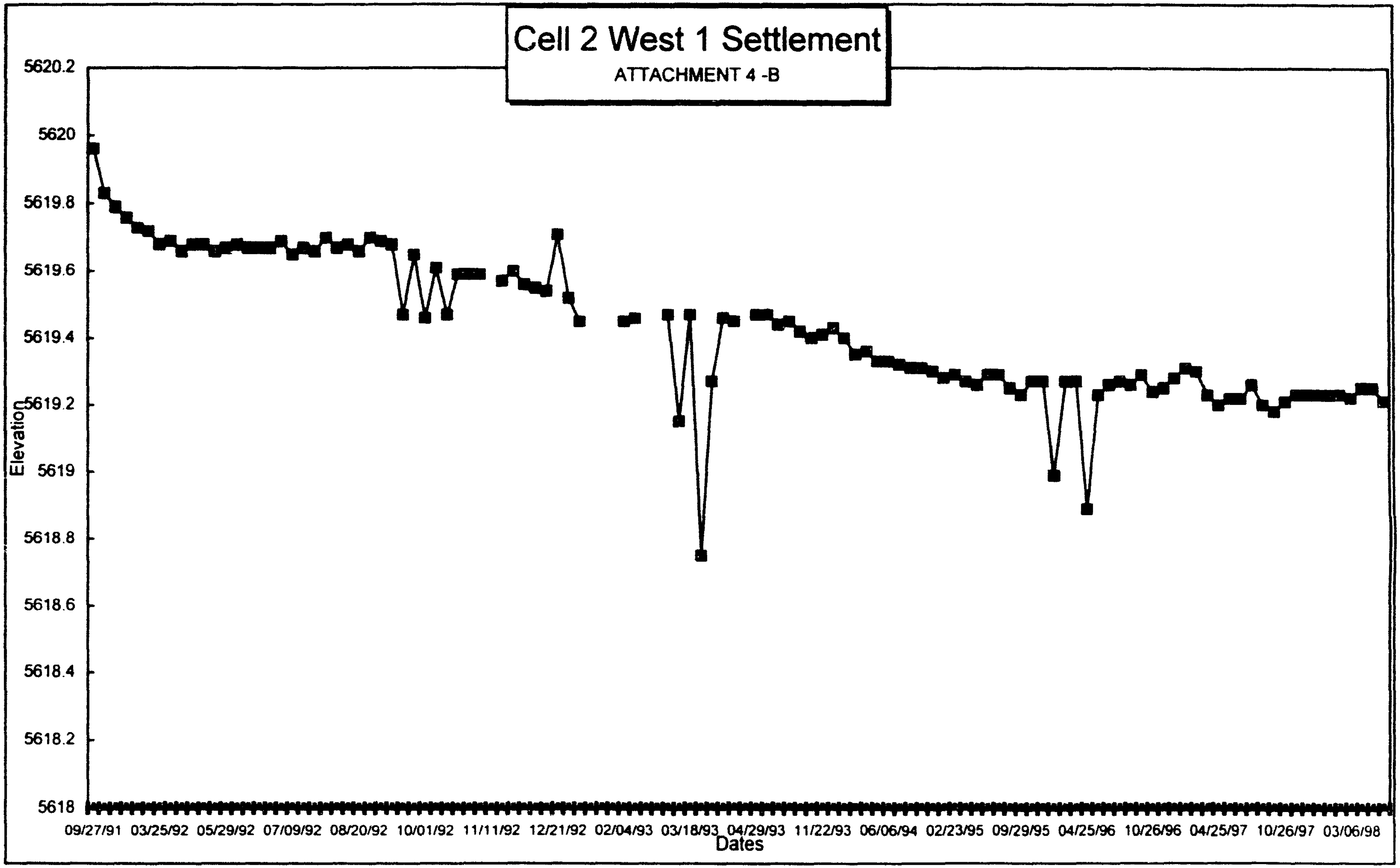
DATE	ELEVATION	
11/25/92	5619 38	Initial
12/03/92	5618 34	
12/10/92	5618 33	
12/16/92	5618 33	
12/21/92	5618 37	
12/31/92	5618 35	
01/07/93	5618 27	
01/14/93		
01/21/93		
01/28/93		
02/04/93	5618 27	
02/11/93		
02/18/93		
02/25/93		
03/03/93	5618 23	
03/10/93	5617 93	
03/18/93	5618 23	
03/25/93	5617 49	
04/01/93	5618 05	
04/08/93	5618 22	
04/15/93	5618 19	
04/22/93	5618 21	
04/29/93	5618 17	
06/30/93	5618 18	
07/29/93	5618 17	
08/26/93	5618 14	
09/30/93	5618 13	
10/27/93	5618 06	
11/22/93	5618 07	
12/28/93	5618 07	
01/27/94	5618 05	
02/24/94	5618 02	
03/31/94	5618 02	
05/05/94	5617 99	
06/06/94	5618 00	
07/29/94	5617 96	
09/29/94	5617 98	
10/28/94	5617 97	
12/22/94		
01/27/95	5617 96	
02/23/95	5617 95	
03/31/95	5617 93	
04/28/95	5617 87	
06/02/95	5617 93	
07/31/95	5618 14	
09/01/95	5617 91	
09/29/95	5617 93	
10/19/95	5617 99	
11/30/95	5617 95	
12/21/95	5617 61	
01/26/96	5617 92	
03/01/96	5617 89	
04/25/96	5617 91	
05/30/96	5617 88	
06/27/96	5617 89	
07/26/96	5617 90	
08/30/96	5617 87	
09/27/96	5617 93	
10/26/96	5617 90	
11/27/96	5617 89	
12/20/96	5617 90	
01/31/97	5617 94	
02/27/97	5617 94	
03/27/97	5617 86	
05/02/97	5617 86	
05/30/97	5617 85	
06/26/97	5617 85	
07/31/97	5617 94	
08/28/97	5617 88	
09/25/97	5617 87	
10/26/97	5617 89	
11/18/97	5617 87	
12/18/97	5617 90	
01/16/98	5617 89	
02/06/98	5617 88	
03/06/98	5617 87	
04/03/98	5617 85	
05/06/98	5617 89	
06/01/98	5617 89	

