40-3453

State of Utah

DEPARTMENT OF ENVIRONMENTAL QUALITY
DIVISION OF RADIATION CONTROL

Michael O. Leavitt
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November 10, 1997 **EXPRESS MAIL**

Joseph J. Holonich
Nuclear Regulatory Commission
Division of Waste Management, Uranium Recovery Branch
Washington, D.C. 20555-0001

Dear Mr. Holonich:

Please find enclosed some information brought to the attention of the Radiation Control Board regarding the Atlas tailings Final Technical Evaluation Report (FTER). At the November 5, 1997 meeting of the Utah Radiation Control Board, a Utah citizen, Peter Heaney, requested that the Board require the State to intervene on some issues relating to the FTER. During the meeting, the Division of Radiation Control (DRC) staff responded to many of Mr. Heaney's concerns and agreed with a majority of the findings in the FTER.

However, a major issue of concern was identified that focuses on the adequacy of the launchable rock apron design such as to protect the pile from the possibility of a probable maximum precipitation event and river migration. The Board was presented with information from three sources: (1) Original information from Peter Heaney which was supplemented at the Board meeting with a U.S. Army Corps of Engineers evaluation of the launchable rock apron design; (2) an evaluation by the Division of Radiation Control staff regarding the information submitted originally by Mr. Heaney (excluding the new Corps information); (3) an evaluation of the information by Atlas Corporation. Since the Board or DRC staff had not had the opportunity to review the latest information and legal counsel was unavailable, this issue was deferred to the December 5, 1997 meeting of the Board.

We are enclosing all information received to date regarding this issue. We would suggest that the NRC review this information to determine the appropriateness of the launchable rock apron design as described in the FTER. We would suggest that a conference call be scheduled as soon as possible at a mutually convenient time with you and appropriate staff to discuss the state concerns regarding this issue as well as the information brought to light as a result of the U.S. Army Corps of Engineers report. Our expectation would be that NRC respond in writing to the issue of the rock apron to the Division prior to the Radiation Control Board meeting on December 5, 1997. You, or anyone from

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your staff, would be welcome to come and attend the Board meeting and speak to the rock apron issue. We look forward to hearing from you regarding this important issue. If you have any questions, please do not hesitate to contact me.

UTAH RADIATION CONTROL BOARD

William J. Sinclair

William J. Sinclair, Executive Secretary

c: Dianne Nielson, Executive Director UDEQ w/o enclosures
Richard Blubaugh, Atlas Corporation w/enclosures
Radiation Control Board members w/o enclosures
Richard Bangart, NRC Office of State Programs w/o enclosures
Charles Hackney, NRC Region IV w/o enclosures

Enclosures

7 Nov 97

Radiation Control Board
State of Utah
Moab meeting

Re: Disparities between NRC Final Technical Evaluation Review Design of Launchable Rock Apron and US Army Corps of Engineers recommendations.

Dear Radiation Control Board:

The following are the major disparities between the NRC Final Technical Evaluation Review Design of the Launchable Rock Apron and the US Army Corps of Engineers recommendations as described in the attached letter report.

1.) US Army Corps of Engineers requires that anticipation of maintenance will be part of the completed project. USACE Engineer Regulation 1110-2-1405, the design presentation must include an operation and maintenance section. The NRC's FTER claims the design will be free of active maintenance for the 200-1000 year design life.

2.) US Army Corps of Engineers recommends a design channel bottom El 3940 due to local bend scour. The NRC's FTER use of a design channel bottom El 3947 is not supported in Appendix O of the Atlas Tailings Pile Reclamation Plan.

3.) US Army Corps of Engineers recommends determining rock size by using river flow rather that overflow, The NRC's FTER recommends determining rock size by using over flow.

4.) US Army Corps of Engineers recommends use of 10.2 ft.sec as the maximum flow velocity in designing the launchable rock apron. The NRC's FTER uses a maximum flow velocity of 7 ft.sec.

5.) US Army Corps of Engineers recommends use of post launch 18" blanket with a D_{50} 9.25" with D85/D15 = 2 to address river flows. The NRC's FTER recommends an 8" blanket with a D_{50} of 11.2".

6.) US Army Corps of Engineers recommends a before launch 1 ft. section 54" thick by 31.3 ft long or 141ft^3/ft. The NRC's FTER recommends a before launch 1 ft section 30" thick by 20 ft long or 50 ft^3/ft.

The differences in Item 6 results in a necessary volume of 13,787 yd^3 of launchable rock riprap in the US Army Corps of Engineers design versus 4889 yd^3 in the NRC approved design.

It is my understanding that the Council on Environmental Quality has funded a study to determine quantitative and qualitative ground water effects on the Colorado River from leeching contaminants. They are proposing to drill some sampling wells to obtain the necessary information.

In addition to my previous request for Utah State intervention pertaining to the launchable rock apron, I request that the State of Utah intervene to obtain the necessary information to define the historical northern boundary of the Colorado River through sediment analysis. I also request the State of Utah intervene to obtain the necessary information to define the subsidence zone at this site including how far the zone extends under the tailings pile itself.

I thank you for your time.

Sincerely,

Peter Haney

COUNTY COUNCIL
Bart Leavitt, Chair
Ray Pene, Vice Chair
Ken Ballantyne
Al McLeod
Harvey Merrell
Dale Mosher
Frank Nelson
Telephone: 801-259-1346

GRAND COUNTY
UTAH

COUNTY ADMINISTRATOR
Earl W. Sires
Telephone: 801-259-1346
Fax: 801-259-2574

Dr. James Houston
Waterways Experiment Station
CEWES-CV-Z
3909 Hall Ferry Road
Vicksburg, MS 39180

October 21, 1997

Dear Dr. Houston,

The Grand County Council hereby requests the U.S. Army Corps of Engineers to review the Nuclear Regulatory Commission's launchable rock apron design. As we understand, the project will include the following:

a. Design channel bottom elevation resulting from local bend scour.
b. Rock size in apron.
c. Before launch apron configuration and rock quantity.
d. Effect of subsidence on rock apron launching.

We understand that the study will not address the expected duration of the proposed design after has launched because no information exists on the long term (250-1,000 years) performance of launchable stone. The study will provide a discussion of what standard Corps practice is for maintaining bank revetments.

Enclosed is the $3,000 fee for the letter report.

Please fax a draft of the letter report for review to 435.259.2574 between the hours of 8 a.m. to 5 p.m. MST or 435.259.2959 thereafter.

Sincerely,

Bart Leavitt

Bart Leavitt, Chairman
Grand County Council

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FACSIMILE TRANSMITTAL HEADER SHEET

For use of this form, see AR 25-11; the proponent agency is ODISC4

	COMMAND/ OFFICE	NAME/ OFFICE SYMBOL	OFFICE TELEPHONE NO. (AUTOVON/Comm.)	FAX NO. (AUTOVON/Comm.)
FROM:	USAEWES	Dr. Stephen T. Maynord CEWES-CN-N	601/634-3284	601/634-3218
TO:	Grand County Council	Mr. Peter Haney	801/259-6131	801/259-5369

CLASSIFICATION	PRECEDENCE	NO. PAGES (Including this Header)	DATE-TIME	MONTH	YEAR	RELEASER'S SIGNATURE
		33	7 10:45	11	97	Debbie Fulcher

REMARKS Attached is letter report. Original will be mailed via federal express this afternoon.

Space Below For Communications Center Use Only

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DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
3909 HALLS FERRY ROAD
VICKSBURG, MISSISSIPPI 39180-6199

REPLY TO
ATTENTION OF

November 7, 1997

Coastal and Hydraulics Laboratory

SUBJECT: Letter Report: "Review of Rock Apron Design, Atlas Uranium Mill, Moab, Utah"

Grand County Council
ATTN: Mr. Peter Haney
125 East Center Street
Moab, Utah 84532

Dear Mr. Haney:

Enclosed is subject Letter Report. Should you have any questions concerning this, please contact Dr. Stephen T. Maynord (601/634-3284).

Sincerely,

James R. Houston

James R. Houston, PhD
Director
Coastal and Hydraulics Laboratory

Enclosure

Review of Rock Apron Design, Atlas Uranium Mill, Moab, Utah

Introduction and Objectives

The comments presented herein address the proposed rock apron design at the Atlas Uranium Mill at Moab, Utah. The Nuclear Regulatory Commission requires that the Atlas proposal meet a design life criteria of 200-1000 years such that ongoing active maintenance is not necessary to preserve isolation. During a 200-1000 year life, many factors can affect the stability of the Atlas mill tailings pile. This review focuses only on the design of the rock launching apron that is placed along the toe of the tailings pile. The rock apron is intended to protect the pile should the Colorado River migrate to the position of the pile. Specifically, the objective of this review is to address the following questions:

1). How does the 1000 year life affect the applicability of US Army Corps of Engineers (USACE) guidance?
2). What has been the historical performance of rock launching aprons and under what conditions have they been successful.
3). What should be the design channel bottom elevation for computing the quantity of rock required in the rock apron?
4). What rock size should be used in the apron?
5). What is the before launch apron configuration and required rock quantity?
6) How would the anticipated subsidence affect the performance of the rock apron?

Description of Project and Proposed Rock Apron

A layout of the tailings pile along the Colorado River is shown in Encl 1. Encl 2 shows cross-section locations near the tailings pile and cross-sections 4-8 are shown in Encls 3-7. Encl 8 shows the alignment and location of the rock apron. Encl 9 shows a section through the tailings pile which includes the rock apron. Appendix O (encl 10), "Calculation Brief, Rock Apron/toe Design", contains details of the design evaluated herein. The rock apron is placed at the toe of the 3V:10H tailings pile side slope in a before launch section 20 ft wide by 2.5 ft thick. The design uses a 1V:2H launch slope and a scour depth of 21 ft which is based on a river thalweg elevation of 3947 and a before launch section bottom elevation of 3968.

Applicability of USACE Guidance

Guidance presented in USACE (1994) is based on the anticipation that surveillance / monitoring and, if necessary, maintenance will be part of the completed project. USACE Engineer Regulation 1110-2-1405 states that the design presentation must include an operation and maintenance section. Projects with a design life of 200-1000 years are well beyond the project life used in USACE bank stabilization projects and have a greater risk of failure before reaching their design life if the same design principles are used. This greater risk can be offset somewhat by greater conservatism in the initial design. Consequently, a project with a 200-1000 year design life should apply USACE guidance in a conservative manner.

The objectives of the Atlas protection is to isolate tailings and contaminants without requiring active maintenance. In 10 CFR Part 40, Appendix A- "Criteria Relating to the Operation of Uranium Mills and the Disposition of Tailings or Wastes Produced by the Extraction or Concentration of Source Material From Ores Processed Primarily for their Source Material Content", under Part IV, Long-Term Site Surveillance, Criterion 12 states "As a minimum, annual site inspections must be conducted by the government agency responsible for long-term care of the disposal site to confirm its integrity and to determine the need, if any, for maintenance and/or monitoring." If the rock apron launches, these annual site inspections will be required to insure the long term performance of the rock apron. Factors such as insuring a reserve quantity of rock in the apron, dealing with concentrated overbank drainage, and the stability of the upstream and downstream ends of the protection should be evaluated.

Historical Performance of Rock Aprons

The rock apron proposed for the Atlas site is broadly classed as launchable stone protection and has been widely used in the USACE. Launchable stone techniques include the following:
A. Windrow revetments- Primarily used on Missouri River. Rock is placed along top bank similar to the proposed Atlas site. Sites installed in the 1970's are performing as designed with a few sites having depleted the supply of rock. One of the characteristics of some of the Missouri River sites is a launch slope much steeper than the 1V:2H slope found in purely non-cohesive material. This steep launch slope is almost certainly due to the presence of cohesive soils. The primary effect of the steep bank angle is the slope effect on the stability of the rock.
B. Trench-fill revetments- Widely used on the Arkansas and Red Rivers, and to a lesser extent on the Mississippi River. Involves placing the stone section at about the low water reference plane preventing significant underwater slope preparation. Above the trench, standard revetment construction techniques are employed. Trench-fill revetments along the Arkansas River were constructed in the 1960's and have performed as designed in conjunction with periodic maintenance.
C. Weighted riprap toe- Riprap is placed at the toe of the eroding bank and the stone launches only in response to bed lowering. Weighted riprap toe was widely used in the USACE(1981) experimental bank protection program with good performance.

Launchable stone functions best when the eroding bank or channel bottom is non-cohesive. Since few riverbanks are purely non-cohesive and most contain layers of cohesive soil, the successful performance of many field installations of launchable stone shows that some layering is allowable. Banks having thick layers of cohesive or highly erosion resistant soil beneath the launchable section will fail in large blocks as they are undermined and the stone will launch in an uneven manner leading to poor slope coverage.

Design Channel Bottom Elevation

Many river engineers believe that scour at the toe of bank protection is one of the most common causes of failure. The design channel bottom elevation at a protection site that has a design life of 200-1000 years must consider not only local bend scour but any channel bed degradation that may occur over the project life. Appendix O gives no supporting information for the selection of El

3947 as the design channel bottom elevation. NUREG(1997), 4.5.1.2.3 Colorado River, states "To estimate the depth of scour associated with migration of the river, the licensee conservatively assumed that the river channel would retain essentially the same elevations and configuration in its migrated state as in its current state." El 3947 is the maximum thalweg elevation between cross-sections 4 and 8. Standard practice is to set the design channel bottom elevation some distance below the minimum thalweg elevation in the reach. This minimum thalweg elevation is almost always measured at low flow conditions and streams generally tend to locally scour bendways during higher flows. The magnitude of "some distance" is generally based on experience with the same or similar streams and can vary from as little as 1-3 ft on a small stream to much larger on large rivers. The design life of 200-1000 years certainly warrants a conservative selection of the design channel bottom elevation.

Musseter and Harvey (1994) was examined for pertinent information concerning selection of the design channel bottom elevation. They conclude that the bed of the river does not generally degrade during a flood event and that long term degradation will not occur because of the bed rock control downstream at the Portal. This does not preclude local bend scour which will be addressed in the following paragraph.

To compute local bend scour, a bend configuration must be defined that is consistent with the Colorado River having migrated over to the tailings pile. One possible mechanism for this migration is for erosion to begin between cross-sections 4 and 5 which has high velocity (Mussetter and Harvey, Fig 4.5), highest right bank shear (Mussetter and Harvey, Fig 5.3), and lowest centerline radius Rc / water surface width W (Mussetter and Harvey, Fig 5.4). Once scour is projected along the right bank between cross-sections 4 and 5, it is likely that the bend that forms to go around the tailings pile could have an Rc/W about equal to the minimum of the present channel which is about 3.0. However the Colorado River finds its way over to the tailings pile, an Rc/W = 3.0 is not overly conservative since it is occurring in the present channel. One additional concern is that if the river finds its way over to the pile, the East point of the tailings pile/rock apron will introduce a significant discontinuity along the outer bank.

In addition to the above channel configuration, a flow condition must be defined that produces the most significant scour. Bankfull is about 40,000 cfs and the Portal produces backwater effects on the right bank at the higher flows with little influence below about bankfull. The guidance used herein for estimating local bend scour is presented in Maynord (1996) and is an empirical method based on flows at about bankful or higher and flows not having significant backwater effects. The hydraulic profile for a discharge of 45,000 cfs was used to define the local bend scour. Higher flows were not used in the scour analysis because the method in Maynord (1996) will overestimate scour if significant backwater effects are present. Using cross-section 4 as representative of the upstream end of the bend, water surface elevation = 3965, main channel average velocity = 5.45 ft/sec, channel area = 45000/5.45 = 8257 sq ft, water surface width W=850 ft, average depth D=8257/850 = 9.7 ft, Rc/W=2.94, and aspect ratio W/D=87.6. Using Maynord (1996) with a safety factor = 1.08 results in a maximum water depth in the bend of about 25 ft or a design channel bottom elevation of 3965-25 = 3940.

A design bed elevation less than the El 3947 used in Appendix O is recommended for the rock apron. Using local bend scour estimating techniques, El 3940 is determined and is about 5 ft below the minimum low flow thalweg elevation (el 3945) in the existing channel reach (cross-sections 4-8).

Rock Size in Rock Apron

Appendix O determines rock size for overflow and for river flow and determines that overflow is the most critical and requires an average stone size of 11.2". Both overflow and river flow will be examined in the following analysis.

The overflow analysis in Appendix O uses Stephenson (1979) to determine the required rock size for flow down the 1V:2H launch slope of the rock apron resulting in an average stone size of 11.2". Stephenson borrows stability coefficients from guidance presented in Olivier (1967) and apparently ran no tests of his own. This reviewer could not find Olivier (1967) but neither Stephenson (1979) nor Olivier (1973) state the slopes for which the equation is valid. However, Knauss (1979) (encl 11) reports that Olivier's data were limited to 20 percent slopes. The derivation by Stephenson results in rock size much larger than Olivier (1973) when both are extrapolated out to a 1V:2H slope. Knauss reports on tests by Hartung and Scheurlein (1970) which address slopes up to 1V:1.5H. Knauss simplified the Hartung/Scheurlein work which showed that the required rock size is less than the size required by extrapolating the Olivier (1973) equation for the steep slope of 1V:2H. The difference is attributed to the presence of entrained air which begins on a rock covered slope at about 15-20 percent. Rock size recommended by Knauss using the Hartung/Scheurlein work for 1V:2H slope and q=0.434 cfs/ft is 0.18 ft and had a unit stone weight close to that proposed for the rock apron. Rock size from Olivier(1973) for the same conditions is 0.39 ft but note again that Olivier's tests were limited to 20 percent slope. Recent experimental work by Robinson, Rice, and Kadavy (1997) evaluated slopes up to 40 percent in a physical model and 16.7 and 33.3 percent in a prototype. Extrapolating their equation up to 50 percent slope (1V:2H) and q=0.434 cfs/ft resulted in an average rock size of 0.21 ft which is quite similar to the Knauus results. Robinson, Rice, and Kadavy used rock having specific gravity of 2.54-2.82 but all but one of the steep slope tests were conducted with specific gravity of 2.54-2.59 which is close to the 2.47 at the proposed rock apron. Data from Abu-Sayf (1976) for a 1V:2H slope and specific gravity of 2.65 were plotted as part of this analysis and show a required average rock size of 0.23 ft when using a rock size 25 percent greater than the size at failure. These three similar results strongly suggest that the Stephenson (1979) method resulting in an average rock size of 11.2" (0.93 ft) is too conservative for a 1V:2H slope.

The selection of a unit q = 0.434 cfs/ft assumes that there is no concentration or channelization of flow. Since this slope will be formed by the launching of the rock apron, the launched stone will not be smooth and uniform. It also seems unlikely that the 3V:10H slope of the tailings pile will introduce flow uniformly to the rock apron slope all throughout the 200- 1000 year life. Abt et al (1988) discusses concentration and channelization of flow on an overflow embankment. Abt recommends a factor of 3.0 for the condition where flow will channelize on the overflow embankment. Considering that this is a slope formed by launching and a design life of 200-1000 years, a factor of 3.0 times the uniform q seems appropriate. Based on Robinson, Rice, and Kadavy (1997) the average stone size should be 0.4 ft for q = 3(0.434) = 1.3 cfs/ft. The Knauss relation results in an average stone size of 0.38 ft. The Abu-Sayf data show an average stone size of 0.39 ft. The lesser unit weight for the proposed rock apron requires a larger stone size of about 15% resulting in a recommended rock size of 0.45 ft to handle overflow.

Rock size in the apron to remain stable against river flow is determined using procedures in USACE (1994). The same bend configuration used in the design channel bottom analysis is used in the riprap design. The riprap analysis uses the same Rc =2500 ft which is the minimum radius in the existing reach. The riprap analysis uses main channel cross-section 4 which is quite similar [illegible]ain channel cross-sections at 7 and 8 and differs from cross-sections 5 and 6 primarily [illegible]se of the presence of the mid channel bar. Main channel average velocities from cross-[illegible]tion 4 were used in the analysis (encl 12). Cross-section 5 yielded similar and only slightly higher velocities. The primary difference between this analysis and the Appendix O analysis is that the large drop in main channel average velocity between cross-sections 5 and 6 (encl 12) is not assumed to last throughout the design 200-1000 year life of the project. Even if one believes this large drop will persist, the hydraulics at cross-section 5 should be used because it is located near the upstream end of the tailings pile. This large drop in main channel velocity is because the left overbank is assumed to convey a significant amount of flow. Whether that remains true for the next 1000 years depends on factors such as the effects of vegetation such as the Tamarisk on blockage of the flow and the land practices of the owners of the left bank, some or all of which has been purchased by the Nature Conservancy, who may change what has been done in the past. Riprap size is determined for flows at or below 70,300 because of the backwater effects from the Portal at higher flows as well as some conveyance of the overbank across from the tailings pile. Results are shown in the following table.

Q, cfs	Water Surface El	main chan avg vel, ft/sec	max vel at outer bank*, ft/sec	depth at outer bank*, ft	width, ft	Rc/W	D50**, ft
20,000	3959.5	3.8	5.7	10.8	600	4.2	0.19
45,000	3965	5.5	8.3	15.2	830	3.0	0.47
70,300	3968	6.8	10.2	16.8	850	2.9	0.77

* velocity and depth at 20% upslope from toe as per USACE(1994)
** Based on gradation having D85/D15=2 and safety factor of 1.1

Guidance in USACE (1994) recommends launchable stone having a D85/D15>= 2.0. Stone used in overflow embankment should have D85/D15<=2.0 as per USACE(1994). A gradation having D85/D15 = 2.0 is recommended. Thickness in Appendix O is 2D50 resulting in a recommended thickness of 2(0.77)=1.54 ft. Use T=18" thick blanket with D85/D15=2 to address river flows.

Based on overflow and river flow, river flow is the most severe stress and requires an 18" blanket thickness.

Before Launch Apron Configuration and Rock Quantity

The design in Appendix O uses an increase of stone volume of 50 percent (uncertainty factor = 1.5) for dry placement. launch depth greater than 15 ft as per USACE (1994). The volume/unit length of bank is determined as

$$volume\ /\ ft = (scour\ depth)\ (\sqrt{5})\ (T)\ (uncertainty\ factor)$$

where T is the required blanket thickness of the riprap if it had been placed by mechanical means rather than by stream launching. Maynord and White (1995) discuss the finding that the stone launches to about 85 percent of T. Guidance in USACE (1994) uses 100 percent of T to compute required stone volume. The use of (8"/12) for T in Appendix O is not correct. Using the parameters from Appendix O with $T = 2D50 = 2(11.2")/12 = 1.87$ ft results in

$$volume\ /\ ft = (21\sqrt{5})\ (1.87)\ (1.5) = 131\ ft^3/ft \tag{2}$$

Using the parameters determined herein, the required volume is

$$volume\ /\ ft = (3968-3940)\ (\sqrt{5})\ (1.5)\ (1.5) = 141\ ft^3/ft \tag{3}$$

Using a before launch section thickness of 3T results in a section 54" thick by 31.3 ft long.

Effect of Subsidence on Performance of Rock Apron

Subsidence of rock aprons is not an issue that has been studied and not normally required for the design life of USACE rock apron structures. This section primarily contains opinions of the reviewer concerning what subsidence scenarios could lead to problems with the rock apron.

Subsidence of the before launch toe section (because of its relatively massive nature) could lead to an unprotected zone between the rock apron and the toe of the tailings pile.

Subsidence of a differential nature along the length of the rock apron could lead to a concentration of runoff down the launched apron slope and fail the launched slope due to the concentrated flow.

Summary and Conclusions

Using Corps of Engineers guidance, the apron design was evaluated as follows:

1) A design channel bottom elevation of 3940 is proposed for determining stone quantity in the rock apron. El 3940 is about 5 ft below the existing low flow thalweg in the reach near the tailings pile.

2) Based on this evaluation, river flows will dominate attack of the proposed rock apron. Rock in the apron should have a D50 of at least 0.77 ft that will result in a layer thickness of 18" after launching. The rock gradation in the apron should have a D85/D15 = 2. Using a before launch section thickness of 3 times the after launch layer thickness, the before launch section should be 54" thick and 3[illegible]2 ft in length.

3) Two potential issues regarding subsidence are presented.

References

Abt, S.R., Wittler, R.J., Ruff, J.F., Lagrone, D.L., Khattak, M.S., Nelson, J.D., Hinkle, N.E., Lee, D.W. (1988). "Development of riprap design criteria by riprap testing in flumes:phase II," NUREG/CR-4651, prepared by Colorado State University and Oak Ridge National Laboratory.

Abu-Sayf, A.Y. (1976). "The stability of stones in steep shallow streams," Phd dissertation, Civil Engineering Department, Imperial College, London.

Hartung, F. And Scheurlein, H. (1970). "Design of overflow rockfill dams,"International Commission on Large Dams, Montreal.

Knauss, J. (1979). "Computation of maximum discharge at overflow rockfill dams," Proceedings of the XIII International Commission on Large Dams, New Delhi.

Maynord, S.T. (1996). "Toe-Scour Estimation in Stabilized Bendways", ASCE Journal of Hydraulic Engineering, Vol 122, No. 8, August.

Maynord, S.T. and White, D.M. (1995). "Toe Scour and Bank Protection Using Launchable Stone", US Army Engineer Waterways Experiment Station, Technical Report IIL-95-11.

Mussetter, R.A., and Harvey, M.D. (1994). "Geomorphic, Hydraulic, and Lateral Migration Characteristics of the Colorado River, Moab, Utah," prepared for Canonie Environmental and Atlas Corporation.

Nuclear Regulatory Commision (1997). "Final evaluation report for the proposed revised reclamation plan for the atlas corporation Moab mill," NUREG-1532.

