Appendix C .....Interior Site Drainage Calculations

Appendix C.1:	Calculation C.1 PMP Calculation - Local Storm
Appendix C.2	Calculation C.2 Design Flowrates and Erosion Protection
Appendix C.3:	Interior Diversion Channel
Appendix C.4:	Interior Discharge Channel
Appendix C.5:	Amendment to Pond 3 Erosion Protection & Erosion Protection for the Area North of Pond 1 Design Report by Maxim Technologies July 28, 2003
Appendix C.6:	Amendments to Erosion Protection Drawings for Pond 3 Area, Task 3 Design Report by Maxim Technologies October 13, 2006

Appendix C.1 .....Calculation C.1 PMP Calculation – Local Storm

Calculation C.1
Rio Algom Mining LLC
Ambrosia Lake, New Mexico
Arroyo Del Puerto
Interior Site Drainage Calculations
PMP Calculation - Local Storm

1. PMP Calculation - Local Storm

Previous calculation performed in the design report titled:
Reclamation Plan for the Disposal of Pond Sediments and ancillary Materials
Tailings Cell 2 Expansion
Rio Algom Mining LLC, Ambrosia Lake, New Mexico
31-May-07

From that Design Report: 1hr-1mi2 PMP (Local) = 9.6 inches

2. Recalculation

See Figure 1.1 - Regions Covered by Generalized PMP Studies (see Sheet 2 of 15)
Reference: Reference: Hydrometeorological Report No. 55A (HMR 55A)
U.S. Dept of Commerce
NOAA, June 1988

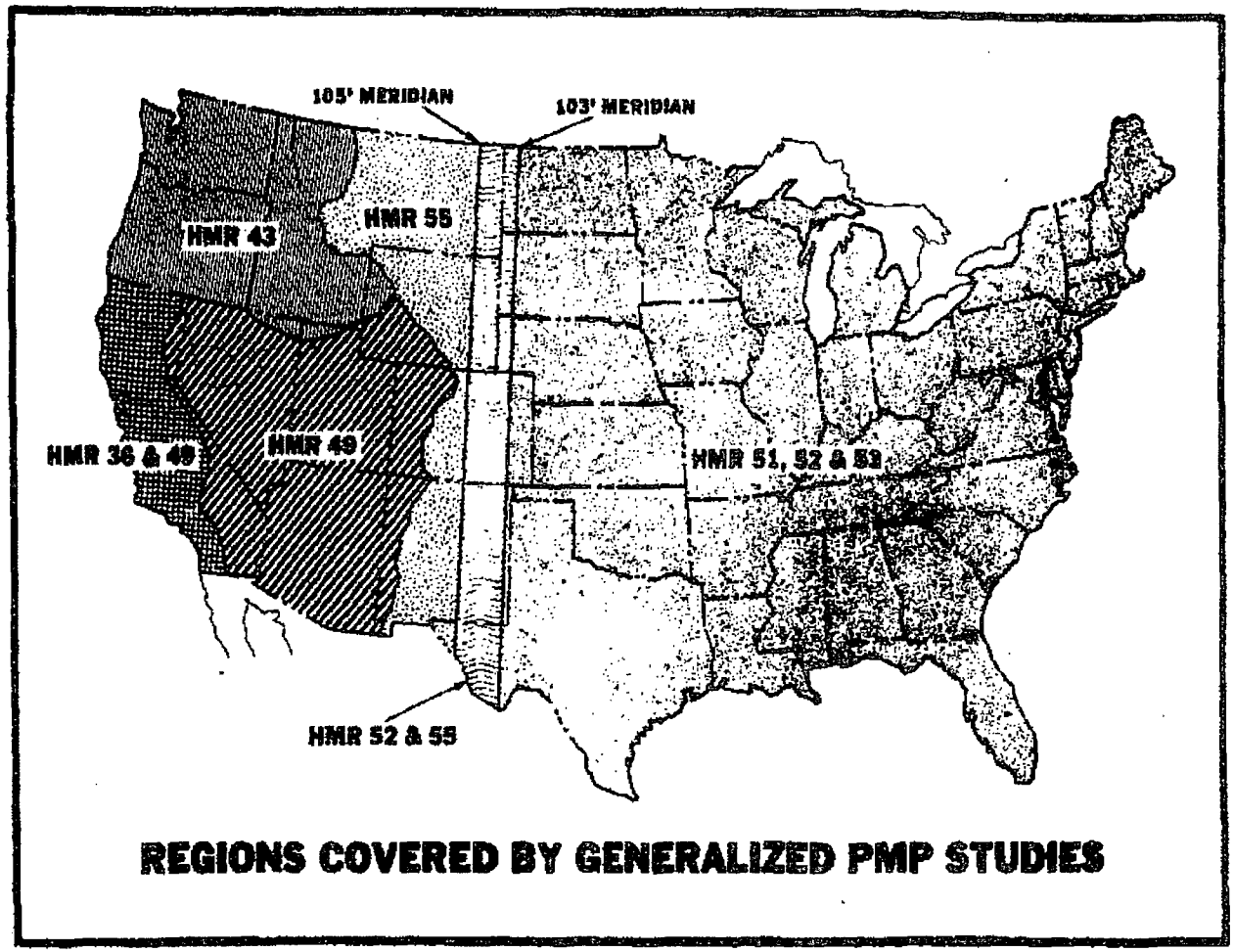

Figure 1.1.—Regions of the conterminous United States for which PMP estimates are provided in indicated Hydrometeorological Reports. See Appendix A for description of geographical region covered and scope of each report.

See the following table indicating the Stepwise Procedure Steps for a Local Storm taken from Section 14.3 of HMR 55A, (as shown on Sheets 4 thru 6 of 15)

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO EROSION PROTECTION MEASURES
HMR 55A[1] PMP LOCAL STORM CALCULATION
STEPWISE PROCEDURE, LOCAL STORM

Procedure Step #	Item Description	Units	Reference	Results
1	Index 1-hr 1-mi^2 PMP Estimate at 5000 ft elevation	inches	See Plate VI c (HMR 55A)(Sheet 7 of 15)	10.6
2	Maximum persisting 12-hr 1000-mb dew points for July	^oF	See Fig 4.11 (Sheet 8 of 15) (HMR 55A)	76.6
	Mean elevation of drainage area	feet	See Design Report Drawing "Sheet 2"	7000
	Adjustment for Mean Elevation of Drainage	%	See Fig. 14.3 (Sheet 5 of 15) (HMR 55A)	90%
3	Index 1-hr 1-mi2 PMP Estimate at 7000 ft elevation of Site	inches	Step #1 times Step #2	9.60
4	Depth-Duration Curve for 1 mi^2	inches	From Table 12.4 (Sheet 9 of 15) (HMR 55A)	See Worksheet Tab "PMP Indexes-55", (calc. Sheets 12 thru 13 of 15)
5	Areal Reduction Factors	%	From Table 12.12 (Sheet 10 of 15) (HMR 55A)	100%
6	PMP Estimates for Basin	inches	Step #4 times Step #5	No reduction to Step 4 values
7	Incremental PMP Amounts	see temporal sequence	See Tables 12.5 & 12.6 (Sheet 11 of 15) (HMR 55A)	See Worksheet Tab "Incremental PMP" for chronological distributions, (Calc Sheets 14 thru 15 of 15)

[1] Hydrometeorological Report No. 55A, U.S. Dept. of Commerce, NOAA, June 1988

14.3 Stepwise Procedure, Local Storm

1. Index 1-hr 1-mi^2 PMP estimate at 5,000-ft elevation

Locate the drainage in Plate VI a-c, and determine the drainage average index 1-mi^2 1-hr PMP in inches at 5,000 ft. This is readily accomplished by eye because of the smooth gradient, and linear interpolation is assumed to apply.

2. Adjustment for mean elevation of drainage

Determine the mean drainage elevation to the nearest 100 ft. An adjustment needs to be determined and applied to the depth from step 1 if this elevation differs from 5,000 ft by more than 1,000 ft. If the mean terrain elevation of the drainage is greater than 6,000 ft or less than 4,000 ft, the correct vertical adjustment factor can be obtained by reference to figure 14.3. This is a nomogram of vertical elevation adjustments as discussed in section 12.3.2.4. To use the nomogram, enter the horizontal scale (abscissa) at the maximum persisting 12-hr 1000-mb dew point obtained from figure 4.11 for the location of the drainage. Move vertically in the figure to intersect the mean elevation of the drainage (to the nearest 100 ft) and read off the adjustment factor on the vertical scale (ordinate).

As an example of this determination, take a drainage that has a mean elevation of 7,800 ft and a maximum persisting 12-hr dew point of 70°F. Entering figure 14.3 at 70° on the abscissa and moving vertically to 7,800 ft, an adjustment factor of 0.82 is read from the ordinate.

3. Index 1-hr 1-mi^2 PMP estimate at mean elevation of drainage

Multiply the adjustment factor determined in step 2, if needed, by the index 1-mi^2 1-hr depth from step 1 to obtain a representative surface adjusted index PMP estimate.

4. Depth-duration curve for 1 mi^2

Refer to table 12.4 to obtain the 1-mi^2 factors for durations up to 6 hr. Multiply these factors by the estimate from step 3. These can be plotted on linear graph paper and a smooth curve drawn to obtain intermediate durational amounts if these are needed for the 1-mi^2 area.

5. Areal reduction factors

To obtain areal reduction factors, use the relations provided in figure 12.20. Find the drainage area on the abscissa and read the corresponding reduction factors as percent of the 1-mi^2 PMP.

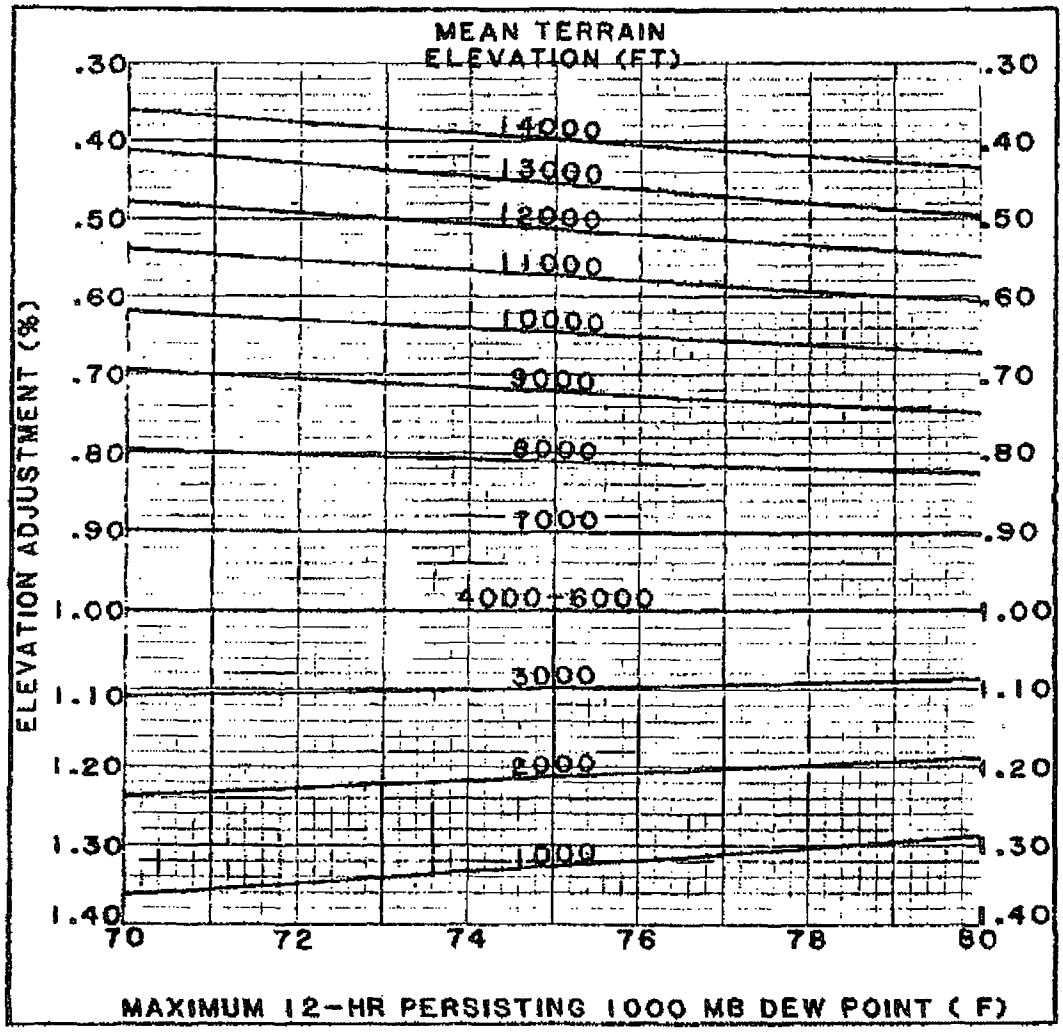

Figure 14.3.—Adjustment for elevation for local-storm PMP based on procedures developed in the report and maximum persisting 12-hr 1000-mb dew point (F).

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6. PMP estimates for basin

Multiply percentages of step 5 by the index PMP amounts from step 4. These values should be plotted on linear graph paper and a smooth curve drawn through the points. Values for the intermediate durations may be determined from this curve.

7. Incremental PMP amounts

If needed, local-storm PMP incremental amounts obtained through subtraction of adjacent amounts in step 6 may be arranged in temporal sequences recommended in tables 12.5 and 12.6.

No example is believed necessary for local-storm PMP determination, as the adjustment for elevation is the only complex element in the determination, and an example calculation of this factor is given in step 2.