Cell 2 West 4

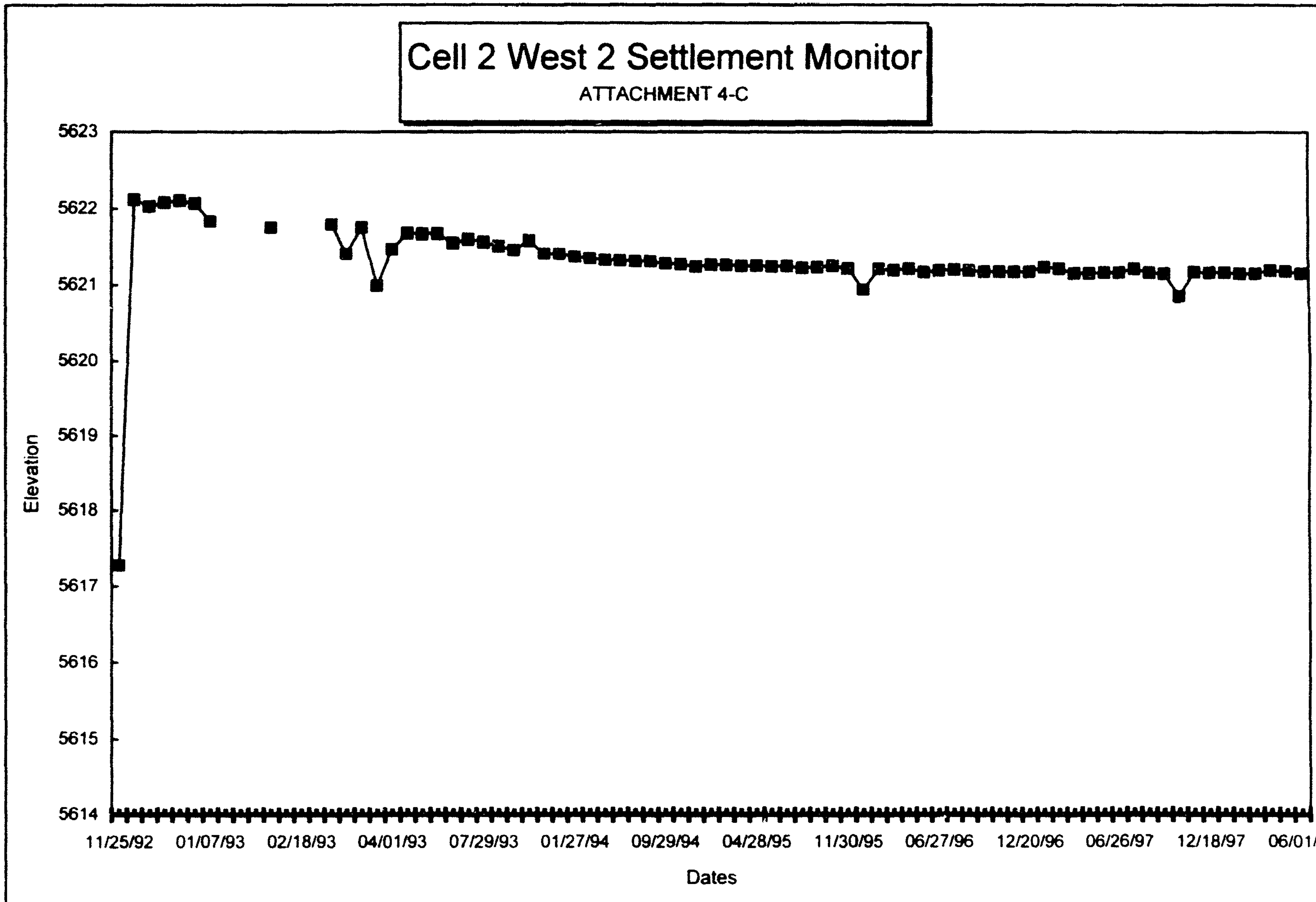
DATE	ELEVATION	
11/25/92	5618 38	Initial
12/03/92	5618 23	
12/10/92	5618 2	
12/16/92	5618 18	
12/21/92	5618 37	
12/31/92	5618 2	
01/07/93	5618 11	
01/14/93		
01/21/93		
01/28/93		
02/04/93	5618 13	
02/11/93		
02/18/93		
02/25/93		
03/03/93	5618 05	
03/10/93	5617 75	
03/18/93	5618 01	
03/25/93	5617 26	
04/01/93	5617 83	
04/08/93	5618 04	
04/15/93	5618 03	
04/22/93	5618 01	
04/29/93	5618 04	
06/30/93	5618 04	
07/29/93	5617 95	
08/26/93	5617 88	
09/30/93	5617 93	
10/27/93	5617 88	
11/22/93	5617 89	
12/28/93	5617 85	
01/27/94	5617 85	
02/24/94	5617 81	
03/31/94	5617 75	
05/05/94	5617 75	
06/06/94	5617 77	
07/29/94	5617 75	
09/29/94	5617 72	
10/28/94	5617 77	
12/22/94	5617 72	
01/27/95	5617 73	
02/23/95	5617 67	
03/31/95	5617 72	
04/28/95	5617 51	
06/02/95	5617 65	
07/31/95	5617 73	
09/01/95	5617 65	
09/29/95	5617 66	
10/19/95	5617 55	
11/30/95	5617 61	
12/21/95	5617 35	
01/26/96	5617 65	
03/01/96	5617 57	
04/25/96	5617 53	
05/30/96	5617 56	
06/27/96	5617 6	
07/26/96	5617 6	
08/30/96	5617 62	
09/27/96	5617 65	
10/26/96	5617 59	
11/27/96	5617 58	
12/20/96	5617 63	
01/31/97	5617 56	
02/27/97	5617 53	
03/27/97	5617 57	
04/25/97	5617 49	
05/30/97	5617 55	
06/26/97	5617 56	
07/31/97	5617 61	
08/28/97	5617 55	
09/25/97	5617 52	
10/26/97	5617 53	
11/18/97	5617 53	
12/18/97	5617 57	
01/16/98	5617 53	
02/06/98	5617 43	
03/06/98	5617 41	
04/03/98	5617 53	
05/06/98	5617 54	
06/01/98	5617 49	