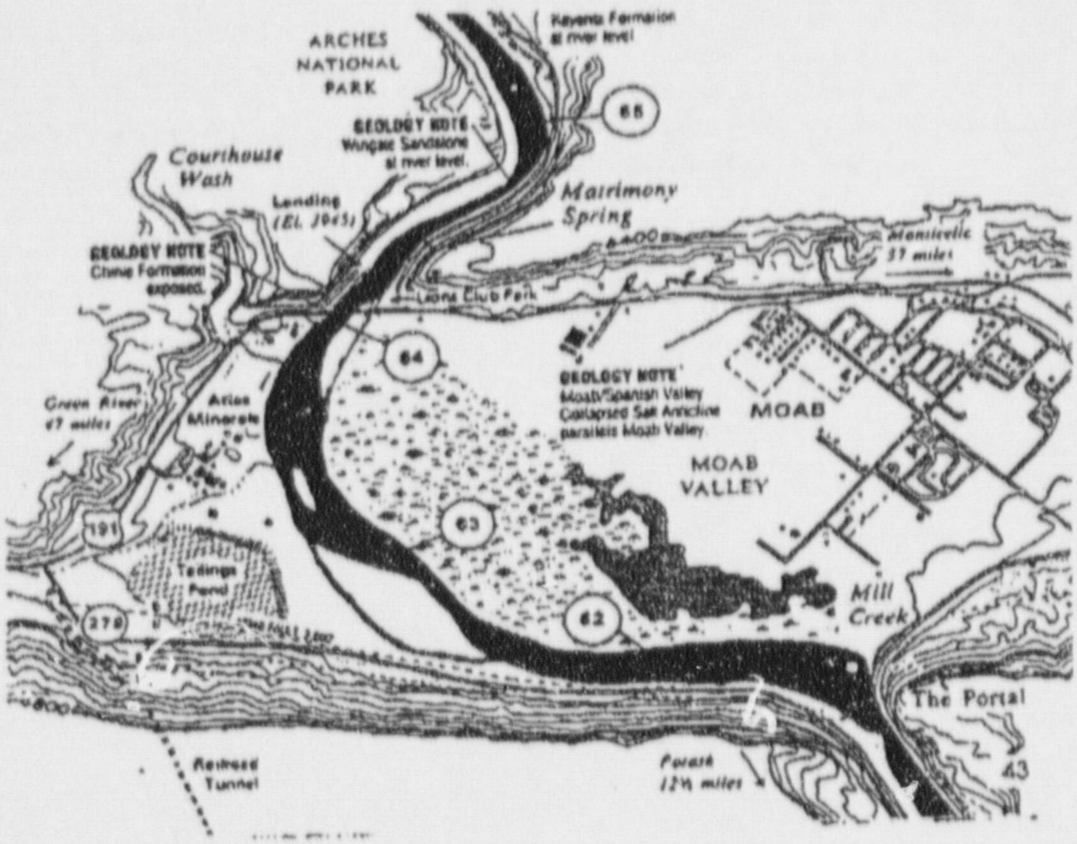
Olivier, H. (1967). "Through and overflow rockfill dams- new design techniques," Proceedings of the ICE, March.

Olivier, H. (1973). "Some aspects of major river diversion during construction," ICOLD Congress, Madrid, 1973.

Robinson, K.M., Rice, C.E., and Kadavy, K.C. (1997). "Rock Chutes for Grade Control," MLDCI Conference, Proceedings paper, Oxford, MS.

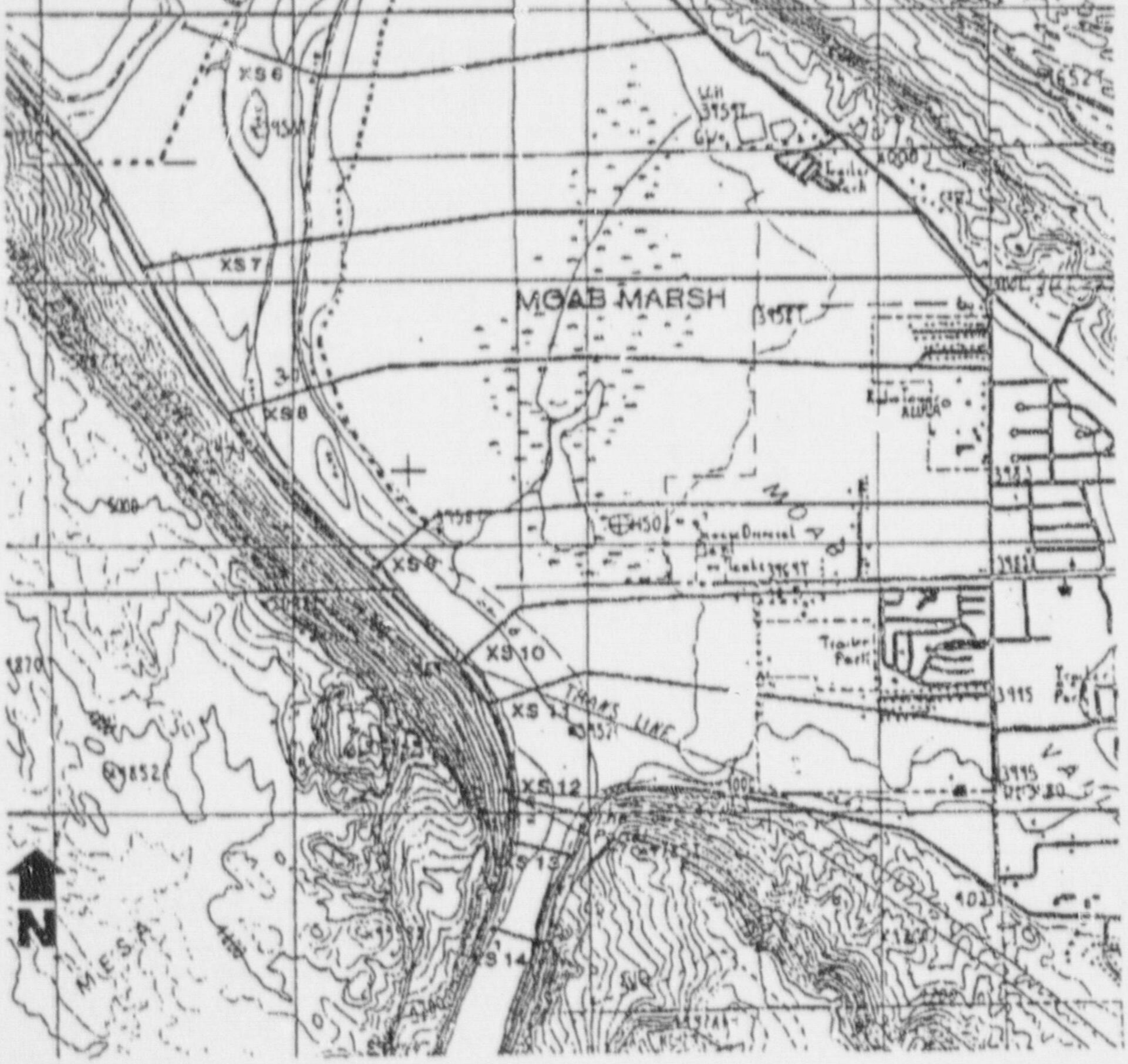
USACE. (1981). "Final Report to Congress, The Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251," Washington, D.C.

USACE. (1994). "Hydraulic design of flood control channels," EM 1110-2-1601, U.S. Government Printing Office, Washington, D.C.



* Circled numbers are river miles upstream of the Colorado/Green River confluence.

Figure 1.1. Location map of the project site (from Belknap, 1991).



SCALE: 1" = 2000'

Figure 2.1. Map of the study reach showing the location of cross sections and other significant features.

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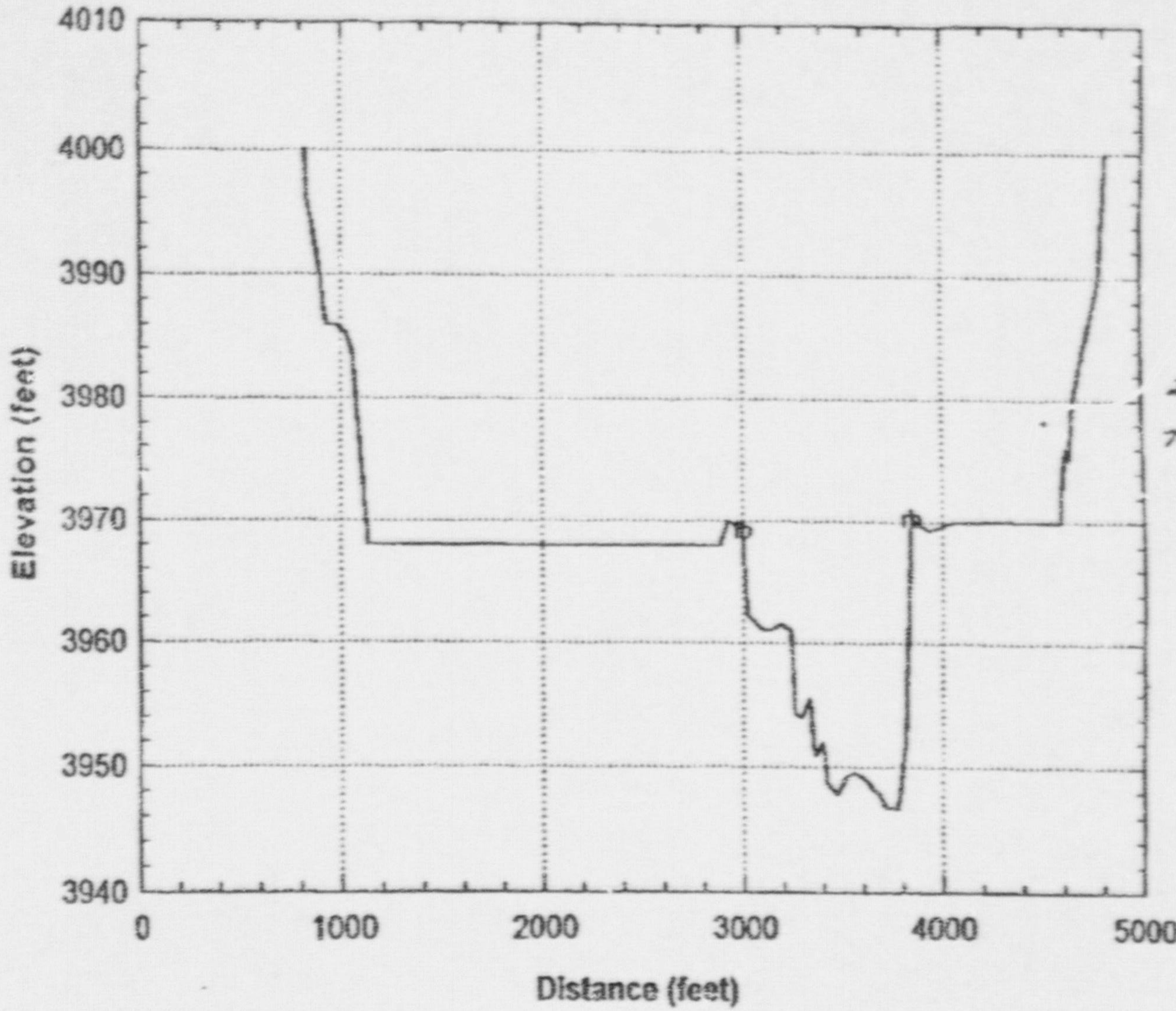
CROSS SECTION 4



Q oL
45,000 = 3965
29,000 = 3959.5
70,300 = 3968

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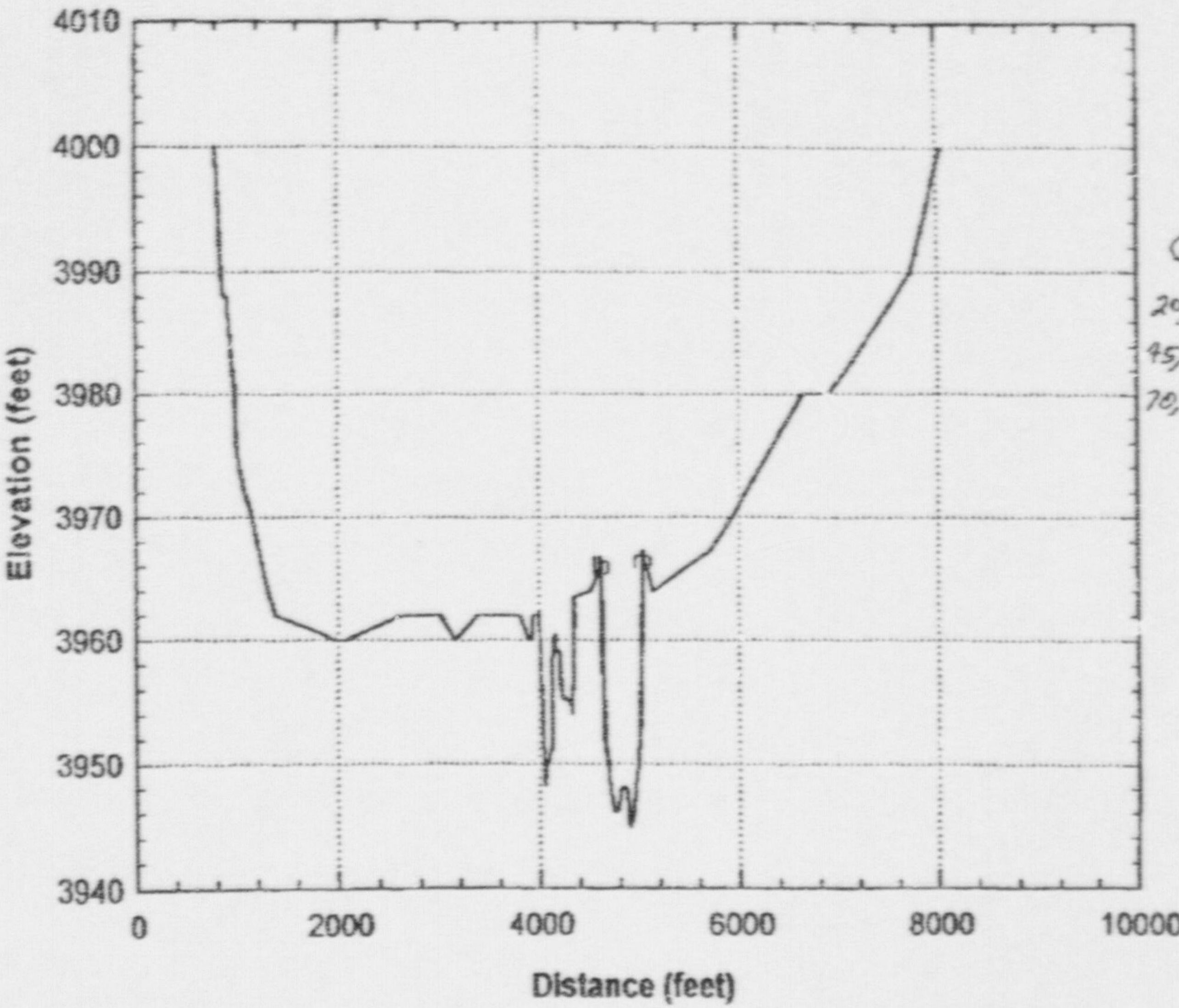
CROSS SECTION 5



Q	el
20,000	3959
45,000	3964.5
70,000	3967.5

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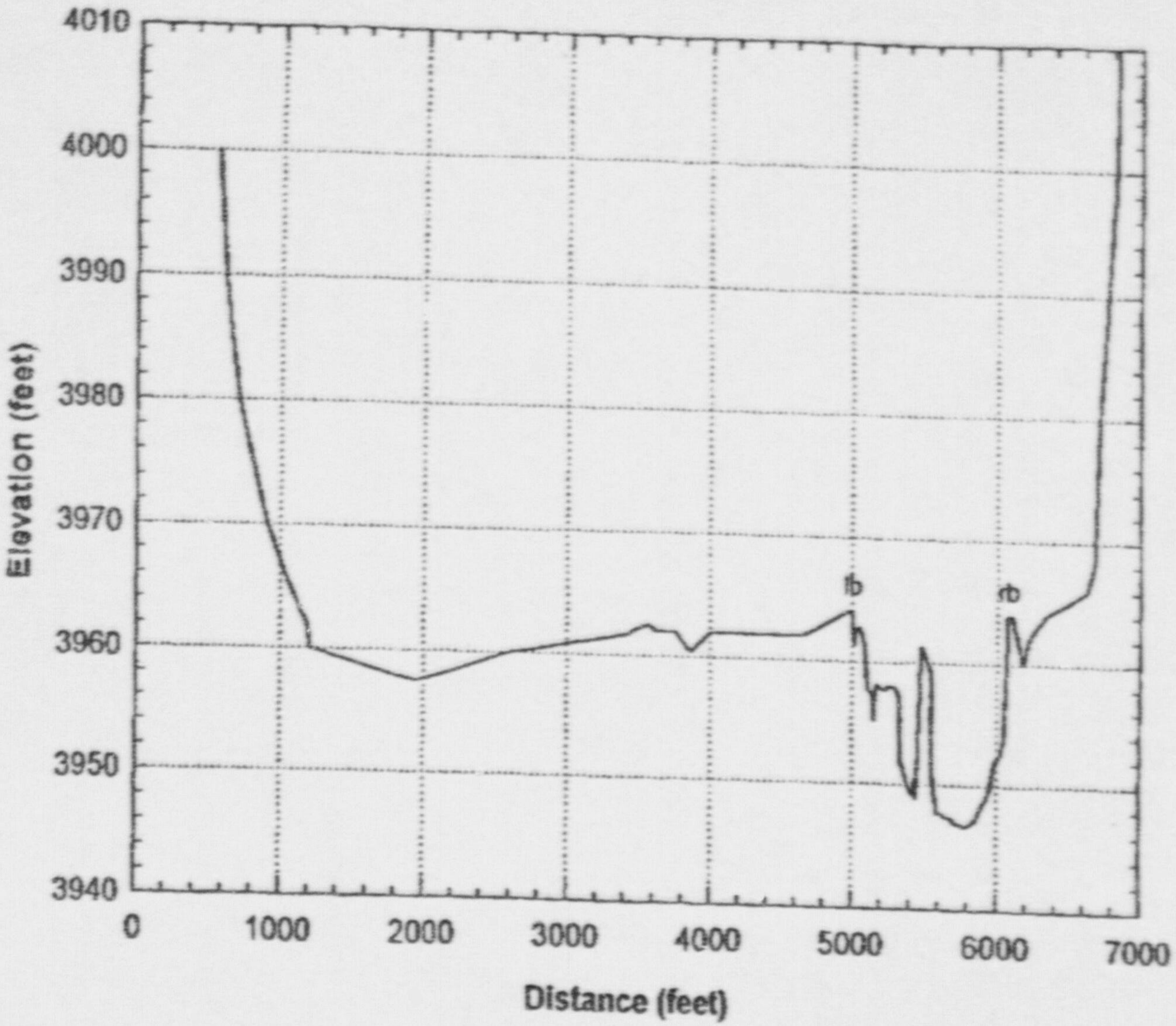
CROSS SECTION 6



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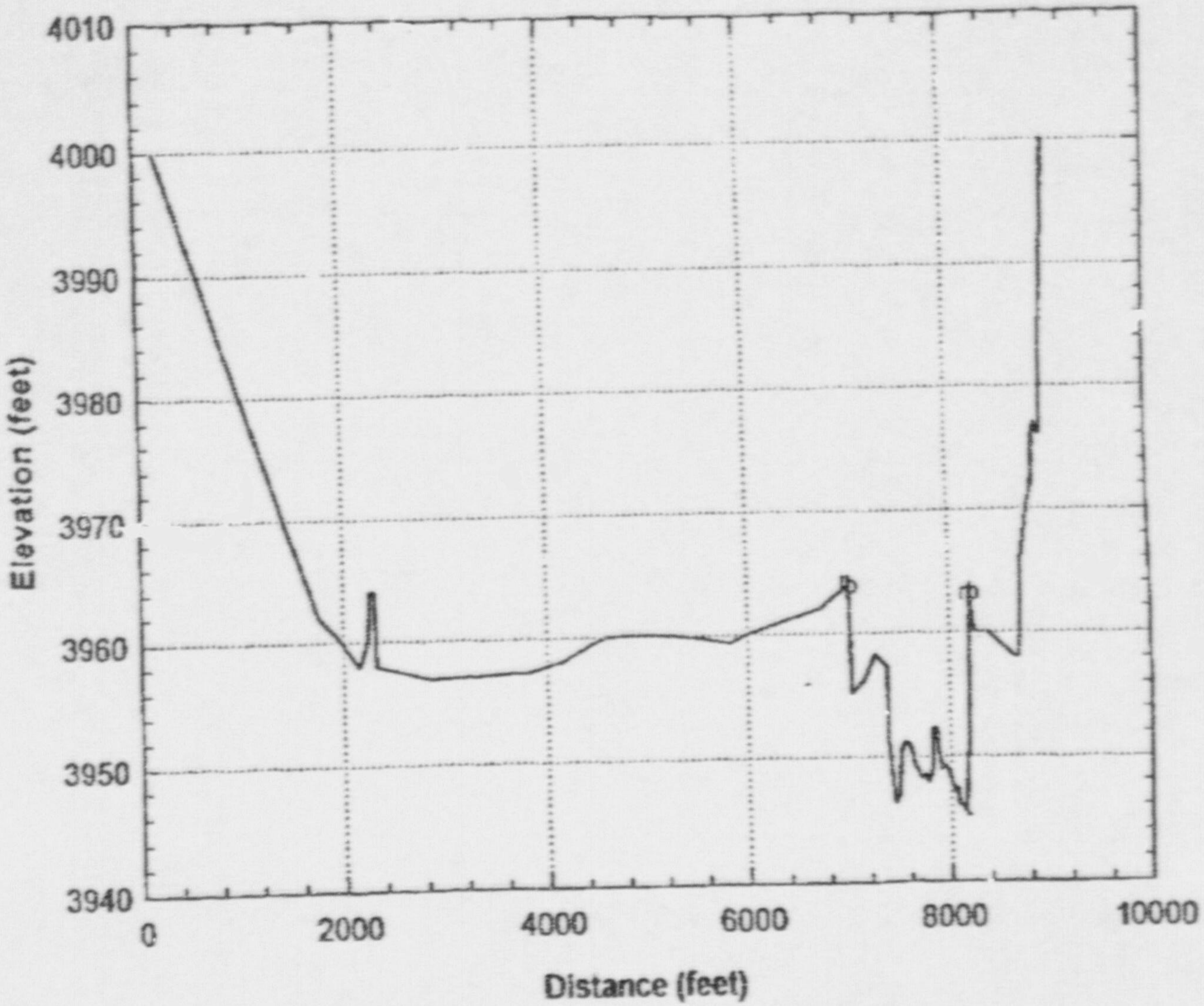
Mussetter Engineering, Inc.
Final Report Nov. 1994

