Department of Energy
Albuquerque Operations Office
P. O. Box 5400
Albuquerque, New Mexico 87115

WM Record File ______ WM Project 58
Docket No.
PDR ✓
LPDR

Distribution: Jean Tribet
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(Return to WM, 623-SS) J3

SEP 06 1985

Mr. Leo Higginbotham, Chief
Low-Level Waste and Uranium
Recovery Projects Branch
Division of Waste Management
United States Nuclear Regulatory Commission
Washington, DC 20555

Dear Mr. Higginbotham:

Enclosed are responses and back-up information in response to NRC comments contained in your letter of May 17, 1985, dealing with the Shiprock Construction documents as well as your letter of June 6, 1985, dealing with the Shiprock Final Remedial Action Plan. Could you please provide your response regarding the acceptability of this information as soon as possible. We require a response prior to October 4, 1985 since the Shiprock Remedial Action is currently in progress.

Should you have any questions on these issues, please contact Mr. David Ball of my staff at (FTS) 844-3941, or Leon Stepp with Jacobs Engineering Group at (FTS) 846-4030.

Sincerely,

John G. Themelis

John G. Themelis, Project Manager
Uranium Mill Tailings Project Office

Enclosure

cc w/enclosures:
R. Hopkins, M-K
R. Williams, JEG
L. Stepp, JEG
D. Gillen, NRC

WM DOCKET CONTROL CENTER
'85 SEP -9 A11:02

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PDR WASTE
WM-58 PDR

RESPONSE TO NRC COMMENTS
OF MAY 17, 1985, ON
SHIPROCK CONSTRUCTION DOCUMENTS - SURFACE WATER HYDROLOGY

GENERAL COMMENTS

NRC Comments, together with responses are presented below. The remedial action design was prepared in accordance with the RAP to meet the EPA Standards. Compliance with certain NRC comments will significantly increase costs because of requirements for larger size erosion protection material. The total cost increase has not been estimated, but the cost of increasing riprap rock size and quantity in the ditches alone is estimated to be about $1,723.000. It is also estimated that erosion protection for side slopes will increase by 57% ($3,054,000). Because the present design uses the maximum rock sizes locally available, any increase in rock size will require that a new source of material be located and developed. Although investigations for another rock source have not been made, a source of material probably does not exist within about 30 miles of the site, and it may be necessary to build or upgrade roads into the source and develop a quarry to provide material that can be processed to obtain larger riprap material.

ANALYSIS OF PRESENT DESIGN

The criteria used in the present design are discussed in the following paragraphs.

The erosion protection for the permanent ditches has been designed for the PMP event (the Design of Small Dams, Figure 20 and NRC Staff Position Paper WM-8201, Part III A.2 were used).

The time of concentration and runoff intensity were calculated using the SCS Curve Number Methods. The peak runoff was then calculated using the Rational Method with coefficient of runoff equal to unity. Through-flow in the rock layer was ignored.

The riprap size was selected using the Safety Factor Method (Rock Riprap Design Methods, etc. NUREG/CR-2684) using Factor of Safety equal to unity and the average boundary shear.

Evaluating the present design (5-inch D_{50} rock) using (local) maximum shear stress as the criteria, Ditches 1, 4, 5 and 6, with 5-inch medium size rock can withstand the maximum shear stress under the Small Dams PMP event. Ditches 2, 3 and 7 with 5-inch D_{50} rock can stand the maximum shear stress under the 1,000-year storm event (NOAA Atlas 2, Volume 4, New Mexico, 1973).

ANALYSIS USING PROPOSED NRC CRITERIA

Several joint NRC/DOE committees are presently reviewing design criteria to be applied to the UMTRA program. One of the considerations being discussed is the cost to implement criteria. Of particular concern is the cost to assure, absolutely, that stability is achieved for 1,000 years without maintenance. It is felt that the present design meets the criteria that stability be achieved for at least 200 years, even when analyzed for peak stress. This design is achieved using the largest rock available within the vicinity of the site.

More conservative criteria can be applied at a greater cost. The additional cost of the proposed NRC criteria is estimated to be in the order of $4,777,000. This is based on an estimated additional cost of $1,723,000 for drainage ditches and $3,054,000 for increased size and thickness of erosion protection materials for the side slopes.

The erosion protection cover for the tailings embankment and the permanent drainage ditches has been re-analyzed using the criteria contained in the NRC Comments, and the total impact of these changes is discussed in the following paragraphs.

For the tailings embankment top, the specified 1-1/2 inch median size is adequate. For side slopes, 8-1/2 inch median size rock is required.

The riprap layer thickness may have to be increased from 12 inches to 18 inches to accommodate the larger rock size on embankment side slopes. A greater thickness of the transition layer (bedding) may be required for the same reason.

Ditches 1, 4, 5 and 6 will require at least 7-inch and possibly 8.5-inch median size rocks. Ditches 2, 3 and 7 will require 12-inch median size rocks and the layer thickness will need to be increased to 18 inches. A thicker transition layer (bedding) will also be required.

The flow depth in the ditches will be increased by about 2 feet in the upper reach and by about 5 feet in the lower reach and at the junction. This will require wider ditches.

Utilization of the proposed NRC criteria will significantly increase the Shiprock Remedial Action costs, possibly as much as 50%, because of the unavailability of larger rocks nearby the site.

NRC Comments 1.A Use of Average Shear Stress for Riprap Design

The size of the riprap for protection of the drainage ditches was determined by estimating the average shear stress over the entire ditch cross-section. Because the adopted design cross-section is basically a rather inefficient one (the hydraulic radius is small relative to the area of the flow, particularly in the outer flow areas away from the center of the channel), the shear stress and resulting riprap size will also be small if the shear forces are averaged across the entire section.

In actuality, most of the flow will be concentrated in the center of the ditch, with the outer edges of the ditch carrying proportionately very little flow. This will produce much higher velocities and shear forces in the center of the ditch. The rock protection for the ditches should be designed for the velocities and the localized shear forces produced at the most critical locations in the ditch, which for this design, is the center of the ditch. The riprap size for the drainage ditches should be re-analyzed in accordance with methods which account for uneven flow distribution over the channel cross-section width and for localized, rather than average, shear forces. Acceptable methods for estimating the variation of velocities, discharges, and shear forces may be found in Corps of Engineers EM 1110-2-1601, "Hydraulic Design of Flood Control Channels" and ETL 1110-2-120, "Additional Guidance for Riprap Channel Protection."

Response:

Use of average shear conforms with the RAP (Attachment A: Calculation Summaries, Section 4.0), where it is implicitly assumed that total collapse occurs when the average boundary shear stress exceeds the critical shear stress and movement of D_{50} particle size occurs when the (local) maximum shear stress exceeds the critical shear stress. The use of maximum shear stress alone, using the design PMP, will increase the D_{50} riprap size for Ditches 1, 4, 5 and 6 from 2.25 inches to 4.14 inches. These ditches were protected with 5 inch D_{50} riprap. For Ditches 2, 3 and 7, the D_{50} riprap size will increase from 3.85 inches to 7.10 inches. The 5 inch D_{50} riprap shown on the design plans is adequate for a 1,000 year storm based on design rainfall and concentration times. The current design provides for riprap with D_{50}= 5 inches on all ditches, because the ditches form a continuation of the embankment side slopes on which 5-inch rock is used.

NRC Comment 1.B Use of Inappropriate Mannings's 'n' Values

The flow velocities, water surface profiles, and shear forces for riprap design for the drainage ditches were estimated using a Manning's 'n' value (a dimensionless measure of the roughness and frictional effects on channel flow) of 0.038. Our review indicates that this estimate may be too high, resulting in flow velocities and riprap sizes which are too small.

Recognizing that estimation of Manning's 'n' values can sometimes be very subjective, there are however, several methods available to directly calculate the 'n' value if the hydraulic radium and average rock size are known. A specific check of the computations (by the NRC staff) of the 'n' value in ditch segment D-7, using Corps of Engineers EM 1110-2-1601 formulae (Plate 4), indicates that the 'n' value in this segment is approximately 0.027. Use of this value would produce much higher velocities in the ditch, resulting in the need for larger rock to be used for erosion protection. (We note that this method was previously used - See sheet 12/24 Calculation No. 04-11-RO-02).

The riprap sizes for each of the diversion ditch segments should be re-evaluated using estimates of Manning's 'n' value which corresponds to the depth of flow and rock size in a particular ditch segment. Acceptable methods for estimating these values may be found in Corps of Engineers EM 1110-2-1601, "Hydraulic Design of Flood Control Channels." It should be noted that several trial-and-error calculations may be necessary to arrive at the final 'n' values and riprap size. It should also be pointed out that Manning's 'n' may be larger than 0.038 in those ditch segments when the depth of flow is low, relative to the rock size.

Response:

For Ditches 1, 2 and 3, where flow depths less than 3.5 feet are expected, riprap was designed using Manning's 'n' = 0.04. For Ditches 5, 6 and 7, where greater depths are expected, 'n' = 0.035 was used (Calculation No. 04-11-RO-02). Because of uncertainties in accurately assessing Manning's coefficient, 'n', an average value of 0.038 was used for all ditches in the design revision, including final design. The coefficient, 'n' has been recalculated using the Corps of Engineers' formula (EM 1110-2-1601) for Ditches 1, 3, 5, and 7 and found to range from 0.029 to 0.031 (based on designed riprap, D_{50} = 5 inches).

The design based on 'n' = 0.038 is on the conservative side, because a higher 'n' gives a greater depth of flow and larger boundary stress than would be the actual case.

NRC Comments 1.C Inadequate Riprap Design at Channel Bends and Junctions

The size of riprap to be placed in curved sections of the channel was concluded to be the same as for straight sections of the ditches. However, there is an apparent incorrect conclusion drawn from the results of calculations presented on Sheet 18/24 - Calculation No. 04-11-RO-02, dated 9/20/84. We conclude that the forces produced in the curved channel portions will be considerably larger than the forces produced in the straight portions. The riprap size in these areas should be increased, accordingly.

We note from a review of Calculation 04-11-RO-02 Sheet 6/24 that there is an intent to design channel junctions with a selected radium-of-curvature to width (r/w) ratio. If the intent is to provide a r/w ratio of at least 2, the selected factor of 1.84 for increasing shear stresses is acceptable.

Otherwise, for different ratios, EM 1110-2-1601 provides acceptable guidance for determining appropriate factors for increasing shear stresses.

In addition, our review of the details of the channel junctions that were provided on Drawing SHP-PS-10-0016, REV A, indicate that the channel junctions, as designed, will be subjected to excessive shear forces that have not been properly accounted for. These junctions are apparently not designed using the proposed r/w ratio of 2 and do not represent adequate transition zones for high velocity flow. We conclude that the channel junctions should be redesigned. EM 1110-2-1601 (pp. 57-62) provides acceptable general guidance for designing channel confluences.

It also appears that the roadway crossings through the diversion ditches will develop very undesirable flow conditions. In any redesign of the channel junctions, consideration should be given to redesigning the roadway crossings so that undesirable flow currents are not created.