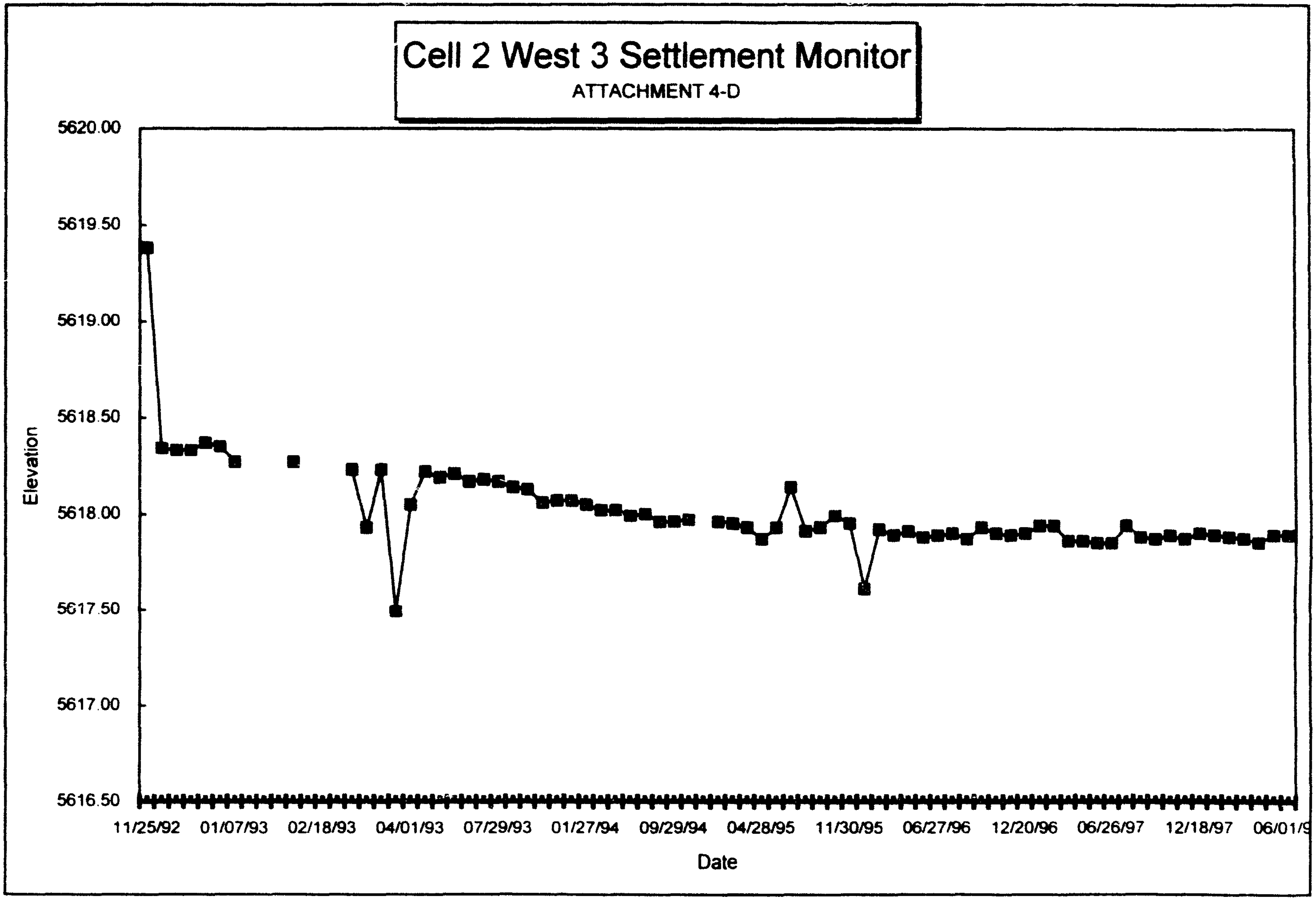
Cell 2 East Settlement Monitor
ATTACHMENT 4-A
5625.2
5625
5624.8
5624.6
5624.4
5624.2
5624
5623.8
5623.6
5623.4
5623.2
Elevation
08/18/89
05/03/90
05/23/91
05/04/92
07/23/92
10/16/92
01/28/93
04/22/93
05/05/94
09/01/95
09/27/96
09/25/97
Dates