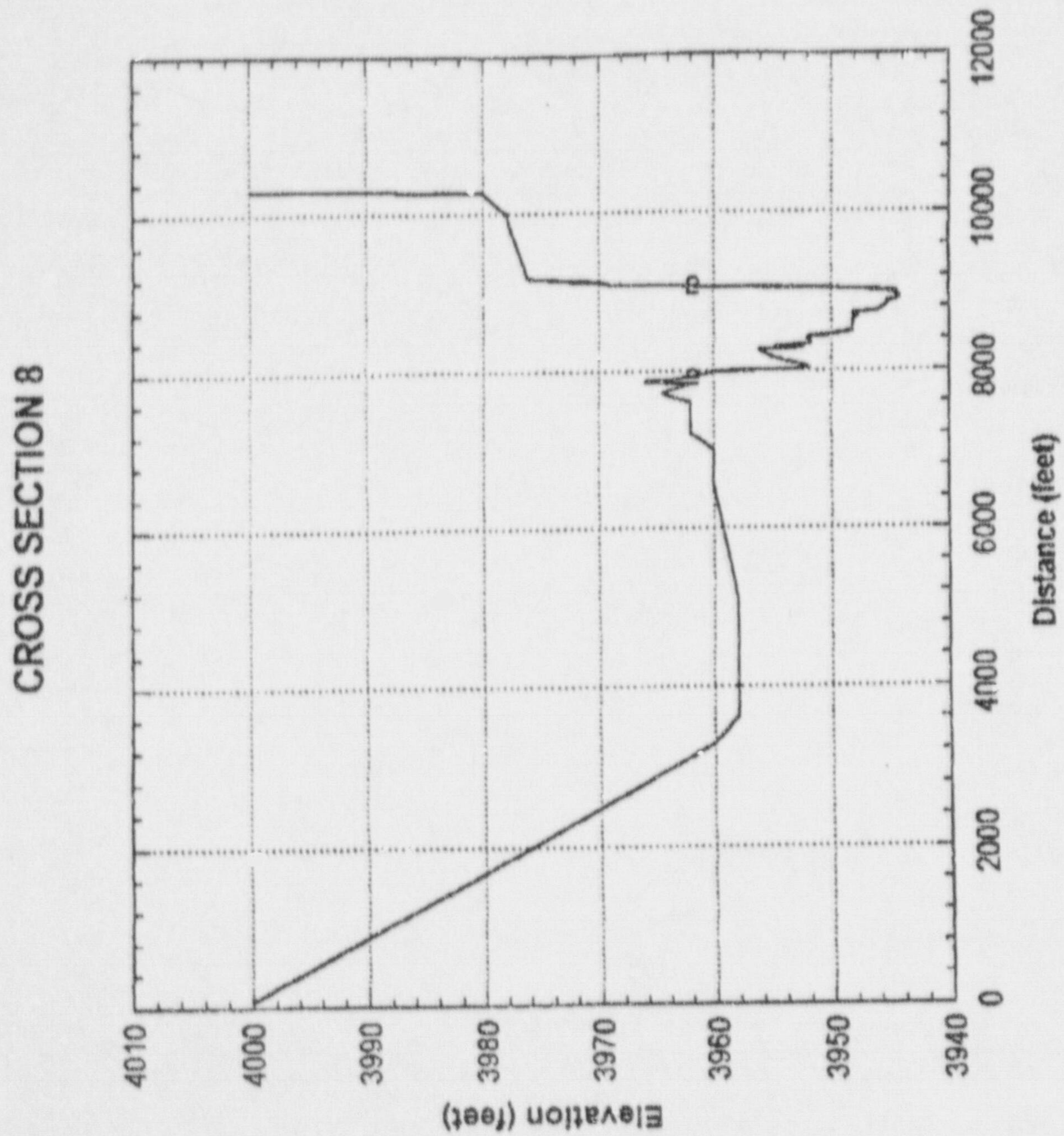
encl 6

CROSS SECTION 7



A.7

Mussetter Engineering, Inc.
Final Report Nov. 1994



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Final Report May, 1994

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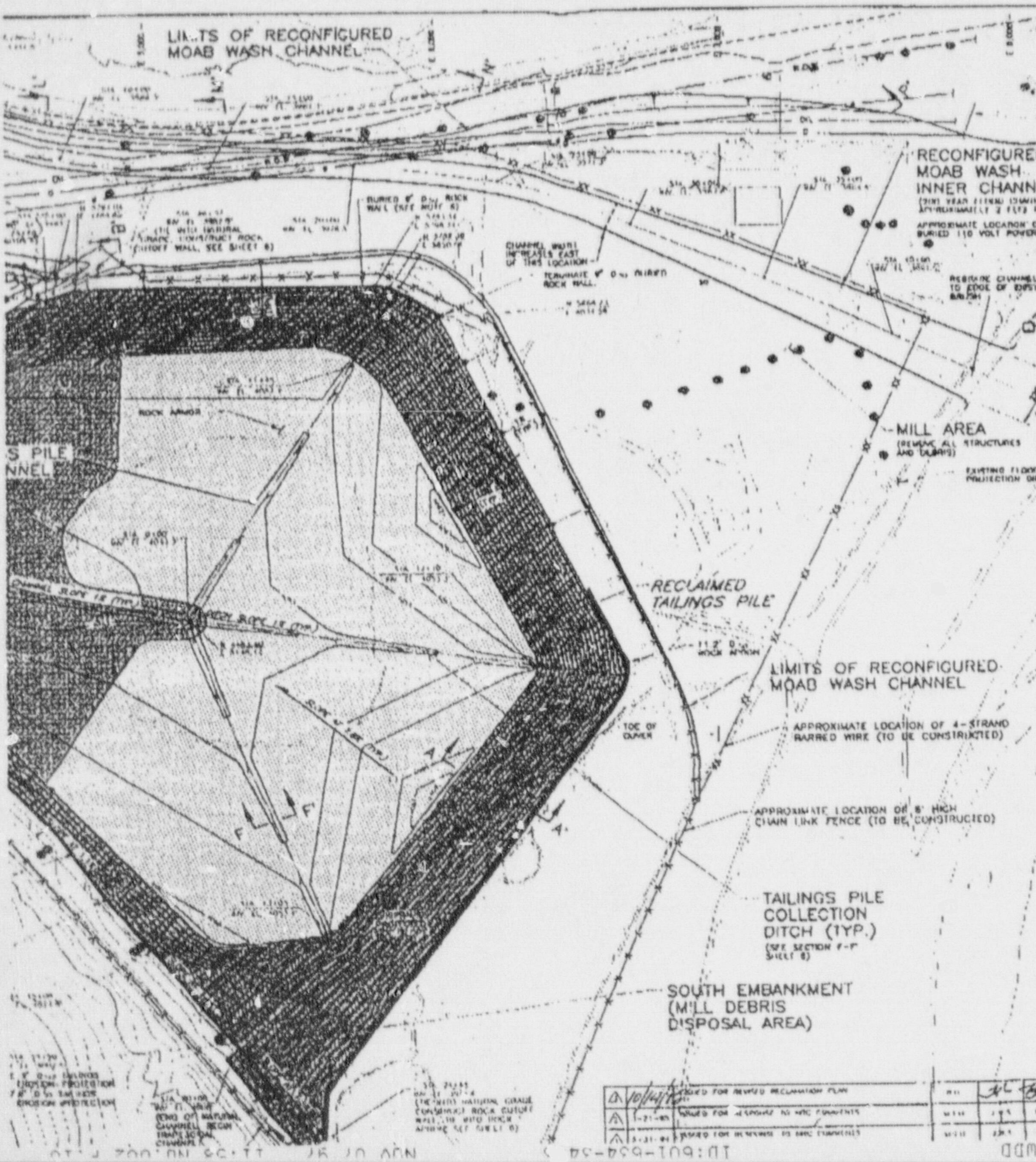
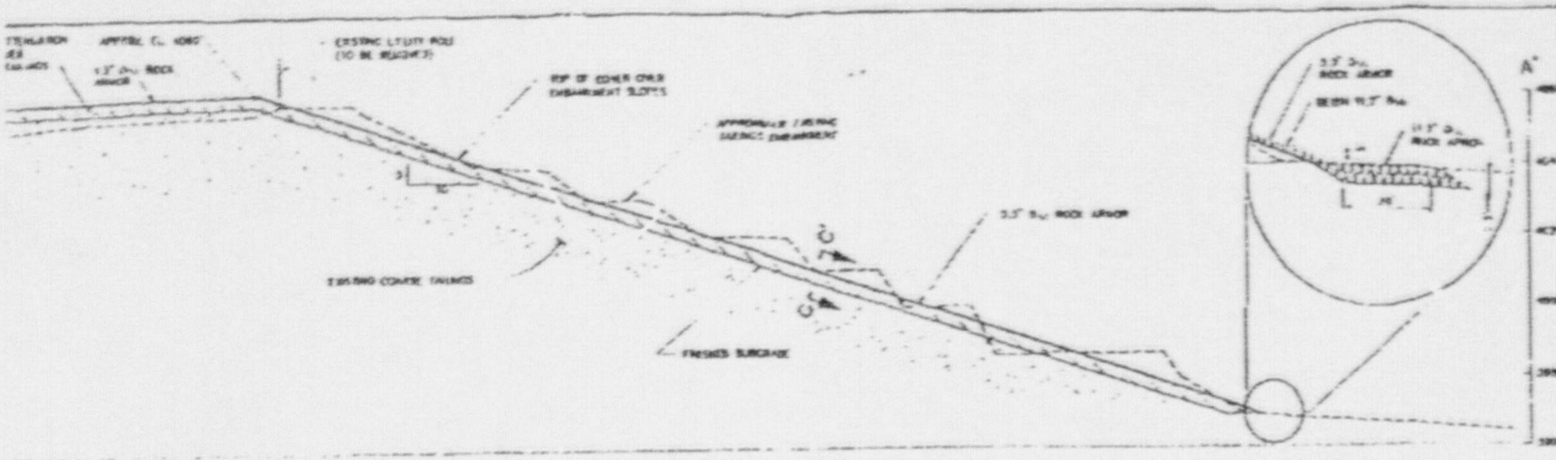
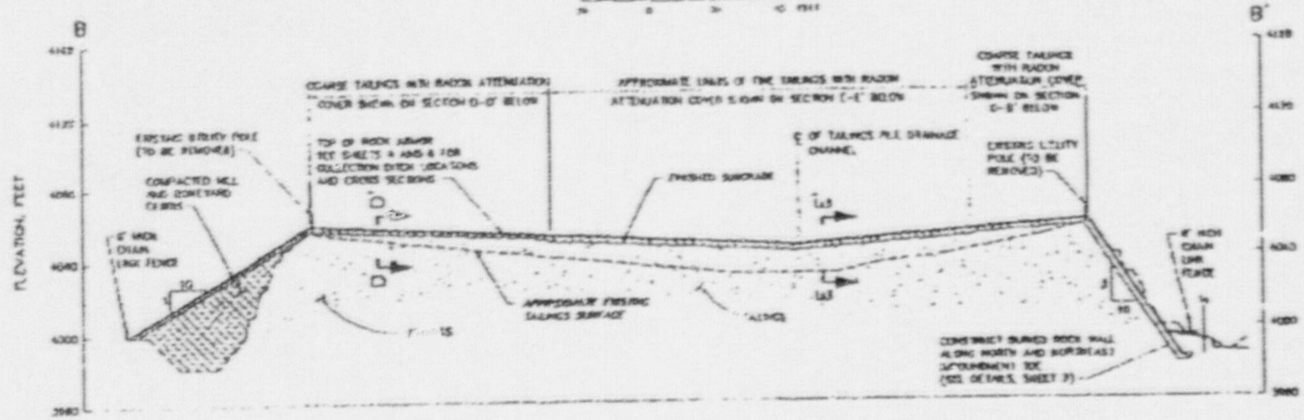

LIMITS OF RECONFIGURED MOAB WASH CHANNEL
RECONFIGURED MOAB WASH INNER CHANN
MILL AREA
(REMOVE ALL STRUCTURES AND DEBRIS)
ROCK ARMOR
RECLAIMED TAILINGS PILE
LIMITS OF RECONFIGURED MOAB WASH CHANNEL
TOE OF COVER
APPROXIMATE LOCATION OF 4-STRAND BARBED WIRE (TO BE CONSTRUCTED)
APPROXIMATE LOCATION OF 6' HIGH CHAIN LINK FENCE (TO BE CONSTRUCTED)
TAILINGS PILE COLLECTION DITCH (TYP.)
(SEE SECTION F-F' SHEET 6)
SOUTH EMBANKMENT (MILL DEBRIS DISPOSAL AREA)



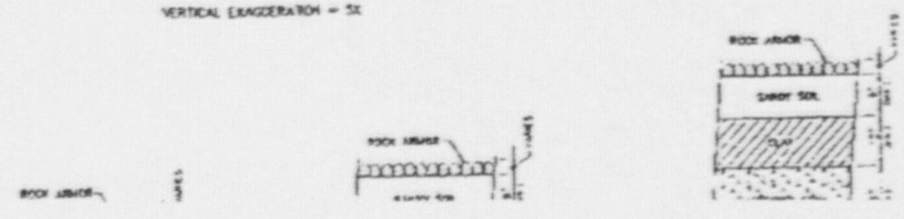
CROSS SECTION A-A'
TYPICAL EMBANKMENT SLOPE
(LOOKING NORTHEAST)



SECTION B-B'
TAILINGS PILE CROSS SECTION
(LOOKING WEST)

VERTICAL EXAGGERATION = 5X

NOTES:
1. IF THE QUALITY OF T... BORROW SOURCE US... ASSUMED IN THE DE... MUST BE CHANGED A...
2. SEE SPECIFICATIONS ... GRADATION AND THI...



b | encl

SMITH

APPENDIX O

CALCULATION BRIEF
ROCK APRON/TOE DESIGN

encl 10

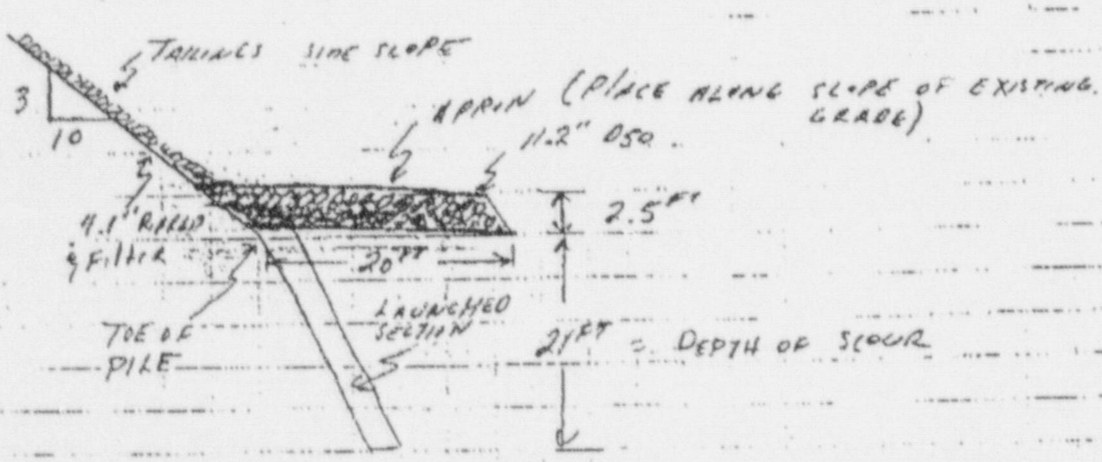
1

CanonieEnvironmental

By JWS Date 11/30/94 Subject ATLAS - APRON/TOE Sheet No. 1 of 14
Chkd. By DR Date 12/29/94 DESIGN Project No. 88-067-21
1/4" X 1/4"

PURPOSE - The purpose of this calculation brief is to design an apron for the EASTERN edge of the ATLAS' Tailings Pile in Moab, UT. The purpose of the Apron is to protect the tailings pile should the Colorado River migrate to the Toe of the Tailings pile as suggested by the NRC, (ATLAS - Uranium Mill, questions and comments, 1994). Figure 1 shows the Tailing Pile, Proposed Location of the tailings pile, and the Colorado River

Results - 5000 CUBIC YARDS OF 11.2" D_{50} APRON is required to protect the eastern edge of the tailings pile should the river migrate to the toe of the tailings pile. A conceptual view of the Apron is shown below:



CanonieEnvironmental

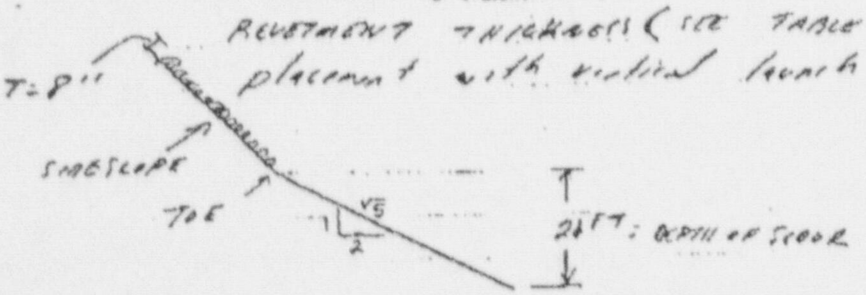
By JWC Date 11/30/94 Subject ATLAS - Apron Design Sheet No. 2 of 14
Chkd. By DR / BWH Date 12/29/94 / 3/11/95 Project No. 88-667-21
1/4" X 1/4"

CALCULATIONS

① DETERMINE LAUNCHABLE VOLUME OF RIPRAP
The procedure given in EM1110-2-1601 (1994 UPDATE) will be followed. - SEE ATTACHMENT A FOR EXCERPTS. SECTION 3-10 "Revetment Toe Protection methods" - method D will be followed.

LAUNCH SLOPE = 1V on 2H
SCOUR DEPTH = 21 FT = TOE ELEVATION (≈3968) - RIVER THALWEG ELEVATION (≈3947) - SEE FIG. 2 TOP X-SEC IS WORK NEAR PILE

THICKNESS AFTER LAUNCHING - INCREASE BY 50% OF BANK REVETMENT THICKNESS (SEE TABLE 3-2 ATTACH. A; dry placement with vertical launch distance = 21 FT > 15 FT)



∴ Volume/ft = Launch Slope Length × Bank Revetment Thickness × % increase.

$= 21\sqrt{5} \times 8''/12 \times 1.5$

$= 47\ FT^3/FT$. - FOR ADDITIONAL FACTOR OF SAFETY INCREASE TO $50\ FT^3/FT$

LENGTH OF APRON ALONG TOE = 2640 FT - From approximately 300 FT. NORTH OF EASTERN MOST EDGE OF PILE SOUTH TO SOUTHERN MOST POINT OF PILE AS SHOWN ON FIGURE 1.
BWH 3/11/95

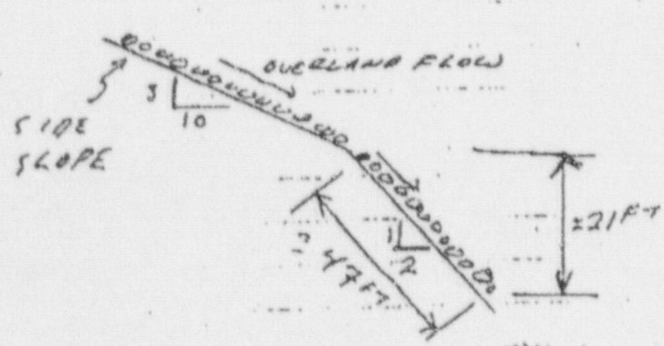
CanonieEnvironmental

By JWC Date 11/20/94 Subject Atlas - Apron Design Sheet No. 3 of 14
Chkd. By DR Date 12/29/94 Project No. 88-067-21
1/4" X 1/4"

TOTAL volume = 50 ft³/ft + 2640 ft
= 4889 CY
or ≅ 5000 CY

② DETERMINE APRON ROCK SIZE

USE STEPHENSON'S METHOD (NUREG-4620, NUREG 4651) Because method is applicable to slopes greater than 10%. Rock will be designed based on final configuration (i.e. after launching on to 1V:2H slope). Overland flow over 3V:10H side slope and 1V:2H apron will be considered in design of rock size.



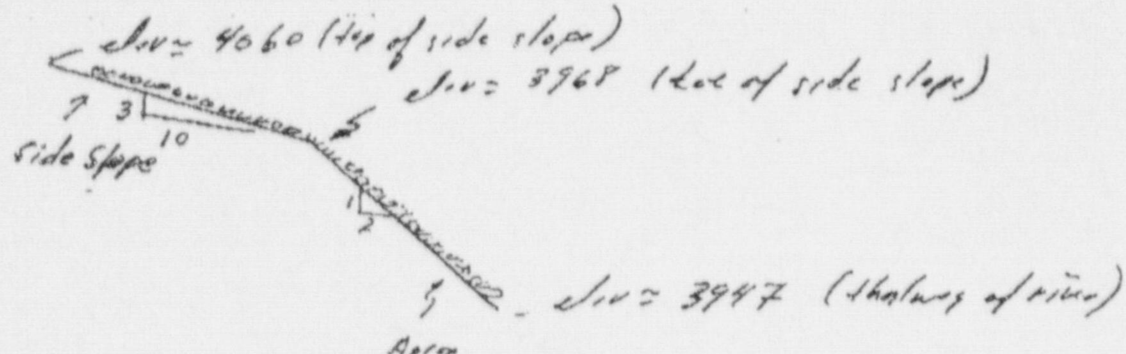
CanonieEnvironmental

By JWS Date 11/30 Subject ATLAS - Apron Design Sheet No. 4 of 14

Chkd. By ___ Date 12/27/94 Project No. 88-007-21

1/4" X 1/4"

Stephenson's method for overland flow
over apron - See Attachment A for excerpts from NUREG 4620



① estimate t_c

$H = 4060 - 3947 = 113$ ft = max. elev. change, ft

$L = 300^{ft} + 47^{ft} = 347^{ft}$ = longest travel path

$t_c = 60(11.9L^3/H)^{0.385}$ t_c in minutes (eq. 4.45 NUREG 4620)

$= 60(11.9(347/5280)^3/113)^{0.385}$ L in miles, H in feet

$t_c \approx 1.08$ minutes

② % of 1hr PMP that would occur during t_c - see table 2.1 NUREG 4620

% of 1hr = 11.88%

③ Max amt. of ppt. in period t_c

$.1188 \times 8.25$ in $= 0.98$ inches

1hr PPT. amount (From REVISED PMP CALC., Canonie, April, 1993)

④ Intensity of ppt. = 0.98 inches / 1.08 minutes × 60 min/hr

= 54.5 in/hr

⑤ Calc. Peak Discharge

CanonieEnvironmental

By JKS Date 11/30/94 Subject Apron Riprap Size Sheet No. 5 of 14

Chkd. By DR Date 12/31/94 Project No. 88.067.21

1/4" X 1/4"

Using Rational Method

$\theta \approx 26.6°$

$Q = ciA$

$A = 347 ft^2 \times \text{unit width} / 43560\ ft^2/ac = 0.0080\ ac$

$Q = (1)(54.5\ in/hr)(.0080\ ac)$ [C=1 per NUREG 4620]

$Q = 0.434\ cfs/ft$

$$D_{50} = \left[\frac{q \tan\theta^{7/6}\, n^{1/6}}{C g^{1/2} \left[(1-n)(S-1)\cos\theta(\tan\phi - \tan\theta)\right]^{5/3}}\right]^{2/3}$$

(NUREG 4620 eqn 4.28)

See Overland Flow CALC., June 1992 for values of n, C, S

$$= \left[\frac{0.434 \tan(26.6°)^{7/6} (0.45)^{1/6}}{0.27 (32.2)^{1/2}\left[(1-0.45)(2.47-1)\cos(26.6°)(\tan(42°)-\tan 26.6°)\right]^{5/3}}\right]^{2/3}$$

$= 0.916\ ft$

$D_{50} = 11''$

oversize by 2% → 11.2" 2% oversize for durability is required - see Rock Quality, Assessment of Oversizing, Atlas Corp. Rec. Plan, Canonie, 1992

Because of flow transition at toe, extend riprap (11.2") approximately 5 ft. up on 3V:10H slope.

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By JES Date 11/30/94 Subject Atlas Apron Design Sheet No. 6 of 14
Chkd. By DR Date 12/29/94 Project No. 88-067-21
1/4" X 1/4"

③ Check Apron Riprap size against River flow using Army Corp of Engineers method

From EM-1110-2-1601

τ = Actual shear stress = $\dfrac{\gamma V^2}{\left(32.6 \log_{10} \dfrac{12.2 y}{D50}\right)^2}$ (EM 1110-2-1601 eqn. 31)

γ = 62.4 lb/ft³ (unit wt. of water)
y = depth of flow, ft
D50 = median rock size, ft
V = velocity of channel, fps

τ allowable bottom = $a(\gamma_s - \gamma)$ D50 (EM 1110-2-1601 eqn. 33)

a = coeff of 0.04
γ_s = unit wt. of stone = 154 lb/ft³ (from surface water calc. June 1992 Atlas Rec. Plan)

τ allowable side = $\tau_{bot}\left(1 - \dfrac{\sin^2\phi}{\sin^2\theta}\right)^{1/2}$

ϕ = side slope angle = $\tan^{-1}(1/2)$ = 26.6°
θ = Angle of repose of riprap = 42° (see June 92 Rec. Plan)

Results are summarized on following page, using V, y from several selected discharges in main river channel from HEC-2 runs performed by Mussetter & Harvey, 1994 along cross-section 6 nearest river. See Attachment B for excerpts from report.

By JKS
11/30/94
CKD BY DR 12/29/94

7/14

Actual and Allowable Shear Stresses for Selected Discharges
In Colorado River using Army Corp of Engineer Method

Q, cfs	V, fps	Y, ft	Actual Shear, psf	Allowable Shear, psf Bottom	Allowable Shear, psf Side
4000	2.25	4.98	0.09	3.42	2.54
20000	3.08	12.05	0.12	3.42	2.54
48900	3.3	17.96	0.11	3.42	2.54
70300	3.3	21.07	0.11	3.42	2.54
123500	3.01	29.24	0.08	3.42	2.54
178000	2.98	36.39	0.07	3.42	2.54
300000	3.09	49.95	0.07	3.42	2.54

D50 =	11.2	in
phi =	26.6	Deg
theta	42	Deg

Notes:

V = Velocity of Main Channel
Y = Depth of Flow in Main Channel
V, Y values are from Mussotter & Harvey, 1994

Results indicate that
$\tau_{actual} < \tau_{allowable}$
for all selected River Discharges
therefore overland flow controls rock size.

COE.XLS 11/30/94

CanonieEnvironmental

By JWS Date 11/30/94 Subject ATLAS-Apron design Sheet No. 8 of 11
Chkd. By DR Date 12/27/94 Project No. 88-067-21
1/4" X 1/4"

6) Notes on soil type where apron will be installed.

EM 1110-2-1601 1994 update recommends that rock should be launched on to noncohesive soil and that launch slope is less predictable if cohesive material is present.

Boring logs in area of apron were analyzed for non-cohesive. Attachment C contains these borings. Boring logs indicate mainly sands and gravels and some silt which would be considered noncohesive and launched rock should be adequate.