Response:

Under present design, the riprap protection for channel bends and junctions can be made adequate by using D_{50} = 7 inches. The maximum shear stress in the bends can be kept equal to (local) maximum shear in the straight channel by providing a minimum radius equal to twice the water surface width. See pages 13, 15, and 18 of Calculation No. 04-07-RO-02.

The curved channel potions are being redesigned using current design criteria. These designs may be modified if different criteria are adopted.

NRC COMMENT 1.D PMP Rainfall Distribution and Reduction of Rainfall Intensity

The peak flow in the drainage ditches was computed based on a reduction of rainfall intensity (see sheet 7/24 - Computation 04-11-RO-02) based on Soil Conservation Service (SCS) methods. The NRC staff concludes that this reduction is not appropriate for rainfall as severe as the PMP. In most accepted computations of the runoff from a PMP, including other accepted SCS methods, it is assumed that the peak burst of rainfall (with a duration corresponding to the time of concentration) occurs at a time when previous rainfall has sufficiently saturated the ground so that nearly total runoff occurs during the most critical period. We conclude that any reductions in intensity are already accounted for by the computation of the time of concentration. Since it appears that peak flow rates in the ditches could be increased by about as much as 1/3 if nearly 100 percent runoff is assumed, we conclude that the calculations should be revised to reflect more severe rainfall intensities. (See also Comment 2.)

Response:

The reduction in PMP rainfall intensity resulting from soil infiltration by use of the SCS Method is equivalent to using the rational formula for peak flow with a runoff coefficient, C = 0.75. For comparable topography (rolling to hilly terrain), the Office of Surface Mining (OSM/TR-82/2, Surface Mining Water Diversion Design Manual) recommends runoff coefficient between 0.70 and 0.82 for tight clay under cultivation. Therefore, the peak runoff estimated in Calculation No. 04-11-RO-04 for ditch erosion protection is reasonably conservative.

The infiltration accounted for in this case will not be affected by the high intensity PMP because of large void sizes and through-flow in the riprap and bedding layers.

NRC Comment 1.E Time of Concentration

The curve number (lag) method of computing the time of concentration for the drainage ditches is not considered conservative for rainfall as severe as the PMP. We conclude that the times of concentration will be significantly smaller if other methods are used.

For example, in the design of Ditch D-1, the time of concentration for flow over and through the rock layer on the 56-foot long, 45 percent slope (595 feet long) is calculated to be 3.69 minutes.

Our review indicates that the latter method of computation is more appropriate for PMP rainfalls and should be adopted for design of the ditches.

Response:

The time of concentration for the present design was developed using the SCS curve number method for overland portion of flow. Hydraulics method was used for channelized flow.

If the time of concentration is calculated using the method of OSM/TC-82/2, the time of concentration will be decreased for all ditches. The time of concentration will change from 15 minutes to 6.5 minutes for Ditches 2, and 3, and from 25 minutes to 18 minutes for Ditch 7. By this shortening of time of concentration alone, the PMP rainfall intensity will be increased by 55 percent (from 14.5 to 22.5 inches/hour) for ditches in the upper reaches, and by 22 percent (from 11.5 to 14.0 inches/hour) for ditches in the lower reaches.

NRC Comment 2.

The methodology for determining rainfall distribution and intensities, as given in NRC Staff Technical Position Paper WM:-8201, has been superseded by that given in the recently published Hydrometerological Report No. 55 (March, 1984). The NRC staff no longer endorses the methodology presented in WM-8201. WM-8201 was developed for use at active uranium mill sites, most of which are located in Wyoming, east of the Continental Divide. At the time of the development of WM-8201,

reasonable guidance for rainfall distributions in that area was unavailable and/or questionable. WM-8201 was formulated to provide that type of general guidance, based on Corps of Engineers rainfall distributions. The recent publication of Hydrometeorological Report No. 55 has indicated that certain areas in Wyoming will be subject to rainfall intensities (especially of short duration) much greater than those given in WM-8201. As a result, the NRC staff intends to make appropriate modifications to WM-8201 to reflect the new data.

The modifications to WM-8201 will include recommendations to use the rainfall distribution guidance that is developed in the Hydrometeorological Report that is appropriate for a given region. These modifications will be applicable to UMTRAP sites in general. For the Shiprock site, in particular, the rainfall distributions given in Hydrometeorological Report No. 49 should be used, since this represents the most current estimates of rainfall potential for this area of the United States.

Extrapolation of the data for time intervals less than 15 minutes will be necessary.

Response:

The PMP magnitude and distribution were calculated for the present design using the Design of Small Dams and WM-8201, as provided for in the RAP.

If the PMP distribution and intensities are recalculated as given in the Hydrometeorological Report No. 49, the following changes would result.

The 1-hour PMP would be changed from 8 inches (Ref. Design of Small Dams) to 9.1 inches (Ref. HMP No. 49). This increase coupled with the change in durational distribution from that in WM-8201 (NRC Hydrologic Design Criteria, 1982) to that in HMR No. 49 would lead to higher PMP intensities in embankment and ditch erosion protection design.

For the embankment, the PMP intensity would increase from 24 inches/hour to 55 inches/hour for the 5-minute duration.

For ditches in the upper reaches, the increase in PMP intensity would be 110 percent (from 22.5 inches/hour to 47.5 inches/hour) and in the lower reaches the increase will be 75 percent (from 14.0 inches/hour to 24.5 inches/hour).

The present design utilizes the erosion and riprap materials available at the site. Other design criteria can be applied, but will result in significant cost increases.

NRC Comment 3.

Our review of the erosion protection to be provided for the filled-in arroyos indicated that the protection may not be adequate to meet EPA long-term stability criteria. We conclude that the 1 vertical (V) on 2 Horizontal (H) fill slopes have not been adequately protected and that the erosion protection for the toe of slopes may not be adequate to resist flood velocities in the San Juan River. We disagree with your conclusions that only minor erosion of the slopes will occur during major rainstorms and that insignificant gully erosion on the slopes should be expected. We conclude that the filled-in arroyos, as designed, will experience significant erosion due to (1) erosion of the toe of the slope during high river stages, and (2) erosion of the slope due to gullying and sheetwash. We further conclude that the arroyos could be severely eroded due to the occurrence of relatively minor flood and precipitation events.

Since it appears that the setback distance, coupled with rerouting of the drainage, is adequate to meet EPA standards in those areas where the existing escarpment is being cut back and re-shaped, additional protection should be needed only in the areas where the arroyos are being filled. This additional protection is needed to assure that the arroyos doe not re-form and eventually, through headcutting, extend into the tailings stabilization area. We conclude that adequate protection has not been provided to prevent these phenomena.

Accordingly, the erosion protection design for the filled-in arroyos should be modified to prevent the re-development and growth of the filled-in arroyos. The 1V on 2H slopes should be designed using similar design methods to the remediated pile slopes, and the toe of the slopes should be designed to resist velocities due to large floods in the San Juan River.

Response:

The present design provides pit run rock fill for arroyos for added protection against erosion. In time the finer particles will be washed away and the remaining larger sized pieces will form a self-armoring rock cover. Our calculation (Calculation No. 04-390-05-00) indicates that a 12-inch thick rock layer of 3/4-inch median sized riprap will completely contain the PMP runoff as throughflow.

Over the 1,000-year design life the escarpment will recede from the San Juan River and form a naturally stable slope. It therefore is not necessary to provide extensive protection for the arroyos as was provided for the tailings embankment.

When DOE/NRC final design criteria and parameters are resolved we will review the design for erosion protection of the filled-in arroyos.

Because of the protection to the toe of the filled-in arroyos provided by the bedrock in the escarpment face, erosion caused by occasional flooding in the San Juan River is not expected to be severe. However, the design of the toe of the filled-arroyo slopes is being reviewed relative to potential flood damage.

NRC Comment 4.

The larger rock that will be placed on the 1 Vertical (V) on 5 Horizontal (H) side slopes of the remediated pile should be extended for a short distance (say 30') up onto the flatter 4 percent slope. Since this transition area represents a very critical area where a significant amount of flow emerges from the rock layer, it is considered to be a prudent measure to provide this extra degree of flood protection.

Response:

The present erosion protection design provides for D_{50}, 5 inch rock on the entire 20 percent slope, but this rock size is required only on the lower portions of the slope, and the rock is larger than is required in the upper part of the slope where it meets the flatter top slope. Therefore, we do not feel that an additional transition to the top slope is required.

If larger rock is extended onto the flatter slope, we suggest that the larger rock be extended only 5 to 10 feet rather than 30 feet onto the 4 percent slope because of the limited availability of larger rock.

NRC Comment 5.

Additional information and design changes should be provided regarding the erosion protection that will be provided at the outlet of Ditch D-7. This area could become unstable, since a large amount of runoff which formerly was discharged elsewhere is not being directed at this area of the escarpment. Provide the details of, and bases for, the erosion protection and transition design in this area.

Response:

Erosion protection at the 5-foot high outlet of Ditch D-7 is only required during construction. Bedrock (Mancos shale) outcrops about 350 east of the outlet, approximately 650 feet west of the toe of the tailings embankment. Just as river and surface flows are not expected to erode the 300-foot wide strip of Mancos shale along the north and east sides of the site during the 200 to 1,000-year design life, the outcrop east of the Ditch D-7 outlet will provide an outfall that will be adequate for the specified design life.

RESPONSE TO NRC GEOTECHNICAL COMMENTS
OF MAY 17, 1985 ON
SHIPROCK CONSTRUCTION DOCUMENTS (SPECIFICATIONS)

Comment 1. Disposal of Demolished Materials

(Section 02050, Demolition, Part 3-Execution, Section 3.3, Page 020508)

"Item 3.3.A(2) requires that non-radioactively contaminated materials and low-level radioactively contaminated materials shall be placed in areas outside the tailings embankment. The low-level radioactively contaminated materials shall be covered with a minimum thickness of 1 foot of earth cover. The contract documents do not indicate the proposed location for such disposal nor do they identify specifications for the type and engineering index properties for earth cover material. These topics should be addressed in the specification/contract documents."

Response:

Specification 02050, Item 3.3.A(2) requires that certain clean and low-level contaminated materials be placed outside the tailings embankment. These materials are actually already in place outside the embankment, and are to remain in place. The construction sub-contractor will be informed of the lccations of these materials and of the fact that they are to remain in their present locations.

The same item requires that the low-level radioactively contaminated materials mentioned above are to be covered with a minimum thickness of one foot of earth cover. These materials will be covered by clean fill placed as part of the site grading already shown on the drawings, so that special materials will not be required.

Comment 2. Radon Barrier Materials

(Section 02200, Earthwork, Part 2-Products, Section 2.2B, Page 02200-3)

"Item 2.2.B specifies that the Radon Barrier materials shall be non-radioactively contaminated sandy SILT available from the designated borrow area, with a gradation of 50-100 percent passing a #200 sieve. This gradation requirement may not be sufficiently restrictive. The specified gradation should be compatible with the gradation used in the radon cover thickness computation, the gradation used in the filter criteria computation in sizing the bedding material beneath rock cover,

and the gradation of the material used in developing the compaction density-moisture criteria sued in the design. The gradation of the material also impacts on compaction characteristic of the material and on the long-term moisture content of the material. These parameters are important in determining the effectiveness of the material as a radon barrier. The above concerns should be considered in finalizing the specification for the radon barrier material."