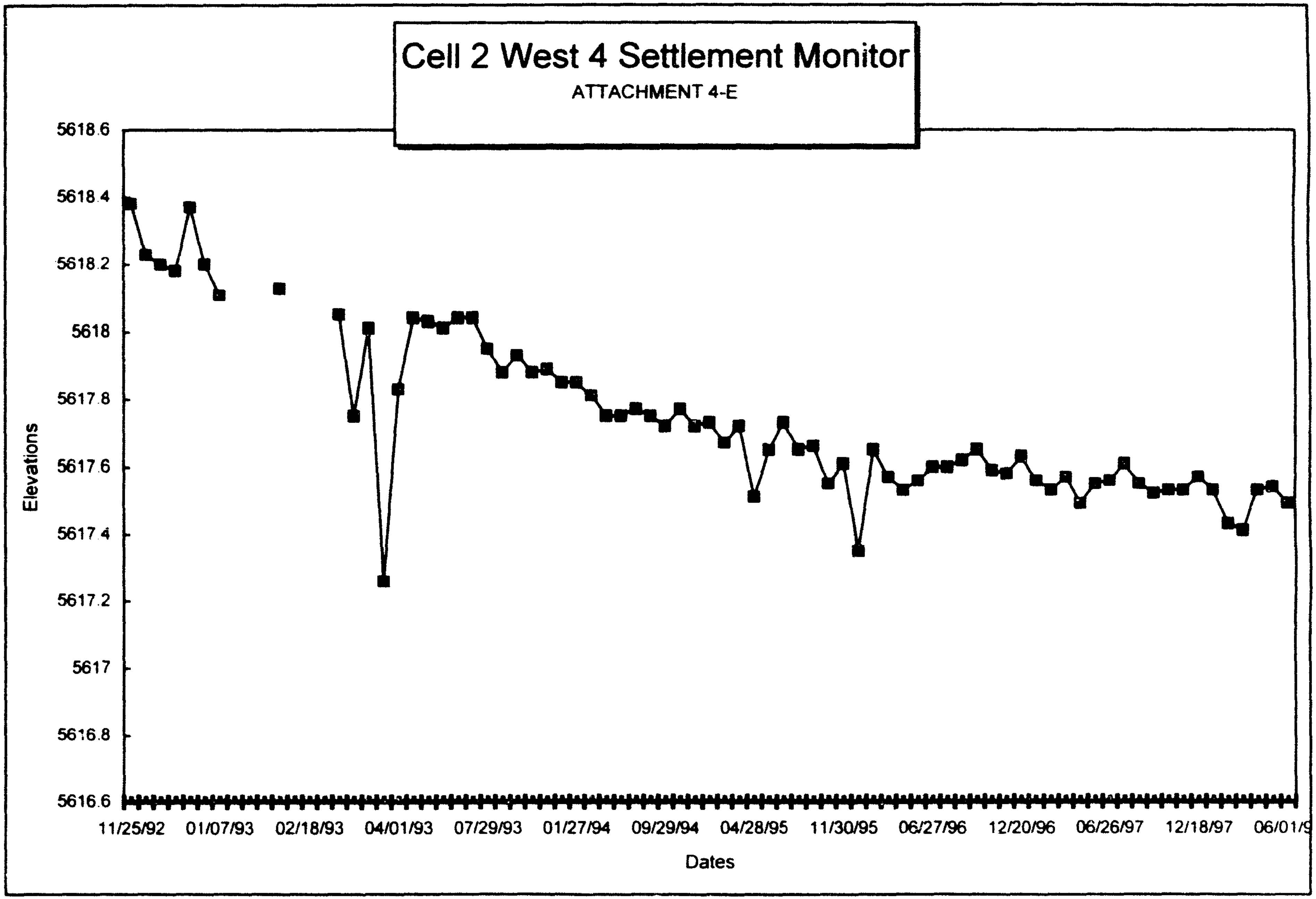
Cell 2 West 1 Settlement
ATTACHMENT 4 -B
5620.2
5620
5619.8
5619.6
5619.4
5619.2
5619
5618.8
5618.6
5618.4
5618.2
5618
Elevation
09/27/91 03/25/92 05/29/92 07/09/92 08/20/92 10/01/92 11/11/92 12/21/92 02/04/93 03/18/93 04/29/93 11/22/93 06/06/94 02/23/95 09/29/95 04/25/96 10/26/96 04/25/97 10/26/97 03/06/98
Dates



