REVIEW OF PROPOSED 5th DAM RAISE

UPPER TAILINGS POND EMBANKMENT SYSTEM

LISBON VALLEY OPERATIONS

RIO ALGOM CORPORATION

by

John D. Nelson and Steven R. Abt

17631

REVIEW OF PROPOSED 5th DAM RAISE
UPPER TAILINGS POND EMBANKMENT SYSTEM
LISBON VALLEY OPERATIONS
RIO ALGOM CORPORATION

for

U.S. Nuclear Regulatory Commission

NMSS

by

John D. Nelson and Steven R. Abt

Geotechnical Engineering Program

Civil Engineering Department

Colorado State University

Fort Collins, Colorado 80523

October 1, 1980

TABLE OF CONTENTS

List of Figures

1. INTRODUCTION

The Rio Algom Lisbon Valley Mine and Mill Facility is located south of La Sal, Utah. This mill was previously licensed and is currently under review for relicensing. The Rio Algom Mill has been depositing tailings in a disposal system consisting of two tailings ponds, referred to as the upper pond and the lower pond. The lower pond has reached its full capacity. All tailings are now being deposited into the upper pond and the pond is nearly full.

The Rio Algom Corporation has proposed that the embankment of the upper pond be raised five feet utilizing the downstream method of construction with slopes of 2.0 horizontal to 1.0 vertical. This five feet raise is proposed as a short term solution for the storing of mill tailings.

This report presents the results of a review of the stability of the embankment system after the proposed five foot raise has been constructed. The five foot expansion is an interim measure until final plans and specifications are presented for an alternate ongoing tailings management plan.

The proposed impoundment expansion (five foot raise only) was reviewed in accordance with USNRC Regulatory Guide 3.11. The review is based upon the documents listed as references in this report and observations made during a visit to the site.

2. DESCRIPTION OF EXISTING CONDITIONS AND RESULTS OF ANALYSES

2.1 Hydrologic Considerations

The required storage capacity of the upper tailings pond was based on considerations of the 100-year storm. The 100-year storm was selected rather than the Probable Maximum Flood (PMF) design series because the proposed five foot expansion is considered an interim measure until a subsequent proposed tailings management plan is implemented.

The principal drainage area tributary to the upper tailings pond is approximately 150 acres. Dames and Moore (ref. 3) computed the 100-year, 7-day rainstorm to produce 4.0 inches of rainfall. Based on 100 percent runoff, a total volume of 50 acre-feet of runoff would need to be stored in the tailings pond area.

In determining the storm water storage capacity of the tailings pond, it was stipulated that the maximum operating pond level would be five feet below the embankment crest elevation. It was computed that after storage of the 100-year storm and allowing for wave runup a minimum freeboard of two feet would remain in the pond at all times.

2.2 Stability Considerations

The existing embankment and soil properties are described in the report by Dames and Moore dated August 22, 1980 (ref. 3). Figure 2.1 shows a plan view of the existing embankment and the proposed five-foot embankment raise. Figure 2.2 shows the critical embankment section through the existing embankment and the proposed five-foot embankment raise.

Borings were advanced at locations 1 and 2 as shown in Figure 2.1.

The logs of those borings as presented by Dames and Moore are shown in Figure 2.3. Samples were taken from depths of 22.5 feet in each boring for laboratory testing and determination of shear strength. Multiple stage triaxial tests were performed on each of those samples. Stress paths and strength envelopes as presented by Dames and Moore (ref. 3) are shown in Figures 2.4 and 2.5. The triaxial tests indicated that for the embankment soil, the angle of internal friction was 34.5° and 38.5° in terms of effective stresses, and 31.0° and 40.0° in terms of total stresses. For one test, a cohesion of 800 psf was indicated whereas for all others the cohesion was zero.

No shear strength was presented for the natural foundation soils. Standard penetration test results taken in the foundation soils indicate blow counts ranging from 12 to 23 blows per foot (ref. 3). Some direct shear test results were presented in the Dames and Moore report dated October 2, 1973 (ref. 1). That report was prepared in conjunction with the design of the upper tailings embankment. Test results presented for samples taken from boring D7 (Page I-A-7; ref. 1) indicate that angles of internal friction of less than 20° may be attained for yield shear strengths.

Stability analyses were conducted by Dames and Moore utilizing the shear strength parameters as shown in Fig. 2.2. The results indicated that the minimum factor of safety for static steady state seepage conditions would be 1.55. For seismic conditions, the minimum factor of safety would be 1.35. For static analyses, effective stress shear strength parameters were utilized whereas for seismic analyses total stress shear strength parameters were used.

3. INDEPENDENT REVIEW OF ANALYSES

3.1 Hydrologic Analysis

A review of the applicant's hydrologic analysis for the proposed expansion was performed. Dames and Moore (Ref. 3) presented the 100-year, 7-day rainfall as 4.0 inches. The staff checked the reported 100-year rainfall with a 24-hour, 100-year rainfall value of 3.0 inches as presented in the U.S. Weather Buearu Technical Paper No. 40. Also, the low hazard probable maximum precipitation (PMF) was computed for the project site in accordance with procedures presented in the U.S. Bureau of Reclamation publication "Design of Small Dams" (Ref. 6). The low hazard thunderstorm yielded a 6-hour storm depth of 2.93 inches while the 24-hour general storm was estimated to be 5.80 inches. Because the low hazard PMF is usually larger than the 100-year storm, the value of 4.0 inches for the 100-year, 7-day storm as reported by the applicant is considered to be reasonable.

The hydrologic maps submitted by the applicant were checked and reviewed by the staff. On the basis of the maps provided, it is estimated that the upper pond will have a surface area of 39 acres after the embankment is raised resulting in an average storage volume of 27.5 acre-feet per foot of freeboard. Although the applicant did not provide an elevation-volume curve for the proposed expansion, it was estimated that the expansion will have a total storage capacity of approximately 138 acre-feet of runoff.