15. FUTURE STUDIES

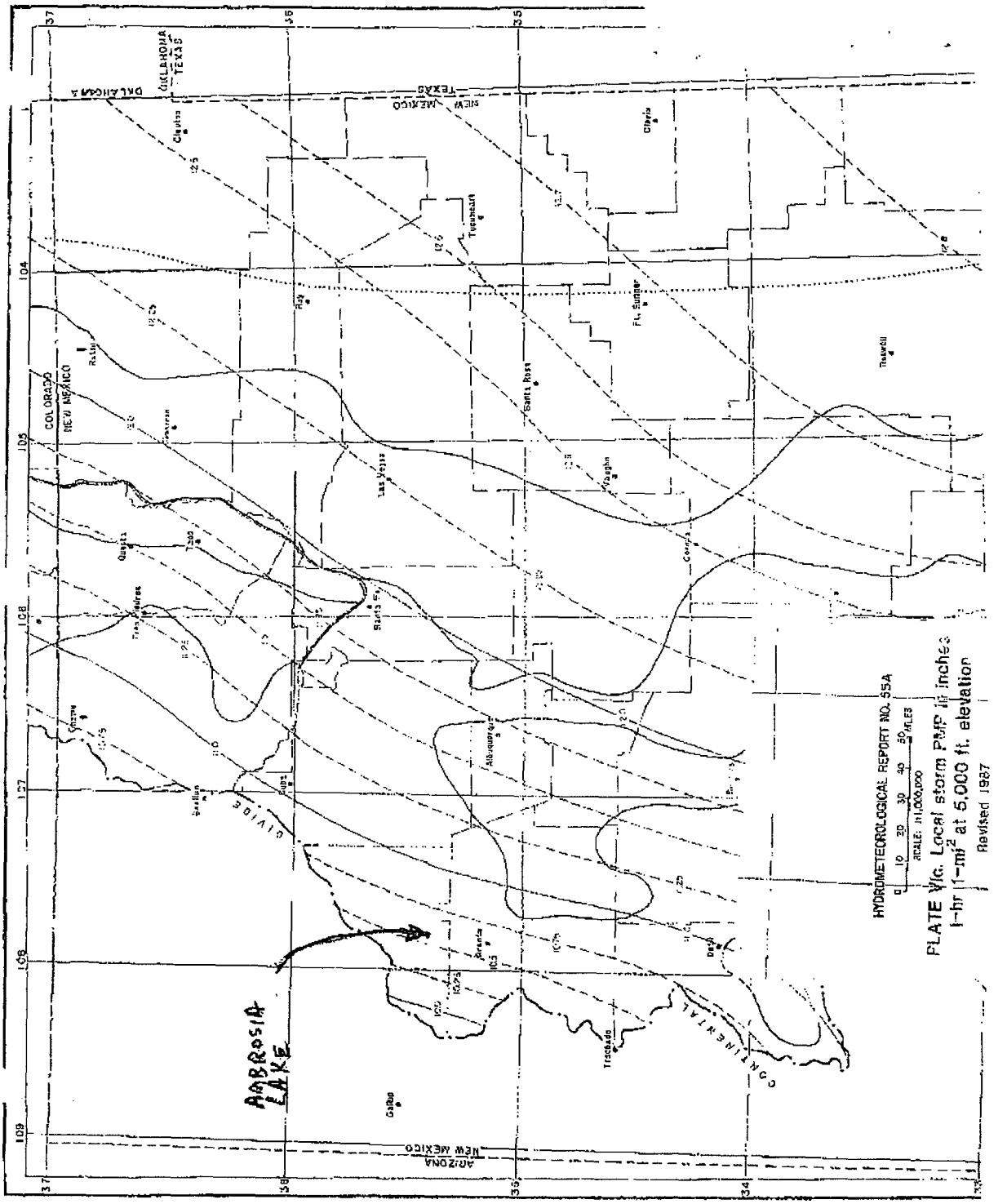
There are several problems involved in the development of design estimates that should be resolved. The purpose of this chapter is to briefly discuss these needed future studies.

15.1 Seasonal Variation

In the present study, it has been possible to develop only all-season PMP estimates. Although no attempt has been made to define the season of occurrence, some observations are possible. In the northern portion of the study region among the more important storms are Gibson Dam, MT (75), June 6-8, 1964; Warrick, MT (10), June 6-8, 1906; Springbrook, MT (32), June 17-21, 1921; and Savageton, WY (38), September 27-October 1, 1923. Through the central portion of the study region, Cherry Creek (47) and Hale (101), CO, May 30-31, 1935, Plum Creek (76), CO, June 13-20, 1965, Big Elk Meadow (77), CO May 4-8, 1969, and Big Thompson, July 31, 1976 are important in determining PMP estimates. In the extreme southern part of the study region, tropical storms or their remnants will be the causative mechanism for the longer duration PMP event. Such storms as Rancho Grande (60), NM, August 26-September 1, 1942, and Meek (27), NM, September 15-17, 1919 are typical of these events. Shorter duration storms similar to that at White Sands, NM, August 19, 1978 are important in this region. These storm dates suggest that the all-season PMP event will occur from early summer through fall. In those portions of the study region where snowmelt can be a critical factor, the probable maximum flood (PMF) may be the result of the lesser magnitude spring PMP event and accompanying snowmelt. The definition of the seasonal variation of PMP is, therefore, a necessary addition to the present report.

15.2 Permissible Snowpack With PMP and Snowmelt Criteria

To adequately evaluate the spring PMF, two additional factors are required. The first is an evaluation of the snowpack that could exist prior to the PMP event. The question to be answered is the depth and extent of the snow cover. Could, for example, the probable maximum snowpack (PMSP) occur just prior to the


HYDROMETEOROLOGICAL REPORT NO. 55A
PLATE VIa. Local storm PMP in inches
1-hr 1-mi² at 5,000 ft. elevation
Revised 1987
ABROSIA LAKE
COLORADO
NEW MEXICO
OKLAHOMA
TEXAS
ARIZONA
DIVIDE
CONTINENTAL

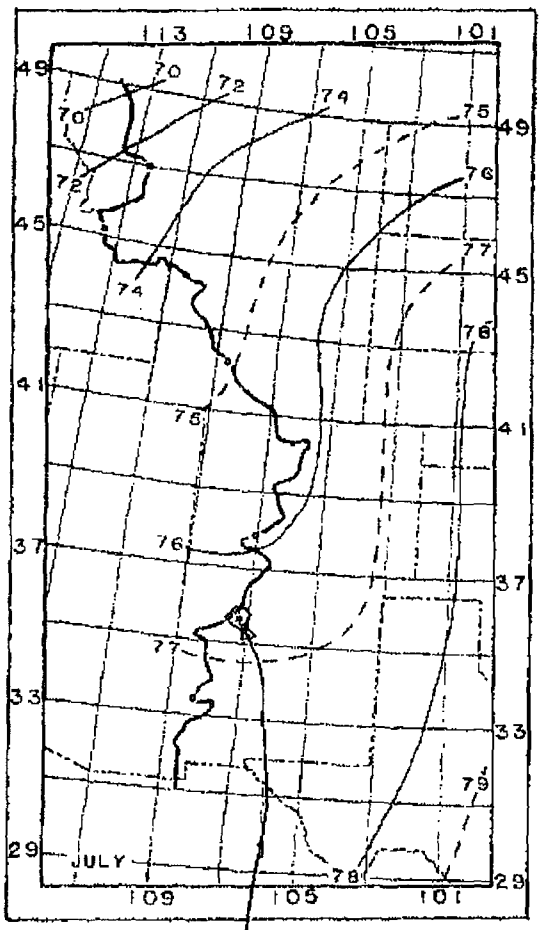


Figure 4.11.--Maximum persisting 12-hr 1000-mb dew points (°F) for July.

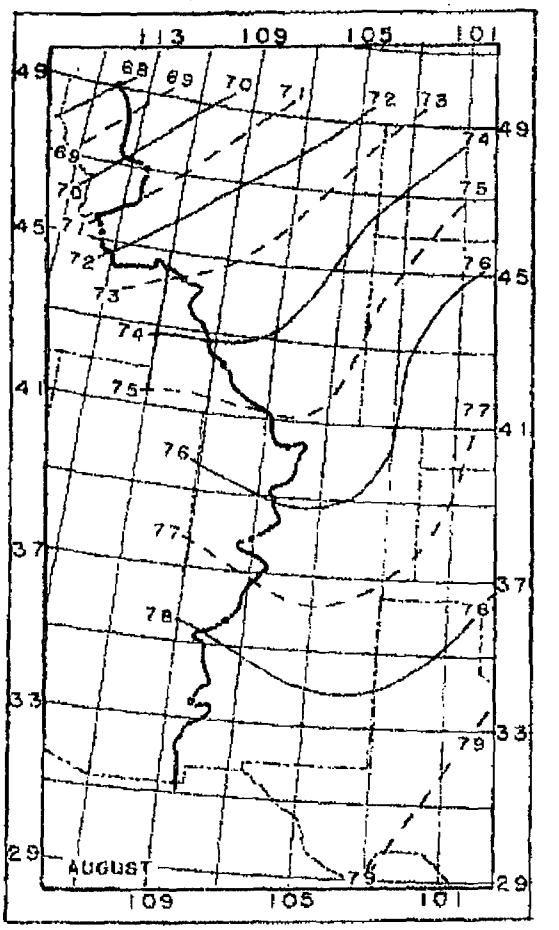


Figure 4.12.--Maximum persisting 12-hr 1000-mb dew points (°F) for August.

AMEROSIA LAKE

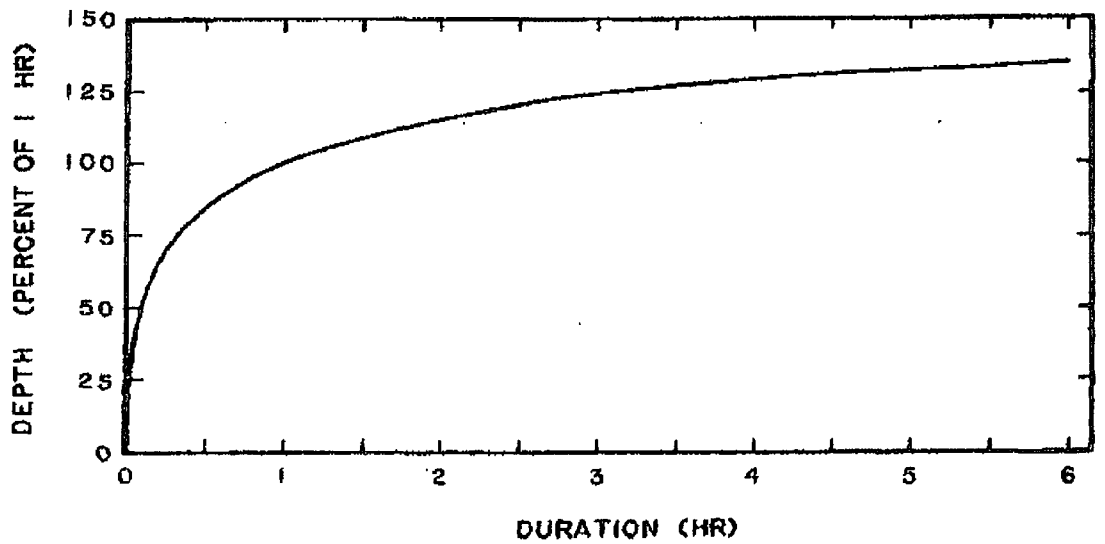


Figure 12.10.—Depth-duration curve for 6-/1-hr ratio of 1.35.

Table 12.4.—Percent of 1-hr local-storm PMP for selected durations for 6-/1-hr ratio of 1.35 (HMR No. 49)

Duration (hr)	Percent of 1 hr
1/4	.68
1/2	.86
3/4	.94
1	1.00
2	1.16
3	1.23
4	1.28
5	1.32
6	1.35

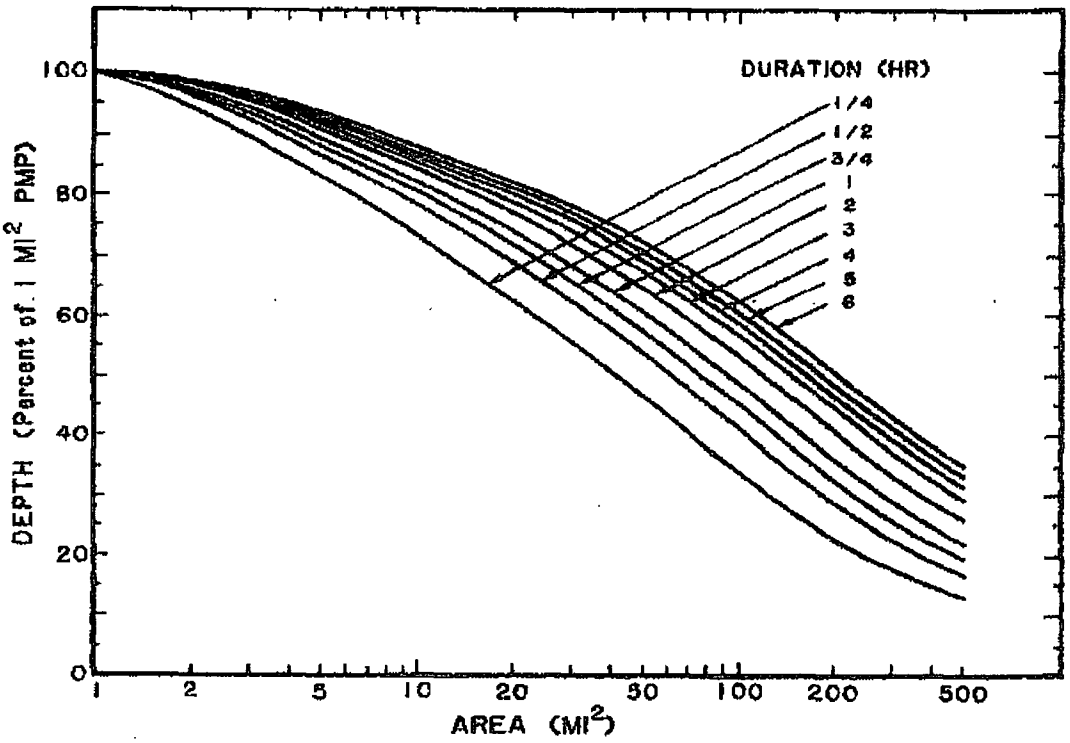


Figure 12.12.—Depth-area relations adopted for local-storm PMP in the CD-103 region (Hansen et al. 1977).

the storm in figure 12.9 that was constructed from a written account of the storm. The sequence of the hourly incremental rainfall for the storm shows that the storm decreased each succeeding hour after the first hour. However, meaningful conclusions cannot be drawn from this one example.