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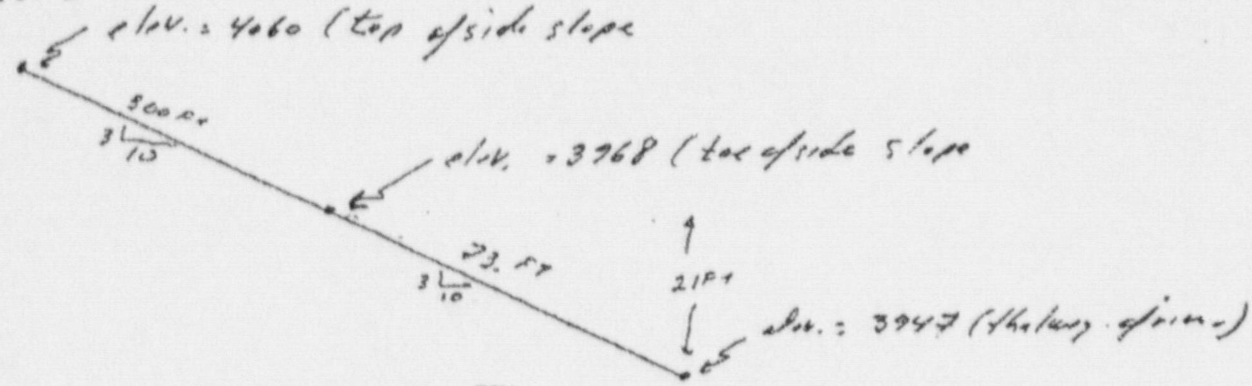
By JWL Date 11/30 Subject Alt - Apron Design Sheet No. 9 of 14
Chkd. By DR Date 12/29/94 Project No. 88-067-21
1/4" X 1/4"

(1) Find Apron Rock Size in flatter slope - use 3V:10H

USE STEPHENSONS METHOD ON FLATTER APRON SLOPE:



(1) estimate $t_c = 60(11.9 L^3/H)^{0.385}$

$L = 373^{ft}$

$H = 4060 - 3947 = 113^{ft}$

$t_c = 60\left(\frac{11.9(373/5280)^3}{113}\right)^{0.385}$

$t_c = 1.18$ minutes

(2) % of 1hr pmp during tc. From table 2.1
% of 1hr = 13%

(3) max. amt. of ppt in tc
.13 × 8.25 in = 1.07 inches

(4) Intensity = 1.07 inches / 1.18 minute × 60 min/hr = 54.4 in/hr

(5) Peak Discharge
Q = CiA

CanonieEnvironmental

By JES Date 11/30/94 Subject Atlas - Apron design Sheet No. 10 of 14
Chkd. By DR Date 12/29/94 Project No. 86-067-21
1/4" X 1/4"

$A = (373^{FT} + 1^{FT}) / 43560 = .0086$ Ac

$Q = (1)(54.4)(.0086) = 0.466$ cfs/ft

$$D_{50} = \left[\frac{q \tan\theta^{7/6} n^{1/6}}{C g^{1/2} \left[(1-n)(s-1)\cos\theta (\tan\phi - \tan\theta) \right]^{5/3}} \right]^{2/3}$$

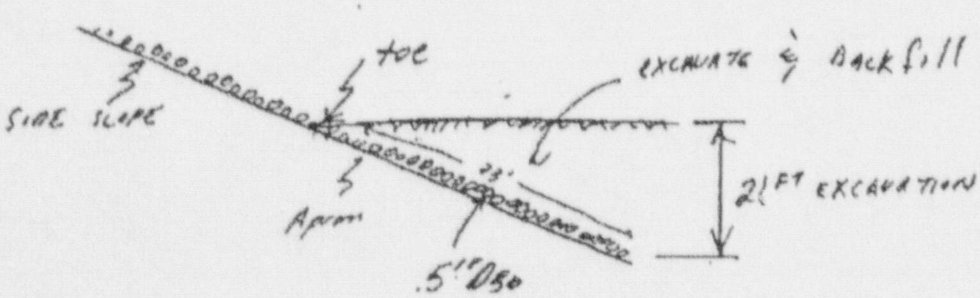
$$= \left[\frac{0.466 \tan(16.7)^{7/6} (0.45)^{1/6}}{0.27 (32.2)^{1/2} \left[(1-0.45)(2.47-1)\cos(16.7^{\circ})(\tan(42) - \tan(16.7)) \right]^{5/3}} \right]^{2/3}$$

$= 0.88^{FT}$

$D_{50} = 4.6$ in

over-size 2% and $D_{50} = 1.02 \times 4.6 = 4.7'' D_{50}$ or say 5" because already have designed riprap gradation.

must construct apron as follows:



Layer thickness $= 2 \times D_{50} = 10''$
Volume $= 10''/12 \times 73 = 61 \; FT^3/FT$

over 2640^{FT} - need $\approx \underline{6000 \text{ cy}}$ of 5" D_{50} with 21^{FT} excavation.

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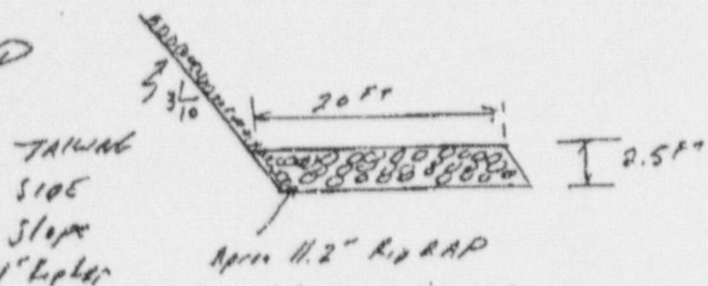
By JES Date 11/30 Subject ATCAT - Apron Design Sheet No. 11 of 14
Chkd. By DR Date 12/29/94 Project No. 88-067-21
1/4" X 1/4"

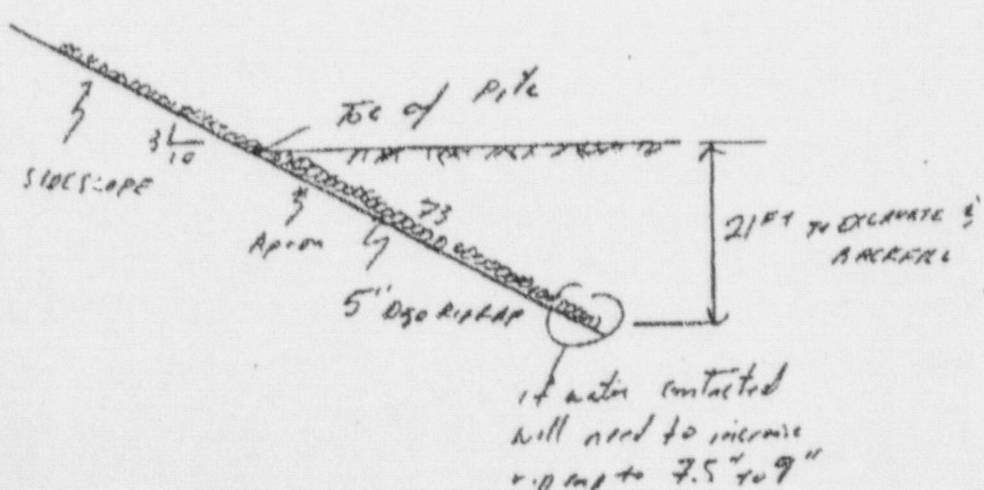
Summary of Options

Option 1



Volume/ft = 50 ft³/ft

Option 2



Volume/ft = 73 ft x 10"/12
= 61 ft³/ft

encl 11

Commission Internationale des Grands Barrages

Treizième Congrès des Grands Barrages
New Delhi, 1979

Q. 50
R. 9

COMPUTATION OF MAXIMUM DISCHARGE AT OVERFLOW ROCKFILL DAMS (A COMPARISON OF DIFFERENT MODEL TEST RESULTS)(*)

J. KNAUSS

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Research Institute for Hydraulic Engineering
of teh Technical University in Munich
Oskar v. Miller-Institute at Obernach

GERMANY, F.R.

1. INTRODUCTION

In 1973 at the ICOLD-Congress in Madrid « a design formula for overtop rockfill » was presented by H. Olivier in R.63 to Q.41 (Ref. [1]). More detailed information about the design procedure was given in his paper « Through and Overflow Rockfill Dams - New Design Techniques » from 1967 (Ref. [2]). The computation-method was based on the results of an experimental investigation done by A. Linford and D.H. Saunders at the BHRA-laboratories (Ref. [3]).

An other computation-method was proposed by F. Hartung and H Scheuerlein in their paper R.35 to Q.36 « Design of Overflow Rockfill Dams » (Ref. [4]) presented at the ICOLD-Congress in Montreal, 1970. The authors apply the hydraulics of steep and rough open channel flow to the construction of overtopped rockfill. An extensive study of the flow in steep rough channels was carried out by H. Scheuerlein (Ref. [5] and [6]) in 1968. In the report of Hartung and Scheuerlein special reference was made to the communication « Construction of Dams by Depositing Rock in Running Water » by S.V. Isbash presented at the ICOLD-Congress in Washington, 1936 (Ref. [7]).

(*) *Calcul du débit maximal en cas de déversement pour des barrages en enrochement (comparaison entre les différents résultats des essais sur modèle).*

Reprint from the Proceedings of the XIII[th] Congress of the International Commission of Large Dams, New Delhi, 1979.

In 1976, H.H. Thomas in his book « The Engineering of Large Dams », chapter 15, « Flow Through and Over Rockfill » (Ref. [8]) gives a short description and comparison of the two computation-methods. He states that the « results are in reasonable agreement » under special assumptions. This statement may be concerned as a very rough estimation.

With references to the subsequent discussion of the two methods an exact comparison, for example, leads to the following figures:
downstream slope of the overtopped rockfill: 1 on 2.5
weight of the average stone (crushed material): 1.75 kN
equivalent diameter of the average stone: 0.5 m
« natural » packing of the surface layer ($p \sim 1.2$ or $\Phi \sim 0.6$)
required number of stones per unit area: 3.33
max discharge after Linford/Saunders/Olivier: 0.6 $m^3/s \cdot m$
max discharge after Hartung/Scheuerlein: 1.4 $m^3/s \cdot m$

The computed discharge varies considerably. Only for a slope of about 1 on 8 (keeping the other data of the example constant) one yields equal discharge (2.3 $m^3/s \cdot m$). The explanation of these facts is the main scope of this paper.

Due to the results of the experiments at BHRA a preliminary remark may be given: the value of the so-called threshold flow is determined only by the dimensions and characteristics of the stones on the surface of the downstream slope and independent of the underlying material. The design equation was obtained from the results of tests with an impervious downstream slope and therefore with no seepage flow. By means of special additional tests, however, the established computation-method has been shown also applicable to structures with seepage flow.

2. DEFINITION OF Fr_s, THE FROUDE NUMBER OF THE STONES

A comparison of the two computation-methods in a direct and comprehensive way is not possible except in special design cases with given data. Besides, the computational procedure is more or less complicated. With the following introduction of a dimensionless approach a more generalized comparison will be available. Using the so-called equivalent diameter of the stones as characteristic length, a special Froude number may be defined as follows:

$$Fr_s = \frac{q}{\sqrt{g} \cdot d_s^{3/2}} \text{ resp. max } Fr_s = \frac{q_c}{\sqrt{g} \cdot d_s^{3/2}} \tag{1}$$

with
Fr_s = Froude number of the stones
d_s = equivalent diameter of the stones at the surface layer of the downstream slope of the rockfill dam, being identical with the diameter of a sphere of the same volume as the average stone within the layer (m)
q_c = max q = specific, critical (or threshold) flow at incipient motion of individual stones ($m^3/s.m$).

After some transformation and rearrangement of the given equations a max Fr_s-formula can be derivend for either computation-method. The possibility of the introduction of Froude's law of similarity into the problem under consideration is very important and advantageous because of the necessary extrapolation of model test results up to more practicable ranges of application not covered by the experiments.

3. DERIVATION OF max Fr_s-FORMULAS

3.1. METHOD INTRODUCED AND INVESTIGATED BY LINFORD/SAUNDERS AND OLIVIER

In their experiments Linford and Saunders observed the begin of damage of the downstream slope at an overstopped model rockfill dam (Ref. [3]). After evaluation of the discharge at this threshold stage in the model they compared their test results with a theoretical approach using the following, well-known equations:

(a) $q_c = h_c \cdot v_c$ - continuity equation (critical values)

(b) $\tau_c = \rho_w \cdot g \cdot h_c \cdot J$ - critical boundary shear stress (hydraulic properties, Du Boys)

(c) $\tau_c = \tau_c^* \cdot (\rho_s - \rho_w) \cdot g \cdot d_s$ - critical boundary shear stress (material properties, Shields)

(d) $v_c = \frac{1}{\sqrt{\lambda_c}} \cdot \sqrt{8\, g \cdot h_c \cdot J}$ - flow equation (flat slopes, Chezy)

(e) $\frac{1}{\sqrt{\lambda_c}} = \frac{C}{\sqrt{8\, g}} \cdot \left(\frac{h_c}{d_s}\right)^{1/6}$ friction factor (Manning-Strickler)

with C = 19 and valid for $\frac{h}{d_s'} > 2.5$

The most important terms are defined as follows:
J = downstream slope of embankment
v_c = mean (critical) velocity at incipient motion (m/s)
τ_c^* = critical shear stress coefficient
ρ_s, ρ_w = mass density (here: $\rho_s = 2.7\ t/m^3$ for heavy stones and $\rho_w = 1.0\ t/m^3$ for water).

Combining these five equations similar to the hydraulics of sediment transport in open channels (traction theory) and introducing the defined characteristic Froude number one finally arrives at:

$$\max Fr_s = \frac{C}{\sqrt{g}} = (\tau_c^*)^{5/3} \left(\frac{\rho_s - \rho_w}{\rho_w}\right)^{5/3} . J^{-7/6} \tag{2}$$

From an over-all evaluation of the test results of Linford and Saunders one can find the following expression for a mean value of the critical shear stress coefficient:

$$\tau_c^* \cong 0.0715 \cdot \left(\frac{1.2}{p}\right) \tag{3}$$

with a packing factor p describing the influence of the varying roughness of the uppermost rockfill layer due to the dam construction. The definition of p will be given later. The most important range of p investigated was : $0.8 \leq p \leq 1.2$.

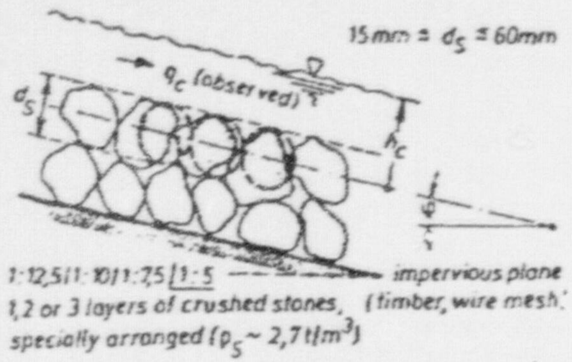


Fig. 1 :

Experimental arrangement used by Linford/Saunders.

Procédé expérimental utilisé par Scheuerlein.

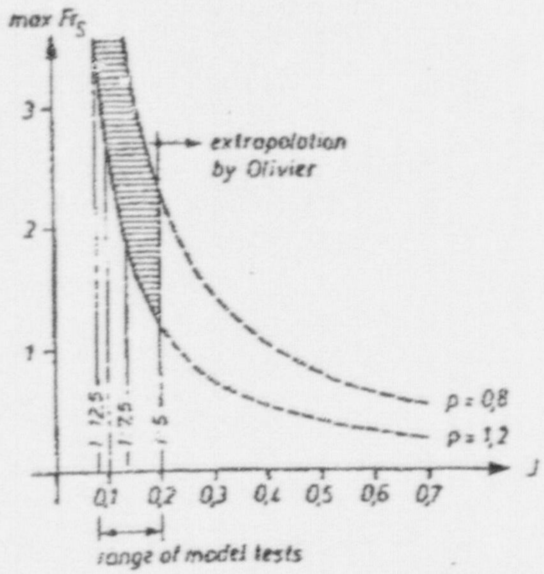


Fig. 2 :

Max Fr_S-formula using the data of Hartung/Scheuerlein.

Formule du Fr_S max d'après Hartung/Scheuerlein.

Fig. 1 shows schematically the experimental arrangement used by Linford and Saunders. Fig. 2 presents a graph of the derived max Fr_S-formula according to equation (2) and (3). The steepest slope investigated in the model was 1 on 5 ($J = 0.2$). The graph points out the extensive extrapolation of test result and theory towards steeper slopes proposed by Olivier in Ref. [1] and [2]. The reason for the extrapolation is evident since in rockfill dam construction steeper slopes are more important than smaller ones. But as shown later, the justification of this extrapolation is very doubtful. Fortunately, the results are laying on the safe side regarding dam stability. However, the use of overtopped rockfill as a spillway normally depends on the maximum admissible discharge per unit width, and consequently on the relation between the design flood and the given length of the dam section suitable for overspilling. If the threshold flow is too small the overflow of rockfill dams for flood control is of little interest. Olivier's extrapolation towards steeper slopes yields maximum discharge values with such a tendency.

In Olivier's ICOLD-report (Ref. [1]) the design formula is written in the way

$$q_c = \text{const.}\left(\frac{\rho_S - \rho_w}{\rho_w}\right)^{5/3} \cdot J^{-7/6} \cdot d_S^{3/2}\ (\text{m}^3/\text{s}\cdot\text{m})$$

for crushed rough stones with const. = 0.235 (p = 1.2).

This type of formula is well-known in bedload theory as the critical discharge equation after Schoklitsch presented at first in 1934 and later rearranged in 1950. Schoklitsch proposed a constant value of 0,26 according to $\tau_c^* = 0{,}076$.

3.2. Method introduced and investigated by Hartung/Scheuerlein

From numerous test-series at the Hydraulic Laboratory at Obernach Scheuerlein developed a theoretical method to describe the mechanics of flow in steep and rough open channels (Ref. [5] and [6]). The most important result was an equation determining the friction factor as a function of relativ roughness, packing of the roughness elements, channel slope and especially of air-entrainment into the extremely turbulent water flow.

In their ICOLD-report (Ref. [4]) Hartung and Scheuerlein made use of these fundamental flow formulas to find a method for the design of overflow rockfill dams. They finally came up with a proposal for the computation of the maximum admissible discharge introducing a criterion for the channel stability. The equations used are given as follows :

(a) $q_c = \sigma_c \cdot h_c \cdot v_c$

- continuity equation including the influence of air-entrainement (critical values)

(b) $v_c = E \cdot \sqrt{2g} \cdot \sqrt{\dfrac{\rho_S - \rho_w}{\sigma_c \cdot \rho_w}} \cdot \sqrt{d_S \cos\varphi}$ with E = 1.2

- critical velocity equation (incipient motion) derived from stability conditions

(c) $v_c = \frac{1}{\sqrt{\lambda_c}} \cdot \sqrt{8\, g \cdot h_c \sin\varphi}$

- flow equation (steep slopes)

(d) $\frac{1}{\sqrt{\lambda_c}} = -A \cdot \log\left(B_c \cdot \frac{k}{4 \cdot h_c}\right)$

- friction factor (Scheuerlein), with the following subsitutions derived from the tests :

$A = -3{,}2$

$k = \frac{d_s}{3}$

$B_c = \sigma_c \cdot (1.7 + 8.1 \cdot \Phi \cdot \sin\varphi)$

$\sigma_c = 1 - 1.3 \cdot \sin\varphi + 0.08 \frac{h_c}{k}$

The most important terms are defined as follows :
σ = aeration factor relating the density of air-water mixture to the density of pure water
E = stability factor
φ = angle of slope
k = mean hydraulic roughness height
Φ = packing factor (definition will be given later)

The combination and rearrangement of these equations and the introduction of the defined characteristic Froude number finally lead to the equation :

$$\max Fr_s = 5 \cdot \sqrt{2}\left(\frac{\rho_s - \rho_w}{\rho_w}\right)^{1/3} \sqrt{\sigma_c} \cdot (\sigma_c - 1 + 1.3 \cdot \sin\varphi) \cdot \sqrt{\cos\varphi} \quad (4)$$

The aeration factor related to critical flow conditions may be computed from a simplified equation, which was derived from the author as a straight forward, but nearly exact approximation to the model test results :

$$\sigma_c \simeq 1.18 + 0.08 \cdot \Phi - 1.44 \sin\varphi \text{ (for } \rho_s = 2.7\ t/m^3) \quad (5)$$

The very important aeration of the flow is highly influenced by the magnitude of the downstream slope and the packing factor of the surface layer of the rockfill. The range of Φ investigated was : $0.75 \leq \Phi \leq 1.125$.

In Fig. 3 the experimental arrangement of Scheuerlein's investigation is shown schematically. Fig. 4 contains a graph of the derived max Fr_S-formula according to equation (4) and (5). The range of slopes experimentally investigated varied from 1 on 10 (rather flat) to 1 on 1.5 (very steep).

For the sake of the comparison of the two computation-methods the author had to extrapolate the packing factor Φ to a lowest value of 0.625 (decreasing roughness). The graph points out the interesting fact, that air-entrainment starts with slopes near 1 on 5, which was the steepest inclination of downstream embankment within the tests from Linford/Saunders.

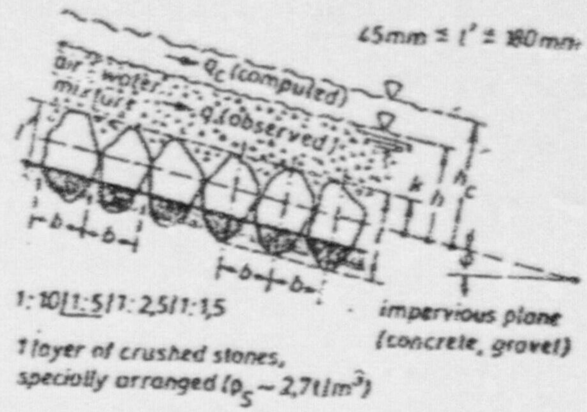


Fig. 3 :

Experimental arrangement used by Scheuerlein.
Procédé expérimental utilisé par Scheuerlein.

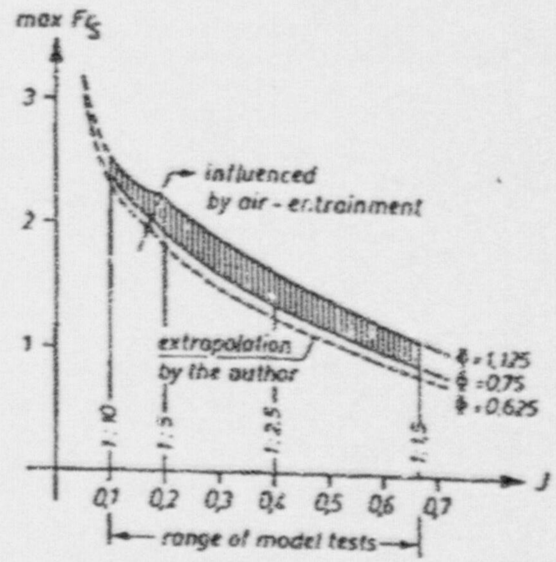


Fig. 4 :

Max Fr_s-formula using the data of Hartung/Scheuerlein.
Formule de Fr_s max d'après Hartung/Scheuerlein.

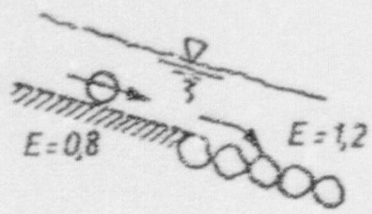


Fig. 5 :

Isbash's results concerning stability factor E.

Résultats d'Isbash sur le facteur de stabilité.

4. COMPARISON OF THE TWO METHODS

4.1. STABILITY FACTOR E

Considering the forces acting on the stones within the surface layer of the overflowed downstream embankment Scheuerlein established a stability condition resulting in his critical velocity equation for incipient motion :

$$v_c = E \cdot \sqrt{2g} \sqrt{\frac{\rho_s - \rho_w}{\sigma_c - \rho_w}} \cdot \sqrt{d_s \cdot \cos\varphi}$$

In this equation the influence of air-entrainment and channel slope is included. For the stability factor he chose the value E = 1.2 according to the test results of Isbash (Fig. 5 and Ref. [7]). Isbash's equation given in 1936 for dumping rock in running water reads :

$$v_c = E \cdot \sqrt{2g} \sqrt{\frac{\rho_s - \rho_w}{\rho_w}} \cdot \sqrt{d_s}$$

From the equations used by Linford and Saunders for their theoretical approach one can derive :

$$v_c = \frac{C}{\sqrt{2g}} \left(\frac{\rho_s - \rho_w}{\rho_w}\right)^{1/6} \cdot (\tau_c^*)^{2/3} \cdot J^{-1/6} \cdot \sqrt{2g} \cdot \sqrt{\frac{\rho_s - \rho_w}{\rho_w}} \cdot \sqrt{d_s}$$

The stability factor is thus discribed theoretically with :

$$E = \frac{C}{\sqrt{2g}} \left(\frac{\rho_s - \rho_w}{\rho_w}\right)^{1/6} \cdot (\tau_c^*)^{2/3} \cdot J^{-1/6} \qquad (6)$$

For the purpose of comparison it is necessary to introduce the effect of air-entrainment according to the test results of Scheuerlein into the last equation and to replace J by sin φ for steep slopes as follows :

$$E^* = \frac{C}{\sqrt{2g}} \left(\frac{\rho_s - \rho_w}{\sigma_c - \rho_w}\right) (\tau_c^*)^{2/3} \cdot (\sin\varphi)^{-1/6} \qquad (7)$$

An evaluation of equation (6) and (7) with C = 19, $\tau_c^* = 0.0715 \cdot (1.2/p)$ and $\sigma_c = 1.18 + 0.08 \cdot \Phi - 1.44 \cdot \sin\varphi$ for $p = \Phi = 0.915$ leads to the graph in Fig. 6.

The final result of this consideration and comparison is therefore an approval of Scheuerlein's assumption for the stability factor with E = 1.2.

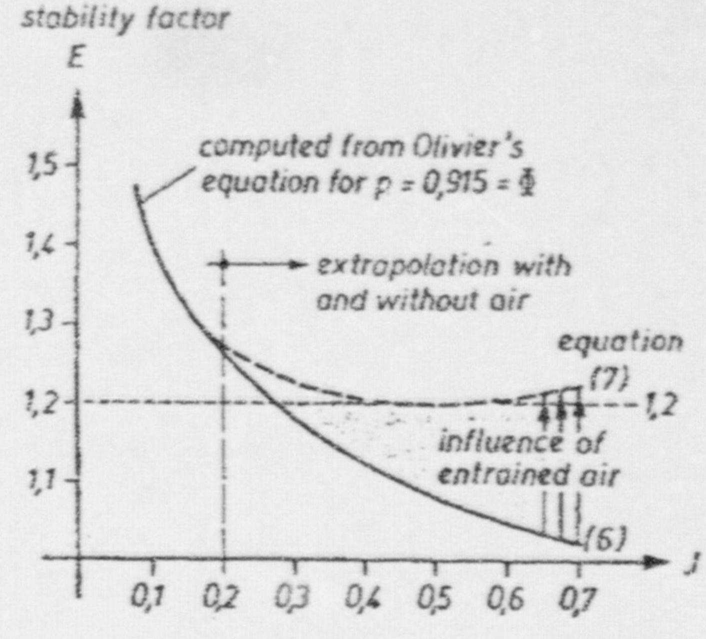


Fig. 6 :

Comparison of Scheuerlein's approach for E (1.2) with the test results from Linford/Saunders.

Comparaison de l'hypothèse de Scheuerlein pour E (1,2) avec les résultats expérimentaux de Linford/Saunders.