Utilizing the 4.0 inch 100-year rainfall and 100 percent runoff, a runoff of 50 acre-feet would have to be stored within the pond freeboard. The addition of 40 acre-feet of runoff to the impoundment would raise the water surface approximately 1.8 feet. Should extreme wind conditions prevail when the 100-year storm occurs, the maximum wave height is estimated to be 2 feet.

Assuming that the 1.8 feet and 2.0 feet of freeboard would be consumed by runoff and wave height respectively, a freeboard of 1.2 feet would remain. Although 1.2 feet of freeboard falls somewhat short of the reported minimum freeboard of 2.0 feet, it would only occur under extreme conditions and is considered to be adequate.

3.2 Stability Analyses

3.2.1 Site Conditions and Material Properties

During the later part of 1979, a site visit was made by Roy Person, NRC and John D. Nelson, CSU. These personnel were accompanied by representatives from Rio Algom and Dames and Moore and walked over both embankments to observe conditions along the crest, the face, and the toe of the embankment. In general, the embankment appeared to be maintained well. There was no evidence of erosion or cracking of the embankment. No evidence of seepage or wet spots along the face of the embankment were observed.

Appendix B of the Dames and Moore report dated August 22, 1980, includes a report of construction inspection and control during the upper tailings embankment. In all cases, the percent compaction that was finally achieved was greater than 90 percent. In a few isolated areas, the actual percent compaction was less than 90 percent. Those areas were reworked and subsequent testing indicated that adequate compaction had been accomplished. On the basis of that report, it may be concluded that the embankment fill material will be of uniform density.

3.2.2 Shear Strength

The report on inspection control (Appendix B, ref. 3) indicates that relatively uniform densities may be expected throughout the embankment. The frequency of sampling and testing that was reported in ref. 3 is therefore considered to be adequate.

Multiple stage triaxial tests were perform d on two samples taken from the embankment. The test results indicated reasonable magnitude of shear strengths in terms of effective stresses. The shear strength parameters in terms of total stress, do not differ greatly from those for effective stresses. This is due to the development of low negative pore water pressures. This is reflected in the shapes of the stress paths and indicate the overconsolidated nature of the compacted embankment material. It is believed, therefore, that the total stress shear strength parameters that are reported, adequately describe the shear strength of the embankment.

Shear strength values for the natural foundation soils as shown in Figure 2.2 were not substantiated with laboratory data. Furthermore, laboratory test data presented in the Dames and Moore report dated October 2, 1973 (ref. 1) indicate that shear strength values considerably lower than those that were assumed for the stability analyses may exist in the foundation soils. The reviewers, therefore, conclude that inadequate shear str h data for the natural foundation soil has been presented.

The boring logs that were presented in reference 3 indicated that bedrock had not been encountered in the borings. A subsequent letter from James R. Boddy of Dames and Moore dated September 11, 1980 to Rio Algom (ref. 5) indicates that bedrock was, in fact, encountered. Furthermore, in reference 3 the critical embankment section (as shown in Figure 2.2 of this report) indicates two strata of natural soils. The basis for representing the natural soil in two distinct strata with different shear strengths is not presented and the boring logs do not indicate any basis for doing so. Also, on page 7 of reference 3 it is stated that a cutoff barrier was extended to competent bedrock. This cutoff barrier is not indicated in the critical embankment for stability analyses. It is concluded, therefore, that the critical embankment section as shown in reference 3 (Figure 2.2 of this report) does not represent the actual existing conditions with regard to geometry and shear strength of the foundation soils.

3.2.3 Seepage Conditions and Location of Phreatic Surface

The phreatic surface within the embankment that was assumed for purposes of stability analyses is shown in Figure 2.2. However, the boring logs indicate that to the entire depth of soil overlying the bedrock, no groundwater was encountered. Furthermore, piezometers placed at depths of 31.5 feet in boring 1 and 28.5 feet in boring 2 indicated no water at the time reference 3 was prepared. The two piezometers, however, were located near the top of the location of the phreatic surface as shown in Figure 2.2. Consequently, they are not capable of locating a phreatic surface even slightly lower than that. It is concluded, therefore, that the locations of the peizometers are such that they would be unable to accurately describe the location of the phreatic surface unless it were relatively high.

The fact that no water was encountered during drilling may be indicative that seepage is existing in the weathered sandstone beneath the foundation soils. However, there is no indication that observations were made in the drill holes after long periods of time after drilling. It is concluded, therefore, that the location of the phreatic suface within the embankment and foundation soils is not adequately addressed.

3.2.4 Stability Analyses

Stability analyses were conducted by Dames and Moore utilizing their in-house computer program. The critical failure surfaces that were determined are shown in Figure 2.2 of this report. The static factor of safety of 1.55 and the seismic factor of safety of 1.35 were indicated.

Stability analyses were checked at Colorado State University using the Computer Program STABL2. The program utilizes a modified Bishop's method similar to that utilized by Dames and Moore. This program computed a factor of safety of only 1.4 for the static condition shown in ref. 3. The STABL2 program generates conservative values and for that reason the actual factor of safety was checked by means of a hand calculation. This resulted in a factor of safety for the static critical section of 1.45. For the seismic analyses, the STABL2 program indicated a factor of safety of 1.14. This value is somewhat lower than that indicated by Dames and Moore. Nevertheless, it is greater than the value required by Regulatory Guide 3.11. An additional analysis was conducted by the review team considering a potential shear surface that extended primarily through the proposed embankment raise. For that shear surface, factors of safety for static conditions of 1.57 and for seismic conditions of 1.08 were computed.