Response:

Specification 02200, Item 2.2.B requires that the Radon Barrier Material be silt having at least 50 percent passing the No. 200 sieve, obtained from an approved source area. The approved borrow area is shown on the Drawings. Grain size distribution curves for all the materials in the approved borrow area meeting the above requirements are attached. All of these materials are well-graded. None are gap-graded. The gradation characteristics shown were used in determining bedding material gradation, compaction criteria, and radon barrier source thickness requirements. Thus all materials from the approved borrow meeting the specification requirements are expected to be satisfactory as radon barrier material. Additional radon attenuation tests are to be made on the approved borrow material and the cover thickness will be adjusted as necessary.

Comment 3. Relocation and Compaction of Slimes

(Section 02200, Earthwork, Part 3-Execution, Section 3.2, Page 02200-7)

"Items 3.2.A.7 & 8 specifies that pockets of slime located in tailings designated to be relocated or otherwise disturbed shall be mixed with sands and placed at the lower part of the tailings embankment. Also, slimes which are not relocated or disturbed during construction shall be mixed with or covered by sands to support the construction equipment. The specification for these items are not sufficiently restrictive. Additional guidance on the minimum or acceptable ratio of the sand-slime mixture and whether slimes and sands have to be premixed prior to placing or placed in alternate layers should be provided. This is an important aspect of the tailings stabilization plan and specifications should be very precise on the work to be performed."

Response:

Specification 02200, Items 3.2.A.7 and 8 require that slimes to be relocated or otherwise disturbed or which will not support construction equipment are to be mixed with sands. Mixing is required only for construction purposes, not for radon attenuation. The construction sub-contractor will be informed that other options, such as drying the slimes, will also be acceptable, so long as the work is not excessively delayed and the other specification requirements, such as compaction, are met. The specifications require that all relocated tailings be compacted, and the development of pockets of uncompacted slimes in the embankment will not be allowed.

Comment 4. Compaction of Radon Barrier Materials

(Section 02200, Earthwork, Part 3-Execution, Section 3.2, Page 02200-8)

"Item 3.2.C specifies that the Radon Barrier material shall be compacted to at least 95 percent of the maximum dry density (ASTM D 698) on the wet side of the optimum moisture content. Your design calculations assume a moisture content 3 percent in excess of the optimum moisture content determined from laboratory tests. The specification should state the acceptable range of moisture content in excess of the optimum moisture content with due consideration of the swelling and shrinkage characteristic of the clay mineral components of the soil."

Response:

Specification 02200, Item 3.2.C requires compaction of the Radon Barrier to at least 95 percent of maximum dry density on the wet side of optimum. The design calculations assume a moisture content of three percent above optimum only for the purpose of computing unit weight and degree of saturation values for the radon barrier layer.

Compaction curves for all the materials in the approved borrow area are attached. As can be seen, the highest moisture content at which these materials can be compacted to 95 percent of maximum dry density averages 3.6 to 3.9 percent above optimum. The effect of these variations on the parameters calculated is not significant.

The question of the swelling and shrinkage characteristics of the Radon Barrier materials was also raised. The materials are silts with low plasticity, having low susceptibility to swelling and shrinkage.

Comment 5. Gradation of Select Rock Material and Select Bedding Material

(Section 02270, Erosion Protection, Part 2-Products, Section 2.1, Page 02270-3)

"Items 2.1.C.4A specifies the gradation for the Type B rock material cover proposed for the tailings embankment side slopes, drainage ditches, and intercept ditches. There is a 6-inch thick bedding material between this rockfill cover and the radon barrier earth cover over the tailings pile. The proposed gradation of the Type B rock material and the select bedding material do not comply with the filter criteria gradation requirement D_{15}/D_{85}-5. This is a design requirement and the specification of all the materials proposed for covering the tailings embankment should comply with the filter gradation requirement."

Response:

Specification 02270, Item 2.1.C.4 gives gradation requirements for Type B rock material and Select Bedding Materials. The requirements do not meet the criteria D_{15}/D_{85} not to exceed five.

The purpose of this criteria is to prevent migration of fines from the bedding material into the overlying rock material. The primary force tending to cause this to happen is the rainfall which flows through the voids in the rockfill.

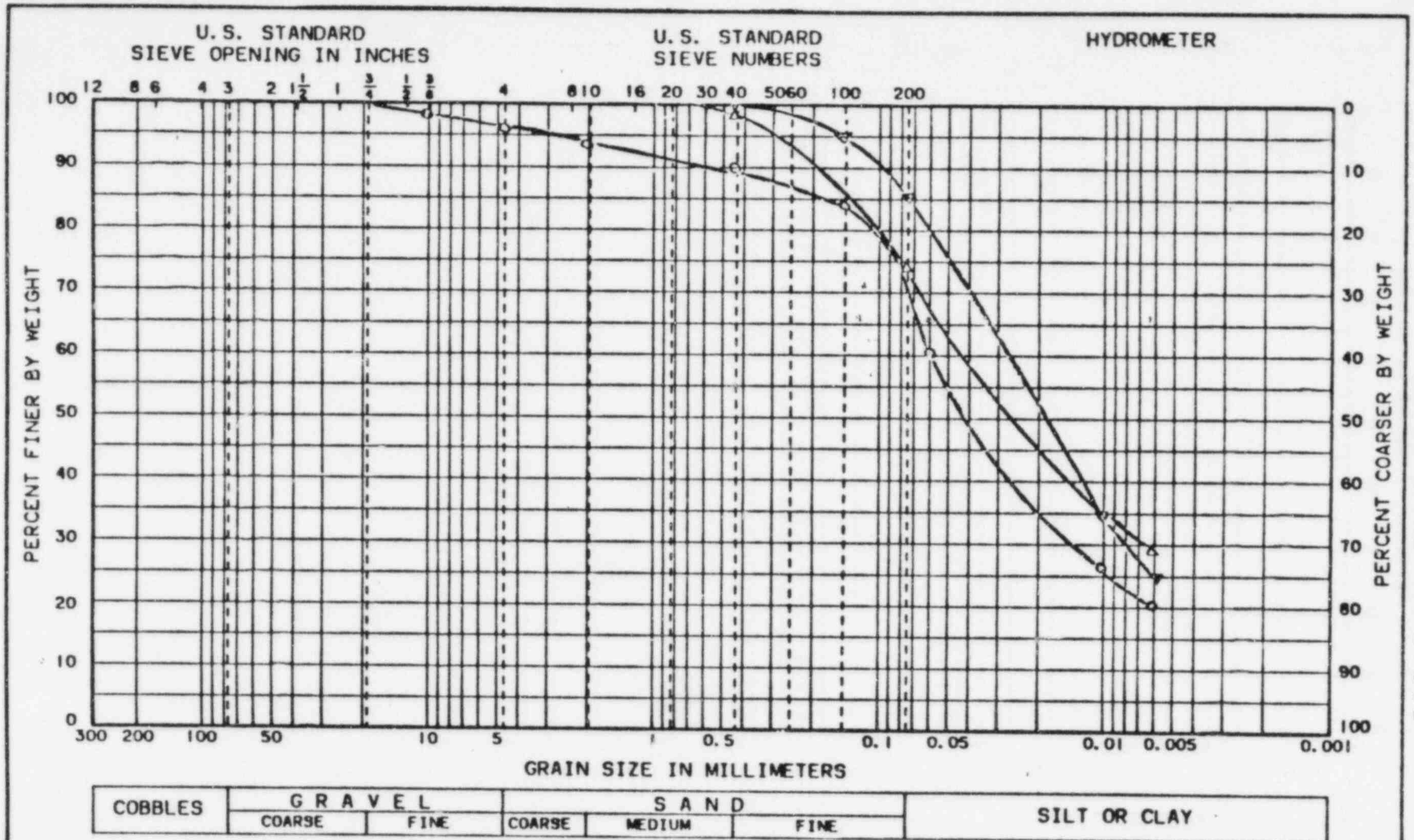
The attached calculation shows that the velocity of water flowing in the voids is less than the velocity required to cause erosion of the bedding material. Therefore, the gradations specified are satisfactory.

Attachments: 1. Grain Size Analysis for Radon Barrier Materials (1 sheet)

2. Compaction Test Data for Radon Barrier Materials (1 sheet)

3. Calculation 04-390-06-00, Evaluation of Bedding Material. Document No. 4005-SHP-C-01-01030-00.

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON-KNUDSEN COMPANY



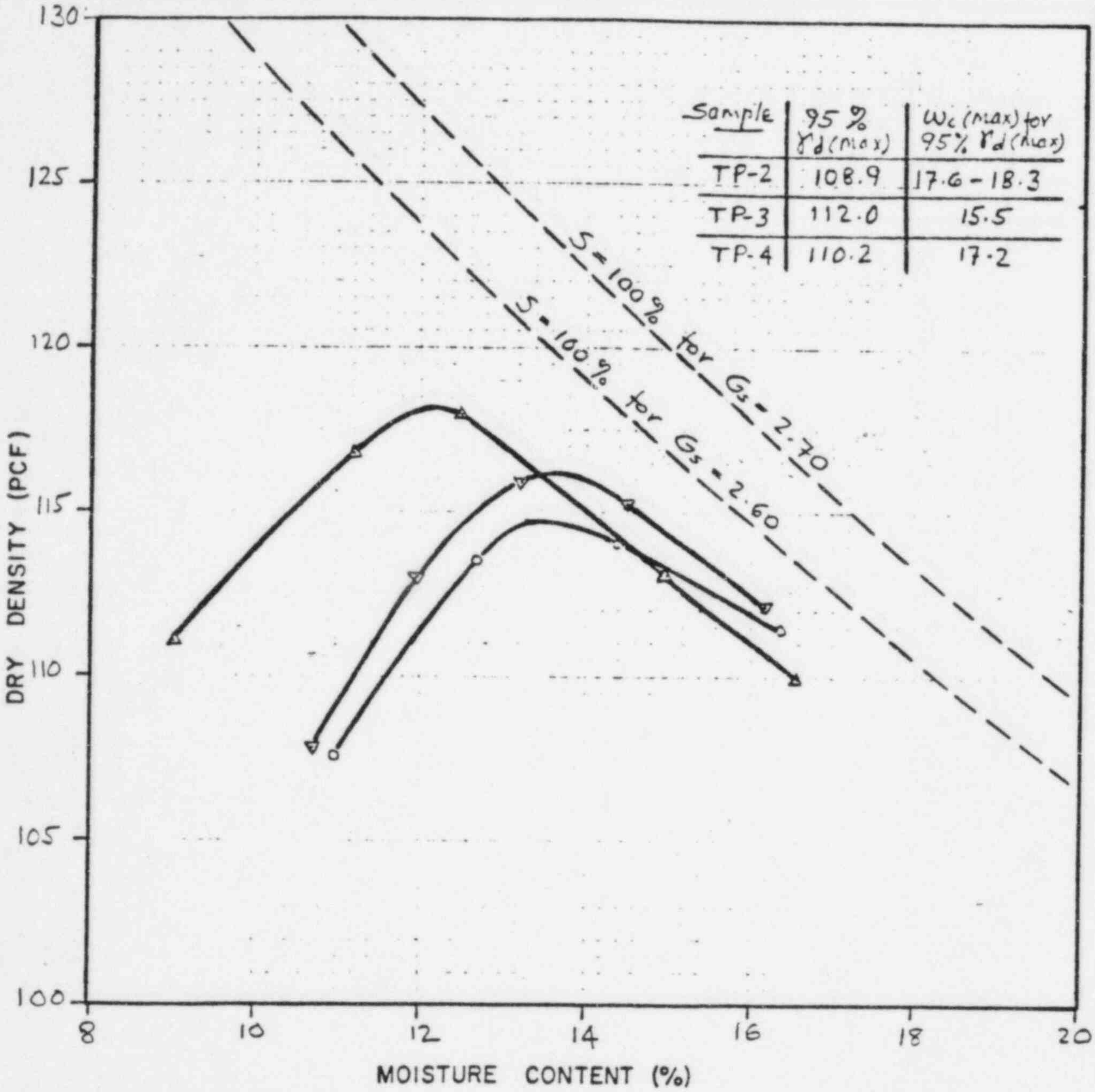
SAMPLE NO.	~~ELEV. OR~~ DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
⊙ SHB-TP2	2.0-7.5 FT	Sandy Silt	5.5	21		2
△ SHB-TP3	11.0-15.0 FT	(ML)	5.3	24		2
▽ SHB-TP4	1.0-10.0 FT	Borrow Area	5.4	22		2