4.2. ROUGHNESS PARAMETTERS, PACKING FACTORS p AND Φ

With reference to Fig. 7 some definitions of the characteristic geometrical dimensions of the stones due to Scheuerlein's experimental arrangements are to be presupposed to the next considerations :

- k = mean hydraulic roughness height
- l' = mean geometric roughness height
- l = total height of the average stone
- b = width of the average stone
- $N = 1/b^2$ = number of stones per unit area
- $c = l'/l$ = parameter of the experimental arrangement
- f = shape factor of the average stone

Equalizing the volume of the average stone ($V_m = f \cdot b^2 \cdot l$) with the volume of a sphere ($V_{sp} = \pi/6 \cdot d_s^3$) one arrives at an equation for the equivalent diameter of the stones :

$$d_s = \left(\frac{\pi}{6} \cdot f \cdot \frac{3k}{N}\right)^{1/3} \qquad (8)$$

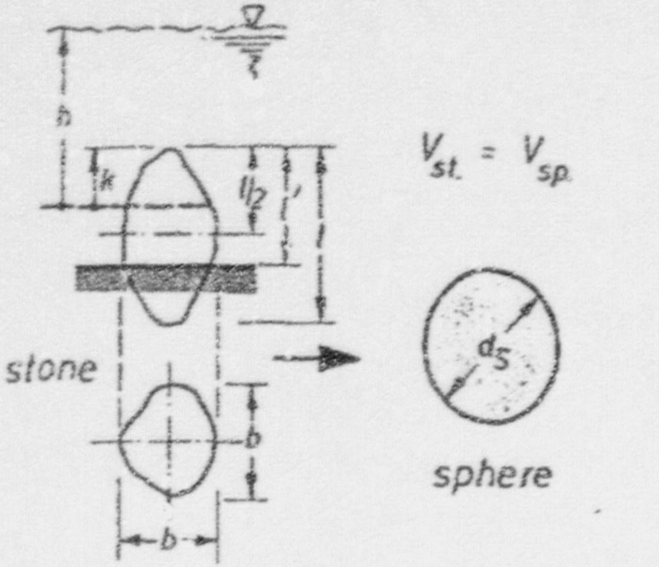


Fig. 7:

Definition of the geometrical dimensions of the stones according to Scheuerlein's experiments.

Définition des dimensions géométriques des pierres dans les expériences de Scheuerlein.

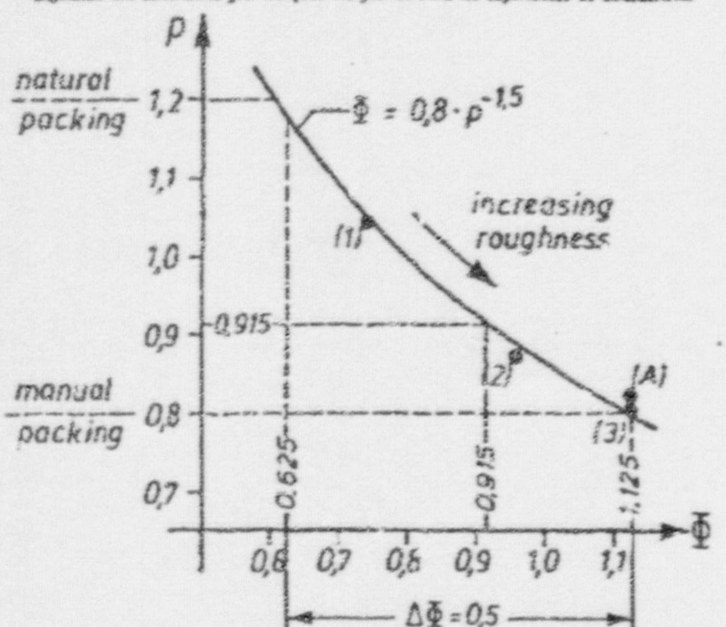


Fig. 8:

Comparison of packing factor p (Linford/Saunders) and Φ (Scheuerlein).

Comparaison des taux de tassement p et Φ.

with $k = 1/3$ as a representative value of the effective hydraulic roughness height determined by volumetric evaluation of the various model roughnesses.

The shape factor of the crushed stones used in the experiments may be determined as follows :

(a) $V'_{st} = f' \cdot b^2 \cdot l'$ - volume of stone above the impervious plane

(b) $V'_{st} = \frac{l' \cdot k}{N}$ - volumetric measurement in the models

(c) $f = \frac{l/2}{l} \cdot f'$ - additional assumption

(d) $\frac{k}{l'} = \frac{1}{3c}$ - introduction of parameter c

Combining these equations one gets :

$$f = \frac{1}{2c} \cdot (1 - \frac{1}{3c}) \tag{9}$$

For the purpose of comparison now the definitions of the different packing factors used in both investigations must be given :

$$p = \frac{1}{d_s^2 \cdot N} \text{ (Linford/Saunders)} \tag{10}$$

which reads : unit area divided by the number of stones in this area, and also divided by the area of the average stone

$$\Phi = l' \sqrt{N} = \frac{l'}{b} \text{ (Scheuerlein)} \tag{11}$$

which reads : mean vertical roughness height divided by the mean horizontal width of the roughness elements.

The combination of the four equations (8) to (11) results in the very interesting relationship for the two packing factors :

$$p^{3/2} \cdot \Phi = \frac{\pi}{6} \cdot \frac{c}{f}$$

Evaluating the right side of this formula according to the measured mean values from Scheuerlein's experiments as shown in the Table one finds :

$$p^{3/2} \cdot \Phi \simeq 0.8 \tag{12}$$

This expression is plotted in Fig. 8 and compared with the exact values determined for the four sizes of stones investigated by Scheuerlein.

Now it is possible to correlate the two investigation-methods with respect to the different construction of the surface layer. The packing with $p \simeq 1.2$ ($\Phi \simeq 0.625$) may be called a « natural » packing according to the denotation of Linford/Saunders and Olivier. The autor thinks that « natural » packing may be adopted for a « dumped » embankment. The value $p \simeq 0.8$ ($\Phi \simeq 1.125$) stands for « manual » packing especially for an artificial arrangement with flat stones placed on edge. This instruction for packing demands a selection of the stone material used for the construction of the surface layer of the overflowed downstream slope of the rockfill dam.

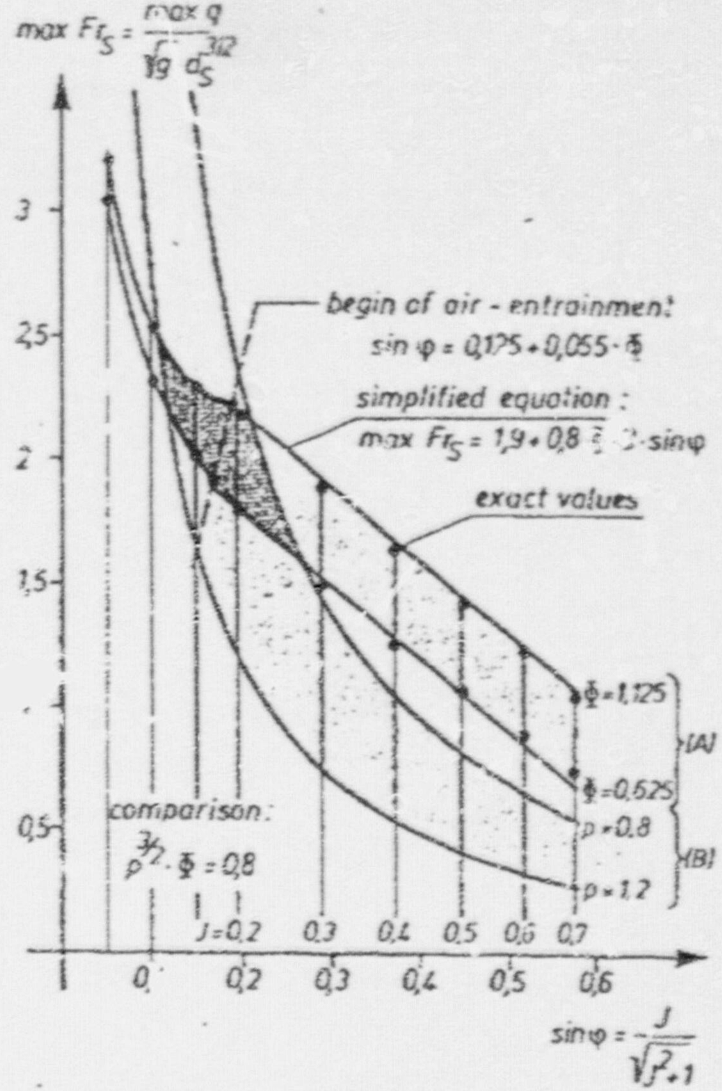
Table : Evaluation of roughness parameters

Tableau : Evaluation numerique du parametre de rugosite

specification of roughness	mean values (measured) (*) k (m)	l' (m)	N (1/m²)	$\Phi = l' \cdot \sqrt{N}$	kind of used stones
1	0,027	0,043	299	0,745	crushed limestones
2	0,061	0,093	106,6	0,96	crushed limestones
3	0,079	0,135	69,5	1,125	crushed limestones
[A]	0,1	0,18	39	1,125	crushed sandstones

(*) Ref. [5], Scheurlein's experiments

specification of roughness	$c = l'/3 \cdot k$	$f = \frac{1}{2 \cdot c} \cdot \left(1 - \frac{1}{3 \cdot c}\right)$	$\frac{c}{f}$	mean value	$p^{1.5} \cdot \Phi = \frac{\pi}{8} \cdot \frac{c}{f}$	mean value
1	0,531	0,3505	1,515	1,525	0,793	0,8
2	0,508	0,3385	1,501	1,525	0,786	0,8
3	0,5695	0,364	1,564	1,525	0,819	0,8
[A]	0,6	0,3705	1,520		0,848	



(A) max $Fr_S = f(J, \Phi)$ by Hartung/Scheuerlein

(B) max $Fr_S = f(J, p)$ by Linford/Saunders and Olivier

Fig. 9 :

Comparison of the different formulas max $Fr_S = f(\sin\varphi, \Phi$ or $p)$ for $\rho_s = 2.7$ t/m³.

Comparaison des différentes formules max $Fr_S = f(\sin\varphi, \Phi$ ou $p)$ pour $\rho_s = 2,7$ t/m³.

3. Comparison of max Fr_s-formulas

In Fig. 9 the derived max Fr_s-formulas are plotted and may now be compared. In the range of slopes from 1 on 10 to 1 on 5 the computed values for both methods are in a more or less reasonable agreement. With steeper slopes a significant difference between the evaluated curves appears. The most important reason for this discrepancy is the omission of air-entrainment in Olivier's extrapolation of the test results. Considering equivalent stability conditions the air-water mixture allows a higher critical velocity and flow depth and therefore an increase of the admissible discharge than pure water. A further reason of smaller but also significant influence must be seen in the necessary replacement of $J = tg\,\varphi$ by $\sin\varphi$ for steeper slopes.

A problem on principle is the use of a flow equation (Manning-Strickler) for the theoretical interpretation fo the test results (Linford and Saunders), which is not valid for the very rough turbulent flow in the case under consideration (relativ roughness $h_c/d_s \leq 1$). As know from fundamental hydraulics the applicability of the Manning-Strickler equation starts with a relativ roughness above $h/d_s > 2.5$.

5. PROPOSED COMPUTATION PROCEDURE

5.1. The authors simplified max FR_s-formula

The author's conclusion emerging from the established discussion of the two methods is to recommend the application of the computation proposal given by Hartung/Scheuerlein, which leads to more practicable maximum discharge values. The complex equations (4) and (5), however, should be replaced by the simplified expression:

$$\max Fr_s = 1.9 + 0.8\cdot\Phi - 3\cdot\sin\varphi \qquad (13)$$

The justification of this simplification is shown in the graph of Fig. 9. The new formula is valid in the interesting range of steeper slopes beginning with 1 on 5 or somewhat smaller.

5.2 Maximum discharge equation

Combining equation (1) and (13) and introducing the weight of the average stone (G_s) instead of the equivalent diameter (d_s) one finds:

$$G_s = \rho_s \cdot g \cdot \frac{\pi}{6} \cdot d_s^3 = 13.865 \cdot d_s^3 \quad \text{in kN} \qquad (14)$$

wiht $\rho_s = 2.7\ t/m^3$ and $g = 9.81\ m/s^2$
and therefore:

$$\max q = 0{,}84 \cdot \sqrt{G_s} \cdot (1.9 + 0.8\cdot\Phi - 3\cdot\sin\varphi) \quad \text{in } m^3/s\cdot m \qquad (15)$$

This final result of the investigation is evaluated and plotted in Fig. 10 for the limiting values of the packing factor and several selected stone sizes. The difference of maximum admissible discharge referring to natural respectively manual packing is described by the term $\Delta q \sim 1/3\ \sqrt{G_s}$.

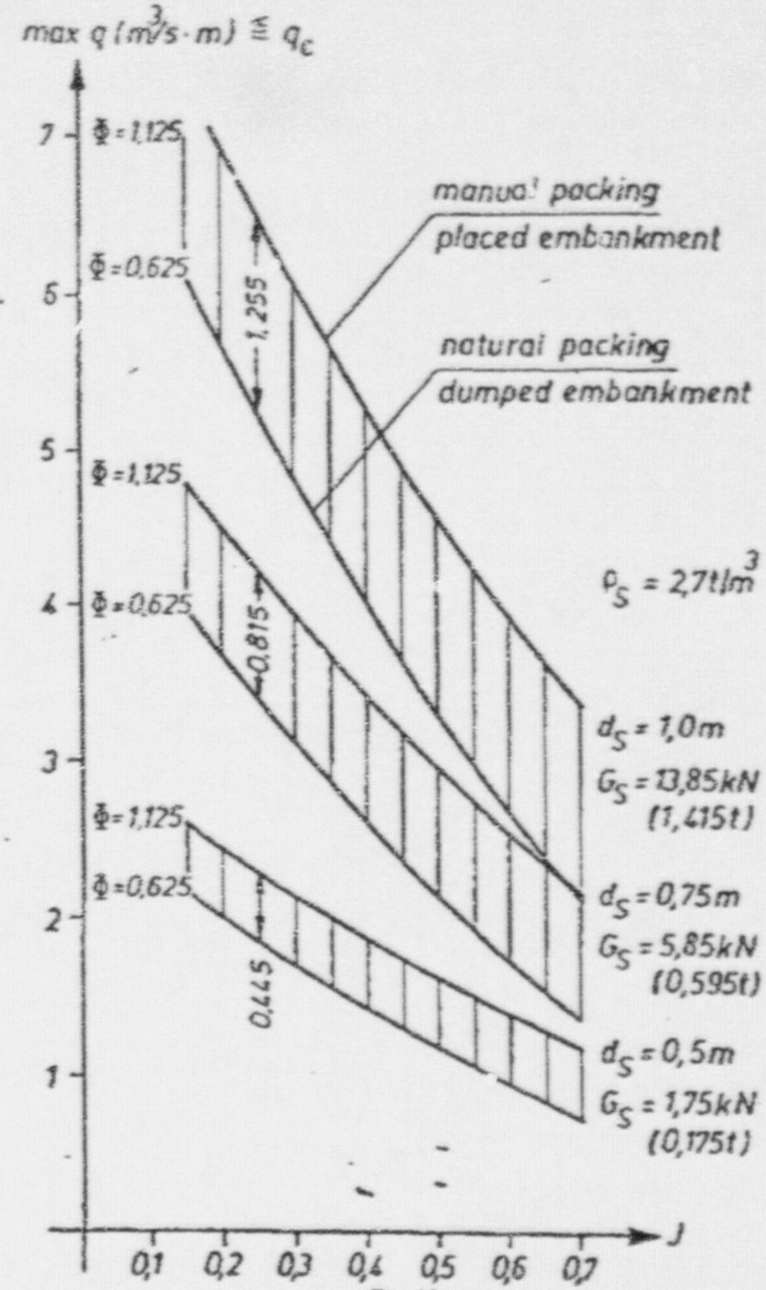


Fig. 10

Maximum discharge versus downstream slope.
size resp. weight of stones and packing factor (final result of investigations)

Débit maximal relatif à l'angle de pente,
à la grandeur et au poids des pierres et au taux de tassement (résultat définitif de la recherche).

5.3. Determination of packing factor Φ and arrangement of stones

It is usefull to determine the packing factor from datas, which may be measured in a more direct way. From a combination of the already introduced equations

$\Phi = \Gamma \cdot N$ - equation (11), with $\Gamma = c \cdot l$

$V_{st} = f \cdot b^2 \cdot l = f \cdot \frac{1}{N}$ - with $N = \frac{1}{b^2}$, and

$G_s = V_{st} \cdot \rho_s \cdot g$

one can find:

$$\Phi = \frac{c}{f} \cdot \frac{1}{\rho_s \cdot g} \cdot G_s \cdot N^{3/2} = 0.0575 \cdot G_s \cdot N^{3/2} \qquad (16\ a)$$

with $c/f \cong 1.525$ according to the evaluation of the test results shown the Table in chapter 4. The packing factor, therefore, can be computed with equation (16 a) after measuring the weight of the average stone within the surface layer of the rockfill and counting the number of stones per unit area.

For a chosen value of factor Φ and for stones with a given weight the necessary arrangement of the stones within the surface layer of the overflowed embankment can be calculated after a convertion of equation (16 a) as follows:

$$N = 6.7 \cdot \left(\frac{\Phi}{G_s}\right)^{2/3} \text{ in stones/m}^2 \qquad (16\ b)$$

Considering the limiting values of natural and manual packing according to the tests under discussion, the required number of stones per unit area increases up to $N_{m.p.} = 1.5 \cdot N_{n.p.}$.

Concluding this chapter reference must be made to the numerous problems involved in the practical application of the above theory to real dam construction. In his paper « Trough and Overflow Rockfill Dams — New Design Techniques » (Ref. [2]) H. Olivier gives a series of very distinct remarks and useful instructions to the necessary specific design considerations and to the construction procedure. Together with his several prototype experiences given to ICOLD in Ref. [1] sufficient information is present to construct the overtopped dam as stable as possible.

REFERENCES

[1] H. Olivier — Some Aspects of Major River Diversion During Construction. R. 63, Q. 41, ICOLD-Congress, Madrid, 1973.

[2] H. Olivier — Through and Overflow Rockfill Dams — New Design Techniques, Proc. I.C.E., 36, Paper No. 7012, 1967.

[3] A. Linford and D.H. Saunders — A Hydraulic Investigation of Through and Overflow Rockfill Dams, Report RR 888, British Hydromechanics Research Association, 1967.

[4] F. Hartung and H. Scheuerlein — Design of Overflow Rockfill Dams. R. 35, Q. 36, ICOLD-Congress, Montreal, 1970.

[5] H. Scheuerlein — Der Rauhgerinneabfluss. Bericht Nr. 14 der Versuchsanstalt für Wasserbau der Techn. Universität München, 1968.

[6] H. Scheuerlein — The Mechanics of Flow in Steep, Rough Open Channels. Paper A 56, IAHR-Congress, Istanbul, 1973.

[7] S.V. Isbash — Construction of Dams by Depositing Rock into Running Water, Communication No. 3, ICOLD-Congress, Washington, 1936.

[8] H.H. Thomas - The Engineering of Large Dams - Chapter 15 - Flow Through and Over Rockfill, John Wiley and Sons, London, Sydney, 1976.

SUMMARY

For overspilling of rockfill dams during construction and after completion two methods are available to determine the maximum admissible discharge capacity. The first one discussed in this paper was presented by Linford/Saunders and Olivier with the resulting equation as follows:

$$\frac{\max q}{\sqrt{g} \cdot d_s^{3/2}} = 0.18 \cdot \left(\frac{1.2}{p}\right)^{5/3} \cdot J^{-7/6} \qquad (I)$$

The second one was developed by Hartung/Scheuerlein and simplified by the author. His final equation reads:

$$\frac{\max q}{\sqrt{g} \cdot d_s^{3/2}} = 1.9 + 0.8 \cdot \Phi - 3 \cdot \sin\varphi \qquad (II)$$

Both the equations are valid for crushed stones with an angular shape and a specific mass density of about 2.7 t/m^3. The packing factors for the construction of the surface layer of the rockfill should be estimated within the range of $0.8 \leqq p \leqq 1.2$ or $1.125 \geqq \Phi \geqq 0.625$.

From the results of the comparison of the two methods, the author suggests the application of the second equation for the design of overflowed rockfill dams with steeper downstream slopes in the range of 1 on 5 to 1 on 1.5. The formula leads to more practicable flow capacities at the overspill section of the dam according to the inclusion of the hydraulic advantages of air-entrainment into the water flow. After evaluating a relationship between the packing factors used in both experiments ($p^{3/2} \cdot \Phi = 0.8$), the more densely compacted rock in Scheuerlein's tests may be compared with the rather coarse compacted rockfill in most of the tests conducted by Linford and Saunders. In order to get a certain packing factor it is necessary to arrange the stones within the surface layer in such a way that one obtains a certain mean number of rocks per unit area. The best stability condition is reached by manually placing of more or less flat stones on edge.

The general purpose of this report was to work out a theoretical background proved by experiments, which enables to estimate wether an overspill of rockfill is suitable for flood control at the dam itself or not. Using very big and heavy boulders on a not too steep downstream slope of the rockfill rather large capacity spillways are possible.

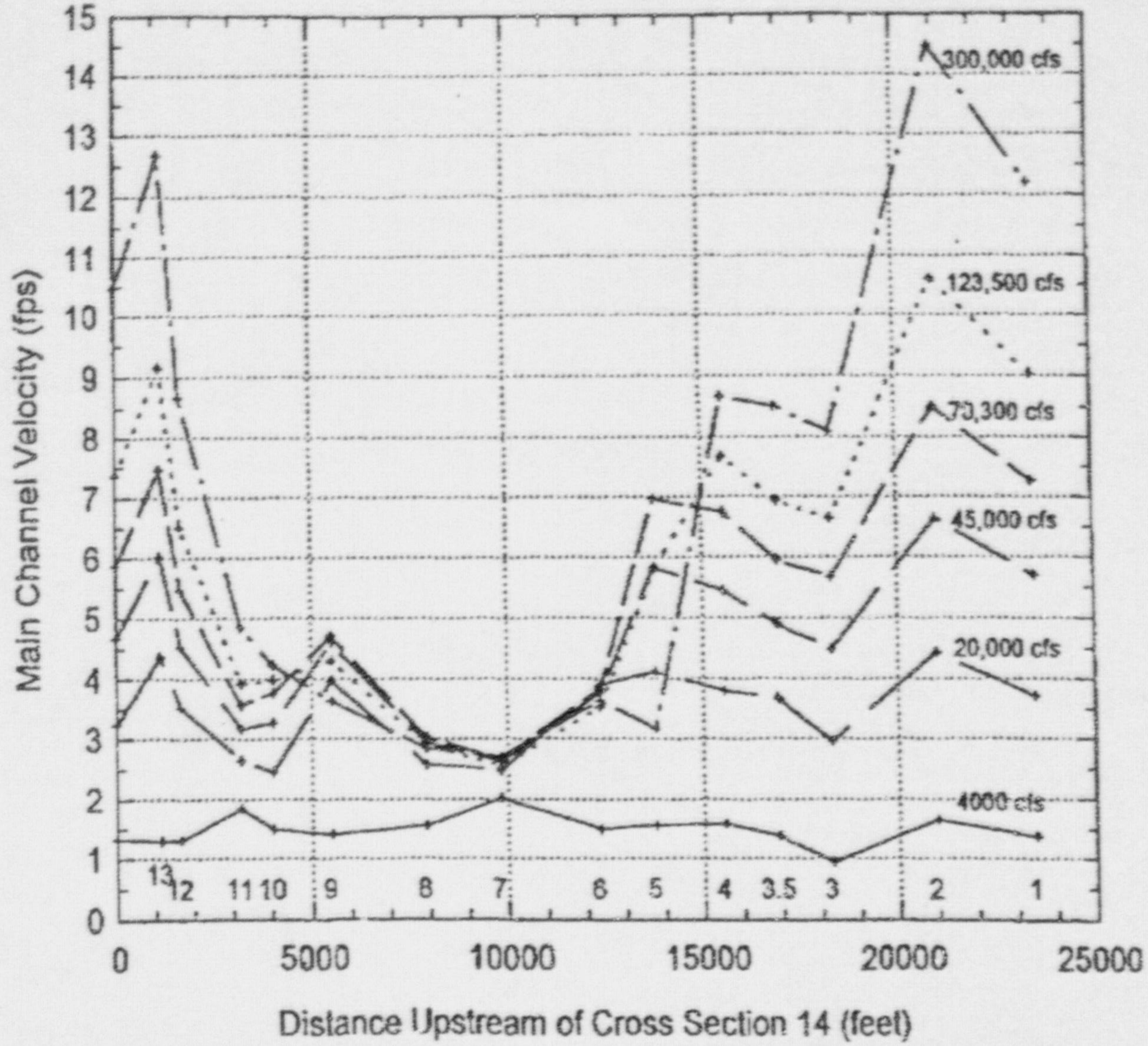


Figure 4.5 Velocity profiles for the study reach for 6 discharges ranging from 4,000 cfs to 300,000 cfs.

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State of Utah
DEPARTMENT OF ENVIRONMENTAL QUALITY
DIVISION OF RADIATION CONTROL

Michael O. Leavitt
Governor

Dianne R. Nielson, Ph.D.
Executive Director

William J. Sinclair
Director

MEMORANDUM

TO: William Sinclair
Executive Director Utah Division of Radiation Control

THROUGH: Dane Finefrock D.F.

FROM: Woodrow Campbell / Woodrow Campbell
Loren Morton Loren Morton

DATE: November 4, 1997

SUBJECT: DRC Staff Response to August 20, 1997 Request for DRC Intervention from Mr. Peter Haney: Atlas Minerals Uranium Tailings Pile near Moab, Utah.

We have completed our evaluation of the questions raised by Peter Haney in a letter dated August 20, 1997. It should be noted that Mr. Haney brought out some very interesting points that should have received more direct attention from the NRC. Mr. Haney's concerns can be organized into five different groups. DRC staff response for each is provided below.