Cell 2 West 2 Settlement Monitor
ATTACHMENT 4-C
Elevation
5623
5622
5621
5620
5619
5618
5617
5616
5615
5614
11/25/92
01/07/93
02/18/93
04/01/93
07/29/93
01/27/94
09/29/94
04/28/95
11/30/95
06/27/96
12/20/96
06/26/97
12/18/97
06/01/9
Dates



Cell 2 West 3 Settlement Monitor
ATTACHMENT 4-D
5620.00
5619.50
5619.00
5618.50
5618.00
5617.50
5617.00
5616.50
Elevation
11/25/92
01/07/93
02/18/93
04/01/93
07/29/93
01/27/94
09/29/94
04/28/95
11/30/95
06/27/96
12/20/96
06/26/97
12/18/97
Date



Cell 2 West 4 Settlement Monitor
ATTACHMENT 4-E
Elevations
5618.6
5618.4
5618.2
5618
5617.8
5617.6
5617.4
5617.2
5617
5616.8
5616.6
11/25/92
01/07/93
02/18/93
04/01/93
07/29/93
01/27/94
09/29/94
04/28/95
11/30/95
06/27/96
12/20/96
06/26/97
12/18/97
Dates

ATTACHMENT 5

TABLE 22

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

(VALUES) x 10E-3 µCi/Kg

	BHV-1		BHV-2		BHV-3		BHV-4		BHV-5	
Date	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	0.600
Dec-81	0.790	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	0.589	0.550	0.461	0.340	0.643	0.340	0.147	0.140
Jan-84	0.713	0.866	0.618	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	0.866	0.382	0.694	0.396	0.826	0.728	0.836
Aug-87	0.600	0.800	1.500	0.900	0.800	0.600	1.200	0.700	1.500	1.300
Aug-88	1.500	1.600	1.300	0.700	0.600	0.900	1.000	1.300	3.800	5.000
Aug-89	1.200	1.600	1.100	3.000	0.800	1.000	1.100	1.400	2.900	5.700
Aug-90	2.900	5.800	1.000	1.400	0.800	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 (Revised Calculation 8/98)

- Initial Survey will be conducted on an area to be determined But approximated by a perimeter approximately 1000' outside of Restricted Area Boundaries.

Area By Cad = 38,728 M sq ft
= Includes, Tails, mill yard, ore Pad.

Area that requires wind blown Survey is:

		Mft^2
Area of Wind blow Survey		38,728
Less	Cell 4A	1,909
	Cell 3	3,234
	Cell 2	2,987
	Cell 1	2,576
	Mill Yard	1,643
	Ore Storage	977
		25,402 Mft^2

Establish Grid –

Assume 10 x 10 meter grid (Standard NRC/EPA) ≐ 1076 ft^2
Survey Crew of 2 Capable of setting 500 grid Points/Day

25,402,000 ft^2 ÷ 1076 ft^2 = 23,600 grid points

23,600 grid points ÷ 500 grid points/day = 47.2 Days
≅ 47 Days Survey

2 man Crew 2 × 8 hrs/Day × 47 /Day = $\boxed{752 \text{ hrs}}$

Total Grid Survey = 752 man hrs

Scanning Crew Consists of 2 – [redacted] one Scanner, one Record Keeper.

Coverage

Assume: Scoping Survey Completed by Scanning with MR Meter Held Close to ground while traversing at ± 0.5 m/sec as per Guidance in NUREG 5849

0.5 m/sec × 60 sec/min × 60 min/hr × 8 hr/day = 14,400 m/day

Assume eff of .8

.8 × 14,400 m/day = 11,520 m/day

Assume 30 m path for each 10 m × 10 m grid to cover 10% of surface area (NUREG 5849)

Scanning Crew Surveys (Scans) 11,520 m/day ÷ 30 = 384 grids/day

Scanning therefore takes:

23600 grids ÷ 384 grids/day = 61.5 days
= 62 days

62 days × 2 men × 8 hrs/day = 992 man hrs

Assume Map Preparation and Data Reduction will take Scanning Crew another 20 days to Complete   20 days × 2 men × 8 hrs/day = 320 man hrs

Total Scanning Mhrs = 1312 mhrs

Sample Crew Cost = $13.00/mhr
$13.00/mhr × 1312 mhrs = $17,056.00

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
97 0% or 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 63
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 4Aeach tanks