PROJECT UMTRA - Shiprock
JOB NO. 4005 - Task 390
AREA Radon Barrier Materials
HOLE NO. Test Pits.
DATE ABH/7-24-85 WYL 7/24/85

CAB110

Reference: UMTRA-Shiprock Calc. No 04-07-R0-04/ Material Properties

ATTACHMENT 1



TEST METHOD: ASTM D 698

SYMBOL	SAMPLE SOURCE	CLASSIFICATION	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (PCF)
⊙	SHB-TP2 (2.0-7.5 FT)	Sandy Silt (ML)	13.6	114.6
△	SHB-TP3 (11.0-15.0 FT)		12.1	117.9
▽	SHB-TP4 (1.0-10.0 FT)		13.7	116.0

Reference: UMTRA Shiprock Calc. No. 04-07-R0-04 / Material Properties.

MORRISON-KNUDSEN ENGINEERS
A MORRISON KNUDSEN COMPANY

COMPACTION TEST DATA
UMTRA SHIPROCK
Radon Barrier Materials (Borrow Area)

DRAWN AB H
APPROVED WYL
PROJ. NO. [illegible]
DATE 7/24/85
FIGURE

Calculation Cover Sheet

Contract No. 4005 Discipline Task 390

Calc. No. 04-390-06-00
No. of Sheets 6

Project
UMTRA - Shiprock

Feature
Embankment Design.

Item
Evaluation of Bedding Material

Sources of Data
(1) UMTRA-Shiprock Phase II Construction Documents
(2) NRC Comments to above (May 17, 1985)
(3) UMTRA-Shiprock Calc. No 04-07-R0-04
(4) UMTRA-Shiprock Calc. No 04-09-R0-02

Sources of Formulae & References
(5) Embankment-Dam Engineering, RC Hirschfeld and SJ Poulos, eds. Wiley (1973)
(6) Rock Riprap Design Methods etc, W.H. Walters. NUREG/CR-2684.

Preliminary Calc. ☐ Final Calc. ☒ Supersedes Calc. No. ______

0	——	AB Hlay	7/24/85	W Y. Lin	7/24/85	[illegible]	7/25/85
Rev. No.	Revision	Calculation By	Date	Checked By	Date	Approved By	Date

IECO Form 42-110

ATTACHMENT 3

INTERNATIONAL ENGINEERING COMPANY, INC.

Project	UMTRA - SHP.	Contract No. 4605	Sheet 1/6
Feature	Task 390 - Engineering Support.	Designed A & H	File No.
			Date 7/23/85
Item		Checked WYL	Date 7/24/85

From (1). UMTRA - Shiprock / Phase II Contract Documents.

(2) NRC Comments of May 17, 1985

Using Lep's method (Embankment - Dam Engineering, "Casagrande Volume", R.C. Hirschfeld & S.J. Poulos, eds., 1973, Wiley - Interscience - pp. 87 - 107, Flow Through Rockfill by T.M. Leps):

Average velocity of water in the void of rockfill is given by

$$V_v = W m^{0.5}\, i^{0.54}$$

For Type B Select Rock (Ref-1 & attached gradation curves) the rock size (assumed to be represented by the median - D_{50} rock size) is found to range from about 115 mm (4.5 inch) to 200 mm (8 inch). The mean rock size (by weight) is then determined to be 6.1 inches

Then from Table 1 (Leps)

$$W m^{0.5} = 28 \text{ inches/sec.}$$
$$= 2.34 \text{ ft/sec}$$

The worst case arises on side slope of the embankment (5:1 slopes) where $i = 0.2$

Then $$V_v = 2.34(0.2)^{0.54} = 0.98 \text{ ft/sec.}$$

The average boundary shear on a typical interstitial flow channel can be represented by

$$\tau_s = \gamma_w m S$$

where m = hydraulic mean radius for interstitial space.

Then applying Manning's formula to interstitial channel flow we can write

$$V = \frac{1.49}{n} m^{2/3} S^{1/2}$$

or $$m^{1/2} S^{1/2} = \frac{nV}{1.49\, m^{1/6}}$$

Substituting in the expression for shear stress, we get

$$\tau_s = \frac{\gamma_w n^2 V^2}{2.22\, m^{1/3}}$$

The Manning's coefficient n can be approximated using Corps of Engineer's formula, with m used instead of R

i.e. $$n = \frac{m^{1/6}}{23.85 + 21.95 \log (m/k)}$$

From Table I (Leps, Ref. 5) the hydraulic radius for the 6.1 inch rock is found to be 0.76 inch. And from the attached gradation curves, the median size for the select bedding materials is found to range from 1 mm (0.04 inch) to 19 mm (0.75 inch)

INTERNATIONAL ENGINEERING COMPANY, INC.

Project	UMTRA - Shiprock	Contract No.	4005	Sheet	3/6
Feature	Task 390 - Engineering Support	Designed	AEH	File No.	
Item		Checked	WYL	Date	7/23/85
				Date	7/24/85

Then using the upper limit (D_{50} = 3/4 inch) - worst case for "n"

$$n = \frac{(0.76/12)^{1/6}}{23.85 + 21.95 \log (0.76/0.75)}$$

$$= 0.026$$

And the average boundary shear stress is

$$\tau_s = \frac{62.4\,(0.026)^2\,(0.98)^2}{2.22\,(0.76/12)^{1/3}}$$

$$= 0.046 \text{ psf}$$

Using the Safety Factor Method (Rock Riprap Design Methods and Their Applicability to Long-Term Protection of Uranium Mill Tailings Impoundments, prepared by WH Walters - NUREG/CR-2684, PNL-4222, 1982):

The stability number is calculated as

$$\eta = \frac{21\,\tau_s}{(S_s - 1)\,\gamma_w\,k}$$

From the median rock size range determined previously for the select bedding material, the mean value (by weight) is calculated for stability calc:

ie $$k = \left(\sqrt{(1\text{mm})^3 \cdot (19\text{ mm})^3}\right)^{1/3}$$

$$= 4.35 \text{ mm } (0.175 \text{ inch})$$

then $$\eta = \frac{21 \times 0.046}{(2.70-1)\,62.4\,(0.175/12)} = 0.624$$

INTERNATIONAL ENGINEERING COMPANY, INC.

Project UMTRA - Shiprock
Feature Task 290 - Engineering Support
Item
Contract No. 4105
Designed ADH
Checked WYL
Sheet 4/6
File No.
Date 7/24/85
Date 7/24/85

The Safety Factor (against erosion by flowing water) is then determined for flow over a plane sloping bed by the eqn:

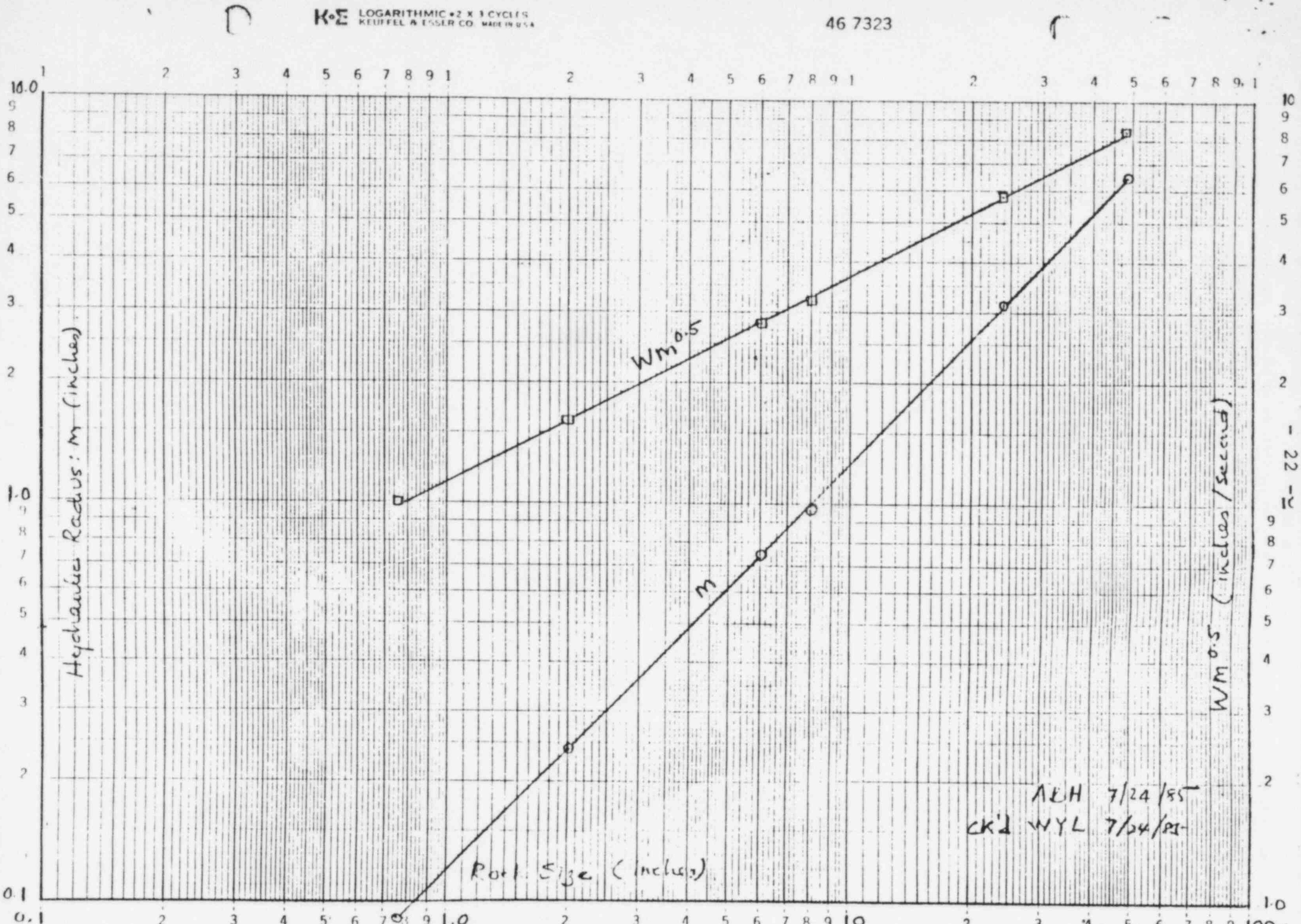
$$SF = \frac{\cos\alpha \tan\phi}{\eta \tan\phi + \sin\alpha}$$