The factors of safety that were computed appear to be adequate. Although the Bishop's method of analysis indicated a factor of safety slightly lower than 1.5 the assumed phreatic surface is believed to be higher than what actually exists, and therefore, would generate factors of safety lower than what actually exists. Some question, however, exists as to the adequacy with which the conditions that were analyzed represent the actual conditions.

4. RECOMMENDATIONS

On the basis of the foregoing review, it is recommended that the following issues be addressed in more detail.

4.1 It is recommended that a revised Appendix A to reference 3 be submitted. This Appendix should include corrected boring logs indicating any variations between the two strata of foundation soils. They should also indicate the depth to bedrock. In addition, this Appendix should include water content and dry density data for the second triaxial test sample.

4.2 The basis on which the shear strength of the natural soils was assumed was not presented. The applicant should show by means of laboratory data or field tests that the assumed values of shear strength are in fact accurate. If this can not be demonstrated, revised slope stability analyses should be conducted.

4.3 Page 7 of reference 3 indicates that a cutoff trench was excavated and recompacted beneath the embankment. However, the critical embankment section that was analyzed and is reproduced as Figure 2.2 in this report does not indicate that cutoff wall. The effect of that cutoff on the stability analyses should be discussed. Plate 4 in reference 3 (reproduced as Figure 2.2 herein) should be revised to indicate that such a cutoff does exist if it does, or else the text should be revised. Because the inspection report as presented in Appendix B of reference 3 indicates that a cutoff trench was excavated, the reviewers believe that Plate 4 (ref. 3) is incorrect and that the stability analyses should be revised.

4.4 The boring logs and the piezometers indicate that the phreatic surface shown in Plate 4 of ref. 3 (see Fig. 2.2) is incorrect. It is not uncommon for the phreatic surface within an embankment to be lowered by a highly permeable natural foundation soil or through highly permeable bedrock. It was indicated in reference 3 that the upper layers of the sandstone bedrock are weathered and it is possible that the seepage is occurring therein. However, it is also possible that seepage could be occurring through a layer in the natural foundation soils causing a zone of high water content with low strength. It is recommended that adequate sampling in the natural foundation soils should be accomplished to show that such a zone of high moisture content does not exist that would adversely affect the stability of the embankment.

4.5 On page 18 of reference 3, it is stated that "it is estimated that a sufficient quantity of material will be available within the immediate vicinity of the impoundment area"...in order to accomplish the proposed five-foot raise. The applicant should demonstrate by means of exploratory borings or test pits that a sufficient quantity of material does in fact exist. Alternatively, it should be demonstrated that alternative sources of borrow with suitable properties can be located in the event that suitable borrow does not exist in the immediate vicinity.

REFERENCES

1. Dames and Moore, "Report of Consulting Services, Tailings Pond Embankment Stability and Groundwater Geohydrology and Seepage Evaluation, Lisbon Valley Mine Tailings Disposal System", Job No. 7144-002-06, October 2, 1973.

2. Dames and Moore, "Report of Preliminary Management Study, Lisbon Valley Mine Tailings Disposal System", Job No. 7144-0106-06, January 18, 1980.

3. Dames and Moore, "Report of Geotechnical Evaluation to Support the Request for a Five-Foot Dam Raise, Upper Tailings Pond Embankment System, Lisbon Valley Operations, for Rio Algom Corporation", Job No. 07144-017, August 22, 1980.