To supplement the lack of available data in the CD-103 region, data from HMR No. 49 was utilized. These data are presented in table 12.5 and include time distribution measurements from 6-hr storms, as utilized by the U.S. Weather Bureau (1947) and by the U.S. Army Corps of Engineers (1965). The choice of which of the two to apply is left to the user, as one sequence may be more critical than the other in a specific case.

There were no data available for the extreme local storms in the CD-103 region from which to determine the sequence of 15-min increments in the 1-hr storm. The 15-min incremental sequence taken from HMR No. 49 is, therefore, recommended. This incremental sequence appears in table 12.6. It is the result of percentages of total rainfall for thunderstorm rainfall determined by the U.S. Weather Bureau (1947).

Table 12.5.—Recommended chronological distribution of 1-hr incremental rainfall amounts for 6-hr local-storm PMP (Hansen et al. 1977)

Increment	Sequence position	
	HMR No. 5*	EM1110-2-1411#
Largest hourly increment	third	fourth
Second largest	fourth	third
Third largest	second	fifth
Fourth largest	fifth	second
Fifth largest	last	last
Least	first	first

* U.S. Weather Bureau 1947
\# U.S. Corps of Engineers, Standard Project Flood Determinations, March 1952, revised March 1965

12.8 Seasonal Distribution

A brief analysis was undertaken to determine the season of occurrence of the local storm in the CD-103 region. The analysis took the form of recording the maximum 1-hr event at recorder stations throughout the CD-103 region (sec. 12.5.1). The period of record totaled 31 years (1948-78); however, many stations had fewer years than this maximum period of record. It was decided to use only stations that had 20 or more years of precipitation record. This removed stations whose data may not have been representative of the true conditions at the station because of an insufficient period of record.

Table 12.7 shows the seasonal distribution of the maximum 1-hr events at selected stations in the CD-103 region. Most of the maxima occur in the summer months of June, July, and August. These months represent the months of greatest potential moisture influx into the region, as shown by the maximum persisting 12-hr 1000-mb dew-point charts of chapter 4. The months of May and September show fewer recorded maximum 1-hr events, while April and October show the least. No other months in the year produced maximum 1-hr events of record for this period. These results are not unlike those found in HMR No. 49.

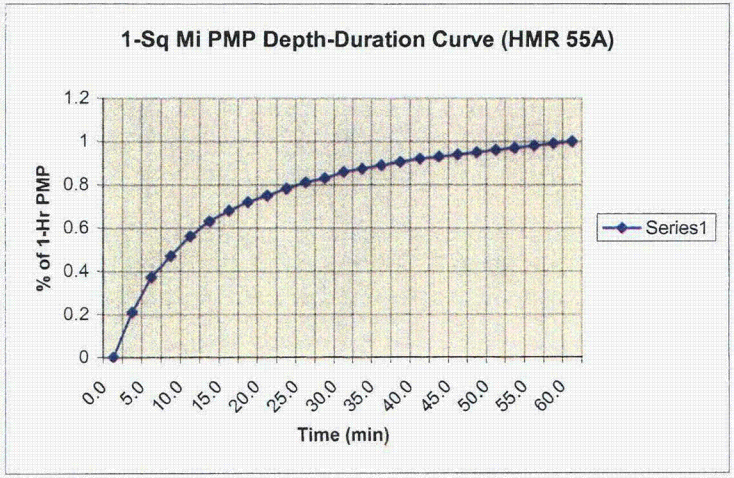
Table 12.6.—Recommended chronological distribution of 15-min incremental rainfall amounts for 1-hr local-storm PMP (Hansen et al. 1977)

Increment	Sequence position
Largest 15-min increment	first
Second largest	second
Third largest	third
Fourth Largest	fourth

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO

1-HOUR PMP DEPTH-DURATION CURVE

Duration (min)	Percent of 1-hr PMP[1]	1-hr 1-mi^2 PMP (inches)	PMP 1-hr Indexes up to 6 hr
0.0	0	9.6	0.00
2.5	21.0%	9.6	2.02
5.0	37.0%	9.6	3.55
7.5	47.0%	9.6	4.51
10.0	56.0%	9.6	5.38
12.5	63.0%	9.6	6.05
15.0	68.0%	9.6	6.53
17.5	72.0%	9.6	6.91
20.0	75.0%	9.6	7.20
22.5	78.0%	9.6	7.49
25.0	81.0%	9.6	7.78
27.5	83.0%	9.6	7.97
30.0	86.0%	9.6	8.26
32.5	87.5%	9.6	8.40
35.0	89.0%	9.6	8.54
37.5	90.5%	9.6	8.69
40.0	92.0%	9.6	8.83
42.5	93.0%	9.6	8.93
45.0	94.0%	9.6	9.02
47.5	95.0%	9.6	9.12
50.0	96.0%	9.6	9.22
52.5	97.0%	9.6	9.31
55.0	98.0%	9.6	9.41
57.5	99.0%	9.6	9.50
60.0	100%	9.6	9.60

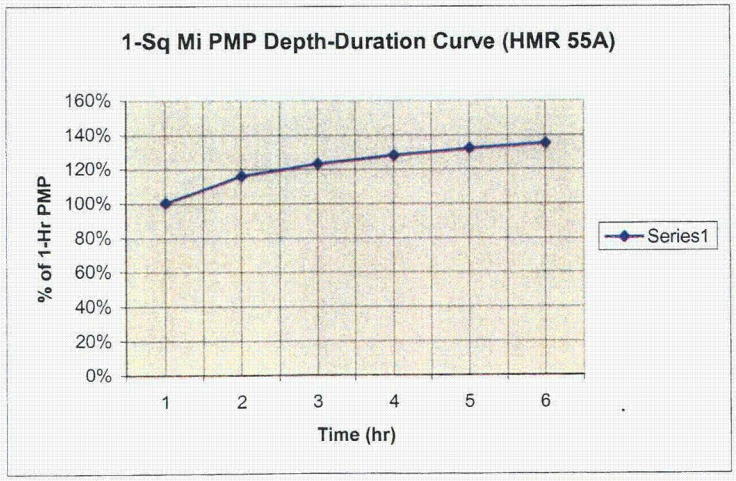


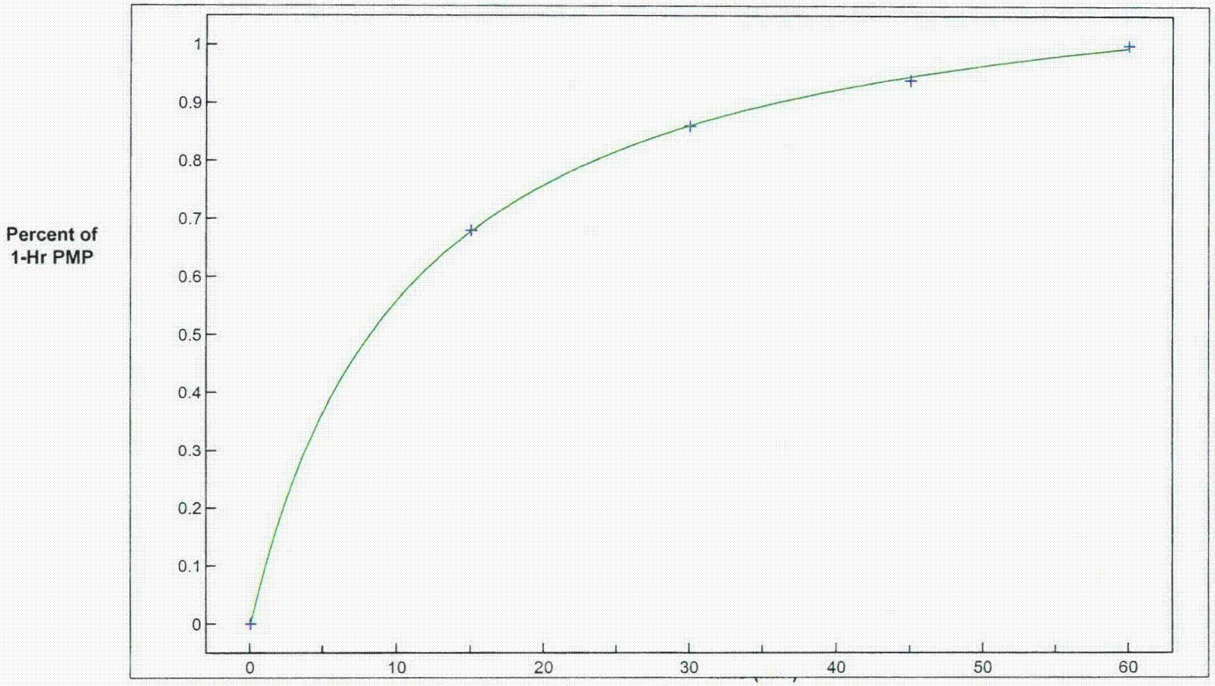
[1] Interpolated Values

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO

6-HOUR PMP DEPTH-DURATION CURVE

Duration (hr)	Percent of 1-hr PMP	1-hr 1-mi^2 PMP (inches)	PMP 1-hr Indexes up to 6 hr
1	100%	9.6	9.60
2	116%	9.6	11.14
3	123%	9.6	11.81
4	128%	9.6	12.29
5	132%	9.6	12.67
6	135%	9.6	12.96





1-Sq Mi PMP Depth-Duration Curve Up to 60 Minutes
Interpreted from HMR 55A Data from Table 12.4

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO

CHRONOLOGICAL DISTRIBUTION OF 15-MINUTE INCREMENTAL RAINFALL AMOUNTS FOR 1-HOUR LOCAL-STORM PMP

Duration (hr)	Percent of 1-hr PMP	1-hr 1-mi^2 PMP (inches)	PMP 1-hr Indexes	Incremental PMP Amounts (inches)	Size of Increment	HMR 5 & 49[1]	
0.00	0.00	9.60	0.00	0.00		Sequence Position	Increment Amount
15.00	0.68	9.60	6.53	6.53	Largest 15-min increment	First	6.53
30.00	0.86	9.60	8.26	1.73	2nd largest	Second	1.73
45.00	0.94	9.60	9.02	0.77	Third Largest	Third	0.77
60.00	1.00	9.60	9.60	0.58	Fourth Largest	Fourth	0.58

[1] HMR 5 (U.S. Weather Bureau, 1947); and HMR 49 (U.S. Weather Bureau, 1977, Reprinted 1984)

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
CHRONOLOGICAL DISTRIBUTION OF 1-HOUR INCREMENTAL RAINFALL AMOUNTS FOR 6-HOUR LOCAL-STORM PMP

Duration (hr)	Percent of 1-hr PMP	1-hr 1-mi^2 PMP (inches)	PMP 1-hr Indexes up to 6 hr	Incremental PMP Amounts (inches)	Size of Increment	HMR 5 & 49[1]		EM1110-2-1411[2]	
0.00	0.00	9.60	0.00	0.00		Sequence Position	Increment Amount	Sequence Position	Increment Amount
1.00	1.00	9.60	9.60	9.60	Largest hourly increment	Third	0.67	Fourth	0.48
2.00	1.16	9.60	11.14	1.54	2nd largest	Fourth	0.48	Third	0.67
3.00	1.23	9.60	11.81	0.67	Third largest	Second	1.54	Fifth	0.38
4.00	1.28	9.60	12.29	0.48	Fourth largest	Fifth	0.38	Second	1.54
5.00	1.32	9.60	12.67	0.38	Fifth largest	Last	0.29	Last	0.29
6.00	1.35	9.60	12.96	0.29	Least	First	9.60	First	9.60

[1] HMR 5 (U.S. Weather Bureau, 1947); and HMR 49 (U.S. Weather Bureau, 1977, Reprinted 1984)
[2] EM1110-2-1411 (U.S. Corps of Engineers, Standard Project Flood Determinations, March 1952, revised March 1965

Appendix C.2 ..... Calculation C.2 Design Flowrates and Erosion Protection

Calculation C.2
Rio Algom Mining LLC
Ambrosia Lake, New Mexico
Arroyo Del Puerto
Interior Site Drainage Calculations
Design Flowrates and Erosion Protection

1. Determine Flowrates - Stepwise Procedure

See the following table indicating the Stepwise Procedure Steps to determine the design flowrates to be used to size erosion protection.