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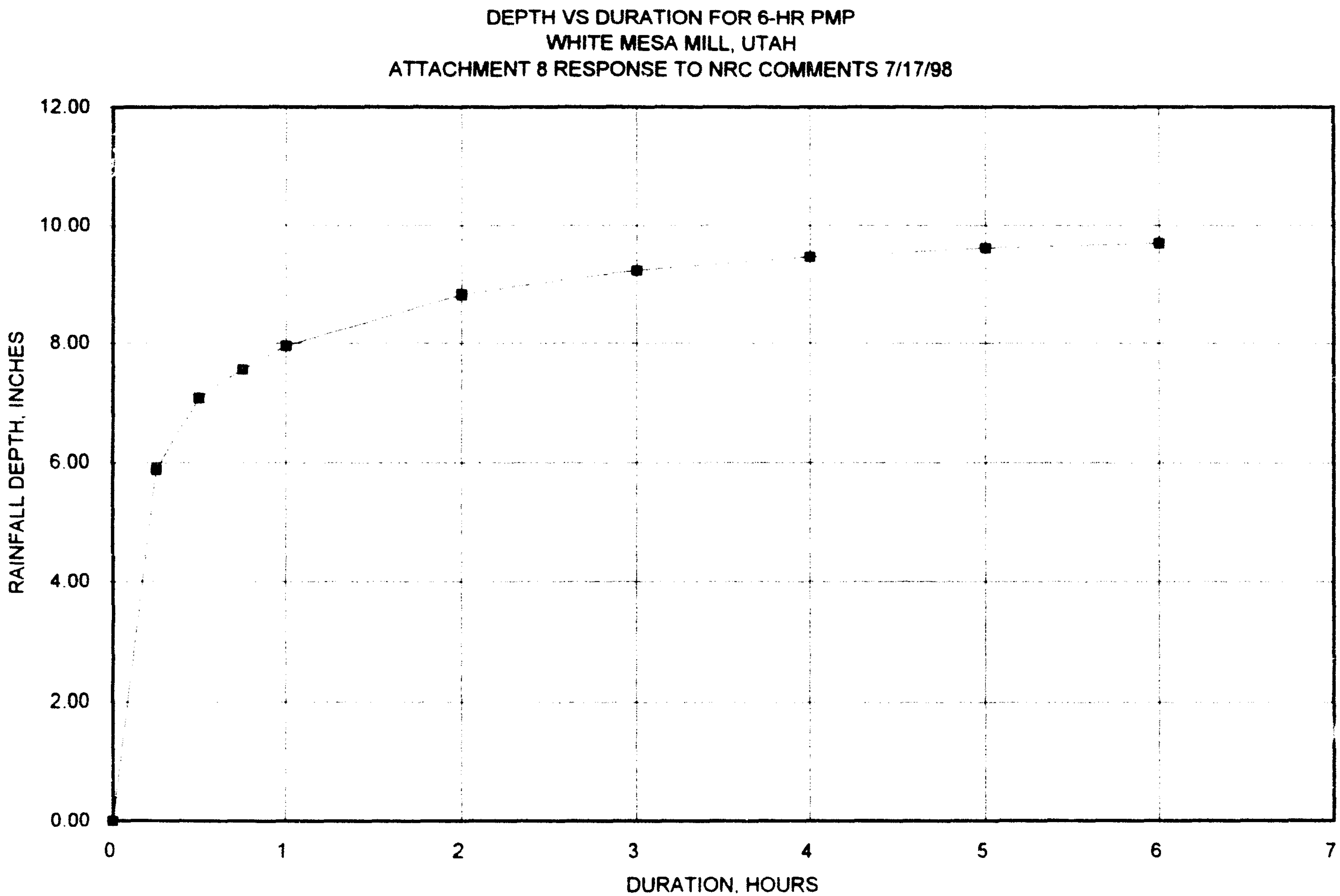
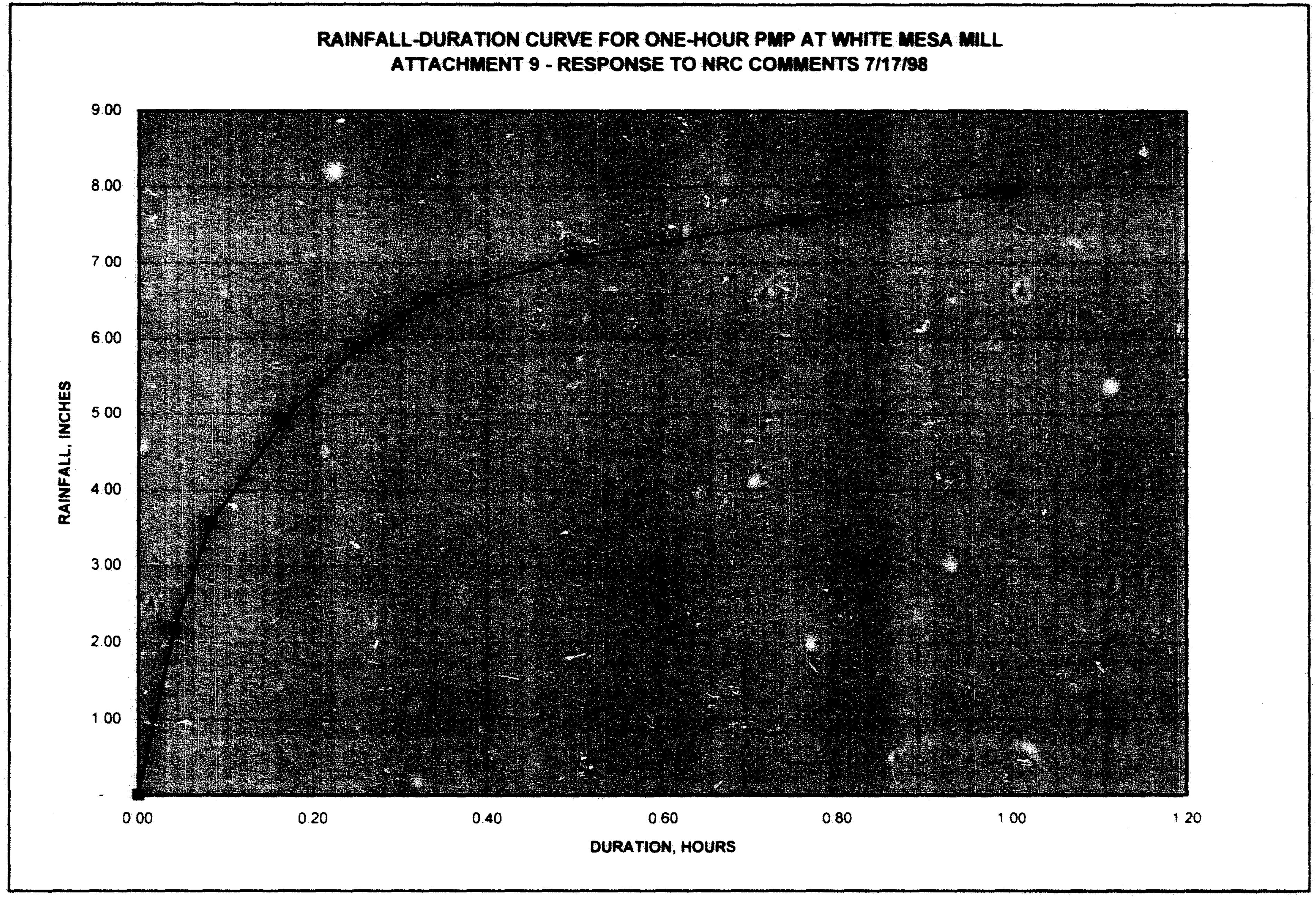
TIME DISTRIBUTION OF FIRST ONE HOUR, OR THE ONE-HOUR PMP
(after Table 2 1, NUREG CR 4620)