From properties summary (Embankment Design Calculation No. 04-07-RC-04, page 2a) and from properties selected for pit run (Embankment Stabilization Calculation No. 04-09-RC-02, page 19) - the friction angle for the select bedding material can be judged to be range from 34 degrees to 38 degrees

Using $\phi = 36°$ we get

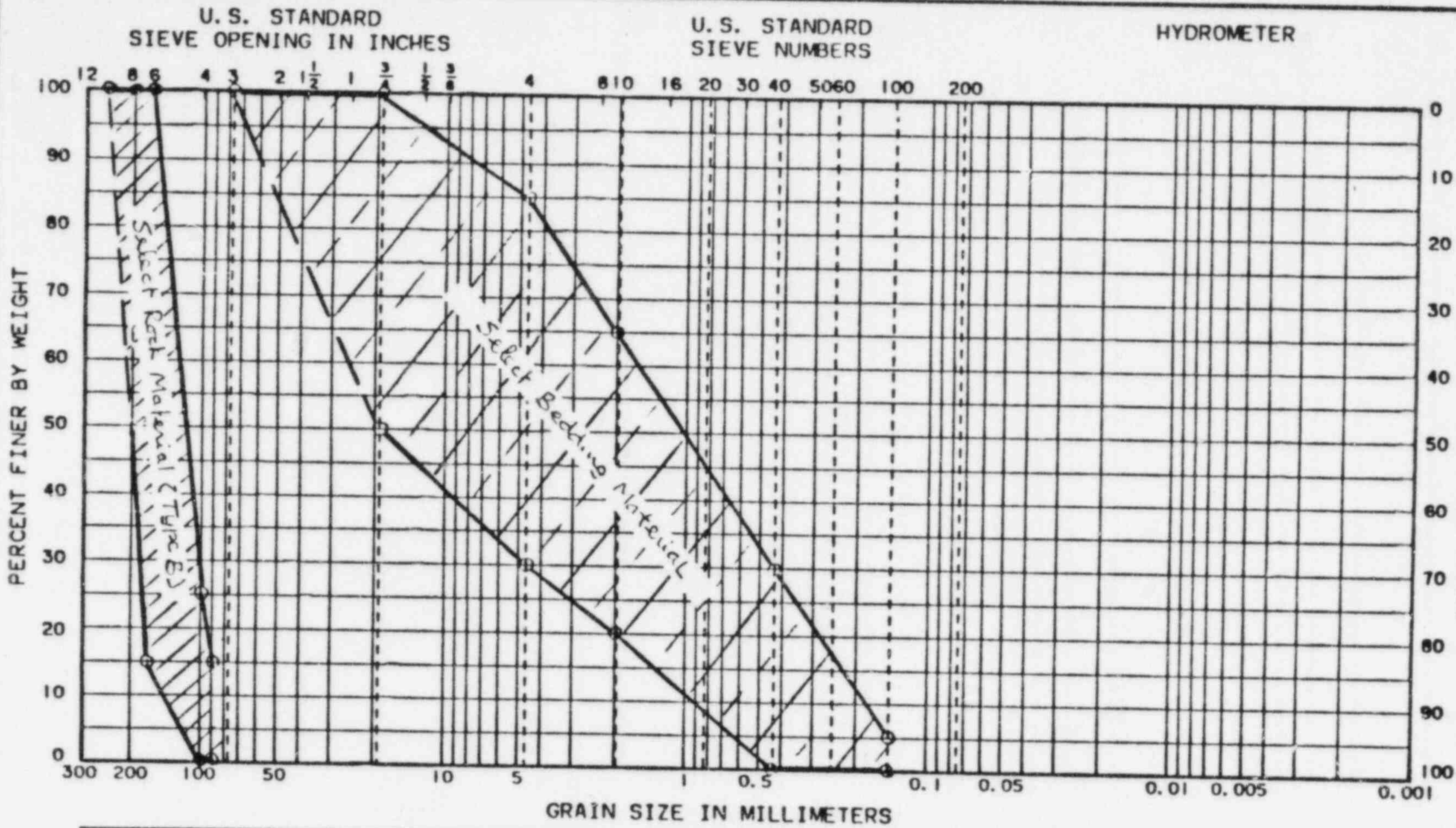
$$SF = \frac{\cos(\tan^{-1} 0.2) \tan 36°}{0.624 \tan 36° + \sin(\tan^{-1} 0.2)}$$

$$= 1.10 > 1.0$$


K•E LOGARITHMIC • 2 X 3 CYCLES KEUFFEL & ESSER CO. MADE IN U.S.A.
46 7323
Wm0.5
m
Hydraulic Radius: m (inches)
Wm0.5 (inches / second)
Rock Size (inches)
AEH 7/24/85
CK'd WYL 7/24/85

INTERNATIONAL ENGINEERING COMPANY, INC.

GRAIN SIZE ANALYSIS



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE NO.	ELEV. OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI

PROJECT UMTRA - SHP
JOB NO. 4005 Task 370
AREA ——
HOLE NO. ——
DATE ABH 7/21/85 WYL 7/2?/85

CAB110

Source: (Ref. 1) UMTRA- Shiprock Phase II Construction Documents

6/6

RESPONSE TO NRC CONDITIONAL CONCURRANCE

ITEMS CONTAINED IN LETTER DATED
JUNE 6, 1985 ON
SHIPROCK REMEDIAL ACTION PLAN

NRC Condition 1.

The first condition involves the characterization of ground-water contamination within the floodplain alluvium of the San Juan River. As discussed in Item 2 of my April 19, 1985 letter to you summarizing open issues, the NRC needs to review the data and proposed actions resulting from the DOE's planned ground-water characterization program in this area.

Response:

Ground-water sampling from well points within the floodplain and within the wash northwest of the mill area has found contaminated ground-water. A field program has been designed as follows to characterize the contamination:

1. To define the lateral and vertical extent of contamination, nests of wells will be completed at multiple depths, and at various locations in order to define lateral variations in water quality.

2. The field program will be flexible in order to achieve an accurate characterization of ground-water contamination. The first well will be drilled to bedrock, and the number of wells and completion depths of follow-on nested wells will be adjusted according to the total thickness of saturated alluvium.

3. To establish background water quality, wells will be installed at the upstream end of the floodplain and possibly on the opposite side of the San Juan River.

4. The source of the floodplain contamination will be determined. The possible sources to investigate include: seepage from the tailings, mill waste water disposed of in the floodplain, a raffinate pond spill that occurred in 1960, and leaching of wind-blown contamination into the floodplain alluvium.

   (a) Wells will be installed in the floodplain at the base of the Mancos Shale escarpment. These wells should provide confirmation of whether contaminated seepage from the tailings into the terrace ground-water system is impacting the floodplain alluvium.

(b) The ground-water characterization is designed to show whether the source of contamination is a concentrated zone of waste water disposed of in the floodplain.

5. Because traces of organic carbons were reported, samples from the wash northwest of the mill will also be analyzed for organic constituents.

NOTE: High water conditions have made access to all areas of the proposed sampling area impossible. A program of approximately 23 wells is currently being implemented. This program was discussed by J. Price with M. Weber on July 18, 1985 and is considered necessary and sufficient to adequately characterize the site. The attached "Scope of Work" (Attachment 1) describes the work to be performed, with the exception of a well to be added by change order on the East side of the river.

NRC Condition 2

The second condition involves rock sizing of the erosion protection features of the Shiprock design (Item 3 of the April 19, 1985 letter). Details of the NRC staff's concerns on the erosion protection design were transmitted by letter dated May 17, 1985. Mr. T. Wathen of Morrison-Knudsen (MK) was given an advance copy of these CONDITIONs. Prior to rock cover being placed on the pile or in the ditches, the DOE will need to receive concurrence from the NRC on the size of rocks to be used.

Response:

Please refer to the enclosed Response to NRC CONDITIONs of May 17, 1985 on Shiprock Construction Documents Surface Water Hydrology.

NRC Condition 3

The third condition concerns the deficiencies in the DOE's characterization of the seismotectonic hazard that were identified in Item 4 of the April 19, 1985 letter. DOE must promptly address these deficiencies, since resolution of the seismotectonic characterization issue may impact dynamic slope stability and design. Although the present slope stability analysis (Item 1 of the April 19, 1985 letter) has been found acceptable based upon the results of our review of information provided by MK, a revised analysis may become necessary if more detailed seismotectonic characterization results in increased design ground acceleration at the site.

Response:

See the attached "Responses to NRC COMMENTS on UMTRAP-SHP Seismic Hazard Assessment".

NRC Condition 4

The fourth condition pertains to details of the radon barrier design. The remedial action plan for Shiprock presents a radon barrier and soil cover designed to be 7 feet thick. During the NRC's recent tour of five sites (including Shiprock on May 9, 1985), a set of radon barrier design calculations were delivered to Banad Jagannath of NRC by T.R. Wathen. These calculations by the Remedial Action Contractor indicate a required average thickness of 8 feet. In addition, the material identified for

the cover has changed from a silty sand (in the RAP) to a sandy silt (in the specifications). The Technical Assistance Contractor staff has indicated in telephone conversations that these design differences have been reconciled, and the NRC has requested the calculations pertinent to the reconciled design. Prior to completion of the radon barrier, the NRC must receive and review this information and provide DOE with concurrance in this aspect of design.

Response:

Please refer to the enclosed Response to NRC Comment 2 of "Response to NRC Geotechnical Comments of May 17, 1985, on Shiprock Construction Documents (Specification)".

CONTRACT NO. ASD-34-6703-S-85-0055
EXHIBIT A
DATED JULY 25, 1985
REVISION A
SCOPE OF WORK
ADDITIONAL SITE CHARACTERIZATION EFFORTS
DATA GAPS -- SHIPROCK MILL TAILINGS SITE

Drilling - Soil Sampling - Monitoring Well Installation

1. Scope and Objectives

This project will enable ground water data to be gathered in the floodplain area downgradient of the Shiprock tailings site, Shiprock, New Mexico for use in evaluating the geohydrology of the area.