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO EROSION PROTECTION MEASURES
HYDROLOGY[1]

STEPWISE PROCEDURE

Procedure Step #	Item Description	Item Variable ID	Units	Results
1	Catchment Areas	A	acres	See Design Report "Drawiing Sheet 2"
2	Maximum Flow Length	L	feet	See Design Report "Drawiing Sheet 2"
3	Upper Elevation of Cell #2 Catchment Area	H_1	feet	See Design Report "Drawiing Sheet 2"
4	Lower Elevation of Cell #2 Catchment Area	H_2	feet	See Design Report "Drawiing Sheet 2"
5	Slope of Cell #2 Catchment Area	S	ft/ft	$(H_1 - H_2)/L$
6	Time of Concentration	T_c	min	NUREG/CR-4620, Equation 4.44
7	% PMP Values from Depth Duration Curve (HMR 55A)	% PMP	%	See curve developed in PMP Local Storm Calculation C.1 and shown again here in Worksheet Tab "PMP Indexes-55" (calc. Sheets 3 thru 4 of 27)
8	PMP Rainfall Depth	D	inches	% PMP x (PMP)
9	Rainfall Intensity	*i*	in/hr	$60D/T_c$
10	Maximum Design Flowrate	Q	cfs	Rational Method: Q = CiA
11	Runoff Coefficient	C	nt	(NUREG/CR-4620)
12	Unit Width Maximum Design Flowrate	q	cfs	$q = CiA_w = L \times (1 \text{ ft width})$

[1] NUREG/CR-4620, Methodologies for Evaluating Long-Term Stabilization Designs of Uranium Mill Tailings Impoundments, June

2. Design Flowrate summary results

See Worksheet Tab "Design Flowrates" (calc. Sheets 5 thru 6 of 27) for summary of catchment areas and applicable design flowrates. Stationing for interior channel shown on Design Report "Drawing Sheet 4", as well as Design Report Plan & Profile Drawings Sheets 13 thru 16.

3. Catchment Area and Flowrate summary at Interior Channel Stationing

See Worksheet Tab "Flow Summary" (calc. Sheet 7 thru 8 of 27) for specific catchment area peak flowrates, overland and channel unit width peak flowrates, and channel peak flowrates at channel stationing.

4. Rock Sizing Methods - see Worksheet Tab "Summary - Rock Methods" (calc. Sheet 9 of 27)

5. Interior Drainage Channel Stationing and Elevation Summary -
see Worksheet Tab "Elevation Summary" (calc Sheets 10 thru 11 of 27)

6. Interior Drainage Channel Manning's Erosion Protection and Scour Depth specific calculations -
see Worksheet Tab "Mannings's Rock-Scour" (calc Sheets 12 thru 15 of 27) for rock size and scour depth derived from channel parameters based on Manning's equation, rock size based on the Abt and Johnson method, and scour depth based on the USACE Equilibrium Scour Depth method.

7. Interior Drainage Channel Rock Size and Placement Summary -
see Worksheet Tab "Rock Summary" (calc. Sheet 16 thru 17 of 27) for the rock size and scour depth derived from channel parameters based on Manning's equation, rock size based on the Abt and Johnson method, and scour depth based on the USACE Equilibrium Scour Depth method.

8. Additional Evaluation Using HEC-RAS -
An additional evaluation was performed of the entire interior drainage channel using HEC-RAS to ensure that the flow was subcritical and doublecheck the results of rock size and scour depth utilizing Manning's equation.
see Worksheet Tab "HEC-RAS Input" (calc. Sheet 18 thru 23 of 27) for the input data from the Interior Channel including channel parameters, channel elevations at stationing, and channel station numbering.
see Worksheet Tab "HC-RAS Output" (calc. Sheet 24 thru 25 of 27) for the HEC-RAS profile output results of the channel stationing.
see Worksheet Tab "HEC-RAS Rock Scour" (calc. Sheet 26 thru 27 of 27) ffor the rock size and scour depth derived from channel parameters from the HEC-RAS modeling, rock size based on both the Abt and Johnson method and the Safety Factors Method (peak shear stress),
and scour depth based on the USACE Equilibrium Scour Depth method.
The results based on the HEC-RAS model are more detailed and consistent then that performed by Manning's equation. The design based on Manning's is more than adequate to satisfy the results.

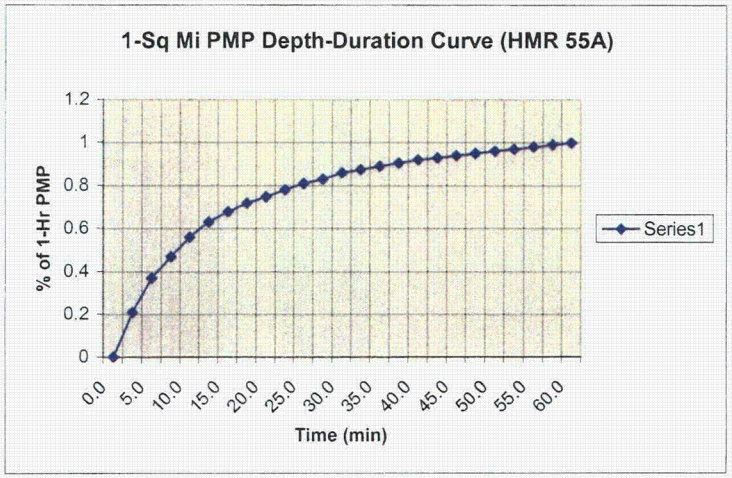
9 Conclusions:

a) The resulting rock sizes indicate D_{50}=7.8-inch rock size to be adequate for the interior drainage channel from Station 7+90 to 55+50 to protect from channel flows, overland flows entering laterally, flows entering from the diversion channel, and flows entering from the discharge channel.
b) Scour depths in the interior drainage channel are generally less than 3.0 feet.
c) The rock sizes in the area of the confluence with the exterior channel are D_{50}=12.0-inch with adequate depth of rock for scour and tied in with rock protection wrapped around the end of the nose of the exterior diversion embankment.
d) Tailings Ponds 4, 5, & 6 are protected adequately with D_{50} = 1.0-inch rock size.
e) The flows in the interior drainage channel maintain a subcritical flow regime with freeboards of 1.0 feet or less.

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO

1-HOUR PMP DEPTH-DURATION CURVE

Duration (min)	Percent of 1-hr PMP	1-hr 1-mi^2 PMP (inches)	PMP 1-hr Indexes up to 6 hr
0.0	0	9.6	0.00
2.5	21.0%	9.6	2.02
5.0	37.0%	9.6	3.55
7.5	47.0%	9.6	4.51
10.0	56.0%	9.6	5.38
12.5	63.0%	9.6	6.05
15.0	68.0%	9.6	6.53
17.5	72.0%	9.6	6.91
20.0	75.0%	9.6	7.20
22.5	78.0%	9.6	7.49
25.0	81.0%	9.6	7.78
27.5	83.0%	9.6	7.97
30.0	86.0%	9.6	8.26
32.5	87.5%	9.6	8.40
35.0	89.0%	9.6	8.54
37.5	90.5%	9.6	8.69
40.0	92.0%	9.6	8.83
42.5	93.0%	9.6	8.93
45.0	94.0%	9.6	9.02
47.5	95.0%	9.6	9.12
50.0	96.0%	9.6	9.22
52.5	97.0%	9.6	9.31
55.0	98.0%	9.6	9.41
57.5	99.0%	9.6	9.50
60.0	100%	9.6	9.60

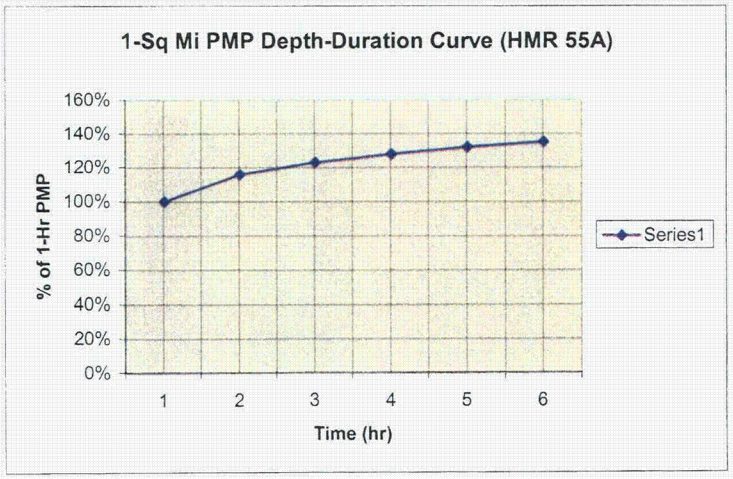


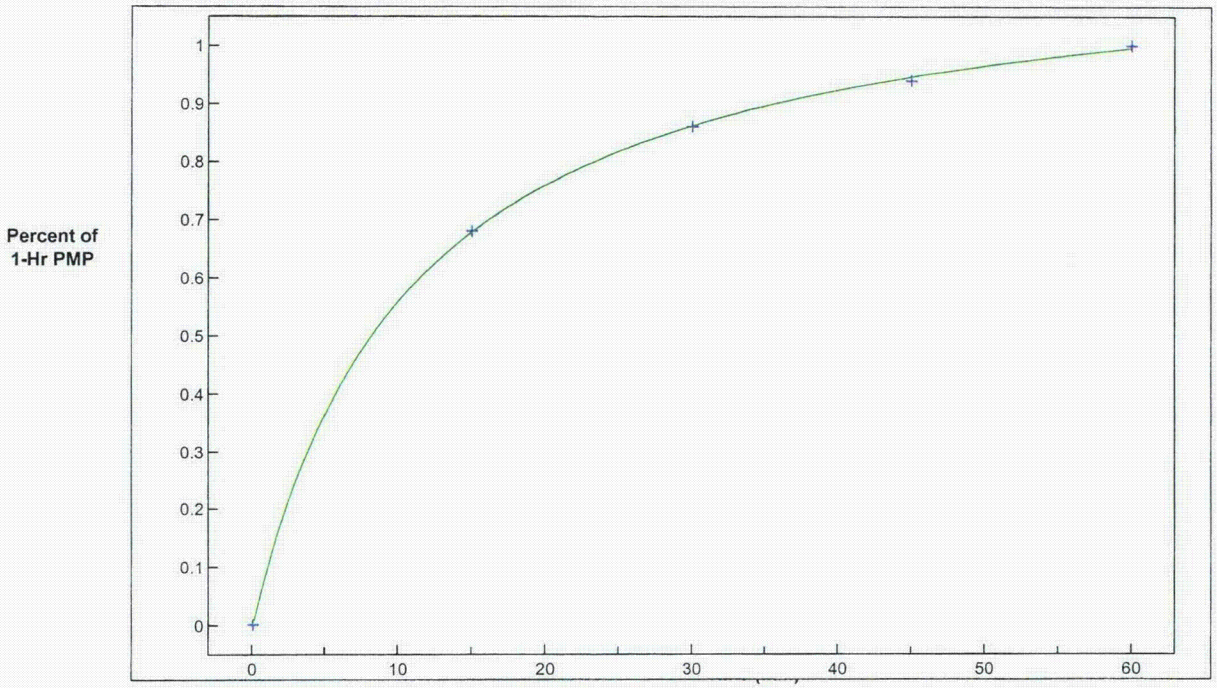
[1] Interpolated Values

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO

6-HOUR PMP DEPTH-DURATION CURVE

Duration (hr)	Percent of 1-hr PMP	1-hr 1-mi^2 PMP (inches)	PMP 1-hr Indexes up to 6 hr
1	100%	9.6	9.60
2	116%	9.6	11.14
3	123%	9.6	11.81
4	128%	9.6	12.29
5	132%	9.6	12.67
6	135%	9.6	12.96





1-Sq Mi PMP Depth-Duration Curve Up to 60 Minutes
Interpreted from HMR 55A Data from Table 12.4

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
DESIGN FLOWRATES - RATIONAL METHOD, AND ROCK SIZING [2, 3]

Catchment Drainage Area No.	A = Area [1] (acres)	PMP = Local 1-Hr 1-Mi^2 PMP Depth (inches)	H_1 = Higher Elevation (feet)	H_2 = Lower Elevation (feet)	H1 - H2	S = Slope (ft/ft)	L = Maximum Slope Length (feet)	A_W = Unit Width Area [1] (acres)	Specific Area Time of Concentration T_c= (0.0078 * ($L^{0.77}$))/ ($S^{0.385}$) (min)
Catchment 1	176.3	9.60	7036	6939	97.00	0.016	6200.0	0.1423	32.2
Catchment 1-Channel Top Edge		9.60	7036	6939	97.00	0.016	6200.0	0.1420	32.2
Catchment 1-Channel Side Slope		9.60	7036	6939	97.00	0.200	6200.0	0.1420	12.1
Catchment 2 - Diversion Channel	15.7	9.60	7026	6926	100.00	0.037	2706.0	0.0621	12.2
Catchment 2: Discharge Channel - Channel Bottom						0.018			
Catchment 3 - Diversion Channel	16.3	9.60	7007	6926	81.00	0.030	2701.0	0.0620	13.2
Catchment 3: Diversion Channel - Channel Bottom						0.009			
Catchment 4	12.4	9.60	7005	6926	79.00	0.029	2700.0	0.0620	13.3
Catchment 4-Channel Top Edge		9.60	7005	6926	79.00	0.029	2700.0	0.0619	13.3
Catchment 4-Channel Side Slope		9.60	7005	6926	79.00	0.200	2700.0	0.0619	6.4
Catchment 5	34.4	9.60	6942	6925	17.00	0.009	2000.0	0.0459	17.0
Catchment 5 - Overland (Tailings Ponds 4 & 5)		9.60	6942	6925	17.00	0.009	2000.0	0.0459	17.0
Catchment 6	86.8	9.60	6940	6920	20.00	0.008	2400.0	0.0551	19.7
Catchment 6 - Overland (Tailings Ponds 4, 5 & 6)		9.60	6940	6920	20.00	0.008	2400.0	0.0551	19.7
Catchment 7	99.2	9.60	6940	6915	25.00	0.016	1600.0	0.0367	11.3
Catchment 7-Channel Top Edge		9.60	6944	6915	29.00	0.018	1600.0	0.0367	10.7
Catchment 7-Channel Side Slope		9.60	6944	6915	29.00	0.200	1600.0	0.0367	4.2
Inside Sideslope of Diversion Berm		9.60			15.00	0.333	45.0	0.0010	0.2

[1] Drawing Sheet 2, Interior Site Drainage Plan

[2] NUREG-1623, U.S. Nuclear Regulatory Commission (NRC), Design of Erosion Protection for Long-Term Stabilization, Final Report, September 2002.