4. Rio Algom Corporation, letter to Mr. John Linehan, dated August 27, 1980.

6. U.S. Bureau of Reclamation (1977), Design of Small Dams, Revised Reprint, A Water Resources Technical Publication.

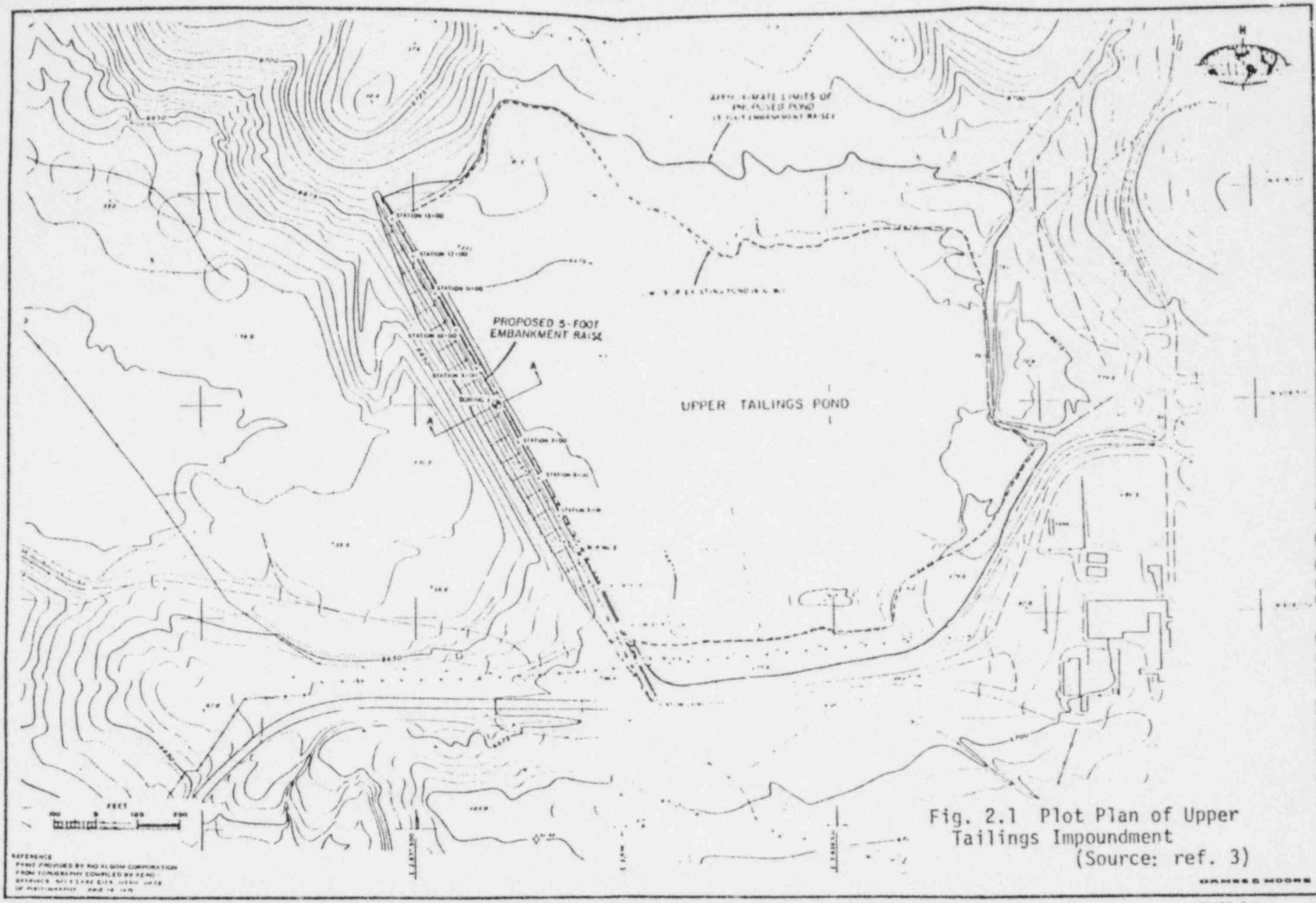


Fig. 2.1 Plot Plan of Upper Tailings Impoundment (Source: ref. 3)

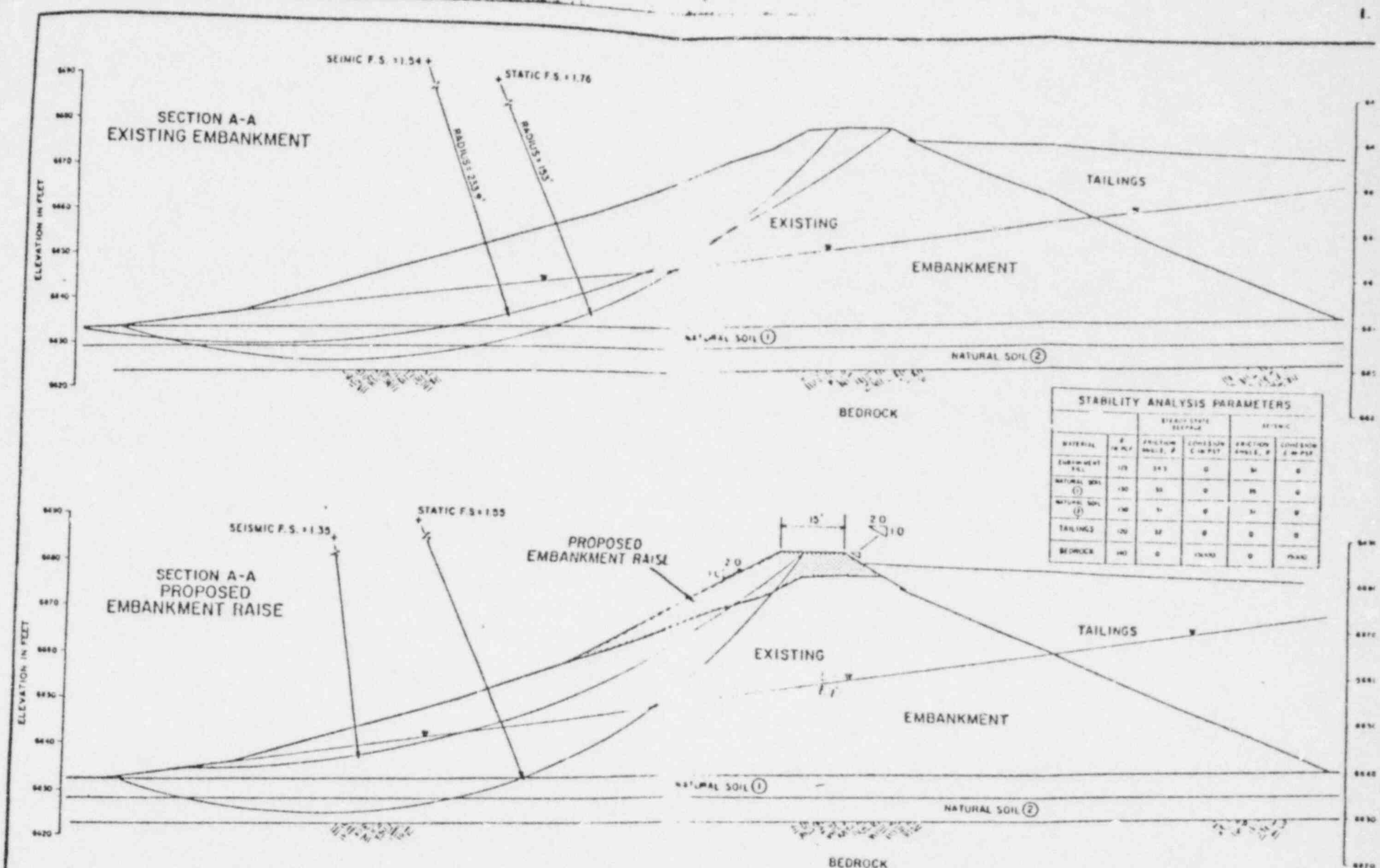
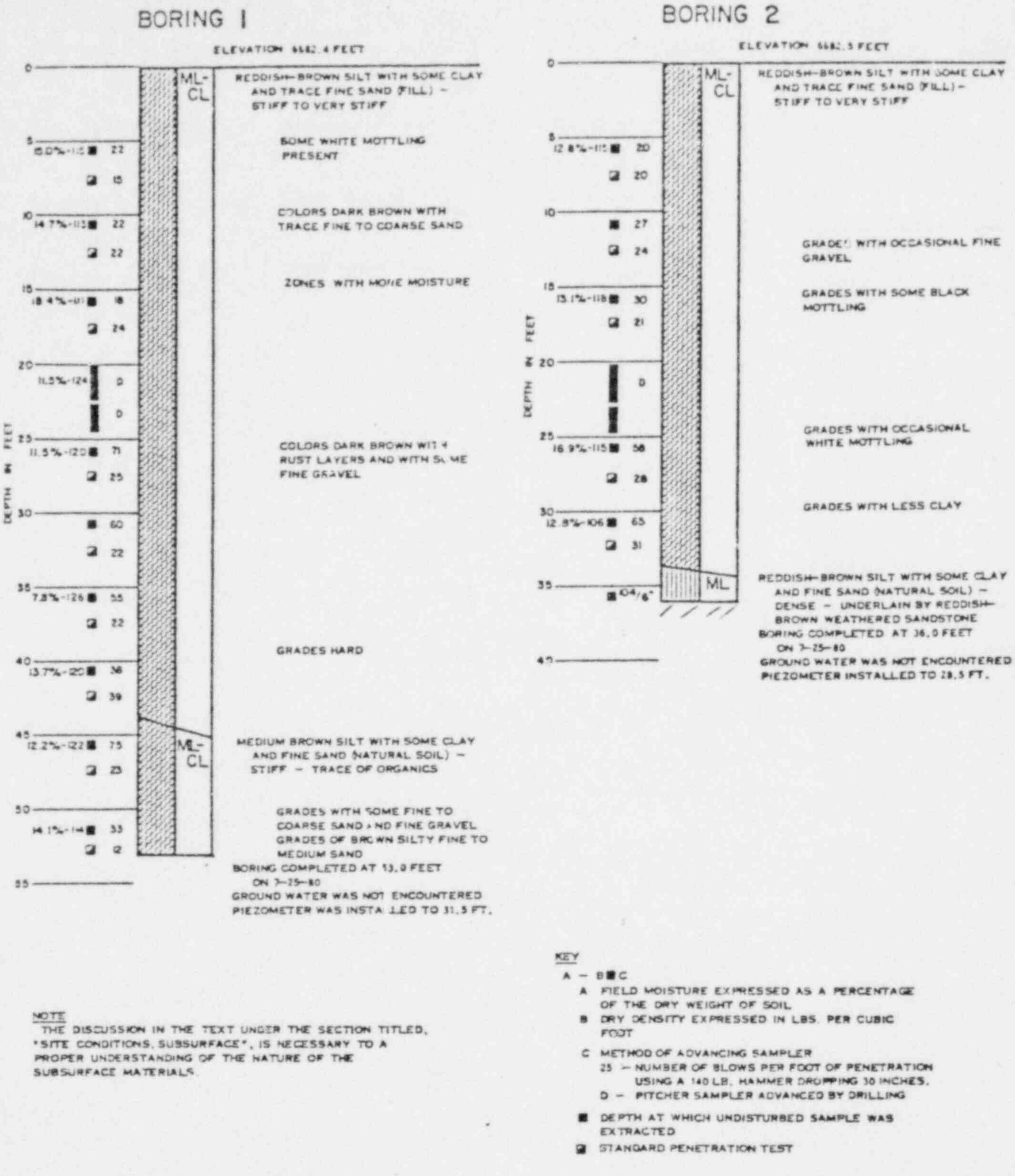