This project is being conducted by Jacobs Engineering Group, Inc. (JEG) in support of its prime contract with the U.S. Department of Energy under the Uranium Mill Tailings Remedial Action (UMTRA) Project.

2. Location

The site is located in the State of New Mexico, approximately one (1) mile immediately south of the City of Shiprock, as shown on Figure 1.

3. Site Access

Figure 2 depicts the site and tentative locations of the proposed boring and boring nests. Legal access to the drill locations will be the responsibility of Jacobs Engineering Group Inc. (JEG). It will be the Subcontractor's responsibility to comply with the terms of the access agreements which will be provided by JEG prior to the commencement of work under this subcontract. It will also be the Subcontractor's responsibility to determine accessibility of the drill sites with the equipment proposed for use on the project as well as to determine the location of all pipes, underground structures, overhead power lines, and other impediments to his operations and to exercise due precautions in conducting operations. The Subcontractor shall be responsible for providing all necessary equipment to perform the work and transport said equipment to the drill locations and for maintaining productivity. (Note: this area may be marshy, brush-covered sandy or with other difficulty of access.) Adjustments in drill site locations may be made, if requested by the Subcontractor and approved by JEG, avoid obstructions or to obtain better accessi-

bility. Drill site restoration to original condition will be the responsibility of the Subcontractor.

4. Supervision

All technical activities shall be under the supervision of the JEG Technical Representative (TR). The TR may designate a Field Technical Representative (FTR) to act in his absence from the site. All drilling and other fieldwork shall be under the immediate supervision of the TR. No work shall commence without the TR's approval. Tentative locations of the drill holes shall be staked in the field by the TR.

5. Personnel

Only experienced, qualified technical and professional personnel familiar with this type of work will be acceptable. The Subcontractor shall notify the TR, prior to its mobilization date, if the Subcontractor intends to substitute any drill rig, equipment and personnel previously proposed and approved by Jacobs for use on the project.

6. Field Documentation

Drilling logs: JEG shall maintain a complete daily drilling log detailing all rig functions, hours, footages, material and water used, and other pertinent data. JEG will maintain the Daily Field Activity Report form to be used (a sample copy is attached). Further the TR will require the Subcontractor to sign the Field activity report daily in order to assure agreement by the parties on the level of effort expended, material used, amount of boring, and standby time incurred etc.

The Subcontractor shall maintain all normal required logs as specified by the State of New Mexico.

7. Drilling and Sampling Methods and Equipment

The nature of the materials at this site is such that drilling fluids may not maintain hole integrity. Simultaneous drilling and installation of casing will be required in order to advance the hole. Cobbles up to 2 feet in diameter may be encountered. A cable-tool rig is preferred. The Subcontractor shall not

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change or substitute the drill rig and equipment proposed for use on the project without the prior approval of the JEG TR. All equipment shall be in good working condition and maintained as such. Only potable water shall be acceptable as a drilling fluid.

Soil samples will be obtained from selected borings at selected depths. Upon notification by the TR, the Subcontractor will bail the hole clean, advance the hole another 1-2 feet, and bail the hole again. The TR will then collect the contents of the bailer to be used as a soil sample. It is estimated that approximately 10-20 samples will be taken.

To aid in determining completion intervals, the electrical conductivity of the ground water will be measured at approximately 10-foot intervals as the well to bedrock and the deeper member of each well nest is advanced. The TR will perform these measurements by lowering a probe down the hole.

All boring location identification and sample numbers will conform to JEG protocols for data collection. The TR shall be responsible for ensuring that these protocols are conformed to. The JEG protocols will be provided by JEG to the Subcontractor prior to the initiation of work.

8. Well Installation

a. General

Approximately twenty-three (23) (but not more than 25) wells will be drilled at locations designated by the TR, as shown in Figure 2. The wells will be installed in triplets, pairs, and singlets, with each well in the pair or triplet being not more than 10 feet horizontally away from any other well in that same group.

b. Drilling Sequence

The first boring drilled shall extend the full thickness of the alluvium. The depth of each boring in the other wells will be based upon the total depth of alluvium. Thereafter, the deepest member of each well set will be drilled first.

c. Well Dimensions

All borings will be at least 8 inches in diameter. Approximately 3 well triplets, 7 pairs, and 9 single wells will be installed. A summary of boring depths is presented in Table 1. Actual boring depths will be determined in the field by

the TR after the depth to bedrock has been determined during the drilling of the first well, and based on the depth to ground water at each location. Additional borings may be added if the depth to bedrock is significantly greater than 20-25 feet. The total estimated boring footage is 485 feet.

d. Monitoring Well Installation and Well Completion

Upon completing each boring, the Subcontractor shall install a monitoring well in the borehole under the direction of the TR/FTR. The monitoring well shall consist of 4-inch, threaded, flush jointed, Schedule 40 PVC pipe. Two feet of blank casing extended from the bottom of the hole, followed by a slotted well screen, 5 feet in length, shall be installed using devices to assure they are centered in the hole. The slotted well screen shall have three rows of slots cut on 120° centers, with 0.051-inch wide slots being 0.25-inch apart. Commercially manufactured well screen shall be used: field slotting of well screen is not acceptable. The Subcontractor will notify the TR of the screen supplier prior to purchase. Adjustments may be made in the well screen as the work progresses based on the gradation of the soils and the thickness of the water producing stratum at the screened interval. The bottom of the PVC casing shall have a flush-jointed plug.

The annular space shall then be backfilled. Backfill material from the bottom of the monitoring well to 2 feet above the screened interval shall consist of sand pack. The sand shall pass the No. 4 sieve and not more than 5 percent shall pass the No. 16 sieve. If sand is procured in bulk quantities, the Subcontractor shall furnish the TR with written documentation that the sand meets the minimum specified requirements. A bentonite seal shall then be installed directly above the graded sand backfill. The bentonite seal shall consist of a 2-foot thickness of bentonite pellets. The pellets shall be no more than one-half inch in diameter, with a minimum purity of 90 percent montmorillonite clay and a minimum dry bulk density of 82 pounds per cubic foot as provided by American Colloid Company, or equal. The annular space shall then be grouted to the surface. The grout shall consist of a neat cement mix with 4 pounds of commercial bentonite and approximately 7-1/2 gallons of water added per 94-pound bag of cement. The grout mix can be adjusted or changed as dictated by field conditions with approval by the TR. Mixing shall be done in a suitable jet or mechanical mixer.

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The grout shall be introduced into the annular space by means of a tremie pipe initially extending to the top of the bentonite seal and slowly raised as the backfilling progresses. Grouting shall be done from this point up in one continuous operation until the annular space is completely filled. The grout shall be allowed to settle for 24 hours and the Subcontractor shall then add any additional grout required to bring the grout level with the ground surface. The wells shall be developed by pumping, bailing or airlifting until clear water is obtained. The top of the PVC monitoring well shall be fitted with a threaded cap. The monitoring well shall be protected with a standard 8-inch ID steel pipe, 4 feet in length, with a locking cap. The locks and 3/4 inch diameter pins will be supplied to the contractor by JEG. A typical monitoring well installation is shown on Figure 3.

9. Project Safety

The Subcontractor shall comply with all applicable Federal, state, and local health and safety regulations and requirements including, but not limited to, those established pursuant to the Occupational Safety and Health Act (OSHA) and the UMTRA Health and Safety Plan (see Appendix A).

10. Project Schedule

Mobilization must be accomplished within one week after receipt of notification of contract award or the notice to proceed is provided, whichever is accomplished at [illegible]'s option. No drilling shall begin until all necessary equipment and materials specified herein are on-site and approval has been given by the TR.

11. Quality Assurance

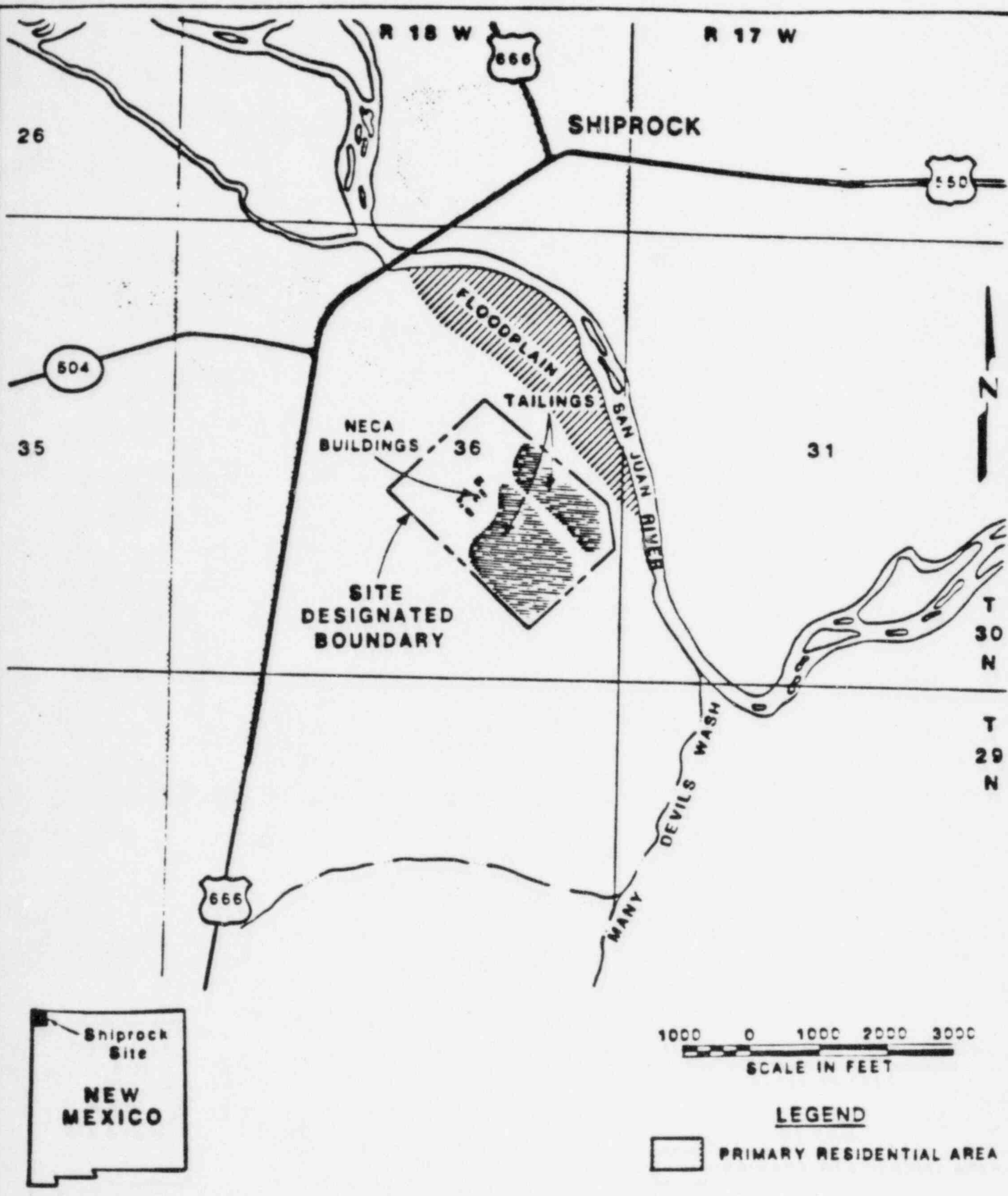
The TR shall direct all field work. This includes all exploratory drilling and monitoring well installation. All work shall be performed in conformance with the specifications and procedures set forth in this scope of work.