[3] Abt, S.R., Johnson, T.L., Thornton, C.I. and Trabant, S.C., Riprap Sizing at the Toe of Embankment Slopes, J. of Hydr. Engr., ASCE, 1998.

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO EROSION PROTECTION MEASURES
INTERIOR SITE DRAINAGE CALCULATIONS
DESIGN FLOWRATES - RATIONAL METHOD, AND ROCK SIZING [2, 3]

Catchment Drainage Area No.	A = Area [1] (acres)	% of PMP Index	D = Rainfall Depth = %PMPxPMP (inches)	i = Rainfall Intensity = $D*60/T_c$ = (inches/hr)	Q = Peak Discharge = CiA (cfs)	Station Location	Cumulative Q in Channel	q_d = Design Unit Width Peak Discharge = CiA_W (cfs/ft)
Catchment 1	176.3	87.3%	8.38	15.63	2756.19	7+90	2756.19	
Catchment 1-Channel Top Edge		87.3%	8.38	15.63				2.22
Catchment 1-Channel Side Slope		87.3%	8.38	41.70				5.92
Catchment 2 - Diversion Channel	15.7	62.2%	5.97	29.35	460.72	27+50	3216.91	
Catchment 2: Discharge Channel - Channel Bottom							498.00	19.92
Catchment 3 - Diversion Channel	16.3	64.4%	6.18	28.10	457.96	27+50	3674.87	
Catchment 3: Diversion Channel - Channel Bottom							497.00	17.75
Catchment 4	12.4	64.7%	6.21	27.94	346.41	27+50	4021.28	
Catchment 4-Channel Top Edge		64.7%	6.21	27.94				1.73
Catchment 4-Channel Side Slope		50.7%	4.87	45.94				2.84
Catchment 5	34.4	71.2%	6.84	24.10	830.00	27+50	4851.28	
Catchment 5 - Overland (Tailings Ponds 4 & 5)		71.2%	6.84	24.10				1.11
Catchment 6	86.8	74.7%	7.17	21.79	1891.18	56+00	6742.46	
Catchment 6 - Overland (Tailings Ponds 4, 5 & 6)		74.7%	7.17	21.79				1.20
Catchment 7	99.2	59.8%	5.74	30.35	3009.83	56+00	9752.29	
Catchment 7-Channel Top Edge		59.4%	5.70	31.96				1.17
Catchment 7-Channel Side Slope		46.5%	4.46	63.02				2.31
Inside Sideslope of Diversion Berm		1.9%	0.18	48.38				0.05

1 Drawing Sheet 2, Interior Site Drainage Plan

2 NUREG-1623, U.S. Nuclear Regulatory Commission (NRC), Design of Erosion Protection for Long-Term Stabilization, Final Report, September 2002.

3 Abt, S.R., Johnson, T.L., Thornton, C.I. and Trabant, S.C., Riprap Sizing at the Toe of Embankment Slopes, J. of Hydr. Engr., ASCE, 1998.

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
SUMMARY: CATCHMENT AREAS & PEAK DISCHARGES

Catchment Drainage Area No.	A = Area [1] (acres)	Time of Concentration (min)	q_d = Design Unit Discharge (cfs/ft)	Channel Bottom Width (ft)	Q = Peak Discharge = CiA (cfs)
Catchment 1	176.3	32.2		N/A	2756.19
Catchment 1-Channel Top Edge	0.1420	32.2	2.22	1 [2]	
Catchment 1-Channel Side Slope	0.1420	12.1	5.92	1 [2]	
Catchment 2 - Diversion Channel	15.7	12.2			460.72
Catchment 2: Discharge Channel - Channel Bottom		N/A	19.92	25.0	
Catchment 3 - Diversion Channel	16.3	13.2			457.96
Catchment 3: Diversion Channel - Channel Bottom		N/A	17.75	28.0	
Catchment 4	12.4	13.3			346.41
Catchment 4-Channel Top Edge	0.0619	13.3	1.73	1 [2]	
Catchment 4-Channel Side Slope	0.0619	6.4	2.84	1 [2]	
Catchment 5	0.0	17.0			830.00
Catchment 5 - Overland (Tailings Ponds 4 & 5)	0.0459	17.0	1.11	1 [2]	
Catchment 6	0.0	19.7			1891.18
Catchment 6 - Overland (Tailings Ponds 4, 5 & 6)	0.0551	19.7	1.20	1 [2]	
Catchment 7	99.2	11.3			3009.83
Catchment 7-Channel Top Edge	0.0367	10.7	1.17	1 [2]	
Catchment 7-Channel Side Slope	0.0367	4.2	2.31	1 [2]	
Inside Sideslope of Diversion Berm	0.0010	0.2	0.05	1 [2]	

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
SUMMARY: CATCHMENT AREAS & PEAK DISCHARGES

Catchment Drainage Area No.	A = Area [1] (acres)	Time of Concentration (min)	q_d = Design Unit Discharge (cfs/ft)	Channel Bottom Width (ft)	Q = Peak Discharge = CiA (cfs)
Inlet Channel at Sta 7+90 - 0.5% Slope	176.3		7.35	375.00	2756.19
Channel at Sta 8+00 - 4% Slope	176.3		22.05	125.00	2756.19
Channel at Sta 10+25 - 4% Slope	176.3		22.05	125.00	2756.19
Channel at Sta 10+25 - 0.5% Slope	176.3		22.05	125.00	2756.19
Channel at Sta 24+00 - 0.5% Slope	188.7		30.13	125.00	3766.60
Channel at Sta 25+00 - 0.5% Slope	205.01		33.80	125.00	4224.56
Channel at Sta 26+00 - 0.5% Slope	205.01		35.12	125.00	4390.56
Channel at Sta 27+50 - 0.5% Slope	221.1		38.81	125.00	4851.28
Channel at Sta 43+00 - 0.5% Slope	270.7		44.25	165.00	7301.79
Outlet Channel Apron at Sta 43+42 - 0.5% Slope	270.7		44.25	165.00	7301.79
Inlet Channel Apron at Sta 55+50 - 0.84% Slope	320.4		51.33	190.00	9752.29
Channel at Sta 56+00 - 0.84% Slope	320.4		51.33	190.00	9752.29

[1] Drawing Sheet 2, Interior Site Drainage Plan
[2] Unit Width

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
DESIGN OF EROSION PROTECTION SUMMARY OF METHODS PER NUREG-1623

NUREG-1623[1] Section #	Item Description	Depth of Flow, Rock Sizing, & Scour Depth Methods	Depth of Flow and Rock Sizing Methods	Equations	Reference
D.2	Overland Flow Slopes	Depth of Flow - Rational Method	Abt and Johnson Method	D_{50} Rock Size : $D_{50} = 5.23\ (S)^{0.43}\ (q_d)^{0.56}$	NUREG-1623[1]
D.3	Diversion Channels	Depth of Flow - Rational Method	Manning's n for steep slope and flow depth small relative to the riprap size	Manning's $n = 0.0456(D_{50} * S)^{0.159}$	NUREG/CR-4651[2]
			Manning's n for shallow slope and flow depth exceeding average size of riprap	Manning's $n = 0.0395(D_{50})^{1/6}$	NUREG/CR-4620[3]
			Manning's n for Open Channels	Manning's n = $R_h^{1/6}/(23.85+21.95Log_{10}(R/k))$	EM1110-2-1601[4]
			Manning's Equation to determine flow depth	Flow Depth: $Q = (1.486/n)AR^{2/3}S^{1/2}$	NUREG/CR-4620[3]
D.3	Diversion Channels	Option 1 - Rock Sizing	Safety Factors Method (peak shear stress)	Actual Shear Stress: $t = W_w yS$	NUREG-1623[1]
				D_{50} Rock Size: $D_{50} = (t)/(a(W_s - W_w)) = t / 4.1$, where $a = 0.04$, $W_s = 165$ pcf, $W_w = 62.4$ pcf	NUREG-1623[1]
D.3	Diversion Channels	Option 2 - Rock Sizing	Abt and Johnson Method	D_{50} Rock Size: $D_{50} = 5.23\ (S)^{0.43}\ (q_d)^{0.56}$, where q_d = Q/(bottom width of trapezoidal channel)	NUREG-1623[1]
D.4	Aprons and Diversion Channel Outlets	Option 2 - Rock Sizing	Abt and Johnson Method	$D_{50} = 5.23\ (S)^{0.43}\ (q_d)^{0.56}$	NUREG-1623[1]
		Scour Depth per Unit Width (h_e)	USACE Equilibrium Scour Depth	$h_e = (0.234*q_e^{8/9}) / [g(Ss-1)]^{4/9} d_e^{1/3}$	USACE
D.6	Toe of Embankment Slopes	Rational Method	Abt and Johnson Method	$D_{50} = 10.46\ (S)^{0.43}\ (C_f * q_d)^{0.56}$	Abt et al. (1998)[5]

[1] NUREG-1623, U.S. Nuclear Regulatory Commission (NRC), Design of Erosion Protection for Long-Term Stabilization, Final Report, September 2002.

[2] NUREG/CR-4651, U.S. Nuclear Regulatory Commission (NRC), Development of Riprap Design Criteria by Riprap Testing in Flumes: Phase II, September 1988.

[3] NUREG/CR-4620, U.S. Nuclear Regulatory Commission (NRC), Methodologies for Evaluating Long-Term Stabilization Designs of Uranium Mill Tailings Impoundments, June 1986.

[4] EM1110-2-1601, U.S.Army Corps of Engineers (USACE), Hydraulic Design of Flood Control Channels, Office of the Chief of Engineers, Washington, DC, 1994

[5] Abt, S.R., Johnson, T.L., Thornton, C.I. and Trabant, S.C., Riprap Sizing at the Toe of Embankment Slopes, J. of Hydr. Engr., ASCE, 1998.

[6] Abt, S.R., Thompson, P.I., and Lewis, T.M., Enhancement of the Culvert Outlet Scour Estimation Equations, Geometric and Other General Design Issues, Transportation Research Record (TRR), No. 1523, pp. 178-184, 1996.

[7] U.S. Department of Transportation (USDOT), Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular (HEC) No. 14, 1983.

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
INTERIOR DRAINAGE CHANNEL STATIONING AND ELEVATION SUMMARY

Center Elev at Ending Station (ft)	Beginning Station (ft)	Ending Station (ft)	Reach Between Stations (ft)	Slope Between Previous Reach (ft/ft)	Channel Elev Drop Previous Reach (ft)	Channel Width Increase Previous Reach (ft)	Fixed design Channel Bottom Widths (ft)	Channel Bottom Width at Endin Station (ft)	Left Channel Elev at Ending Station (ft)	Center Elev at Ending Station (ft)	Right Channel Elev at Ending Station (ft)
6939.00		800					165.00				
6930.00	800.00	1025.00	225.00	4.00%	9.00	-40.00	125.00	125.00	6929.06	6930.00	6930.94
6929.63	1025.00	1100.00	75.00	0.50%	0.38	0.00	125.00	125.00	6928.69	6929.63	6930.56
6929.13	1100.00	1200.00	100.00	0.50%	0.50	0.00	125.00	125.00	6928.19	6929.13	6930.06
6928.63	1200.00	1300.00	100.00	0.50%	0.50	0.00	125.00	125.00	6927.69	6928.63	6929.56
6928.13	1300.00	1400.00	100.00	0.50%	0.50	0.00	125.00	125.00	6927.19	6928.13	6929.06
6927.63	1400.00	1500.00	100.00	0.50%	0.50	0.00	125.00	125.00	6926.69	6927.63	6928.56
6927.13	1500.00	1600.00	100.00	0.50%	0.50	0.00	125.00	125.00	6926.19	6927.13	6928.06
6926.63	1600.00	1700.00	100.00	0.50%	0.50	0.00	125.00	125.00	6925.69	6926.63	6927.56
6926.13	1700.00	1800.00	100.00	0.50%	0.50	0.00	125.00	125.00	6925.19	6926.13	6927.06
6925.63	1800.00	1900.00	100.00	0.50%	0.50	0.00	125.00	125.00	6924.69	6925.63	6926.56
6925.13	1900.00	2000.00	100.00	0.50%	0.50	0.00	125.00	125.00	6924.19	6925.13	6926.06
6924.63	2000.00	2100.00	100.00	0.50%	0.50	0.00	125.00	125.00	6923.69	6924.63	6925.56
6924.13	2100.00	2200.00	100.00	0.50%	0.50	0.00	125.00	125.00	6923.19	6924.13	6925.06
6923.63	2200.00	2300.00	100.00	0.50%	0.50	0.00	125.00	125.00	6922.69	6923.63	6924.56
6923.13	2300.00	2400.00	100.00	0.50%	0.50	0.00	125.00	125.00	6922.19	6923.13	6924.06
6922.63	2400.00	2500.00	100.00	0.50%	0.50	0.00	125.00	125.00	6921.69	6922.63	6923.56
6922.13	2500.00	2600.00	100.00	0.50%	0.50	0.00	125.00	125.00	6921.19	6922.13	6923.06
6921.63	2600.00	2700.00	100.00	0.50%	0.50	0.00	125.00	125.00	6920.69	6921.63	6922.56
6921.38	2700.00	2750.00	50.00	0.50%	0.25	0.00	125.00	125.00	6920.44	6921.38	6922.31
6921.13	2750.00	2800.00	50.00	0.50%	0.25	1.29	126.29	126.29	6920.18	6921.13	6922.07
6920.63	2800.00	2900.00	100.00	0.50%	0.50	2.58	128.87	128.87	6919.66	6920.63	6921.59
6920.13	2900.00	3000.00	100.00	0.50%	0.50	2.58	131.45	131.45	6919.14	6920.13	6921.11
6919.63	3000.00	3100.00	100.00	0.50%	0.50	2.58	134.03	134.03	6918.62	6919.63	6920.63
6919.13	3100.00	3200.00	100.00	0.50%	0.50	2.58	136.61	136.61	6918.10	6919.13	6920.15
6918.63	3200.00	3300.00	100.00	0.50%	0.50	2.58	139.19	139.19	6917.58	6918.63	6919.67
6918.13	3300.00	3400.00	100.00	0.50%	0.50	2.58	141.77	141.77	6917.06	6918.13	6919.19