Fig. 2.2 Critical Embankment Section Analyzed (Source: ref. 3)



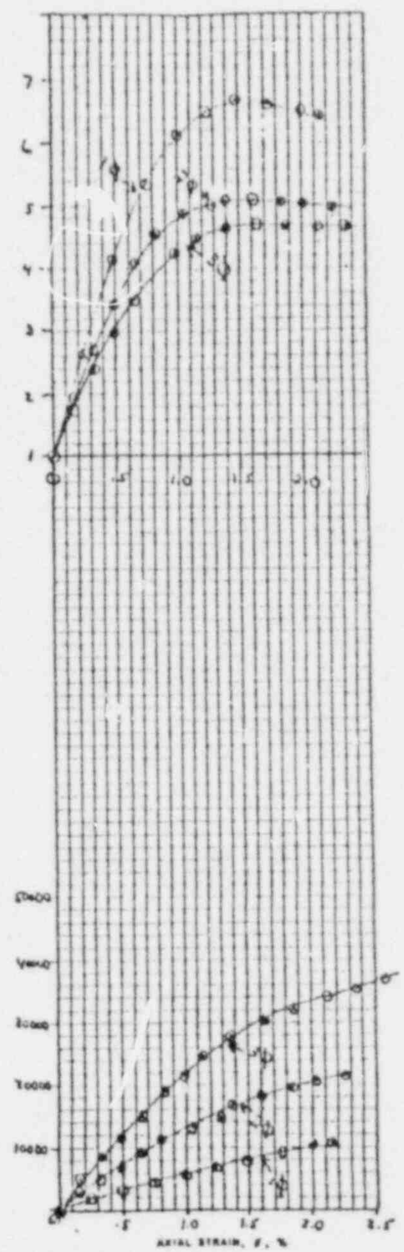
POOR ORIGINAL

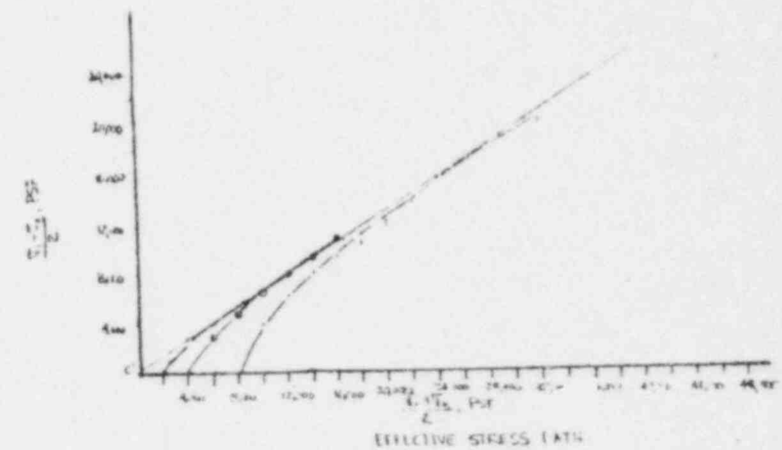
Fig. 2.3 Boring Logs
(Source: ref. 3)

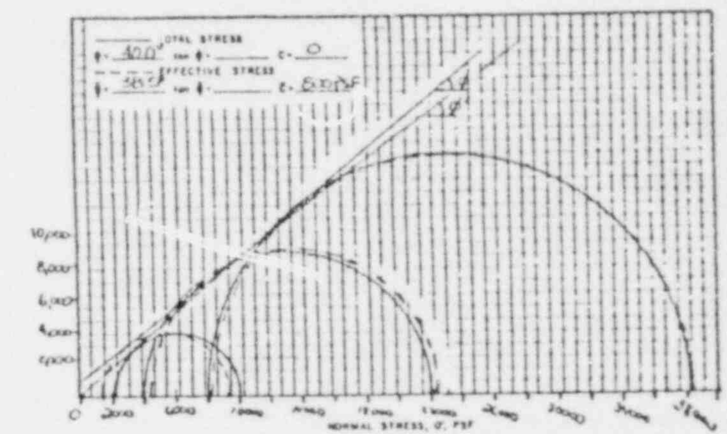
DAMES & MOORE

Multiphase

		①		②		③		◯	
Key		①		②		③			
Boring		1		1		1			
Sample		7		7		7			
Depth (feet)		22½'		22½'		22½'			
Initial	w, %	11.5							
	γ_d, pcf	123.8							
	e_o								
	S	.97		.97		.97			
Final	w, %					12.2			
	γ_d, pcf					127.6			
	e_f								
	S								
Back pressure (psi)		97.0		97.0		97.0			
Strain rate (inches/minute)		.005		.005		.005			
Stress condition		Peak $\sigma_1-\sigma_3$	Max $\bar\sigma_1/\bar\sigma_3$	Peak $\sigma_1-\sigma_3$	Max $\bar\sigma_1/\bar\sigma_3$	Peak $\sigma_1-\sigma_3$	Max $\bar\sigma_1/\bar\sigma_3$	Peak $\sigma_1-\sigma_3$	Max $\bar\sigma_1/\bar\sigma_3$
Total stress	ε, %		1.497		1.614		1.634		
	Time to fail (min)		19		20		20		
	σ_3, psf		2000		4000		8000		
	$\sigma_1-\sigma_3$		7978		18105		30396		
	σ_1, psf		9978		22105		38396		
	$\frac{1}{2}(\sigma_1-\sigma_3)$		4989		9053		15198		
	$\frac{1}{2}(\sigma_1+\sigma_3)$		5989		13053		23198		
	u, psf		590		-432		-216		
	$u/(\sigma_1-\sigma_3)$		.074		-.024		-.007		
Effective stress	ε, %		1.497		1.614		1.634		
	Time to fail (min)		19		20		20		
	$\bar\sigma_3$, psf		1410		4432		8216		
	$\bar\sigma_1-\bar\sigma_3$		7978		18105		30396		
	$\bar\sigma_1$, psf		9387		22537		38612		
	$\frac{1}{2}(\bar\sigma_1-\bar\sigma_3)$		4989		9053		15198		
	$\frac{1}{2}(\bar\sigma_1+\bar\sigma_3)$		5399		20321		23414		
	u, psf		590		-432		-216		
	$u/(\bar\sigma_1-\bar\sigma_3)$		.074		-.024		-.007		
	$\bar\sigma_1/\bar\sigma_3$		6.66		5.09		4.70		
Remarks									







MOHR ENVELOPE – MAXIMUM EFFECTIVE STRESS RATIO

TRIAXIAL COMPRESSION TEST REPORT

TYPE OF TEST TX-CU-PP-MQ

TYPE MATERIAL Reddish Brown Silt with some Clay

SAMPLE DESCRIPTION

CLASSIFICATION (CL/ML)