1. Embankment Protection, Riprap Design, and River Migration -after completing our evaluation so that we would have a better idea of what questions to ask, we contacted Ted Johnson of the NRC to ask specific questions about the Atlas/Canonie (A/C) calculations and the NRC Final Technical Evaluation Report (FTER). Instead of answering specific questions the response given by Mr. Johnson was in more general terms. He indicated that the probability of the river moving so that it would directly impinge on the embankment were very remote. He also indicated that the calculations especially concerning the use of 11.2 inch rock and a thickness of 8 inches were clearly conservative and therefore concluded that the present design is adequate. As a result, the NRC concluded that there is no reason to modify the design or the FTER.

   In general we agree with Mr. Johnson and the NRC that the probability of the river moving to the point that it impinges on the embankment is very low. However, we agree with the NRC that is difficult to quantify this probability for a 1,000 year design life. The low probability of pile undercutting is based on the following arguments:

   a) The Colorado River is controlled by the opening at Matrimony Springs upstream of the embankment and more importantly by the Portal downstream which can act as a funnel slowing flood waters,
   b) The river would need to move a minimum of 500 feet to contact the pile, and

c) The Matheson Marsh and to a lesser extent the riparian zones near the embankment help to slow and store flood waters and also help to protect the banks from erosion.

A. Toeslope Riprap Apron Rock Size - in contrast with the NRC, we d[illegible]t agree that the calculations for riprap size and thickness are conservative. The A/C calculations use the Stephenson Method as found in Nelson, et. al. (NUREG/CR-4620). The following are excerpts from that NRC guidance document:

1) Page 41 - "Rock riprap is one of the most economical materials that is commonly used to provide for cover and slope protection. Factors to consider when designing rock riprap are: (1) rock durability, density, size, shape, angularity, and angle of repose; (2) water velocity, depth, shear stress, and flow direction near the riprap; and (3) the **slope** of the embankment or cover to be protected."

2) Page 43 - "Since the rock protection requirements are significantly different on various locations on the cover, it should be apparent that each riprap design procedure available was formulated to address a specific application."

3) Page 43 - "The sizing of the stable stone or rock requires the designer to determine the maximum flow rate per unit width (q), the rockfill porosity (n), the acceleration of gravity (g), the relative density of the rock (s), the **angle of the slope** measured from the horizontal (θ), the angle of friction (ϕ), and the empirical factor (C)."

As indicated in these excerpts the slope angle is very important in the calculation of the rock size. The A/C calculations show the side slope as being 10 (horizontal) to 3 (vertical) with the toe slope after launching (COE) being 2 to 1. To calculate the rock size using the Stephenson method A/C basically averaged the two slopes together; resulting in under-sized rock for the toeslope segment. This approach would be conservative if looking only at the sideslope, since it would be slightly steeper using the average. However, use of the averaged slope to represent the steeper toeslope is not conservative.

We completed an independent evaluation of the needed toeslope rock size using the actual design slopes. This DRC calculation shows a D50 rock size of 12.57 inches is needed. After correction for rock quality, using the same A/C correction factor, a D50 rock diameter of 12.8 inches is apparent. This change represents a 14% increase in needed D50 rock diameter.

B. Rock Apron Layer Thickness - the thickness evaluation performed by A/C uses a rock size D50 of 4 inch and the thickness T equals 2 times the D50 (T=8 in.=2x4 in.)

or 8 inches for the side slope. This same thickness is used for the toe slope and is increased by 50% (multiplied the thickness by 1.5) as suggested as a safety factor by the COE to account for stone lost during launching (Maynord and White, p. 21). Calculating a volume using a length of 2640 ft, a width of 50 feet (47 feet plus a 3 foot factor of safety) and a thickness of 1 foot (1.5 x 8 inches) indicates a volume of 4889 cubic yards and this was rounded up to 5000 cy as an added factor of safety.

As pointed out by Peter Haney this is very confusing especially when A/C calculated a D50 of 11.2 inches on the next page. Table 4-4 Riprap Sizes and Thicknesses (page 4-21 FTER) indicates that for the Southwest Drainage Channel the D50 is 11.2 inches and a thickness of 17 inches. Using this same thickness (which is a little over the 1.5 x D50 of 11.2 as suggested by the COE) the volume would be [2640x47x(17/12)]/27 = 6510 cy.

Based on DRC staff calculations above, and using a D50 of 12.8 inches and a safety factor of 1.5 the volume of rock needed at the toeslope would be [2640x47x1.5x(12.8/12)]/27=7350 cy.

C. Substrate for Rock Apron and Vicinity - sheet 2 of 10 of the Atlas Corporation Final Reclamation Plan Volume 1 shows an area near the toe of the embankment that is contaminated that will require excavation. This area will need to be backfilled and if possible revegetated to help prevent the river from moving in that direction towards the pile.

2. Salt Dissolution Related Subsidence - the NRC has concluded that an acceptable design basis for the Atlas pile will include a salt related dissolution subsidence rate of 1 meter in 1,000 years (FTER, p. 2-22). No independent analysis of this subsidence rate was undertaken by DRC staff. However, comparisons of relative elevations before and after such subsidence were made, as described below.

Atlas engineering design drawings suggest the base of the launchable stone will be found at the toeslope of the pile at an elevation of about 3,965.5 ft amsl; assuming a 2.5 foot thickness for the launchable stone, set below a native grade found at about 3,968 ft amsl (see 10/14/96 Smith Technologies Report, figures, Sheet 5 of 10, Cross-section A-A').

In April, 1994, nearby river elevation was measured at about 3,952 ft amsl (Mussetter Engineering, Figures 2.1 and 4.1). This places the toeslope rock apron at about 13.5 feet above the river elevations measured. If salt dissolution causes the land surface near the pile to subside about 1 meter (3 feet) during the next 1,000 years, the toeslope's riprap apron (launchable stone) will still be about 10 feet above the river's elevation. Should the river migrate against or near the pile, the launchable stone will continue to armor the slope found south of the pile and above normal river stage (elevation).

3. <u>Effect of Tamarisk on River Migration</u> - Mr. Haney has asked that the Board formally request the NRC respond to four previous letters regarding the effects of tamarisk on river migration. Review of the October 10, 1997 NRC letter to Mr. Haney shows no response was provided Mr. Haney on this issue. Such a request of the NRC can be made by the Board.

DRC staff consideration of tamarisk growing on the Atlas side of the river suggests such riparian vegetation will help prevent erosion of the riverbank. Due in large part to:

A. Binding Action - of roots that reinforce riverbank soils and increase the bank's resistance to erosional forces.

B. Water Velocity Reduction - the tamarisk and other riparian vegetation found above the riverbank will resist over-bank floods and thereby reduce floodstage water velocities.

As for the Moab side of the river, tamarisk and other riparian growth would encourage deposition of sediment, thereby shifting the river channel northwest-ward toward the pile. This migration tendency could be offset by tamarisk growth on the Atlas side of the river. However, the pile's design basis already presumes the worse case scenario where river migration places the cutbank in contact with the embankment. Adequate engineering design of launchable stone in the toeslope's riprap apron can protect the pile for this probability.

4. <u>Effect of Isostatic Adjustment of Tailings Over Salt</u> - Mr. Haney's August 20, 1997 letter outlines two issues: 1) effect of loading caused by the pile on the Paradox Salt formation at depth, and 2) changes to loading caused by addition of new cover materials to the pile. Both of these issues are described below:

A. <u>Pile Loading Effects on the Paradox Salt at Depth</u> - effects of a static load on a soil or rock foundation can be estimated using Boussinesq's Equation (Lindeburg, p. 10-10). For the worst case scenario, immediately underneath the tailings pile, the change in pressure (ΔP) in the foundation due to the added load can be calculated as follows:

$$\Delta P = \frac{3 * \text{Load}}{2\pi * z^2}$$

where: Load = force caused by the tailings (lbs)
z = depth in foundation below the load (ft).

Because this is a inverse distance squared relationship, the added pressure to the foundation is quickly dissipated with depth. DRC staff estimates suggest that a static load of 100 feet of tailings accumulation, as found near the outside of the topslope area, will generate a load of 11,000 lbs/ft^2 at the base of the tailings. This load would be quickly depleted to less than 1.0 lb/ft^2 at a depth of 73 feet below the tailings pile. Considering that the Paradox Salt formation is more than 300 feet below the tailings in the vicinity of the river, and deeper yet at northern locations under the Atlas pile; it appears the additional load caused by the tailings would have little, if any effect, on deep salt movement or displacement.

B. Effects of Incremental Load Increases Caused by Cover System - review of the proposed cover design, indicates that the thickest part of cover system will be constructed across the inside area of the topslope where tailings are finest (Smith Technology Corporation, Sheet 5 of 10, Section E-E'). At this location, the cover system will be about 3.5 feet thick. This added thickness represents an increase in static load of only 3.5% above that calculated for the tailings column described above. For some reason, if the cover system were to be increased to 7 feet of thickness, this change would constitute only a 7% increase in static load. Based on the Boussinesq calculations above, this small increase in static load would be quickly dissipated within a depth of 75 feet into the foundation materials.

As a result of these DRC staff considerations, it appears that the tailings pile and cover system will have little effect on deep salt movement or displacement inside the Paradox Salt formation.

5. Pile Exemption Under Atomic Energy Act (AEA) Section 84(c) - the final NRC staff technical position paper attached to Mr. Haney's August 20, 1997 letter, outlines an exemption option for Title II piles that cannot meet a 200-year stability requirement (NRC Option 5). DRC staff review of the proposed design, and suggested changes made above, indicate that the pile can be stable for more than 200 years. On this basis, it appears that the AEA Section 84(c) exemption is not warranted. Furthermore, such an exemption could only serve to decrease the engineering design requirements, and thereby offer less protection of human health and the environment.

References

Lindeburg, M.R., 1986, Civil Engineering Reference Manual, 4th Ed., Professional Publications Inc., Belmont, CA, 614 pp. (approx.).

Maynord, S.T. and D.M. White, September, 1995, "Toe Scour and Bank Protection Using Launchable Stone", U.S. Army Corps of Engineers Technical Report HL-95-11, 26 pp. with tables, charts, and attachments.

Mussetter Engineering, May, 1994, "Final Report Geomorphic, Hydraulic and Lateral Migration Characteristics of the Colorado River Moab, Utah", unpublished consultants report, 43 pp. with figures and appendices.

Nelson, J.D., S.R. Abt, R.L. Volpe, D. van Zyl, N.E. Hinkle, W.P. Staub, June, 1986, "Methodologies for Evaluating Long-Term Stabilization Designs of Uranium Mill Tailings Impoundments", NUREG/CR-4620, U.S. Nuclear Regulatory Commission, 145 pp.

Smith Technology Corporation, October 14, 1996, "Final Reclamation Plan, Volume 1 - Text, Tables, Drawings, Atlas Corporation Uranium Mill Tailings Disposal Area", unpublished consultants report, 50 pp. with references, tables and figures.

WC/LBM:lm

cc: Ted Johnson, NRC
Mike Fliegel, NRC

F:\haney.mem
File: Atlas Public Inquiries

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RICHARD E. BLUBAUGH
Vice President Environmental
And Governmental Affairs

November 7, 1997

William J. Sinclair
Executive Secretary
Utah Radiation Control Board
168 North 1950 West
P.O. Box 144850
Salt Lake City, Utah 84114-4850

RE: Citizen Request for State Intervention in NRC's Final Technical Evaluation Report Decision to Cap the Atlas Tailings Pile in Place

Dear Mr. Sinclair:

Thank you for the opportunity to address the Utah Radiation Control Board in response to the issues raised by Mr. Peter Haney in his August 20, 1997 letter to you.

As you know, the licensing action at issue has been under review for many years, since 1988 if one considers the initial filing of the modified reclamation plan. There have been several opportunities for interested parties to raise issues of concern. The request by Mr. Haney appears to be either premature (since the final licensing action has not been made by NRC) or so late as to be irrelevant.

However, in order to clarify the misunderstanding and subsequent misrepresentations concerning the issues raised, Atlas Corporation wishes to offer comments on the following for review and consideration by this Board:

- Potential for the Colorado River to migrate toward the Atlas tailings pile
- Collapsible (self-launching) rock apron
- *Final Staff Technical Position, Design of Erosion Protection Covers for Stabilization of Uranium Mill Tailings Sites*
- Radiation exposure and risks associated with the Atlas tailings pile.

In advance of any final ruling, Atlas appreciates the consideration given our comments by you and the Radiation Control Board members.

Yours very truly,

Richard E. Blubaugh

Richard E. Blubaugh

Enclosures

Potential for the Colorado River to Migrate toward the Atlas Tailings Pile

Mr. Haney refers to page 4-13 of the FTER and quotes the following:

The licensee has indicated that the potential for migration of the river is very low and that there are several bases supporting this low probability. The staff requested that Atlas provide quantitative evidence to support this conclusion; however Atlas was not able to do so. Therefore, Atlas intends to provide a large rock apron at the toe of the disposal cell to protect the pile from erosion.

With all due respect to Mr. Haney and other actively interested parties, there is a considerable amount of background information and knowledge that is not readily available to those who have not been directly involved with this licensing process. With respect to this particular issue, Atlas' contractor, Smith Technologies (formerly Canonie Environmental) contracted Mussetter Engineering, Inc. of Fort Collins, Colorado to perform a detailed study addressing the potential of the Colorado River to migrate toward the Atlas tailings pile. This work was performed in 1994 and is the basis for NRC's conclusions also discussed on page 4-13 and elsewhere in the FTER. After reviewing the Mussetter report NRC requested detailed quantitative proof that the river would not migrate and threaten the stability of the tailings pile, or, alternatively, an engineering design modification that would protect the pile if the river did migrate. However, upon learning that the cost of the studies necessary to obtain such quantitative proof was at least as costly as the engineering remedy for the concern, Atlas elected to propose the engineered collapsible rock apron rather than undertake additional studies that would not only be costly and time-consuming but would also be subject to further criticism, potentially resulting in demands for more data and yet more studies…and more delay.

Included for your reference is the summary and conclusions from the Mussetter Engineering report (Attachment 1). Mussetter concludes:

Since the flows that occur in the river on a frequent basis are responsible for virtually all of the work done on the channel banks and the physical factors that cause higher flows to exert very little stress on the right bank are permanent features related to the geology of the site, there is no reason to believe that a tendency for lateral migration of the river toward the tailings pile will occur in the future.

The complete report titled, "Geomorphic, Hydraulic and Lateral Migration Characteristics of the Colorado River, Moab, Utah" will be made available if requested.

We believe that the remainder of the section of the FTER referenced by Mr. Haney (enclosed as Attachment 2) is at least as important as the statements quoted. This section concludes with the following statements:

In summary, the staff concludes that it is unlikely that the river will migrate as far as the tailings pile within the next 200-1000 years. However, because quantitative proof of bank stability was not provided, it is prudent to design the pile for such an occurrence. The licensee intends to provide an erosion protection apron for the pile and this measure is considered by the staff to be a conservative method for addressing Colorado River erosion concerns.

Mr. Haney, in his evaluation of lateral migration of the Colorado River using a multiple hypothesis approach to the problem, asks why the Mussetter report only addresses the period through 1985, asserting that the period from 1985 to 1995 was excluded from the study. The implication is that the exclusion was intentional to avoid having to deal with evidence that would conflict with the remainder of the findings developed during the study. In fact, the Mussetter study included actual site data collected through April 1994 and topographical records based on aerial photography taken in January 1994 (Mussetter, pp. 1.1 - 1.4, Attachment 1).

Collapsible (self-launching) Rock Apron

Atlas' contract engineer (Smith/Canonie) designed the collapsible rock apron according to methods provided by the U.S. Corps of Engineers (COE). The rock apron is also referred to as a launchable or self-launching rock apron. As discussed in Section 4.5.1.2.3 of the FTER (Attachment 3), the rock (also referred to as riprap) will be placed along the sides and toe of the pile (Figure 1). The design included consideration of the assumed future location of the river channel, the estimated depth of scour and the volume and size of the rock. It was assumed that the new location of the channel would be immediately adjacent to the toe of the southeastern side slope embankment. The apron will be installed from the mouth of the southwest drainage channel northeastward to the point where it joins the Moab Wash toe protection (Figure 2).

The following statements reflect the NRC's conclusions concerning the rock apron (FTER, p. 4-18):

The staff reviewed computations provided by the licensee. Based on this review, the staff concludes that the proposed apron length and thickness will provide an adequate volume of rock to protect the side slope from further migration of the Colorado River.

To provide the required protection, the licensee used the Stephenson Method to determine that the riprap apron will need an average rock size of 11.2 inches. Based on the review of the computations provided by the licensee, the staff concludes that this rock size is acceptable.

In response to Mr. Haney's August 24, 1997 letter to Mr. Ted Johnson of the NRC staff, NRC's October 10, 1997 letter (Attachment 3) stated:

To directly address your assertion that there are errors in the licensee's calculations, staff reviewed the calculations and concludes that no mistakes were made by the licensee in calculating the volume of rock needed.

NRC further the clarified the apparent misunderstanding as follows:

Evaluation of the COE design procedures indicates that the volume of rock needed is based on the thickness (T) of the rock layer to be placed on the side slope of the cell: the volume of rock for the apron is not based on the average size (D50) of the rock (11.2 inches) in the apron, as you suggest. The proposed thickness of the side slope rock is 8 inches, with a D50 of 4.1 inches. Therefore, the licensee's use of a rock layer thickness of 8 inches is correct.

Finally, the October 10, 1997 NRC response to Mr. Haney concludes:

In summary, the staff concludes that the licensee's calculations are correct and that significant additional conservatisms, beyond that called for in the COE design procedure, have been included in the design of the rock apron. We conclude that the design is conservative and will protect the tailings pile from erosion, if the river channel should migrate all the way to the toe of the tailings pile. As a result, there is no reason to modify the FTER..

Final Steff Technical Position, Design of Erosion Protection Covers For Stabilization of Uranium Mill Tailings Sites

In his August 20, 1997 letter, Mr. Haney observes that the NRC chose Option 4 (sacrificial soil covers designed to permit controlled erosion) of the *Final Staff Technical Position, Design of Erosion Protection For Stabilization of Uranium Mill Tailings Sites*. He further asserts that NRC should have chosen Option 5 (Exemption of the site under 84 (c) of the Atomic Energy Act based on licensee justification of inability to meet the primary regulations).

First, NRC does not choose the option, the licensee selects the option that it believes works best for the site specific conditions and circumstances. NRC reviews the licensee's proposal and either approves, rejects or approves with modifications.

Second, the option that best describes the Atlas proposal is Option 3, which is considered to be the most effective method of assuring long-term stability (Staff Technical Position, p. 12). Option 3 is described as: *Soil covers totally protected by a layer of rock riprap on both the top and side slopes* (Staff Technical Position, p. 11). The Atlas proposal does not include a sacrificial soil cover designed to permit controlled erosion.

Third, Option 5 is the least desirable of the options presented by the staff since it applies to those sites where designs are not able to meet the minimum long-term stability requirement of 200 years. It is intended be the option of last resort where active maintenance is clearly contemplated. Option 5 provides for an exemption of reclamation design not an exemption of the site.

Radiation Exposure and Risks Associated with the Atlas Tailings Pile

NRC calculates the current radiation exposure from the site to the maximally exposed individual results in a total effective dose equivalent of approximately 70% of the NRC limit of 100 mrem/yr. This is generally confirmed by actually monitoring results (Attachment 4). If the tailings pile were capped as proposed, the post reclamation dose to the same individual would be about 7% of the 10 CFR Part 20 limit. The radiation risk associated with this site can be significantly reduced with the implementation of the proposed reclamation plan.

An independent analysis of the health risks comparing Atlas's proposed plan and the alternative of moving the pile was conducted in 1995 by SENES Consultants Limited. Figure 3.1 from the SENES report (enclosed as Figure 3) shows that the lowest exhalation rates of radon are those associated with onsite reclamation.

We respectfully suggest that this Board, rather than intervene and delay final reclamation even further than it has been to date, consider the fact that radon is the primary source of potential radiation exposure to the citizens of Utah and visitors to the area and urge NRC to move expeditiously to approve Atlas's proposed reclamation plan so that radiation exposures can be reduced to the required levels at the earliest opportunity.

URANIUM MILL TAILINGS UPDATE

Board information item

Receipt of a request to intervene in the Atlas tailings
Final Technical Evaluation Report (FTER) decision

Enclosed in the packet is a letter from the Executive Secretary to the concerned citizen outlining the process for trying to resolve the issues raised in his letter (also enclosed). Bill Sinclair will inform the Board of progress to date in resolving this issue.

State of Utah

DEPARTMENT OF ENVIRONMENTAL QUALITY
DIVISION OF RADIATION CONTROL

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September 16, 1997

Peter Heaney
1991 Cedar Hills Drive
Moab, UT 84532

Dear Mr. Heaney:

Thank you for your letter dated August 20, 1997 which requested State intervention in the NRC's Final Technical Evaluation Report decision to cap the Atlas tailings pile in place. In the correspondence, you raised several technical engineering and site stability questions that you feel have not sufficiently been addressed by Atlas Corporation or the Nuclear Regulatory Commission. The letter was followed up with a meeting between you, Loren Morton of my staff, and myself on September 4, 1997. At that meeting, we discussed your concerns and tried to clarify the issues.

After our meeting, I asked my staff to review your request. My staff has initially reviewed your request and has informed me that they would like some additional time to study the issues so as to make a recommendation to me as how we could proceed on this issue, with or without involvement of the Board. Since your request also involves potential legal action and filing by the State, I have informed the Attorney General's Office that I would need their input as to the appropriateness of your request. Since both of the activities will constitute research of the part of the legal and technical staff, it will not be possible to schedule your issue for the October Board meeting. However, I have recommended to and received concurrence from Dr. Sunderland, Chairman of the Radiation Control Board that if necessary, we would place your item on the Board agenda for the November 7, 1997 meeting. This may prove more convenient to you since the meeting is to be held in Moab at the Senior Citizen's Center beginning at 9 a.m..

As soon as I have a staff and legal recommendation as to how to proceed, I will be back in contact with you to arrange a convenient time to discuss the matter. If I can be of further assistance, do not hesitate to contact me.

Sincerely,

William J. Sinclair
Executive Secretary
Utah Radiation Control Board

c: Norm Sunderland, Chairman, Utah Radiation Control Board
Denise Chancellor, Utah Attorney General's Office
Joseph Holonich, NRC Uranium Recovery Branch
Richard Blubaugh, Atlas Corporation
Dianne Nielson, Ph.D., Executive Director, UDEQ

20 August 97

William Sinclair, Director
Division of Radiation Control
State of Utah
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Re: Request for Utah State Intervention in the NRC's Final Technical Evaluation Report decision to cap the Atlas Tailing Pile in place pursuant to, 2.715(c). (Rules of Practice for Domestic Licensing Proceedings, page 2-23, attached.)

Dear Bill:

We have promised the citizens of Grand County that science will provide the answers to the questions concerning the stability of the Atlas Tailings pile in the floodplain of the Colorado River. With the issuance of the Nuclear Regulatory Commission's (NRC's) Final Technical Evaluation Report (FTER) there are many science questions that were not answered or even addressed in the FTER despite public comment and NRC promises.

The NRC has not fulfilled its regulatory obligation concerning the safety of Utah's citizens, downstream citizens, and our environmentl; it has only professed that some launchable rocks should protect the pile through 1000 years. Page 4-13 of the FTER states, "The licensee has indicated that the potential for migration of the river is very low and that there are several bases supporting this low probability. The staff requested that Atlas provide quantitative evidence to support this conclusion; however Atlas was not able to do so. Therefore, Atlas intends to provide a large rock apron at the toe of the disposal cell to protect the pile from erosion."

The NRC has chosen Option 4 (Sacrificial soil covers designed to permit controlled erosion) of the Final Staff Technical Position Design of Erosion Protection For Stabilization of Uranium Mill Tailings Sites. (Portions enclosed, its full length is 100+ pages) With the evidence presented to date the NRC should have chosen Option 5 (Exemption of the site under 84(c) of the Atomic Energy Act based on licensee justification of inability to meet the primary regulations) (See attached NRC position paper).

How can the NRC conclude that it has no "quantitative evidence to support this conclusion" that there is a low probability for the potential migration of the Colorado River and then determine that it can have a "planned failure" mechanism (the launchable rock apron) to protect the pile in 1000 years? How can one plan to intervene in 1000 years in processes that they don't understand today?

The NRC has determined by its issuance of the Final TER that the Atlas Tailings Pile will be protected from erosion by the Colorado River for at least 1000 years. This determination is clearly in error because none of the following hypotheses for erosion processes were even marginally addressed.

<u>Hypotheses</u> = H_x

H_1 → Migration Pattern of river channel

H_2 → Effects of salt dissolution subsidence

<u>Problem</u> →
(Colorado river erosion)

H_3 → Effects of Tamarisks

H_4 → Isostatic adjustment of tailings pile over salt causing active diaprism

The following excerpts are from <u>To Interpret the Earth</u> <u>*Ten ways to be wrong*</u> By Stanley Schumm, Cambridge University Press, 1991.

> ...., the problems that are process controlled cannot be solved without an understanding of the processes operating. (Schumm, p. 102)
>
> The experiments with which geological history confronts us are neither reversible nor repeatable, and they are accomplished on a scale of time and space that precludes as a matter of course exact reproduction. Moreover, they cannot be directly observed; but they must be reconstructed historically (Bubnoff, 1963, p.3). (Schumm, p. 4)
>
> Any of the problems that are considered to be relevant to a research problem must be resolved before a satisfactory explanation can be developed and certainly before extrapolation is attempted. (Schumm, p.98)
>
> Recognition of the problem will lead to more thorough research plans. Consideration of the problems may, therefore, be time-consuming, difficult and expensive, but **never as expensive as failure**. The development of the required understanding of natural systems will be intellectually rewarding, and it will be **cost effective**. (Schumm, p. 119) (bold added)

A final quotation in Schumm from a <u>1897</u> paper by Chamberlain is instructive:

> *The studies of the geologist are peculiarly complex. It is rare that his problem is a simple unitary phenomenon explicable by a single simple cause. Even when it happens to be so in a given instance, or at a given stage of work, the subject is quite sure, if pursued broadly, to grade into*

some complication or undergo some transition. He must therefore ever be on the alert for mutations and for the insidious entrance of new factors. If, therefore, there are any advantages in any field in being armed with a full panoply of working hypotheses and in habitually employing them, it is doubtless the field of the geologist. (Schumm, p. 22)

The NRC has accepted non answers to its and the public's questions concerning the processes involved at this site. It then went on to utilize an unproven technology. The technology that the NRC is using to protect the tailings pile was defined by the US Army Corps of Engineers and published in September, 1995. It is unproven to protect for 25 years, and can not be extrapolated to the 1000 years required by regulations.