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
INTERIOR DRAINAGE CHANNEL STATIONING AND ELEVATION SUMMARY

Center Elev at Ending Station (ft)	Beginning Station (ft)	Ending Station (ft)	Reach Between Stations (ft)	Slope Between Previous Reach (ft/ft)	Channel Elev Drop Previous Reach (ft)	Channel Width Increase Previous Reach (ft)	Fixed design Channel Bottom Widths (ft)	Channel Bottom Width at Endin Station (ft)	Left Channel Elev at Ending Station (ft)	Center Elev at Ending Station (ft)	Right Channel Elev at Ending Station (ft)
6917.63	3400.00	3500.00	100.00	0.50%	0.50	2.58	144.35	144.35	6916.54	6917.63	6918.71
6917.13	3500.00	3600.00	100.00	0.50%	0.50	2.58	146.94	146.94	6916.02	6917.13	6918.23
6916.63	3600.00	3700.00	100.00	0.50%	0.50	2.58	149.52	149.52	6915.50	6916.63	6917.75
6916.13	3700.00	3800.00	100.00	0.50%	0.50	2.58	152.10	152.10	6914.98	6916.13	6917.27
6915.63	3800.00	3900.00	100.00	0.50%	0.50	2.58	154.68	154.68	6914.46	6915.63	6916.79
6915.13	3900.00	4000.00	100.00	0.50%	0.50	2.58	157.26	157.26	6913.95	6915.13	6916.30
6914.63	4000.00	4100.00	100.00	0.50%	0.50	2.58	159.84	159.84	6913.43	6914.63	6915.82
6914.13	4100.00	4200.00	100.00	0.50%	0.50	2.58	162.42	162.42	6912.91	6914.13	6915.34
6913.63	4200.00	4300.00	100.00	0.50%	0.50	2.58	165.00	165.00	6912.39	6913.63	6914.86
6913.13	4300.00	4400.00	100.00	0.50%	0.50	1.92	166.92	166.92	6911.87	6913.13	6914.38
6912.63	4400.00	4500.00	100.00	0.50%	0.50	1.92	168.85	168.85	6911.36	6912.63	6913.89
6912.13	4500.00	4600.00	100.00	0.50%	0.50	1.92	170.77	170.77	6910.84	6912.13	6913.41
6911.63	4600.00	4700.00	100.00	0.50%	0.50	1.92	172.69	172.69	6910.33	6911.63	6912.92
6911.13	4700.00	4800.00	100.00	0.50%	0.50	1.92	174.62	174.62	6909.82	6911.13	6912.43
6910.63	4800.00	4900.00	100.00	0.50%	0.50	1.92	176.54	176.54	6909.30	6910.63	6911.95
6910.13	4900.00	5000.00	100.00	0.50%	0.50	1.92	178.46	178.46	6908.79	6910.13	6911.46
6909.63	5000.00	5100.00	100.00	0.50%	0.50	1.92	180.38	180.38	6908.27	6909.63	6910.98
6909.13	5100.00	5200.00	100.00	0.50%	0.50	1.92	182.31	182.31	6907.76	6909.13	6910.49
6908.28	5200.00	5300.00	100.00	0.84%	0.84	1.92	184.23	184.23	6906.90	6908.28	6909.67
6907.44	5300.00	5400.00	100.00	0.84%	0.84	1.92	186.15	186.15	6906.05	6907.44	6908.84
6906.60	5400.00	5500.00	100.00	0.84%	0.84	1.92	188.08	188.08	6905.19	6906.60	6908.01
6905.76	5500.00	5600.00	100.00	0.84%	0.84	1.92	190.00	190.00	6904.33	6905.76	6907.18
6904.92	5600.00	5700.00	100.00	0.84%	0.84	0.00	190.00	190.00	6903.49	6904.92	6906.34
6904.08	5700.00	5800.00	100.00	0.84%	0.84	0.00	190.00	190.00	6902.65	6904.08	6905.50

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
INTERIOR DRAINAGE CHANNEL STATIONING AND ELEVATION SUMMARY

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
NTERIOR DRAINAGE CHANNEL STATIONING AND ELEVATION SUMMAR

	Manning's 'n'					Manning's Equation $Q=(1.486/n)AR^{2/3}S^{1/2}$				
Rock Placement Area[1]	Assume D_{50} = k = rock size		R_h = hydraulic radius = depth of flow (ft)	"With Rock" Shallow slope and flow depth > D_{50}; Manning's 'n' = $0.0395(D_{50})^{1/6}$	"Without Rock" Open Channels; Manning's 'n' = $R_h^{1/6}/(23.85+21.95\ Log_{10}(R/k))$	Design Flowrate Q_d (cfs)	Assume y depth (ft)	b = bottom width (ft)	S = channel slope (ft/ft)	Calculated Flow Depth: $Q=(1.486/n)AR^{2/3}S^{1/2}$
	Inches	Feet								
Catchment 1-Channel Top Edge								1.0	0.016	
Catchment 1-Channel Side Slope								1.0	0.200	
Catchment 2: Discharge Channel - Channel Bottom								1.0	0.018	
Catchment 3: Diversion Channel - Channel Bottom								1.0	0.009	
Catchment 4-Channel Top Edge								1.0	0.029	
Catchment 4-Channel Side Slope								1.0	0.200	
Catchment 5 - Overland (Tailings Ponds 4 & 5)								1.0	0.009	
Catchment 6 - Overland (Tailings Ponds 4, 5 & 6)								1.0	0.008	
Catchment 7-Channel Top Edge								1.0	0.018	
Catchment 7-Channel Side Slope								1.0	0.200	
Inside Sideslope of Diversion Berm								1.0	0.333	

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	Manning's 'n'					Manning's Equation $Q=(1.486/n)AR^{2/3}S^{1/2}$				
Rock Placement Area[1]	Assume D_{50} = k = rock size		R_h = hydraulic radius = depth of flow (ft)	"With Rock" Shallow slope and flow depth > D_{50}; Manning's 'n' = $0.0395(D_{50})^{1/6}$	"Without Rock" Open Channels; Manning's 'n' = $R_h^{1/6}/(23.85+21.95\ Log_{10}(R/k))$	Design Flowrate Q_d (cfs)	Assume y depth (ft)	b = bottom width (ft)	S = channel slope (ft/ft)	Calculated Flow Depth: $Q=(1.486/n)AR^{2/3}S^{1/2}$
	Inches	Feet								
Inlet Channel at Sta 7+90 - 0.5% Slope	7.80	0.65	1.65	0.0368	0.0332	2756.19	1.65	375.00	0.0050	2758.7
Channel at Sta 8+00 - 4% Slope	7.80	0.65	1.80	0.0368	0.0329	2756.19	1.80	125.00	0.0400	2758.4
Channel at Sta 10+25 - 4% Slope	7.80	0.65	1.80	0.0368	0.0329	2756.19	1.80	125.00	0.0400	2758.4
Channel at Sta 10+25 - 0.5% Slope	7.80	0.65	3.31	0.0368	0.0310	2756.19	3.31	125.00	0.0050	2755.3
Channel at Sta 24+00 - 0.5% Slope	7.80	0.65	3.97	0.0368	0.0306	3766.60	3.97	125.00	0.0050	3767.5
Channel at Sta 25+00 - 0.5% Slope	7.80	0.65	4.24	0.0368	0.0305	4224.56	4.24	125.00	0.0050	4226.6
Channel at Sta 26+00 - 0.5% Slope	7.80	0.65	4.34	0.0368	0.0304	4390.56	4.34	125.00	0.0050	4392.5
Channel at Sta 27+50 - 0.5% Slope	7.80	0.65	4.59	0.0368	0.0303	4851.28	4.59	125.00	0.0050	4852.2
Channel at Sta 43+00 - 0.5% Slope	7.80	0.65	5.00	0.0368	0.0302	7301.79	5.00	165.00	0.0050	7299.1
Outlet Channel Apron at Sta 43+42 - 0.5% Slope	7.80	0.65	4.48	0.0368	0.0304	7301.79	4.48	165.00	0.0050	7302.5
Inlet Channel Apron at Sta 55+50 - 0.84% Slope	12.00	1.00	4.48	0.0395	0.0337	9752.29	4.48	190.00	0.0084	9752.4
Channel at Sta 56+00 - 0.84% Slope	12.00	1.00	4.91	0.0395	0.0334	9752.29	4.91	190.00	0.0084	9751.8

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INTERIOR DRAINAGE CHANNEL STATIONING AND ELEVATION SUMMARY

	Abt and Johnson Method $D_{50}=5.23(S)^{0.43}(q_d)^{0.56}$		USACE Equilibrium Scour Depth (ft per unit width) $h_e=(0.234*q_e^{8/9})/[g(Ss-1)]^{4/9}d_e^{1/3}$				
Rock Placement Area[1]	q_d = Design Unit Width Peak Discharge (cfs/ft)	$D_{50} = 5.23\ (S)^{0.43}\ (q_d)^{0.56}$ (inches)	he = Equilibrium Scour Depth (ft)	qe = equilibrium discharge (cfs/ft)	g (ft/sec2)	Ss = Sediment Specific Gravity	de = median grain-size diameter (mm)
Catchment 1-Channel Top Edge	2.22	1.37	0.19	2.22	32.2	2.65	0.074
Catchment 1-Channel Side Slope	5.92	7.09	0.46	5.92	32.2	2.65	0.074
Catchment 2: Discharge Channel - Channel Bottom	19.92	4.90	1.36	19.92	32.2	2.65	0.074
Catchment 3: Diversion Channel - Channel Bottom	17.75	3.45	1.23	17.75	32.2	2.65	0.074
Catchment 4-Channel Top Edge	1.73	1.56	0.16	1.73	32.2	2.65	0.074
Catchment 4-Channel Side Slope	2.84	4.70	0.24	2.84	32.2	2.65	0.074
Catchment 5 - Overland (Tailings Ponds 4 & 5)	1.11	0.71	0.10	1.11	32.2	2.65	0.074
Catchment 6 - Overland (Tailings Ponds 4, 5 & 6)	1.20	0.74	0.11	1.20	32.2	2.65	0.074
Catchment 7-Channel Top Edge	1.17	1.02	0.11	1.17	32.2	2.65	0.074
Catchment 7-Channel Side Slope	2.31	4.18	0.20	2.31	32.2	2.65	0.074
Inside Sideslope of Diversion Berm	0.05	0.61	0.01	0.05	32.2	2.65	0.074

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	Abt and Johnson Method $D_{50}=5.23(S)^{0.43}(q_d)^{0.56}$		USACE Equilibrium Scour Depth (ft per unit width) $h_e=(0.234*q_e^{8/9})/[g(Ss-1)]^{4/9}d_e^{1/3}$				
Rock Placement Area[1]	q_d = Design Unit Width Peak Discharge (cfs/ft)	$D_{50} = 5.23\ (S)^{0.43}\ (q_d)^{0.56}$ (inches)	he = Equilibrium Scour Depth (ft)	qe = equilibrium discharge (cfs/ft)	g (ft/sec2)	Ss = Sediment Specific Gravity	de = median grain-size diameter (mm)
Inlet Channel at Sta 7+90 - 0.5% Slope	7.35	1.64	0.56	7.35	32.2	2.65	0.074
Channel at Sta 8+00 - 4% Slope	22.05	7.41	1.49	22.05	32.2	2.65	0.074
Channel at Sta 10+25 - 4% Slope	22.05	7.41	1.49	22.05	32.2	2.65	0.074
Channel at Sta 10+25 - 0.5% Slope	22.05	3.03	1.49	22.05	32.2	2.65	0.074
Channel at Sta 24+00 - 0.5% Slope	30.13	3.61	1.97	30.13	32.2	2.65	0.074
Channel at Sta 25+00 - 0.5% Slope	33.80	3.85	2.18	33.80	32.2	2.65	0.074
Channel at Sta 26+00 - 0.5% Slope	35.12	3.93	2.26	35.12	32.2	2.65	0.074
Channel at Sta 27+50 - 0.5% Slope	38.81	4.16	2.46	38.81	32.2	2.65	0.074
Channel at Sta 43+00 - 0.5% Slope	44.25	4.47	2.77	44.25	32.2	2.65	0.074
Outlet Channel Apron at Sta 43+42 - 0.5% Slope	44.25	4.47	2.77	44.25	32.2	2.65	0.074
Inlet Channel Apron at Sta 55+50 - 0.84% Slope	51.33	6.08	3.16	51.33	32.2	2.65	0.074
Channel at Sta 56+00 - 0.84% Slope	51.33	6.08	3.16	51.33	32.2	2.65	0.074