LIQUID LIMIT ____ PLASTIC LIMIT ____ SPECIFIC GRAVITY, G_s ____

PROJECT Rio Algom

LOCATION

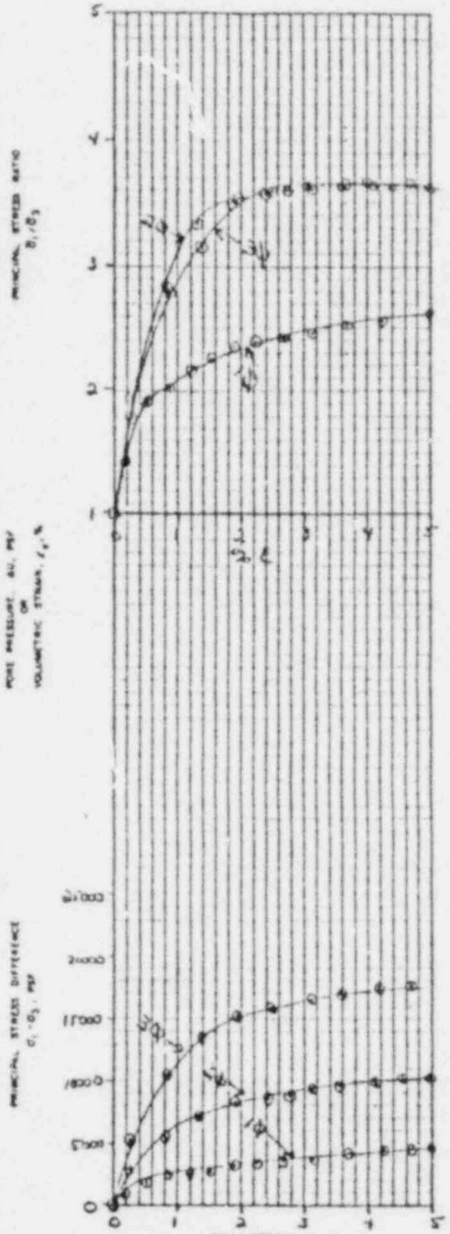
JOB NO. 07144-017 PREPARED BY RGS 7/29/80

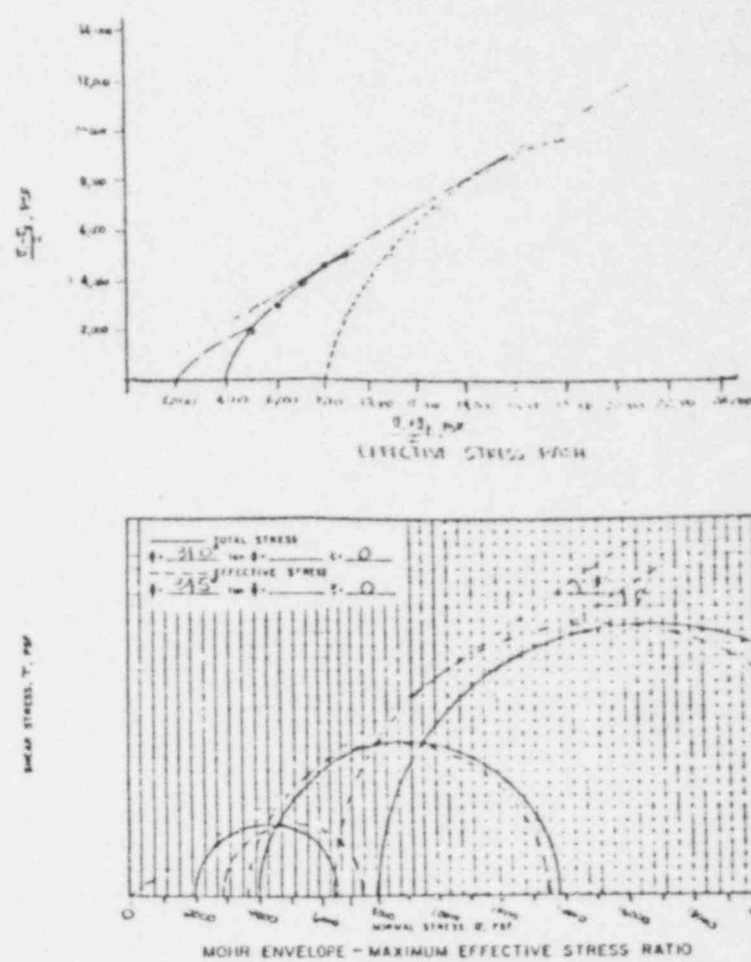
CHECKED BY

PLATE A-5

Fig. 2.4 Triaxial Test Results
(Source: ref. 3)

		①		②		③		◯	
KEY		Phases							
BORING		2		2		2			
SAMPLE		7		7		7			
DEPTH (FEET)		22½'		22½'		22½'			
INITIAL	w, %								
	γ_d, PCF								
	e_i								
	B	0.96		0.96		0.96			
FINAL	w, %								
	γ_d, PCF								
	e_f								
	S								
BACK PRESSURE (PSI)		78.0		78.0		78.0			
STRAIN RATE (INCHES / MINUTE)		0.002		0.002		0.002			
STRESS CONDITION		PEAK $\sigma_1-\sigma_3$	MAX $\bar{\sigma}_1/\bar{\sigma}_3$	PEAK $\sigma_1-\sigma_3$	MAX $\bar{\sigma}_1/\bar{\sigma}_3$	PEAK $\sigma_1-\sigma_3$	MAX $\bar{\sigma}_1/\bar{\sigma}_3$	PEAK $\sigma_1-\sigma_3$	MAX $\bar{\sigma}_1/\bar{\sigma}_3$
TOTAL STRESS	ϵ, %		5.00		4.12		4.19		
	TIME TO FAIL (MIN)		146		115		113		
	σ_3, PSF		2000		4000		8000		
	$\sigma_1-\sigma_3$		4632		9882		17358		
	σ_1, PSF		6632		13882		25358		
	$\frac{1}{2}(\sigma_1-\sigma_3)$		2316		4941		8679		
	$\frac{1}{2}(\sigma_1+\sigma_3)$		4316		8941		16679		
	u, PSF		-864		274		1498		
	A, $u/(\sigma_1-\sigma_3)$		?		.028		.086		
EFFECTIVE STRESS	ϵ, %		5.00		4.12		4.19		
	TIME TO FAIL (MIN)		146		115		113		
	$\bar{\sigma}_3$, PSF		2864		3726		6502		
	$\bar{\sigma}_1-\bar{\sigma}_3$		4632		9882		17358		
	$\bar{\sigma}_1$, PSF		7496		13608		23861		
	$\frac{1}{2}(\bar{\sigma}_1-\bar{\sigma}_3)$		1432		4941		8679		
	$\frac{1}{2}(\bar{\sigma}_1+\bar{\sigma}_3)$		5180		8667		151825		
	u, PSF		-864		274		1498		
	A, $u/(\bar{\sigma}_1-\bar{\sigma}_3)$		?		.028		.086		
	$\bar{\sigma}_1/\bar{\sigma}_3$		2.618		3.652		3.67		
REMARKS									