1.) Given that the monitoring wells drilled by Atlas were meant to interpret water quality and not site stability, I respectfully request that the State of Utah intervene and drill enough borings to determine the answers NRC's questions concerning the impacts of H_1, H_2, and H_4. Just because Atlas has not answered the NRC's questions does not imply that the NRC's questions are difficult to answer. In fact, I have collected data that directly addresses H_1 myself, (see enclosure).

2.) Concerning the effect that tamarisk (H_3) may have on river migration, I request that you to officially ask Ted Johnson, Senior Hydraulic Engineer for the NRC, to answer the questions posed by my letters and in person to the NRC last spring concerning the tamarisk impact. At the May 96 meeting on the Atlas site he said my concerns would be specifically addressed in the FTER, they were not.

3.) Lastly, I do not contest the NRC's legal ability to exempt sites that do not meet their regulations; but I do not know of any regulation that permits the NRC to ignore processes that will ultimately cause this tailings pile's integrity to fail. Why shouldn't this site be exempted according to Section 84(c) of the Atomic Energy Act?

I thank you for your time,

Sincerely,

Peter Haney

Peter Haney

cc: Dr. Jackson, NRC Chairwoman
Senator Orrin Hatch
Senator Robert Bennett
Congressman Chris Cannon
Radiation Control Board
Grand County Council
Ted Johnson, NRC Hydraulic Engineer

to the factors set forth in paragraph (d)(1) of this section. The staff may file such an answer within fifteen (15) days after service of the petition or supplement.

(d) The Commission, the presiding officer, or the Atomic Safety and Licensing Board designated to rule on petitions to intervene and/or requests for hearing shall permit intervention, in any hearing on an application for a license to receive and possess high-level radioactive waste at a geologic repository operations area, by the State in which such area is located and by any affected Indian Tribe as defined in part 60 of this chapter. In all other circumstances, such ruling body or officer shall, in ruling on—

(1) A petition for leave to intervene or a request for a hearing, consider the following factors, among other things:

(i) The nature of the petitioner's right under the Act to be made a party to the proceeding.

(ii) The nature and extent of the petitioner's property, financial, or other interest in the proceeding.

(iii) The possible effect of any order that may be entered in the proceeding on the petitioner's interest.

(2) The admissibility of a contention, refuse to admit a contention if:

(i) The contention and supporting material fail to satisfy the requirements of paragraph (b)(2) of this section; or

(ii) The contention, if proven, would be of no consequence in the proceeding because it would not entitle petitioner to relief.

(e) If the Commission or the presiding officer determines that any of the admitted contentions constitute pure issues of law, those contentions must be decided on the basis of briefs or oral argument according to a schedule determined by the Commission or presiding officer.

(f) An order permitting intervention and/or directing a hearing may be conditioned on such terms as the Commission, presiding officer or the designated atomic safety and licensing board may direct in the interests of: (1) Restricting irrelevant, duplicative, or repetitive evidence and argument, (2) having common interests represented by a spokesman, and (3) retaining authority to determine priorities and control the compass of the hearing.

(g) In any case in which, after consideration of the factors set forth in paragraph (d)(1) of this section, the Commission or the presiding officer finds that the petitioner's interest is limited to one or more of the issues involved in the proceeding, any order allowing intervention shall limit his participation accordingly.

(h) A person permitted to intervene becomes a party to the proceeding, subject to any limitations imposed pursuant to paragraph (f) of this section.

(i) Unless otherwise expressly provided in the order allowing intervention, the granting of a petition for leave to intervene does not change or enlarge the issues specified in the notice of hearing.

(j) The provisions of this section do not apply to license applications docketed under subpart J of this part.

§ 2.714a Petitions for review of certain rulings on petitions for leave to intervene and/or requests for hearing.

(a) Notwithstanding the provisions of § 2.730(f), an order of the presiding officer or the atomic safety and licensing board designated to rule on petitions for leave to intervene and/or requests for hearing may be appealed, in accordance with the provisions of this section, to the Commission within ten (10) days after service of the order. The appeal shall be asserted by the filing of a notice of appeal and accompanying supporting brief. Any other party may file a brief in support of or in opposition to the appeal within ten (10) days after service of the appeal. No other appeals from rulings on petitions and/or requests for hearing shall be allowed.

(b) An order wholly denying a petition for leave to intervene and/or request for a hearing is appealable by the petitioner on the question whether the petition and/or hearing request should have been granted in whole or in part.

(c) An order granting a petition for leave to intervene and/or request for a hearing is appealable by a party other than the petitioner on the question whether the petition and/or the request for a hearing should have been wholly denied.

§ 2.715 Participation by a person not a party.

(a) A person who is not a party may, in the discretion of the presiding officer, be permitted to make a limited appearance by making oral or written statement of his position on the issues at any session of the hearing or any prehearing converence within such limits and on such conditions as may be fixed by the presiding officer, but he may not otherwise participate in the proceeding.

(b) The Secretary will give notice of a hearing to any person who requests it prior to the issuance of the notice of hearing, and will furnish a copy of the notice of hearing to any person who requests it thereafter. When a communication bears more than one signature, the Commission will give the notice to the person first signing unless the communication clearly indicates otherwise.

(c) The presiding officer will afford representatives of an interested State, county, municipality, and/or agencies thereof, a reasonable opportunity to participate and to introduce evidence, interrogate witnesses, and advise the Commission without requiring the representative to take a position with respect to the issue. Such participants may also file proposed findings and exceptions pursuant to §§ 2.754 and 2.762 and petitions for review by the Commission pursuant to § 2.786. The presiding officer may require such representative to indicate with reasonable specificity, in advance of the hearing, the subject matters on which he desires to participate.

(d) If a matter is taken up by the Commission pursuant to § 2.786 or sua sponte, a person who is not a party may, in the discretion of the Commission, respectively, be permitted to file a brief "amicus curiae". A person who is not a party and desires to file a brief must submit a motion for leave to do so which identifies the interest of the person and states the reasons why a brief is desirable. Except as otherwise provided by the Commission such brief must be filed within the time allowed to the party whose position the brief will support. A motion of a person who is not a party to participate in oral argument before or the Commission will be granted at the discretion of the Commission.

§ 2.715a Consolidation of parties in construction permit or operating license proceedings.

On motion or on its or his own initiative, the Commission or the presiding officer may order any parties in a proceeding for the issuance of a construction permit or an operating license for a production or utilization facility who have substantially the same interest that may be affected by the proceeding and who raise substantially the same questions, to consolidate their presentation of evidence, cross-examination, briefs, proposed findings of fact, and conclusions of law and argument. However, it may not order any consolidation that would prejudice the rights of any party. A consolidation under this section may be for all purposes of the proceeding, all of the issues of the proceeding, or with respect to any one or more issues thereof.

§ 2.716 Consolidation of proceedings.

On motion and for good cause shown or on its own initiative, the Commission or the presiding officers of each affected proceeding may consolidate for hearing or for other purposes two or more proceedings, or may hold joint hearings with interested States and/or other federal agencies on matters of concurrent jurisdiction, if it is found that such action will be conducive to the proper dispatch of its business and to the ends of justice and will be conducted in accordance with the other provisions of this subpart.

FINAL
STAFF TECHNICAL POSITION
DESIGN OF EROSION PROTECTION COVERS FOR
STABILIZATION OF URANIUM MILL TAILINGS SITES

1. INTRODUCTION

Criteria and standards for environmental protection may be found in the Uranium Mill Tailings Radiation Control Act (UMTRCA) of 1978 (PL 95-604) (see Ref. 1) and 10 CFR Section 20.106, "Radioactivity in Effluents to Unrestricted Areas." In 1983, the U. S. Environmental Protection Agency (EPA) established standards (40 CFR Part 192) for the final stabilization of uranium mill tailings for inactive (Title I) and active (Title II) sites. In 1980, the United States Nuclear Regulatory Commission (NRC) promulgated regulations (10 CFR Part 40, Appendix A) for active sites and later revised Appendix A to conform to the standards in 40 CFR Part 192. These standards and regulations establish the criteria to be met in providing long-term stabilization.

These regulations also prescribe criteria for control of tailings. For the purpose of this staff technical position (STP), control of tailings is defined as providing an adequate cover to protect against exposure or erosion of the tailings. To help licensees and applicants meet Federal guidelines, this STP describes design practices the NRC staff has found acceptable for providing such protection for 200 to 1000 years and focuses principally on the design of tailings covers to provide that protection.

Presently, very little information exists on designing covers to remain effective for 1000 years. Numerous examples can be cited where covers for protection of tailings embankments and other applications have experienced significant erosion over relatively short periods (less than 50 years). Experience with reclamation of coal-mining projects, for example, indicates that it is usually necessary to provide relatively flat slopes to maintain overall site stability (Wells and Jercinovic, 1983, see Ref. 2).

Because of the basic lack of design experience and technical information in this area, this position attempts to adapt standard hydraulic design methods and empirical data to the design of erosion protection covers. The design methods discussed here are based either on: (1) the use of documented hydraulic procedures that are generally applicable in any area of hydraulic design; or (2) the use of procedures developed by technical assistance contractors specifically for long-term stability applications.

It should be emphasized that a standard industry practice for stabilizing tailings for 1000 years does not currently exist. However, standard practice does exist for providing stable channel sections. This practice is widely used to design drainage channels that do not erode when subjected to design flood flows. Since an embankment slope can be treated as a wide channel, the staff concludes that the hydraulic design principles and practice associated with

The technical basis for using rock for long time periods is well-developed. Jahns (1982, see Ref. 23) points out that many kinds of rocks are relatively resistant to weathering. Most of these more resistant rock types have long been used as construction materials, in monuments, or for decorative purposes, with varying degrees of success. However, it must be recognized that there are limitations associated with procedures that are used to assess rock performance for a 1000-year period.

Determining the quality of riprap needed for long-term protection and stability can therefore be a somewhat difficult and subjective task. Very little design guidance is available to assess the degree of oversizing needed for a particular rock type to survive for long periods, based on its physical properties.

In assessing the long-term durability of erosion protection, the NRC staff has relied on the results of durability tests performed at several uranium mill sites and on information and analyses developed by technical assistance contractors, which provide methods for assessing rock oversizing requirements to meet long-term stability criteria. These procedures have also considered actual field data from several sites and have been modified to provide flexibility to meet construction requirements.

3. REGULATORY POSITION

In accordance with 40 CFR 192, Subparts A, B, and C, and 10 CFR Part 40, Appendix A, the design of protective covers should provide reasonable assurance of long-term stability. The design should provide for control of tailings for 1000 years, if reasonably achievable, but, in any case, for at least 200 years.

Several methods have been developed for designing unprotected soil covers or soil covers with some vegetation, to prevent the development and inhibit the growth of gullies. These methods, illustrated in Appendix A to this STP, are based on staff licensing and review experience and applicable hydraulic engineering principles. The computational procedures outlined in Appendix A were developed based on NRC staff experience with damage to erosion-protection structures during the occurrence of relatively minor storm events. Of necessity, these procedures attempt to account for the limited quantitative data base available to document long-term degradation and the questionable ability of vegetated soil covers to be effective in arid areas. Reasonable and conservative engineering judgment has been used, after evaluating the results of the various methods, to decide on the best estimate of the stable slope.

Methods are also presented for the design of sacrificial soil slopes (Appendix B), for evaluation of feasibility of covers (Appendix C), and for the design of riprap (Appendix D).

The aforementioned design procedures are concerned only with surface water erosion of the cover. The additional soil cover needed to account for wind erosion or sheet erosion needs to be factored into the soil cover design. Procedures discussed by Nelson, et al. (1986, see Ref. 24) may be used to determine the additional cover requirements.

In designing a protective cover, there are many options and design combinations that may be used. There are, in fact, an infinite number and variety of designs, and their selection will depend on site-specific conditions and phenomena. In general, however, cover designs fall into several broad categories. Based on NRC licensing experience with Title I and Title II sites, various options are normally employed to design cover systems:

Option 1	Soil covers designed to be stable for 1000 years.
Option 2	Combinations of soil covers on the top slopes and rock-protected soil covers on the side slopes, both designed to be stable for 1000 years.
Option 3	Soil covers totally protected by a layer of rock riprap on both the top and side slopes.
Option 4	Sacrificial soil covers designed to permit controlled erosion.
Option 5	Designs that are not able to meet the minimum long-term stability requirement of 200 years. Such designs may be exempted under Section 84(c) of the Atomic Energy Act (see Ref 25) for Title II sites and under the supplemental standards of 40 CFR Part 192 for Title I sites. Such exemptions may be granted, based on licensee justification of inability to meet primary regulations.

The preferred options to design a cover system are Options 1,2, and 3; such designs will be stable and will be effective for a 1000-year period. Option 4 is not considered to be a preferred design option; this option should be used only when detailed justification can be provided to demonstrate that designing for time periods greater than 200 years is not reasonably achievable.

Option 1 can generally be implemented only for very short slope lengths, or where significant credit can be given for vegetation. Discussion of unprotected stable soil covers may be found in Section 3.2.1, p. 14; design guidance may be found in Appendix A.

Option 2 may be implemented if Option 1 is impractical due to pile height, size, or topography. In these cases, combinations of stable soil covers over flatter areas and rock-protected soil covers over steeper areas should be considered as possibilities in meeting the 1000-year stability requirement. Discussion of combination covers may be found in Section 3.2.2, p. 16. Design guidance may be found in Appendix A (for soil top slopes) and in Appendix D (for rock-protected side slopes).

Option 3 may be implemented in those cases where rock riprap is available. The placement of riprap protected covers is considered by the NRC staff to be the most effective method of assuring long-term stability. Discussion of riprap cover design is provided in Section 3.3, p. 17. Design guidance may be found in Appendix D.

Option 4 may be implemented if providing combined stable soil top slopes and/or rock-protected side slopes is not practicable or is excessively costly. In such cases, sacrificial side slopes that permit controlled erosion may be acceptable, provided that the tailings will not be exposed or eroded. In general, this option should be considered only when tailings are not placed directly under the soil slope. The staff considers that such designs should be adopted only when licensees or the U. S. Department of Energy (DOE) can provide detailed justification that designing for a 1000-year stability period is not reasonably achievable and that designing for a 200-year period is the only reasonably achievable design option. Discussion of sacrificial side slopes, where tailings are not placed under embankment outslopes, may be found in Section 3.2.4, p. 16; design guidance may be found in Appendix B. Discussion of the detailed justification needed to demonstrate that other designs are not reasonably achievable may be found in Appendix C.

Option 5 may be implemented in those cases where designing for a 200-year stability period is not reasonably achievable. Where DOE or licensees can document the clear impracticability of such designs, they will be considered on a case-by-case basis, considering the possibility of alternatives under Section 84(c) of the Atomic Energy Act for commercial processing sites, or under the supplemental standards of 40 CFR Part 192, for inactive sites.

For the convenience of licensees and designers, Table 1, "Summary of Design Guidance," may be used to direct attention to appropriate sections of this STP and to provide guidance in the design of various features, according to the design option selected.

An evaluation of lateral migration of the Colorado River using a multiple hypothesis approach to the problem.

The following quote from Schumm (1991) is applicable to the current problem.

When only one hypothesis is generated and an attempt is made to demonstrate its correctness, it becomes a 'ruling hypothesis', which dominates the thinking of an investigator and may lead to serious error. The preferred method then is to develop as many explanations of a phenomenon as possible. Through the process of data collection these hypotheses are either modified or eliminated until a solution is developed, or perhaps until a solution is developed, or perhaps until multiple explanations or hypotheses are combined to obtain a composite solution or theory. (Schumm, p. 11)

The following must be considered relative to the Colorado River erosion and migration.

H_1 →Migration Pattern of river channel

H_2 →Effects of salt dissolution subsidence

Problem →
(Colorado river erosion) H_3 →Effects of Tamarisks

H_4 →Isostatic adjustment of tailings over salt with possible diaprism

Of the 4 hypotheses pertaining to the problem of lateral migration of the Colorado River, the NRC did not rule any in or out.

H_1 →Migration Pattern of river channel

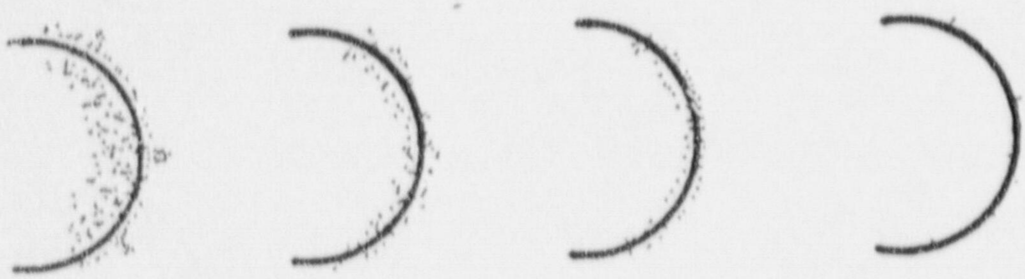
The Mussetter paper only addresses the period through 1985. Why was the period from 1985-1995 excluded from the study? Could it be that the period was one of erosion on the north bank, significant enough to erase all aspects of the aggradation of the period before? This is in fact true, the river has eroded away all aspects of previous historical aggradation and is currently further North than any known historical time. The main channel in now adjacent to the North bank.

Also, for the period 1905-1970 the south bank had been stable when viewed in historical photos and maps. With the invasion and colonization of tamarisks of this river reach, the south bank has currently migrated further north than where the North bank was in 1970. What will stop this current process?

Mussetter Engineering has put forward the hypothesis that the Colorado River has not been north of its present location in the recent past, and that the north bank of the river is composed of fan sediments from Courthouse and Moab Washes. If this is true the sediments on the north bank should have the mineralogic composition of the nearest wash rather than that of the river.

To test this hypothesis I sampled sand from the river, sand from Courthouse Wash, and using a hand auger/soil sampler I drilled and sampled a hole (T-1) on Tex McClatchy's property about 150 feet north of the river and about two tenths of a mile west of where Courthouse Wash enters the river. I was able to collect samples from the hole (T-1) down to about 14 and a half feet. I looked at the samples with a hand lens and (for all sand samples) rolled a small magnet around in the sand and looked at the edge of the magnet with a hand lens.

I found that the sand from Courthouse Wash was almost all quartz with *very minor* magnetite, only a few grains of magnetite stuck to the magnet. The Colorado River sand, in addition to quartz contained dark mineral grains, some mica and *abundant* magnetite. The magnet was almost completely covered with magnetite grains. The sand samples from T-1 were finer grained and included quite a bit of silt, but magnetite was still generally *common* in spite of the smaller grain size.



ABUNDANT (Colorado River) | COMMON (T-1) | MINOR | VERY MINOR (Courthouse Wash)

Magnetite grains on edge of magnet at 20 x.

This proves to me that the river has been north of its present location in the recent geologic past, and therefore could migrate laterally toward the pile in the future.

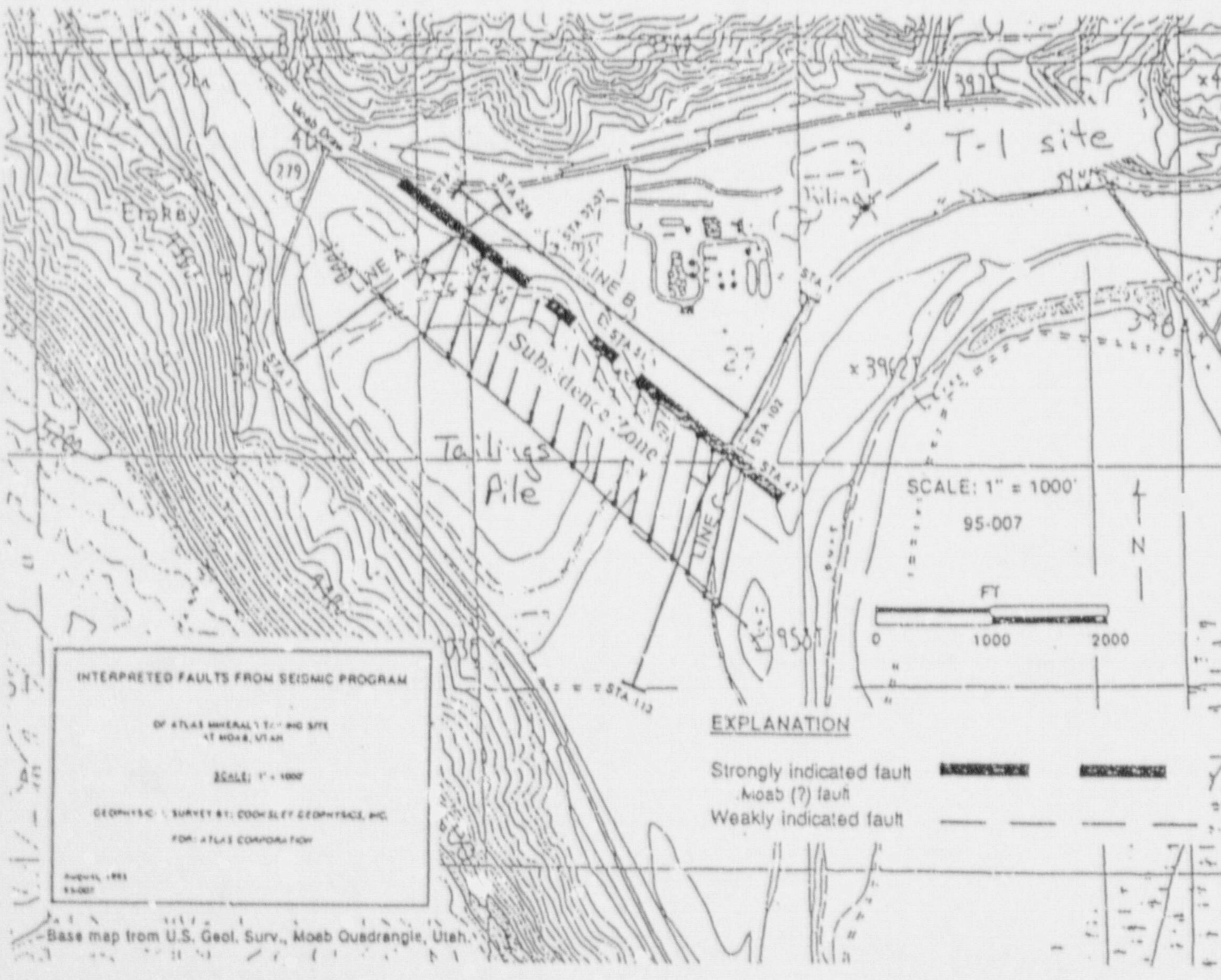
H_2 →Effects of salt dissolution subsidence

Question 6 of the NRC Nov 4, 1994 questions clearly asks "Effects of salt dissolution and/or subsidence on the location of the river channel." The Mussetter response is to call subsidence/ salt dissolution at this site, "some very speculative conclusions by structural geologists...." However the subsidence/salt dissolution model is presented by both Woodward/Clyde and Cooksley Geophysics and demonstrates that subsidence/salt dissolution is an active process at this site. The conclusion that subsidence needs to be episodic to be of consequence to the tailings pile is not correct. The tailings pile could sink relative to river level at a fairly constant rate of subsidence. If the earth that is holding the tailings pile up is sinking, as is documented in the Cooksley Geophysics report, then what will keep the pile out of the river? If the rock apron designed to protect the pile also sinks, then what will protect the pile?

From Page 13 of Cooksley Geophysics, Inc. report, August, 1995:

Smaller faults along which subsidence has taken place are well indicated on sections A and C. Such faults can not be interpreted from Line B which is very close to and parallel to the Moab fault. These smaller faults, some of which may be no more than fractures, probably strike northwesterly similar to the Moab fault. Doelling, (1985), attributes the formation of these structures to the dissolution of the salt and subsequent collapse of the overlying strata. He describes the fault blocks and their attendant faults as elongate in the direction of the valley. In the case of the site surveyed, their strike would be similar to that of the Moab fault. It is probably not possible to map all these smaller structures, but the more obvious ones are denoted on the seismic sections. An overprint of tilting and apparent folding has resulted from the collapse of the rock strata overlying the dissolution areas. Most, if not all of these faults have limited vertical extent.

Page 17 of Cooksley Geophysics, Inc. report, August, 1995 with subsidence zone overlay:



H_3 →Effects of Tamarisks

Despite my repeated requests: April 96, May 96, July 96, Jan 97 letters to the NRC, the issue of how tamarisk will impact Colorado river migration and tailings pile stability has never been addressed. It is and has been my contention that the recent lateral movement of the river channel towards the tailings pile is primarily due to the invasion and colonization by tamarisk of the Colorado river system. This has not been addressed in the FTER.