12. Permits

The Subcontractor shall abide by the requirements contained in permits, rights-of-entry agreements, letters of authorization, and environmental laws which are applicable to the data collection program. Copies of the documents will be transmitted to the Subcontractor by the CR prior to the start of fieldwork.

13. Drill Hole Abandonments

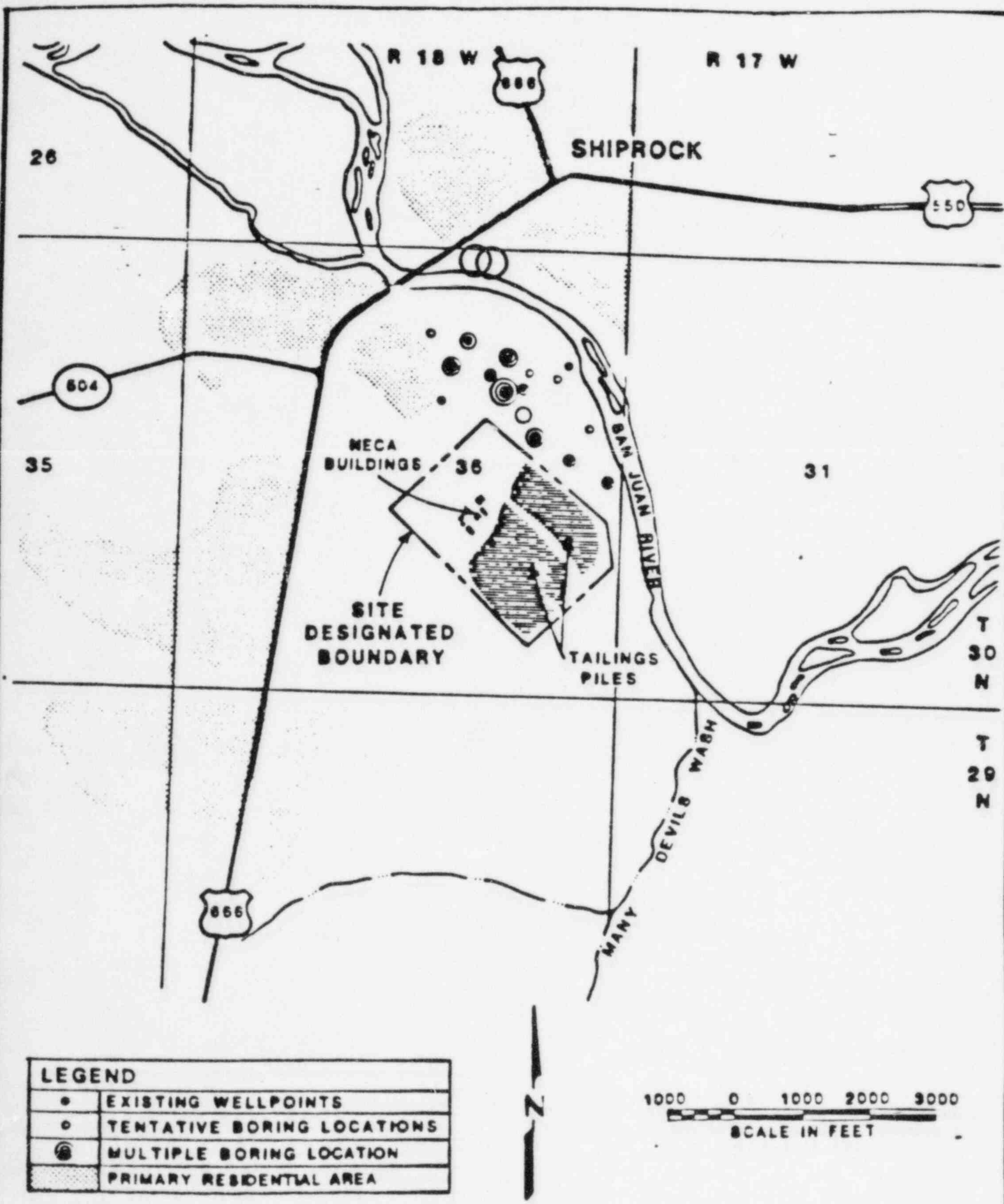
The Subcontractor shall plug or fill all drill holes as directed by the TR/FTR. All drill holes will be abandoned in compliance with New Mexico State regulations and/or directives (see Appendix B).



CONTRACT NO. ASD-34-6703-S-85-0055

EXHIBIT A

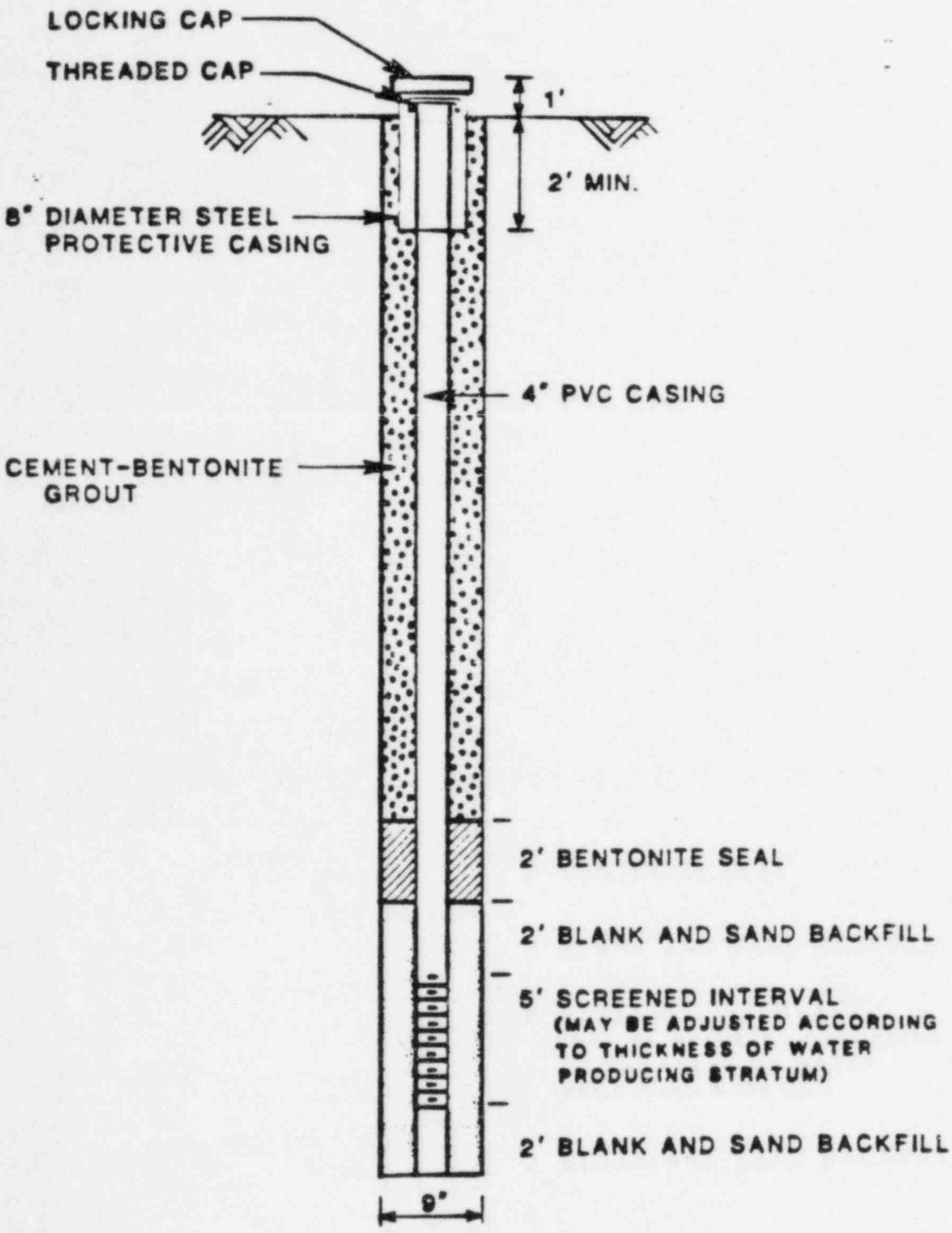
FIGURE 1
SHIPROCK SITE LOCATION MAP



CONTRACT NO. ASD-34-6703-S-85-0055
EXHIBIT A

FIGURE 2 SITE LOCATION AND TENTATIVE BORING LOCATIONS

Revision A



CONTRACT NO. ASD-34-6703-S-85-0055

EXHIBIT A

FIGURE 3 TYPICAL OBSERVATION WELL

CONTRACT NO. ASD-34-6703-S-85-0055

TABLE 1
REVISION A
TO
EXHIBIT A
SHIPROCK MILL TAILINGS SITE
SITE CODE = SHP01
DRILLING SUMMARY

Number of Borings	Estimated Depth (ft)	Minimum Hole Diameter (in.)	Monitoring Well Diameter (in.)
1	35	8	4
6	22	8	4
10	17	8	4
8	11	8	4

ATTACHMENT 2

RESPONSES TO NRC COMMENTS ON UMTRAP-SHP SEISMIC HAZARD ASSESSMENT

Within the content of a March, 1985 NRC memorandum to Leo Higginbotham from Malcom R. Knapp, several issues are discussed concerning the seismic hazard evaluation for the UMTRAP Shiprock site. The objective of this discussion is to clarify these issues and indicate what subsequent actions will be taken to satisfy NRC concerns on the adequacy of the seismic hazard analysis.

1.0 DEFINITION OF ACTIVE FAULT

As stated in our draft position paper on Seismic Hazard Assessments for UMTRAP, the TAC has been applying, and we propose the continued use of an active fault definition provided by Slemmons and McKinney (1977). This definition is as follows:

"An active fault that has slipped during the present seismotectonic regime and is therefore likely to have renewed displacement in the future. The fault activity may be indicated by historic, geologic, seismologic, geodetic, or other geophysical evidence. The most widely used definition in current engineering practice is for faults with evidence of Holocene displacement (approximately the last 10,000 years)."

The Holocene represents the period of time which has elapsed since the last Pleistgocene glaciation (i.e., about 10,000 to 12,000 years). It therefore may not accurately represent the duration of the "present seismotectonic regime." However, surficial deposits of Holocene age can often be distinguished from older units in the field, and reasonably accurate estimates of their ages can be made. Without extremely elaborate field studies, age estimates of Pleistocene and older units are generally subject to greater uncertainty. The Slemmons and McKinney definition quoted above provides a criterion which is applicable within the context of the UMTRAP seismic studies. These studies normally do not include the level of effort required to fully document the Pleistocene displacement history of a particular fault.