TRIAXIAL COMPRESSION TEST REPORT

TYPE OF TEST Tx-CU-PP-MA

TYPE MATERIAL Reddish Brown Silt with some Clay

SAMPLE DESCRIPTION

CLASSIFICATION (CL/ML)

LIQUID LIMIT ____ PLASTIC LIMIT ____ SPECIFIC GRAVITY, G_s ____

PROJECT Rio Algom

LOCATION

JOB NO 07144-017 PREPARED BY RCS , 8/ /80

CHECKED BY

PLATE A-6

Fig. 2.5 Triaxial Test Results (Source: ref. 3)

POOR ORIGINAL

APPENDIX

HYDROLOGIC COMPUTATIONS

Probable Maximum General Storm

1. Probable maximum 6-hour point rainfall for the Rio Algom area is 5 in (Fig 17 P50)

2. $\frac{\text{Area rainfall}}{\text{Point rainfall}} = 1.0$ Assumed

3. Accumulative 24-hour probable maximum rainfall at 1-hour intervals for Zone B is shown in Table 1 Col.(4)
   Col. (3) from Fig 18 P.51 and table 1 P.52.
   Col. (4) = Col. (3) x 5

4. Arrange order of Col. (6) is from P. 88

5. For assumption B the reduction factor from Fig 22 P. 56 is 1.75

6. Col (9) = Col (8)/1.75

POOR ORIGINAL

Table 1 – Probable maximum general storm

1 Duration hours	2 Assigned order	3 Ratio to amount of 6-hour	4 Accumulated 18-hour PMHR (in)	5 1-hour incremental rainfall (in)	6 Arranged order	7 Arranged hourly incremental rainfall (in)	8 Cumulative Design rainfall (in)	9 Reduced rainfall for assumption B (in)
1	1	0.20	1.50	1.50	6	0.55	0.55	0.31
2	2	0.48	2.40	0.90	5	0.60	1.15	0.66
3	3	0.63	3.15	0.75	4	0.70	1.85	1.06
4	4	0.77	3.85	0.70	3	0.75	2.60	1.49
5	5	0.89	4.45	0.60	2	0.90	3.50	2.00
6	6	1.00	5.0	0.55	1	1.50	5.0	2.86
7	1	1.09	5.45	0.45	6	0.45	5.45	3.11
8	2	1.18	5.90	0.45	4	0.45	5.90	3.37
9	3	1.27	6.35	0.45	3	0.45	6.35	3.63
10	4	1.36	6.80	0.45	1	0.45	6.80	3.89
11	5	1.45	7.25	0.45	2	0.40	7.20	4.11
12	6	1.53	7.65	0.40	5	0.45	7.65	4.37
13	1	1.60	8.00	0.35	1	0.35	8.00	4.57
14	2	1.66	8.30	0.30	2	0.30	8.30	4.74
15	3	1.71	8.55	0.25	5	0.30	8.60	4.91
16	4	1.77	8.85	0.30	3	0.25	8.85	5.06
17	5	1.82	9.10	0.25	4	0.25	9.10	5.20
18	6	1.87	9.35	0.25	6	0.25	9.35	5.34
19	1	1.91	9.55	0.20	1	0.20	9.55	5.46
20	2	1.95	9.75	0.20	2	0.20	9.75	5.57
21	3	1.99	9.95	0.20	3	0.20	9.95	5.69
22	4	2.03	10.15	0.20	4	0.20	10.15	5.80
23	5	2.07	10.35	0.20	5	0.20	10.35	5.91
24	6	2.10	10.50	0.15	6	0.15	10.15	5.80

Probable Maximum Thunderstorm

1. Probable maximum thunderstorm 1-hour point precipitation is 7 in (From Fig 20) Zone II

2. $\frac{\text{Area rainfall}}{\text{Point rainfall}} = 1.0$ Assumed

3. Percentages of 1-hour rainfall from Table 2 P. 52
The values for Zones I and III are used here

4. Accumulative rainfall
Col. (4) = Col. (3) x 7 /100

5. Arrangement of the 15-minute incremental rainfall with order sequence given in Table A-8 App A P. 87

POOR ORIGINAL

PADMASTER® / Made in U.S.A.

Table 2 - Probable maximum thunderstorm

1 Assigned order	2 Duration (min)	3 Percent of 1-hour value	4 Accumulative rainfall (in)	5 15-minute incremental amount (in)	6 order of Arrangement	7 Arranged 15-minute incremental rainfall (in)	8 incremental design rainfall P (in)	9 Reduced rainfall for assumption B (in)
1	15	48	3.36	3.36	4	0.84	0.84	0.26
2	30		4.97	1.61	3	1.19	2.03	0.63
3	45		6.16	1.19	2	1.61	3.64	1.14
4	60	100	7.00	0.84	1	3.36	7.00	2.19
5	75	110	7.70	0.70	5	0.70	7.70	2.41
6	90	117	8.19	0.49	6	0.49	8.19	2.56
7	105	122	8.54	0.35	7	0.35	8.54	2.67
8	120	126	8.82	0.28	8	0.28	8.82	2.76
9	135	129	9.03	0.21	9	0.21	9.03	2.82
10	150	131.5	9.21	0.18	10	0.18	9.21	2.88
11	165	133	9.31	0.10	11	0.10	9.31	2.91
12	180	134	9.38	0.07	12	0.07	9.38	2.93