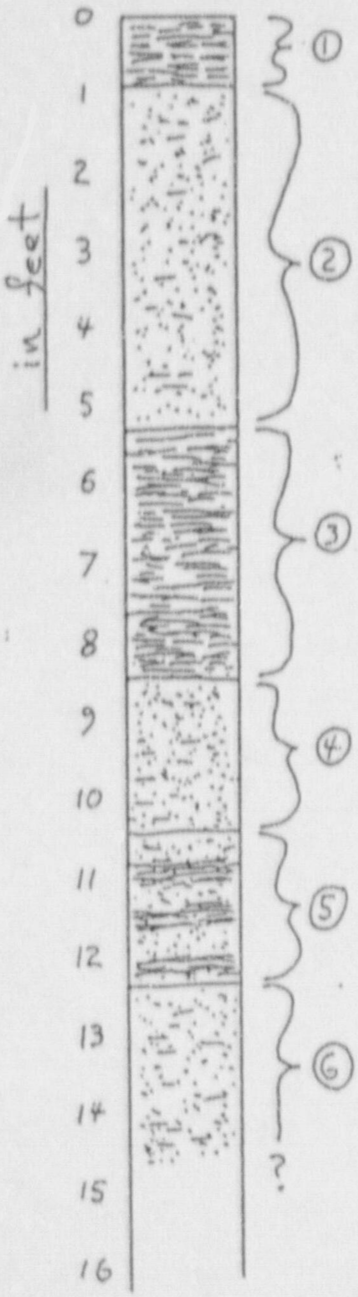
H_4 →Isostatic adjustment of tailings over salt with possible diaprism

> ***Sensitivity** - propensity of a system to respond to a minor external change. The change occurs at a threshold, which when exceeded produces a significant adjustment.* (Schumm, 1991)
>
> ***Complexity** - composed of numerous interconnected parts. Natural systems are inherently complex, but the complexity referred to here is the complex response that results when the system is perturbed. The complex system when interfered with or modified is unable to adjust in a progressive and systematic fashion, and its response can be complex.* (Schumm, 1991)

What have been the isostatic adjustment effects of loading the salt structure beneath this site with more than 10 million tons of tailings? What will be the cumulative effect of adding more than 1 million tons of additional cover material and rock armor?

Has that threshold for isostatic adjustment been exceeded by a triggerable process? Could the 1995 Level 1 survey by Grand County be indicative of that adjustment? That survey measured several inches of increased elevation over the past 40 years of the gas pipeline east of the site since a Level 1 survey in 1955.

Log of hole T-1.



1 Surface muds from historic floods.

2 Light greyish yellow-brown subangular quartz sand
1/4 mm or less plus silt
Common fine grained magnetite
Little or no iron stain on sand grains

3 Clay rich muds - very plastic when wet, similar to 1

4 Light greyish red-brown subangular quartz sand
About 1/8 mm plus silt
Minor fine grained magnetite
Minor iron stain on sand grains

5 Interbedded clay rich mud and fine sand

6 Light greyish red-brown subangular quartz sand
About 1/8 mm plus silt
Magnetite is minor to common and fine grained
Color gets browner at bottom of the hole

Colorado River

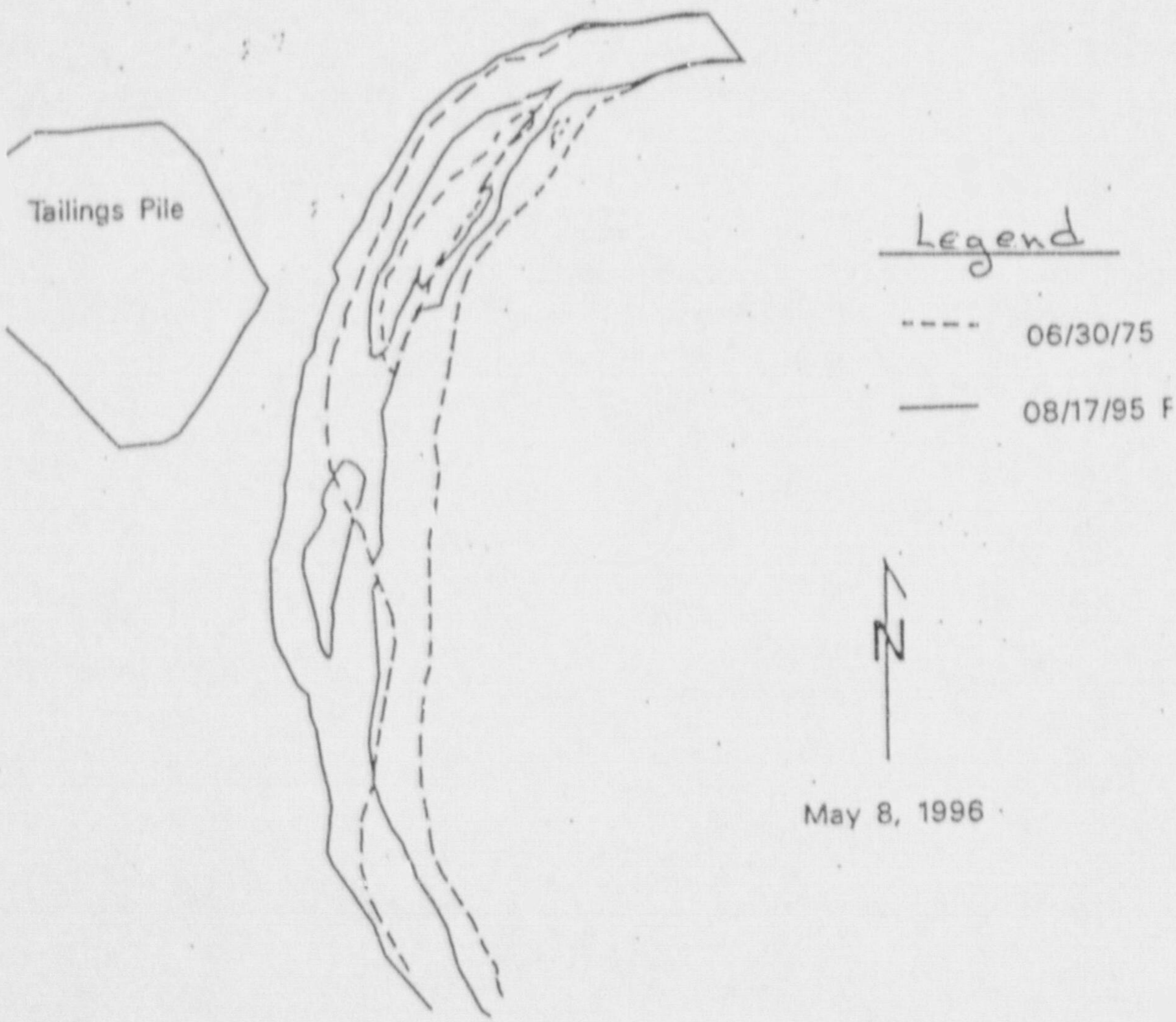
Medium grey brown
subangular sand 1/4 mm or less
Abundant magnetite
Some iron stained sand grains
Minor mica and dark mineral grains
Minor feldspar

Courthouse Wash

Pale reddish tan
subangular - sub ounded quartz 1/4 mm or less
Well sorted - little or no fines
Very minor magnetite
Very light iron stain on most grains
No feldspar, mica or dark mineral grains

Recent Lateral Migration of the Colorado River.

No scale



This data was digitized from scanned aerial photos (150 dpi) and rectified using Arc/Info. The data was then matched at the Colorado River bridge to allow for better matching.

24 Aug 97

Ted Johnson
Nuclear Regulatory Agency
Mail Stop 5-E-4
Washington D.C. 20555

Re: Calculations of Rock Apron Volume Requirements

Dear Ted:

I was reviewing the Atlas Reclamation Plan, Oct 96, while composing the accompanying letter. On page 1 of Appendix O, Calculation Brief - Rock Apron/Toe Design, the D_{50} of the proposed rock apron equals 11.2". Yet on page 2, T (thickness) = 8" was used in the calculations to determine necessary volume per linear foot. Shouldn't 11.2" have been used in the calculations as this is the T (thickness) of the Bank Revetment after launch? This would result in $21\sqrt{5}$ x 11.2"/12 x 1.5 instead of $21\sqrt{5}$ x 8"/12 x 1.5. This results in a linear foot volume requirement of 65.74 ft^3/ft instead of 47 ft^3/ft as calculated on page 2. This results in Rock Apron total volume requirements being 6427.9 yd^3 rather than the 4889 yd^3 or 5000 yd^3 (a difference of 30%) shown in the Calculation Brief on page 3, and the Final Technical Evaluation Report, Nureg - 1532.

This would also require a different construction design of the launchable rock apron. The 50 ft^3/ft configuration of 20' by 2 1/2' would be inadequate.

If my calculations are correct, then please revise the FTER to account for this discrepancy.

Sincerely,

Peter Haney

Peter Haney

1991 Cedar Hills Dr
Moab, UT 84532

cc: Dr. Jackson, NRC Chairwoman
Senator Orrin Hatch
Senator Robert Bennett
Congressman Chris Cannon
Utah Radiation Control Board
Grand County Council

been designed with a launchable rock apron to protect the pile if the river migrates laterally toward the pile.

5.6.4 *Embankment Toe and Rock Apron Design*

The tailings pile toe has been designed to accommodate the erosive forces which would be produced should the Colorado River migrate toward the tailings pile to such an extent that the river channel is located directly against the slope of the pile. The design provides for protection of the toe by use of a launchable rock apron (COE, 1991). As scour and erosion occur, the rock in the apron is undermined and launched, rolling or sliding down the slope of the eroded bank to protect against additional erosion. The location of the apron is shown on Sheet 4. A design detail of the toe of the pile slope and the rock apron is shown on Sheet 5.

The rock apron consists of 11.2-inch D_{50} riprap which is adequately sized to be stable against both overland flow and river velocities. The apron is 2.5 feet thick and 20 feet wide, which provides an adequate volume of rock to collapse into a stable configuration in an eroded channel. To account for turbulence at the toe, the 11.2-inch D_{50} rock has been extended approximately 5 feet up the tailing pile side slope. The apron will be constructed at the same slope as the existing ground adjacent to the tailings pile to minimize flow concentration and erosion. The design calculations for the rock apron/toe design are provided in Appendix O.

SMITH

APPENDIX O

CALCULATION BRIEF
ROCK APRON/TOE DESIGN

/

Canonie Environmental

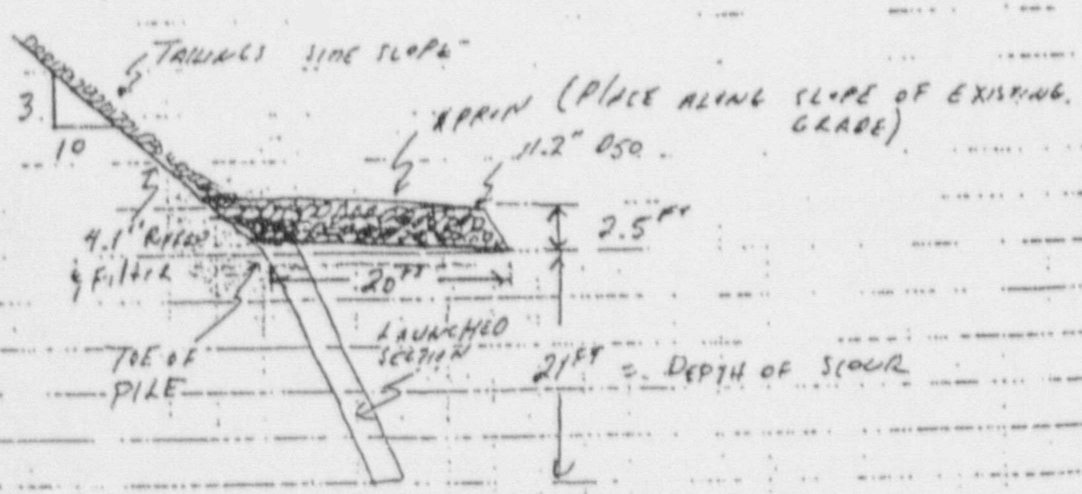
By JWS Date 11/30/94 Subject ATLAS - APRON/TOE DESIGN Sheet No. 1 of 14

Chkd. By DR Date 12/29/94 Project No. 88-007-21

1/4" X 1/4"

PURPOSE - The purpose of this calculation brief is to design an apron @ the "EASTERN" edge of the ATLAS Tailings Pile in Moab, UT. The purpose of the Apron is to protect the tailings pile should the Colorado River migrate to the Toe of the Tailings pile as suggested by the NRC, (ATLAS-Uranium Mill, questions and comments, 1994). Figure 1 shows the Tailing Pile, Proposed Location of the tailings pile, and the Colorado River

Results - 5000 CUBIC YARDS OF 11.2" D50 RIPRAP is required to protect the eastern edge of the tailings pile should the river migrate to the toe of the tailings pile. A conceptual view of the Apron is shown below:



CALCULATIONS

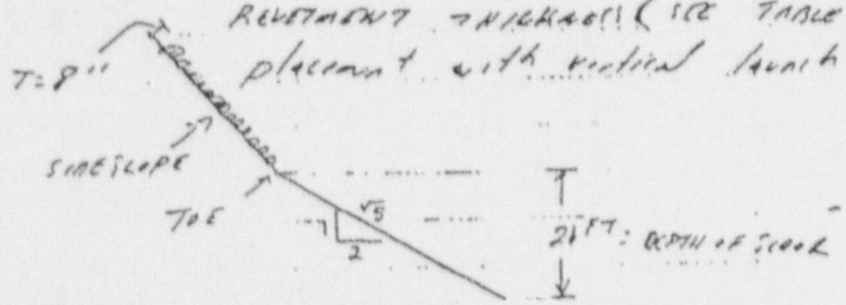
1) DETERMINE LAUNCHABLE VOLUME OF RIPRAP

The procedure given in EM 1110-2-1601 (1991 UPDATE) will be followed. - SEE ATTACHMENT A FOR EXCERPTS. SECTION 3-10 "Revetment Toe Protection methods" - method D will be followed.

LAUNCH SLOPE = 1V on 2H

SCOUR DEPTH = 21 FT = TOE ELEVATION (≈3968) - RIVER THALWEG ELEVATION (≈3947) - SEE FIG. 2 FOR RISES IN RIVER NEAR PILE

THICKNESS AFTER LAUNCHING - INCREASE BY 50% OF BANK REVETMENT THICKNESS (SEE TABLE 3-2 ATTACH. A; dry placement with vertical launch distance: 28 FT > 15 FT)



∴ Volume/ft = Launch Slope Length x Bank Revetment Thickness x % increase

$$= \overset{46.95}{21\sqrt{5}} \times \overset{11.2/12 = .9333}{8/12} \times 1.5 \qquad \frac{1.14}{\cancel{1.06}}$$

= [illegible] FT³/FT. - FOR ADDITIONAL FACTOR OF SAFETY INCREASE TO 50 FT³/FT

LENGTH OF APRON ALONG TOE = 2640 FT - FROM APPROXIMATELY 300 FT. NORTH OF EASTERN MOST EDGE OF PILE SOUTH TO SOUTHERN MOST POINT OF PILE AS SHOWN ON FIGURE 1.

BWH 3/11/95

CanonieEnvironmental

By JWC Date 11/30/94 Subject Atlas - Apron Design Sheet No. 3 of 14

Chkd. By DR Date 12/29/94 Project No. 88-067-21

1/4" X 1/4"

65.7H

6427.9 yd^3

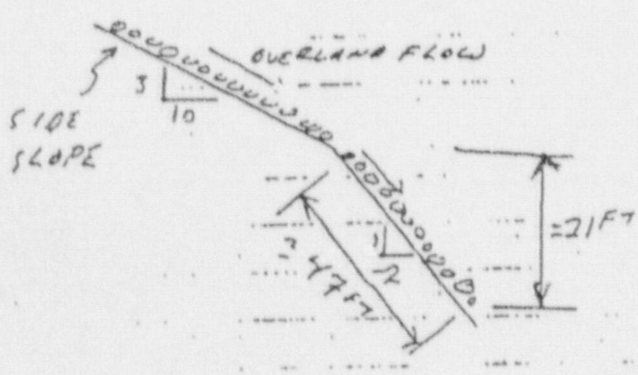
TOTAL VOLUME = 50 ft^3/ft x 2640 ft

= 4889 CY

or ≈ 5000 CY

② DETERMINE APRON ROCK SIZE

USE STEPHENSON'S METHOD (NUREG-4620, NUREG 4651) BECAUSE method is applicable to slopes greater than 10%. Rock will be designed based on final configuration (i.e. after launching on to 1V:2H slope). Overland flow over 3V:10H side slope and 1V:2H apron will be considered in design of rock size.



Sept 1, 1997

The Honorable John McCain
United States Senate
241 Russell
Washington, D.C. 20510-0303

Re: Concerns of NRC's responses to Senator McCain's concerns regarding the Colorado River's effects on the Atlas Tailings Pile. Request for U. S. Army Corps of Engineers (USACE) evaluation of NRC proposed "launchable rock apron" erosion protection design.

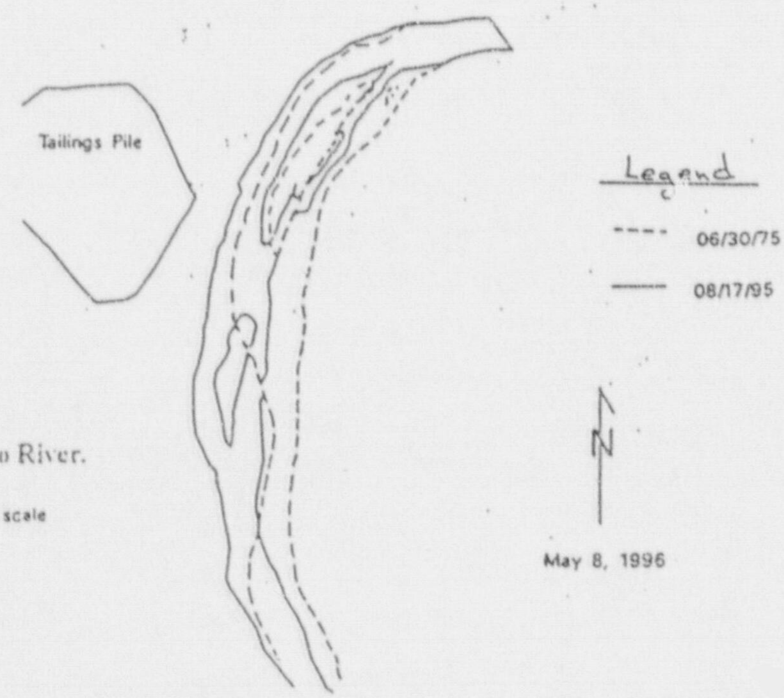
Dear Senator McCain:

I thank you for your interest in the interesting problem of the Atlas Tailings Pile in Moab, UT.

In the NRC's June 4, 1997 responses to your April 25, 1997-letter, NRC Chairwoman Jackson wrote, "However, even if the 1984 flood were exceeded, no erosion damage to the pile is anticipated because there is a temporary cover that will avoid tailings contact with the river, and as stated above, floods on this reach of the Colorado River are non-erosive. It should be noted that the 1984 flood reached the toe of the tailings pile with no adverse consequences."

I believe the "temporary cover" referred to was for wind blown tailings and was not related to erosion protection. I know of no recent measures to "avoid tailings contact with the river . . . "

Between 1975 and 1995 there was considerable erosion along this reach of the Colorado River with the vast majority occurring during the 1983 and 1984 floods. The NRC received this same information documented with aerial photographs in April, 1996. (See also an attached letter to Bill Sinclair.)



Recent Lateral Migration of the Colorado River.

This data was digitized from scanned aerial photos (150 dpi) and rectified using Arc/Info. The data was then matched at the Colorado River bridge to allow for better matching.

The NRC is utilizing a U.S. Army Corps of Engineers design, *Toe Scour and Bank Protection Using Launchable Stone*, Technical Report HL-95-11, September 1995, to theoretically afford the necessary protection for the tailings pile. This report is just two years old, and is still just theory and not a proven technology.

I, therefore, respectfully request the Senator ask the US Army Corps of Engineers at their Flood Control Structures Research Program of the Coastal and Hydraulics Laboratory at the Waterways Experiment Station in Vicksburg, Mississippi; to review the adequacy of the NRC's "launchable rock apron" erosion protection design described in the Final Technical Evaluation Report, NUREG-1532, as compared to the theoretical USACE design, HL-95-11.

If and when this review might happen, I would respectfully request a copy of the USACE findings also be sent to me.

I thank you for your continuing interest in this problem.

Sincerely,

Peter Haney

Peter Haney
1991 Cedar Hills
Moab, UT 84532

cc: Dr. Jackson, NRC Chairwoman
Senator Orrin Hatch
Senator Robert Bennett
Congressman Chris Cannon
Bill Sinclair, Utah Radiation Control Board
Grand County Council
Ted Johnson, NRC Hydraulic Engineer

Flood Control Structures
Research Program

Technical Report HL-95-11
September 1995

Toe Scour and Bank Protection Using Launchable Stone

by Stephen T. Maynord, Douglas M. White

U.S. Army Corps of Engineers
Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

Coastal & Hydraulics Laboratory

Final report

Approved for public release; distribution is unlimited

Prepared for U.S. Army Corps of Engineers
Washington, DC 20314-1000

URANIUM MILL TAILINGS UPDATE

Board information item

Request to intervene in the Atlas Tailings Final Technical Evaluation Report decision

As indicated at the last Board meeting, a citizen (Peter Heaney) of Moab has requested the State to intervene on his behalf in the decision regarding the Atlas Tailings Final Technical Evaluation Report. In the last Board packet, you were provided with a copy of the letter outlining Mr. Heaney's concerns (I have enclosed it again). The Board cannot make a final decision in this matter at the Board meeting because of the unavailability of both of our legal counsel. This issue requires both technical and legal considerations. To take advantage of our meeting in Moab, I have requested Mr. Heaney to come and summarize his issues to the Board. I have also directed the staff to prepare and report the DRC technical position relative to Mr. Heaney's issues, and thirdly, I have invited Atlas representatives to provide their prospective to Mr. Heaney's concerns. I have advised each group to try to limit their prospective to 20 minutes or less on this issue. Following the three discussions, the Board will be able to address questions to any of the parties. I have also encouraged the parties to provide the Board any appropriate information to you at the time of the Board meeting. Finally, since a legal decision cannot be rendered at the meeting, I will set the final decision on this item for the December agenda which will include discussion by the Utah Attorney General's Office of the legal issues.

State of Utah
DEPARTMENT OF ENVIRONMENTAL QUALITY
DIVISION OF RADIATION CONTROL

Michael O. Leavitt
Governor
Dianne R. Nielson, Ph.D.
Executive Director
William J. Sinclair
Director

168 North 1950 West
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Salt Lake City, Utah 84114-4850
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October 24, 1997

Peter Heaney
1991 Cedar Hills Drive
Moab, UT 84532

Dear Mr. Heaney:

This is a follow-up to your letter dated August 20, 1997 which requested State intervention in the NRC's Final Technical Evaluation Report decision to cap the Atlas tailings pile in place. In the correspondence, you raised several technical engineering and site stability questions that you feel have not sufficiently been addressed by Atlas Corporation or the Nuclear Regulatory Commission. As a result of your letter and issues raised, I have scheduled the issue for the next meeting of the Utah Radiation Control Board to be held at the Senior Citizen's Center in Moab beginning at 9:30 a.m. A copy of the tentative agenda is enclosed.

At the Moab meeting, I am requesting that you take twenty minutes or less to address your issues to the Board. This will be followed by a report by my staff on the issues. Atlas Corporation also requested time at our last Board meeting to address the issue and they will also be afforded the same amount of time. Unfortunately, the Attorney General staff is not available to attend the meeting. Without their legal advice, I feel that a final decision on this issue would be premature. However, this meeting will give the Board the opportunity to hear the various technical issues from all sides such that they can render a final decision at the December Board meeting to be held December 5, 1997 in Salt Lake City. This will also allow the Board time to study the various technical issues and information before making a final decision.

Please feel free to provide the Board with any handouts or other information you feel is appropriate regarding your intervention request. We look forward to hearing from you at our Moab meeting. If you have any questions, please do not hesitate to contact me.

Sincerely,

William J. Sinclair
Executive Secretary
Utah Radiation Control Board

c: Norm Sunderland, Chairman, Utah Radiation Control Board
Denise Chancellor, Utah Attorney General's Office
Joseph Holonich, NRC Uranium Recovery Branch
Richard Blubaugh, Atlas Corporation
Dianne Nielson, Ph.D., Executive Director, UDEQ

OCT 1997
Received
Division Of
Radiation
Control

12 October 97

William Sinclair, Director
Division of Radiation Control
State of Utah
P.O. Box 144850
168 North 1950 West
Salt Lake City, UT 84115-4850

Re: November Radiation Control Board Meeting in Moab. Response to Citizen Request for Utah State Intervention in the NRC's Final Technical Evaluation Report decision to cap the Atlas Tailing Pile in place pursuant to, 2.715(c) of NRC Rules of Practice for Domestic Licensing Procedures.....

Dear Bill:

Thank you for your response dated September 16, 1997. Please confirm if the concerns addressed in my letter, 20 August 97, will be on the November Board meeting agenda, your letter says "if necessary". I still feel that answers are necessary.

Again, I would like to invite you and the Board to examine directly my concerns. This would be accomplished by a jet boat tour of the Colorado River adjacent to the Atlas Tailings Site Both shores could then be examined at yours and the Boards leisure for aggradation and erosion. Either the afternoon of November the 6th or 7th or both are available. Any experts Atlas or the NRC wishes to send, who could help explain the evidence, are welcome to join us.

I thank you for your time.

Sincerely,

Peter Haney

Peter Haney
1991 Cedar Hills Dr
Moab, UT 84532

cc: Dianne Nielson, Ph.D., Executive Director, DEQ
Norman Sunderland, Ph.D. Chairman of DRC Board

2.715(c) The presiding officer will afford representatives of an interested State, county, municipality, and/or agencies thereof, a reasonable opportunity to participate and to introduce evidence, interrogate witnesses, and advise the Commission without requiring the representative to take a position with respect to the issue. Such participants may also file proposed findings and exceptions pursuant to §§2.754 and 2.762 and petitions for review by the Commission pursuant to §2.786. The presiding officer may require such representative to indicate with reasonable specificity, in advance of the hearing, the subject matters on which he desires to participate.