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
RAINFALL DEPTH, INCHES
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0
1
2
3
4
5
6
7
DURATION, HOURS

ATTACHMENT 9


RAINFALL-DURATION CURVE FOR ONE-HOUR PMP AT WHITE MESA MILL
ATTACHMENT 9 - RESPONSE TO NRC COMMENTS 7/17/98
RAINFALL, INCHES
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
-
0.00
0.20
0.40
0.60
0.80
1.00
1.20
DURATION, HOURS

ATTACHMENT 10

ATTCHMENT 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	tc hours	RAINFALL WITHIN tc (1)	i in/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	5599	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	Qn/1 49*s^ 5	**Flow Depth, y ft**	Cross Section Area of Flow a ft^2	Hydraulic Radius R ft	a(R)^ 67	**Velocity v fps**	Allowable Peak velocity fps (COE 1970)	Riprap Size d50 inches (ref 1)
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	408 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	408 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^0 24		
	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^0 666/Fbo^0 333 =	4 22	3 99
	Fbo = zero bed factor = 1 0 ft/s^2 for fine sand		
	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 =	8 68	8 00
	Vc =	2	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

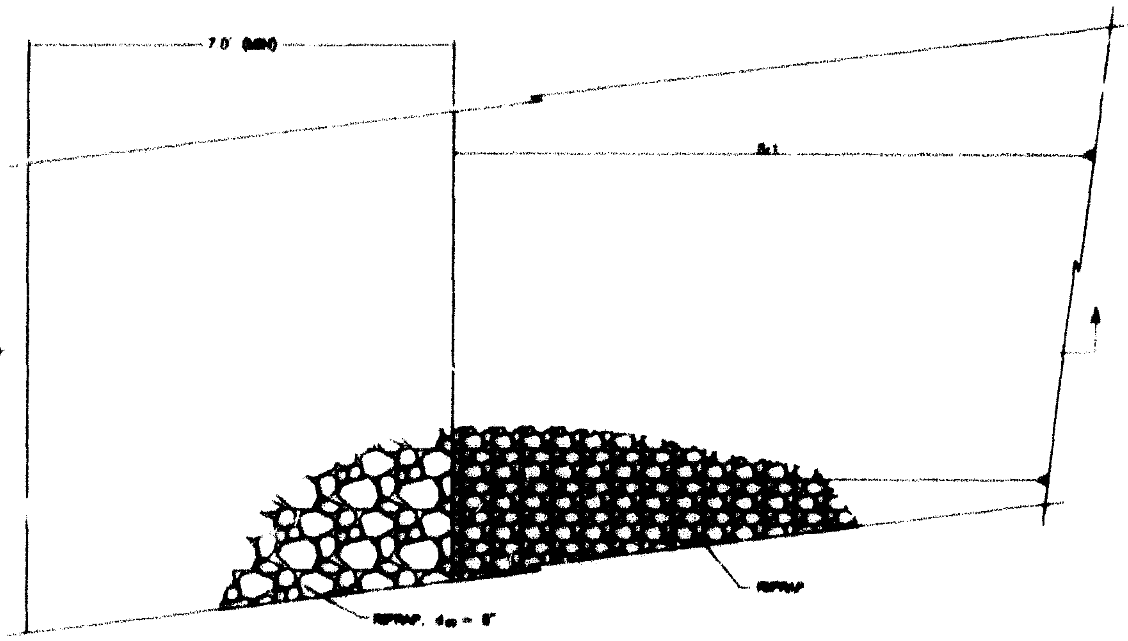
RATIONAL METHOD CALCULATION OF PMF PEAK DISCHARGE, VELOCITY AND DEPTH THROUGH CELL #1 DISCHARGE CHANNEL