It is stressed that, in our view, the first criterion of the stated definition takes precedence over the need to have Holocene displacement in order to classify a fault as active. We recognize the difficulties in fully documenting the tectonic history of an area and the displacement history of faults. We will qualitatively classify a fault as active if it is believed that the structure has experienced displacement during the present seismotectonic regime.

In conclusion, the definition of an active fault quoted above will be applied to all seismic hazard evaluation for UMTRAP. We believe this definition is in keeping with the objectives of the EPA standards.

2.0 ADDITIONAL STUDIES TO CHARACTERIZE SEISMOTECTONIC SETTING

In order to support the previous assumption that no active faults exist in the near vicinity of the SHP site, the following studies will be completed.

- All pertinent geological, geophysical, geomorphological (including soils) mapping and data will be reinterpreted. These data will include existing maps which delineate active faults and bedrock faults of any age in the site region, and published discussions pertaining to the regional seismotectonic setting.
- A reassessment of instrumentally and historically recorded earthquake files will be completed. An epicentral map showing the geographic distribution of all known earthquakes within 120 miles of the SHS site will be compiled. The 120-mile radius has been selected so that any seismic event which could have caused detectable on-site ground motion will be included in the compiled record.
- If available, epicentral listing from state-maintained seismic nets, and available microseismic data will be obtained and evaluated.
- A selected suite of remote sensing imagery and conventional aerial photography at suitable scales will be acquired and analyzed. All photogeologic lineaments or geomorphic features indicative of an active seismic setting will be plotted. Specific attention will be paid to any active fault or bedrock fault traces identified by previous investigators. The photo coverage and analysis will encompass an area within a 12-mile radius of the SHP site, plus selected strip coverage of terrain which may contain active faults, up to a distance of 40 miles from the site.
- Utilizing the findings of the previous efforts, ground and aerial observations of known faults and suspect terrain indicative of an active seismic setting will be performed. All aerial reconnaissance missions will be completed under low-sun-angle conditions. The areas within a 12-mile radius of the SHP site will be thoroughly reconnaissanced and any outlying features which could influence the derived regional maximum credible earthquake (MCE) or influence the earthquake design parameters will be studied in the field.
- The findings of the efforts discussed above will be compiled into a series of maps which depict the distribution of active faults and known earthquake epicenters. A detailed discussion of the seismotectonic setting will be developed. If specific seismogenic sources can be identified, the MCE for each fault system will be estimated using the fault length versus magnitude relationships developed by Slemmons (1977, 1982). By applying deterministic and/or probabilistic methods, an estimate of on-site ground motion will be developed. Currently, acceptable attenuation relationships will be applied.

3.0 ACCELERATION & ATTENUATION

3.1 Terminology

It is agreed that the terms "sustained peak acceleration," "peak horizontal acceleration" and "effective peak acceleration" have distinct meanings, and that they should be explicitly and consistently used. Since the acceleration is determined from an attenuation relationship based on distance and magnitude, the appropriate term is also dependent on the attenuation relationship. Typically, most relationships provide predictions of peak horizontal acceleration. In future work, usage of this term will be strictly adhered to, and use of any other terms will be preceded by a definition.

It is of some importance to note that the term "peak horizontal acceleration" is not always strictly defined. Joyner and Boore (1981), for horizontal components of a record, regardless of instrument location or site geology. Campbell (1981) defines it as the mean of the peaks of the two horizontal components, noting that if a peak horizontal acceleration independent of horizontal direction is required, the mean should be multiplied by a factor of 1.13. As with Joyner and Boore (1981), the term is explicitly defined with reference to the record of the motion. Donovan (1973) uses the term "peak ground acceleration," however, it is not fully defined. This term could be interpreted to be the peak of the horizontal and radial components, but to use the adjective "ground" is misleading, since most instruments are located in or on buildings, dams or other structures. Schnabel and Seed (1973) use the term "maximum rock acceleration" primarily because the records used were the sites underlain by rock and not soil. Donovan and Bornstein (1978) use the term "peak horizontal ground acceleration," but do not provide a complete definition.

3.2 Attenuation Relationships

There are a large number of acceleration attenuation relationships available for use. We have selected eight such relationships for comparison. The seven expressions and one graphical procedure represent a reasonable cross section of available attenuation functions. Source distance (near-field versus far-field events), earthquake location (California, worldwide or United States events), the sophistication of the regression analysis employed in analyzing the data, and the number of records used are key elements in selecting an attenuation relationship to be used.

3.2 Attenuation Relationships - Continued

A comparison of the eight relationships is provided in Table 1, which lists acceleration values predicted by each relationship for magnitude 6 through 7 events at distances of 5 to 100 kilometers. A comparison of the acceleration values indicates that the largest differences occur for larger magnitude events in the near-field. Bolt and Abrahamson (1982) and Campbell (1981) specifically attempted to consider the fault mechanism effect in the near-field in their analyses, the latter by developing an alternate constrained relationship. Since Campbell's data base is broad, and his analysis attempts to consider earthquake location, his constrained relationship might be most appropriate for use in the near-field.

Blume (1977) has considered a large data base, but is limited to events in the United States, which would likely be biased by California events. The only relationships that do not consider mostly California events are those of Campbell (1981) and Donovan (1973). These relationships predict very similar acceleration values, which may be a reflection of their nonbias as regards to event location. Since the UMTRAP sites are located outside the California-Nevada area, either of these relationships might be most appropriate for use.

Considering that Campbell (1981) also has provided a more complete analysis of available data than the other articles, it is our position that either of his attenuation relationships should be used for the UMTRAP sites. It is noted that Campbell (1981) also considers in his analysis the variable location of the recording instrument and variable geologic site conditions.

3.3 Confidence Level

Table 1 also addresses the issue of confidence level by comparing 84 to 50 percentile acceleration predictions for several of the relationships considered. The differences are typically large, increasing with increasing magnitude and decreasing distance. Thus, as has been suggested, it might be most appropriate to use the 84 percentile value in design. However,it should be recognized that these relationships predict a peak acceleration value, which only represents one peak cycle of several hundred cycles recorded during an event.

To address the potential effect of using a peak value, Bolt and Abrahamson (1982) analyzed 62 records, finding that the 90, 95 and 99 percentile levels of acceleration differed remarkably. For one given record, the associated acceleration values were 0.23g, 0.28g and 0.62g. Because of this degree of difference, the authors suggest a percentile acceleration be used that is appropriate for the degree of risk involved.

Applying an 84 percentile bound in an attenuation relationship in conjunction with user of a peak value (which represents a 99+ percentile for a given record) is unduly conservative. It is our position that the mean value of a relationship based on peak values be used for the UMTRA project sites, as it represents a reasonable level of conservatism.

TABLE 1

Comparison of Acceleration Values
From Selected Attenuation Relationships

Reference	M = 6.0					M = 6.5					M = 7.0				
	5[1]	10	20	50	100	5	10	20	50	100	5	10	20	50	100
Joyner & Boore (1981)															
50 percentile	0.32[4]	0.22	0.12	0.044	0.016	0.43	0.30	0.16	0.058	0.022	0.57	0.40	0.22	0.078	0.029
84 percentile	0.58	0.41	0.23	0.080	0.030	0.78	0.54	0.30	0.106	0.040	1.03	0.72	0.40	0.14	0.053
Campbell (1981) (unconstrained)															
50 percentile	0.26	0.16	0.091	0.038	0.018	0.34	0.22	0.13	0.056	0.028	0.42	0.29	0.18	0.083	0.042
84 percentile	0.38	0.24	0.13	0.054	0.027	0.49	0.32	0.19	0.081	0.040	0.61	0.43	0.26	0.12	0.061
Campbell (1981) (constrained)															
50 percentile	0.29	0.18	0.094	0.029	0.010	0.34	0.24	0.14	0.048	0.018	0.38	0.29	0.19	0.074	0.031
84 percentile	0.42	0.27	0.14	0.043	0.015	0.50	0.35	0.20	0.071	0.027	0.56	0.43	0.27	0.11	0.046
Donovan (1973)[2]	0.25	0.20	0.13	0.062	0.028	0.33	0.26	0.18	0.083	0.038	0.44	0.35	0.24	0.11	0.051
Schnabel & Seed (1973)[3, 2]	0.49	0.35	0.20	0.07	0.02	0.57	0.40	0.25	0.09	0.025	0.63	0.48	0.32	0.15	0.035
Blume (1977)															
50 percentile	0.15	0.11	0.069	0.026	0.010	0.25	0.18	0.12	0.044	0.017	0.51	0.37	0.23	0.081	0.029
84 percentile	0.37	0.28	0.17	0.066	0.025	0.62	0.47	0.29	0.11	0.043	0.92	0.68	0.41	0.15	0.052
Bolt (1982)[2]	0.24	0.17	0.090	0.013	0.00	0.43	0.35	0.23	0.063	0.007	0.28	0.26	0.23	0.13	0.049
Donovan & Bornstein (1978)															
50 percentile	0.39	0.25	0.13	0.050	0.022	0.47	0.31	0.18	0.074	0.035	0.56	0.40	0.25	0.11	0.056
84 percentile	0.53	0.34	0.21	0.081	0.036	0.63	0.42	0.27	0.12	0.057	0.75	0.54	0.35	0.17	0.091

[1]Distance in kilometers
[2]Mean (50 percentile) value
[3]Graphical relationship
[4]Acceleration in gravity units

REFERENCES

Blume, J.A., 1977, The SAM Procedure for Site-Acceleration-Magnitude Relationships, Proc. of Sixth World Conference on Earthquake Engineering, New Delhi, India.

Bold, B.A. and Abrahamson, N.A., 1982, New Attenuation Relations for Peak and Expected Accelerations of Strong Ground Motion, Bulletin of the Seismological Society of America, Vol. 72.

Campbell, K., 1982, Near-Source Attenuation of Peak Horizontal Acceleration, Bulletin of the Seismological Society of America, Vol. 71.

Donovan, N.C., 1973, Earthquake Hazards for Buildings, in Building Practices for Disaster Mitigation, National Bureau of Standards Building Science Series 46.

Donovan, N.C. and Bornstein, A.E., 1978, Uncertainties in Seismic Risk Procedures, Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, No. GT-7, July.

Joyner, W.B. and Boore, D.M., 1981, Peak Horizontal Acceleration and Velocity from Strong-Motion Records Including Records from the 1979 Imperial Valley, California, Earthquake, Bulletin of the Seismological Society of America, Vol. 71.

Schnable, P.B. and Seed, H.B., 1973, Accelerations in Rock for Earthquakes in the Western United States, Bulletin of the Seismological Society of America, Vol. 63, No. 2.

Slemmons, D.B., 1977, Faults and Earthquake Magnitude: U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, State-of-the-Art for Assessing Earthquake Hazards in the United States Series, Report 6, Miscellaneous Paper S-73-1.

Slemmons, D.B. and McKinney, R., 1977, Definition of "Active Fault": U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, Miscellaneous Paper S-77-8.

Slemmons, D.B., 1982, Determination of Design Earthquake Magnitude for Microzonation, in Proceedings of the Third International Earthquake Microzonation Conference, Seattle, Washington, Vol. 1, pp. 119-130.