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INTERIOR SITE DRAINAGE CALCULATIONS
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Rock Placement Area[1]	Min D_{50} (in) Abt and Johnson Method	Oversize Min D_{50} 4% (in)	Min Required D_{50} Available to Use (in)	Proposed D_{50} to Use (in)	Max Scour Depth (ft)	Min Thickness Ratio	Min Thickness to Use (in)	Min Apron Thickness = 3x D_{50} (in)	Apron Thickness to Use (in)	Min Side Slope Apron Width = 15 x D_{50} (ft)	Minimum Full Channel Apron Width to Use (ft)
Catchment 1-Channel Top Edge	1.37	1.42	3.2	7.8	0.19	2	16				
Catchment 1-Channel Side Slope	7.09	7.37	7.8	7.8	0.46	2	16				
Catchment 2: Discharge Channel - Channel Bottom	4.90	5.10	7.8	7.8	1.36	2	16				
Catchment 3: Diversion Channel - Channel Bottom	3.45	3.59	7.8	7.8	1.23	2	16				
Catchment 4-Channel Top Edge	1.56	1.62	3.2	7.8	0.16	2	16				
Catchment 4-Channel Side Slope	4.70	4.89	7.8	7.8	0.24	2	16				
Catchment 5 - Overland (Tailings Ponds 4 & 5)	0.71	0.74	1.0	1.0	0.10	2	3				
Catchment 6 - Overland (Tailings Ponds 4, 5 & 6)	0.74	0.77	1.0	1.0	0.11	2	3				
Catchment 7-Channel Top Edge	1.02	1.06	3.2	7.8	0.11	2	16				
Catchment 7-Channel Side Slope	4.18	4.35	7.8	7.8	0.20	2	16				
Inside Sideslope of Diversion Berm	0.61	0.63	1.0	1.0	0.01	2	16	3.0	6	4.0	

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Rock Placement Area[1]	Min D_{50} (in) Abt and Johnson Method	Oversize Min D_{50} 4% (in)	Min Required D_{50} Available to Use (in)	Proposed D_{50} to Use (in)	Max Scour Depth (ft)	Min Thickness Ratio	Min Thickness to Use (in)	Min Apron Thickness = 3x D_{50} (in)	Apron Thickness to Use (in)	Min Side Slope Apron Width = 15 x D_{50} (ft)	Minimum Full Channel Apron Width to Use (ft)
Inlet Channel at Sta 7+90 - 0.5% Slope	1.64	1.70	1.0	7.8	0.56	2	16				
Channel at Sta 8+00 - 4% Slope	7.41	7.70	7.8	7.8	1.49	2	16				
Channel at Sta 10+25 - 4% Slope	7.41	7.70	7.8	7.8	1.49	2	16				
Channel at Sta 10+25 - 0.5% Slope	3.03	3.15	3.2	7.8	1.49	2	16				
Channel at Sta 24+00 - 0.5% Slope	3.61	3.75	7.8	7.8	1.97	2	16				
Channel at Sta 25+00 - 0.5% Slope	3.85	4.00	7.8	7.8	2.18						
Channel at Sta 26+00 - 0.5% Slope	3.93	4.09	7.8	7.8	2.26						
Channel at Sta 27+50 - 0.5% Slope	4.16	4.32	7.8	7.8	2.46	2	16				
Channel at Sta 43+00 - 0.5% Slope	4.47	4.65	7.8	7.8	2.77	2	16				
Outlet Channel Apron at Sta 43+42 - 0.5% Slope	4.47	4.65	7.8	7.8	2.77	2	16	33.2	36	9.8	30.0
Inlet Channel Apron at Sta 55+50 - 0.84% Slope	6.08	6.33	7.8	12.0	3.16	2	24	37.9	39		30.0
Channel at Sta 56+00 - 0.84% Slope	6.08	6.33	7.8	12.0	3.16	2	24				

[1] Drawing Sheet 2, Interior Site Drainage Plan

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CHANNEL PARAMETERS

River Station	Station Number	Center Channel Elev at Ending Station (ft)	Slope Between Previous Reach (ft/ft)	Channel Elev Drop Previous Reach (ft)	Channel Width Increase Previous Reach (ft)	Fixed Design Channel Bottom Widths (ft)
	800.00	6939.00	0.50%			165.00
49.00	1025.00	6930.00	4.00%	9.00	-40.00	125.00
48.00	1100.00	6929.63	0.50%	0.38	0.00	125.00
47.00	1200.00	6929.13	0.50%	0.50	0.00	125.00
46.00	1300.00	6928.63	0.50%	0.50	0.00	125.00
45.00	1400.00	6928.13	0.50%	0.50	0.00	125.00
44.00	1500.00	6927.63	0.50%	0.50	0.00	125.00
43.00	1600.00	6927.13	0.50%	0.50	0.00	125.00
42.00	1700.00	6926.63	0.50%	0.50	0.00	125.00
41.00	1800.00	6926.13	0.50%	0.50	0.00	125.00
40.00	1900.00	6925.63	0.50%	0.50	0.00	125.00
39.00	2000.00	6925.13	0.50%	0.50	0.00	125.00
38.00	2100.00	6924.63	0.50%	0.50	0.00	125.00
37.00	2200.00	6924.13	0.50%	0.50	0.00	125.00
36.00	2300.00	6923.63	0.50%	0.50	0.00	125.00
35.00	2400.00	6923.13	0.50%	0.50	0.00	125.00
34.00	2500.00	6922.63	0.50%	0.50	0.00	125.00
33.00	2600.00	6922.13	0.50%	0.50	0.00	125.00
32.00	2700.00	6921.63	0.50%	0.50	0.00	125.00
31.00	2750.00	6921.38	0.50%	0.25	0.00	125.00
30.00	2800.00	6921.13	0.50%	0.25	1.29	126.29
29.00	2900.00	6920.63	0.50%	0.50	2.58	128.87
28.00	3000.00	6920.13	0.50%	0.50	2.58	131.45
27.00	3100.00	6919.63	0.50%	0.50	2.58	134.03

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CHANNEL PARAMETERS

River Station	Station Number	Center Channel Elev at Ending Station (ft)	Slope Between Previous Reach (ft/ft)	Channel Elev Drop Previous Reach (ft)	Channel Width Increase Previous Reach (ft)	Fixed Design Channel Bottom Widths (ft)
26.00	3200.00	6919.13	0.50%	0.50	2.58	136.61
25.00	3300.00	6918.63	0.50%	0.50	2.58	139.19
24.00	3400.00	6918.13	0.50%	0.50	2.58	141.77
23.00	3500.00	6917.63	0.50%	0.50	2.58	144.35
22.00	3600.00	6917.13	0.50%	0.50	2.58	146.94
21.00	3700.00	6916.63	0.50%	0.50	2.58	149.52
20.00	3800.00	6916.13	0.50%	0.50	2.58	152.10
19.00	3900.00	6915.63	0.50%	0.50	2.58	154.68
18.00	4000.00	6915.13	0.50%	0.50	2.58	157.26
17.00	4100.00	6914.63	0.50%	0.50	2.58	159.84
16.00	4200.00	6914.13	0.50%	0.50	2.58	162.42
15.00	4300.00	6913.63	0.50%	0.50	2.58	165.00
14.00	4400.00	6913.13	0.50%	0.50	1.92	166.92
13.00	4500.00	6912.63	0.50%	0.50	1.92	168.85
12.00	4600.00	6912.13	0.50%	0.50	1.92	170.77
11.00	4700.00	6911.63	0.50%	0.50	1.92	172.69
10.00	4800.00	6911.13	0.50%	0.50	1.92	174.62
9.00	4900.00	6910.63	0.50%	0.50	1.92	176.54
8.00	5000.00	6910.13	0.50%	0.50	1.92	178.46
7.00	5100.00	6909.63	0.50%	0.50	1.92	180.38
6.00	5200.00	6909.13	0.50%	0.50	1.92	182.31
5.00	5300.00	6908.28	0.84%	0.84	1.92	184.23
4.00	5400.00	6907.44	0.84%	0.84	1.92	186.15
3.00	5500.00	6906.60	0.84%	0.84	1.92	188.08
2.00	5600.00	6905.76	0.84%	0.84	1.92	190.00
1.00	5700.00	6904.92	0.84%	0.84	0.00	190.00
0.00	5800.00	6904.08	0.84%	0.84	0.00	190.00

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CHANNEL STATION ELEVATIONS

River Station	Station Number	Down Stream Reach	Q (cfs)	Left Overbank -2	Left Overbank -1	Left Top of Slope	Left Channel Base of Slope	Right Channel 15.0' out from Base of Slope	Right Channel Base of Slope	Right Top of Slope	Right Overbank -1	Right Overbank -2
	800.00	225.00										
49.00	1025.00	75.00	2756.19	6935.94	6934.94	6934.94	6929.06	6930.71	6930.94	6934.94	6934.94	6935.94
48.00	1100.00	100.00	2811.30	6935.56	6934.56	6934.56	6928.69	6930.34	6930.56	6934.56	6934.56	6935.56
47.00	1200.00	100.00	2884.79	6935.06	6934.06	6934.06	6928.19	6929.84	6930.06	6934.06	6934.06	6935.06
46.00	1300.00	100.00	2958.27	6934.56	6933.56	6933.56	6927.69	6929.34	6929.56	6933.56	6933.56	6934.56
45.00	1400.00	100.00	3031.75	6934.06	6933.06	6933.06	6927.19	6928.84	6929.06	6933.06	6933.06	6934.06
44.00	1500.00	100.00	3105.24	6933.56	6932.56	6932.56	6926.69	6928.34	6928.56	6932.56	6932.56	6933.56
43.00	1600.00	100.00	3178.72	6933.06	6932.06	6932.06	6926.19	6927.84	6928.06	6932.06	6932.06	6933.06
42.00	1700.00	100.00	3252.21	6932.56	6931.56	6931.56	6925.69	6927.34	6927.56	6931.56	6931.56	6932.56
41.00	1800.00	100.00	3325.69	6932.06	6931.06	6931.06	6925.19	6926.84	6927.06	6931.06	6931.06	6932.06
40.00	1900.00	100.00	3399.18	6931.56	6930.56	6930.56	6924.69	6926.34	6926.56	6930.56	6930.56	6931.56
39.00	2000.00	100.00	3472.66	6931.06	6930.06	6930.06	6924.19	6925.84	6926.06	6930.06	6930.06	6931.06
38.00	2100.00	100.00	3546.15	6930.56	6929.56	6929.56	6923.69	6925.34	6925.56	6929.56	6929.56	6930.56
37.00	2200.00	100.00	3619.63	6930.06	6929.06	6929.06	6923.19	6924.84	6925.06	6929.06	6929.06	6930.06
36.00	2300.00	100.00	3693.11	6929.56	6928.56	6928.56	6922.69	6924.34	6924.56	6928.56	6928.56	6929.56
35.00	2400.00	100.00	3766.60	6930.06	6929.06	6929.06	6922.19	6923.84	6924.06	6929.06	6929.06	6930.06
34.00	2500.00	100.00	4224.56	6929.56	6928.56	6928.56	6921.69	6923.34	6923.56	6928.56	6928.56	6929.56
33.00	2600.00	100.00	4390.56	6929.06	6928.06	6928.06	6921.19	6922.84	6923.06	6928.06	6928.06	6929.06
32.00	2700.00	50.00	4851.28	6928.56	6927.56	6927.56	6920.69	6922.34	6922.56	6927.56	6927.56	6928.56
31.00	2750.00	50.00	4851.28	6928.31	6927.31	6927.31	6920.44	6922.09	6922.31	6927.31	6927.31	6928.31
30.00	2800.00	100.00	4930.33	6928.07	6927.07	6927.07	6920.18	6921.85	6922.07	6927.07	6927.07	6928.07
29.00	2900.00	100.00	5088.43	6927.59	6926.59	6926.59	6919.66	6921.37	6921.59	6926.59	6926.59	6927.59
28.00	3000.00	100.00	5246.52	6927.11	6926.11	6926.11	6919.14	6920.89	6921.11	6926.11	6926.11	6927.11
27.00	3100.00	100.00	5404.62	6926.63	6925.63	6925.63	6918.62	6920.41	6920.63	6925.63	6925.63	6926.63