FLOW PATH ELEMENT	ELEMENT LENGTH [illegible]	MAX ELEV	MIN ELEV	GRADIENT [illegible]	SLOPE ANGLE degrees	[illegible] hours	RAINFALL WITHIN [illegible]	[illegible]	SURFACE AREA acres	PEAK DISCHARGE [illegible]
LONGEST	4800	5655	5610	0.0094	0.54	0.54	7.20	13.43	[illegible]	1344

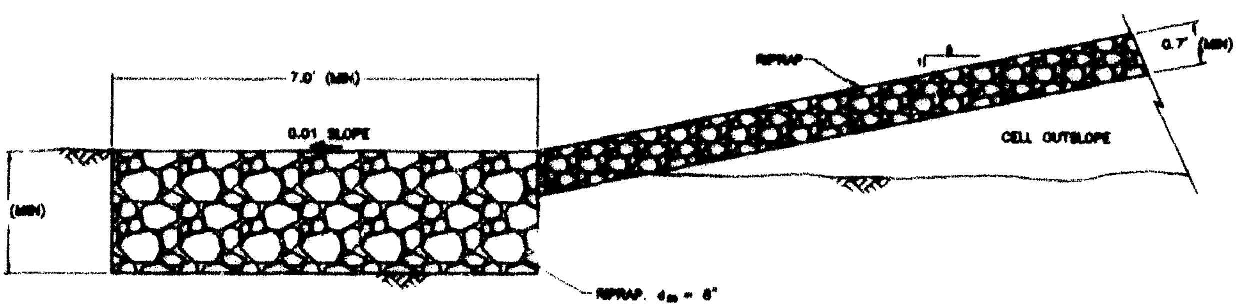
FLOW PARAMETERS IN CELL #1 DISCHARGE CHANNEL AT PEAK PMF DISCHARGE

	Channel Bottom Width (ft)	Channel Side Slopes	Channel Gradient (ft/ft)	Manning Coeff (n)	[illegible]	Flow Depth (ft)	Cross Section Area of Flow [illegible]	Hydraulic Radius R [illegible]	[illegible]	Velocity [illegible]	Allowable Peak Velocity fps [illegible]
Bedrock Channel	100	3:1	0.0100	0.025	226	1.62	169.9	1.54	226.95	[illegible]	8.10
Bedrock Channel	120	3:1	0.0100	0.025	226	1.45	180.3	1.40	225.46	[illegible]	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
CHKD BY:	DATE: 8/17/98	1 of 1
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	ELEMENT WIDTH W	GRADIENT S	SLOPE ANGLE	tc (minimum is 0.042)	RAINFALL WITHIN tc	INTENSITY	Peak Unit Discharge q	d50
	ft	ft	ft/ft	degrees	hours	inches	in/hr	cfs/ft	inches
APRON	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	12.07	1.80	**7.3**

Notes
The top cover element length is 2450 ft. This was used in the calculation 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 Abt, S.R. and Johnson, T.L. "Riprap Design for Overtopping Flow," ASCE Journal of Hydraulic Engineering, 1991.
The d50 for the apron was calculated per Abt, S.R., Johnson, T.L., Thornton, C.I. and Trabant, S.C. "Riprap Sizing at Toe of Embankment Slopes," ASCE Journal of Hydraulic Engineering, July 1998.

DEPTH OF SCOUR AT DOWNSTREAM EDGE OF TOE 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*q^0.24**
K = constant, 2.45
q = 1.81 cfs/ft

ds = **2.82 ft**

Method 2 **ds = 0.25 dm**
dm = mean water depth at design discharge

ds = 0.22 ft

Method 3 **ds = 0.6*dfo**
dfo = q^0.666/Fbo^0.33
Fbo = zero bed factor = 1.0 ft/s^2 for fine sand

ds = **0.09 ft**

Method 4 **ds = 0.25 * dma**
dma = unit cross section of flow = 0.87 ft

ds = 0.22 ft

Method 5 **ds = dma*((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