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CHANNEL STATION ELEVATIONS

River Station	Station Number	Down Stream Reach	Q (cfs)	Left Overbank -2	Left Overbank -1	Left Top of Slope	Left Channel Base of Slope	Right Channel 15.0' out from Base of Slope	Right Channel Base of Slope	Right Top of Slope	Right Overbank -1	Right Overbank -2
26.00	3200.00	100.00	5562.72	6926.15	6925.15	6925.15	6918.10	6919.92	6920.15	6925.15	6925.15	6926.15
25.00	3300.00	100.00	5720.81	6925.67	6924.67	6924.67	6917.58	6919.44	6919.67	6924.67	6924.67	6925.67
24.00	3400.00	100.00	5878.91	6925.19	6924.19	6924.19	6917.06	6918.96	6919.19	6924.19	6924.19	6925.19
23.00	3500.00	100.00	6037.01	6924.71	6923.71	6923.71	6916.54	6918.48	6918.71	6923.71	6923.71	6924.71
22.00	3600.00	100.00	6195.11	6924.23	6923.23	6923.23	6916.02	6918.00	6918.23	6923.23	6923.23	6924.23
21.00	3700.00	100.00	6353.20	6923.75	6922.75	6922.75	6915.50	6917.52	6917.75	6922.75	6922.75	6923.75
20.00	3800.00	100.00	6511.30	6923.27	6922.27	6922.27	6914.98	6917.04	6917.27	6922.27	6922.27	6923.27
19.00	3900.00	100.00	6669.40	6922.79	6921.79	6921.79	6914.46	6916.56	6916.79	6921.79	6921.79	6922.79
18.00	4000.00	100.00	6827.50	6922.30	6921.30	6921.30	6913.95	6916.08	6916.30	6921.30	6921.30	6922.30
17.00	4100.00	100.00	6985.59	6921.82	6920.82	6920.82	6913.43	6915.60	6915.82	6920.82	6920.82	6921.82
16.00	4200.00	100.00	7143.69	6921.34	6920.34	6920.34	6912.91	6915.12	6915.34	6920.34	6920.34	6921.34
15.00	4300.00	100.00	7301.79	6920.86	6919.86	6919.86	6912.39	6914.64	6914.86	6919.86	6919.86	6920.86
14.00	4400.00	100.00	7506.00	6920.38	6919.38	6919.38	6911.87	6914.15	6914.38	6919.38	6919.38	6920.38
13.00	4500.00	100.00	7710.20	6919.89	6918.89	6918.89	6911.36	6913.67	6913.89	6918.89	6918.89	6919.89
12.00	4600.00	100.00	7914.41	6919.41	6918.41	6918.41	6910.84	6913.18	6913.41	6918.41	6918.41	6919.41
11.00	4700.00	100.00	8118.62	6918.92	6917.92	6917.92	6910.33	6912.70	6912.92	6917.92	6917.92	6918.92
10.00	4800.00	100.00	8322.83	6918.43	6917.43	6917.43	6909.82	6912.21	6912.43	6917.43	6917.43	6918.43
9.00	4900.00	100.00	8527.04	6917.95	6916.95	6916.95	6909.30	6911.72	6911.95	6916.95	6916.95	6917.95
8.00	5000.00	100.00	8731.25	6917.46	6916.46	6916.46	6908.79	6911.24	6911.46	6916.46	6916.46	6917.46
7.00	5100.00	100.00	8935.46	6916.98	6915.98	6915.98	6908.27	6910.75	6910.98	6915.98	6915.98	6916.98
6.00	5200.00	100.00	9139.67	6916.49	6915.49	6915.49	6907.76	6910.27	6910.49	6915.49	6915.49	6916.49
5.00	5300.00	100.00	9343.88	6915.67	6914.67	6914.67	6906.90	6909.44	6909.67	6914.67	6914.67	6915.67
4.00	5400.00	100.00	9548.09	6914.84	6913.84	6913.84	6906.05	6908.61	6908.84	6913.84	6913.84	6914.84
3.00	5500.00	100.00	9752.29	6914.01	6913.01	6913.01	6905.19	6907.79	6908.01	6913.01	6913.01	6914.01
2.00	5600.00	100.00	9752.29	6913.18	6912.18	6912.18	6904.33	6906.96	6907.18	6912.18	6912.18	6913.18
1.00	5700.00	100.00	9752.29	6912.34	6911.34	6911.34	6903.49	6906.12	6906.34	6911.34	6911.34	6912.34
0.00	5800.00		9752.29				6902.65		6905.50			

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
HEC-RAS INPUT DATA

CHANNEL STATION NUMBERING (LEFT TO RIGHT)

River Station	Station Number	Left Overbank -2	Left Overbank -1	Left Top of Slope	Left Channel Base of Slope	Right Channel 15.0' out from Base of Slope	Right Channel Base of Slope	Right Top of Slope	Right Overbank -1	Right Overbank -2
	800.00									
49.00	1025.00	0	5	15	44.375	154.375	169.375	194.375	204.375	209.375
48.00	1100.00	0	5	15	44.375	154.375	169.375	194.375	204.375	209.375
47.00	1200.00	0	5	15	44.375	154.375	169.375	194.375	204.375	209.375
46.00	1300.00	0	5	15	44.375	154.375	169.375	194.375	204.375	209.375
45.00	1400.00	0	5	15	44.375	154.375	169.375	194.375	204.375	209.375
44.00	1500.00	0	5	15	44.375	154.375	169.375	194.375	204.375	209.375
43.00	1600.00	0	5	15	44.375	154.375	169.375	194.375	204.375	209.375
42.00	1700.00	0	5	15	44.375	154.375	169.375	194.375	204.375	209.375
41.00	1800.00	0	5	15	44.375	154.375	169.375	194.375	204.375	209.375
40.00	1900.00	0	5	15	44.375	154.375	169.375	194.375	204.375	209.375
39.00	2000.00	0	5	15	44.375	154.375	169.375	194.375	204.375	209.375
38.00	2100.00	0	5	15	44.375	154.375	169.375	194.375	204.375	209.375
37.00	2200.00	0	5	15	44.375	154.375	169.375	194.375	204.375	209.375
36.00	2300.00	0	5	15	44.375	154.375	169.375	194.375	204.375	209.375
35.00	2400.00	0	5	15	49.375	159.375	174.375	199.375	209.375	214.375
34.00	2500.00	0	5	15	49.375	159.375	174.375	199.375	209.375	214.375
33.00	2600.00	0	5	15	49.375	159.375	174.375	199.375	209.375	214.375
32.00	2700.00	0	5	15	49.375	159.375	174.375	199.375	209.375	214.375
31.00	2750.00	0	5	15	49.375	159.375	174.375	199.375	209.375	214.375
30.00	2800.00	0	5	15	49.472	160.762	175.762	200.762	210.762	215.762
29.00	2900.00	0	5	15	49.665	163.536	178.536	203.536	213.536	218.536
28.00	3000.00	0	5	15	49.859	166.310	181.310	206.310	216.310	221.310
27.00	3100.00	0	5	15	50.052	169.085	184.085	209.085	219.085	224.085

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
HEC-RAS INPUT DATA

CHANNEL STATION NUMBERING (LEFT TO RIGHT)

River Station	Station Number	Left Overbank -2	Left Overbank -1	Left Top of Slope	Left Channel Base of Slope	Right Channel 15.0' out from Base of Slope	Right Channel Base of Slope	Right Top of Slope	Right Overbank -1	Right Overbank -2
26.00	3200.00	0	5	15	50.246	171.859	186.859	211.859	221.859	226.859
25.00	3300.00	0	5	15	50.440	174.633	189.633	214.633	224.633	229.633
24.00	3400.00	0	5	15	50.633	177.407	192.407	217.407	227.407	232.407
23.00	3500.00	0	5	15	50.827	180.181	195.181	220.181	230.181	235.181
22.00	3600.00	0	5	15	51.020	182.956	197.956	222.956	232.956	237.956
21.00	3700.00	0	5	15	51.214	185.730	200.730	225.730	235.730	240.730
20.00	3800.00	0	5	15	51.407	188.504	203.504	228.504	238.504	243.504
19.00	3900.00	0	5	15	51.601	191.278	206.278	231.278	241.278	246.278
18.00	4000.00	0	5	15	51.794	194.052	209.052	234.052	244.052	249.052
17.00	4100.00	0	5	15	51.988	196.827	211.827	236.827	246.827	251.827
16.00	4200.00	0	5	15	52.181	199.601	214.601	239.601	249.601	254.601
15.00	4300.00	0	5	15	52.375	202.375	217.375	242.375	252.375	257.375
14.00	4400.00	0	5	15	52.519	204.442	219.442	244.442	254.442	259.442
13.00	4500.00	0	5	15	52.663	206.510	221.510	246.510	256.510	261.510
12.00	4600.00	0	5	15	52.808	208.577	223.577	248.577	258.577	263.577
11.00	4700.00	0	5	15	52.952	210.644	225.644	250.644	260.644	265.644
10.00	4800.00	0	5	15	53.096	212.712	227.712	252.712	262.712	267.712
9.00	4900.00	0	5	15	53.240	214.779	229.779	254.779	264.779	269.779
8.00	5000.00	0	5	15	53.385	216.846	231.846	256.846	266.846	271.846
7.00	5100.00	0	5	15	53.529	218.913	233.913	258.913	268.913	273.913
6.00	5200.00	0	5	15	53.673	220.981	235.981	260.981	270.981	275.981
5.00	5300.00	0	5	15	53.817	223.048	238.048	263.048	273.048	278.048
4.00	5400.00	0	5	15	53.962	225.115	240.115	265.115	275.115	280.115
3.00	5500.00	0	5	15	54.106	227.183	242.183	267.183	277.183	282.183
2.00	5600.00	0	5	15	54.250	229.250	244.250	269.250	279.250	284.250
1.00	5700.00	0	5	15	54.250	229.250	244.250	269.250	279.250	284.250
0.00	5800.00									

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
HEC-RAS CHANNEL OUTPUT DATA

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Depth above Right Channel Bottom (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/sec)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
Lower	49	PF 1	2756	6929.06	6933.41	2.47		6933.9	0.004362	5.58	493.52	162.22	0.56
Lower	48	PF 1	2811	6928.69	6933.08	2.52		6933.57	0.004384	5.63	499.21	162.69	0.57
Lower	47	PF 1	2885	6928.19	6932.63	2.57		6933.13	0.004393	5.68	507.49	163.26	0.57
Lower	46	PF 1	2958	6927.69	6932.18	2.62		6932.69	0.004395	5.73	515.8	163.83	0.57
Lower	45	PF 1	3032	6927.19	6931.73	2.67		6932.25	0.0044	5.79	524.05	164.4	0.57
Lower	44	PF 1	3105	6926.69	6931.28	2.72		6931.81	0.004396	5.83	532.49	164.97	0.57
Lower	43	PF 1	3179	6926.19	6930.83	2.77		6931.37	0.004393	5.88	540.97	165.55	0.57
Lower	42	PF 1	3252	6925.69	6930.39	2.83		6930.93	0.004372	5.91	549.95	166.16	0.57
Lower	41	PF 1	3326	6925.19	6929.95	2.89		6930.49	0.004335	5.94	559.79	166.82	0.57
Lower	40	PF 1	3399	6924.69	6929.51	2.95		6930.06	0.004256	5.95	571.3	167.6	0.57
Lower	39	PF 1	3473	6924.19	6929.1	3.04		6929.64	0.004133	5.94	585.17	168.53	0.56
Lower	38	PF 1	3546	6923.69	6928.7	3.14		6929.24	0.003942	5.88	602.68	169.69	0.55
Lower	37	PF 1	3620	6923.19	6928.33	3.27		6928.85	0.003681	5.79	625.06	171.17	0.53
Lower	36	PF 1	3693	6922.69	6928	3.44		6928.49	0.003344	5.65	653.97	173.06	0.51
Lower	35	PF 1	3767	6922.19	6927.7	3.64		6928.17	0.003003	5.54	679.97	170.73	0.49
Lower	34	PF 1	4225	6921.69	6927.25	3.69		6927.83	0.003641	6.14	688.32	171.22	0.54
Lower	33	PF 1	4391	6921.19	6926.88	3.82		6927.47	0.003569	6.18	710.9	172.53	0.54
Lower	32	PF 1	4851	6920.69	6926.29	3.73		6927.04	0.004638	6.97	696.19	171.68	0.61
Lower	31	PF 1	4851	6920.44	6926.07	3.76		6926.81	0.004554	6.93	700.47	171.92	0.6
Lower	30	PF 1	4930	6920.18	6925.83	3.76		6926.58	0.004578	6.96	708.44	173.32	0.61
Lower	29	PF 1	5088	6919.66	6925.36	3.77		6926.12	0.004595	7.01	725.89	176.19	0.61
Lower	28	PF 1	5247	6919.14	6924.88	3.77		6925.66	0.004618	7.06	743.06	179.03	0.61
Lower	27	PF 1	5405	6918.62	6924.41	3.78		6925.19	0.00464	7.11	760.03	181.86	0.61

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
HEC-RAS CHANNEL OUTPUT DATA

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Depth above Right Channel Bottom (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/sec)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
Lower	26	PF 1	5563	6918.1	6923.94	3.79		6924.73	0.004636	7.15	778.29	184.71	0.61
Lower	25	PF 1	5721	6917.58	6923.46	3.79		6924.26	0.004651	7.19	795.55	187.53	0.62
Lower	24	PF 1	5879	6917.06	6922.98	3.79		6923.8	0.004663	7.23	812.87	190.35	0.62
Lower	23	PF 1	6037	6916.54	6922.51	3.80		6923.33	0.004677	7.27	829.93	193.15	0.62
Lower	22	PF 1	6195	6916.02	6922.03	3.80		6922.86	0.004692	7.31	846.94	195.94	0.62
Lower	21	PF 1	6353	6915.5	6921.55	3.80		6922.39	0.004708	7.35	863.77	198.71	0.62
Lower	20	PF 1	6511	6914.98	6921.07	3.80		6921.92	0.004729	7.4	880.22	201.47	0.62
Lower	19	PF 1	6669	6914.46	6920.58	3.79		6921.44	0.004761	7.44	895.97	204.18	0.63
Lower	18	PF 1	6828	6913.95	6920.08	3.78		6920.96	0.004856	7.52	907.96	206.81	0.63
Lower	17	PF 1	6986	6913.43	6919.56	3.74		6920.46	0.004989	7.62	917.26	209.21	0.64
Lower	16	PF 1	7144	6912.91	6919	3.66		6919.94	0.005269	7.78	918.02	211.21	0.66
Lower	15	PF 1	7302	6912.39	6918.34	3.48		6919.37	0.005986	8.14	896.8	212.19	0.7
Lower	14	PF 1	7506	6911.87	6917.73	3.35		6918.85	0.004425	8.51	882.16	212.92	0.74
Lower	13	PF 1	7710	6911.36	6917.24	3.35		6918.4	0.004529	8.63	893.62	215.02	0.75
Lower	12	PF 1	7914	6910.84	6916.73	3.32		6917.93	0.004692	8.78	900.93	216.78	0.76
Lower	11	PF 1	8119	6910.33	6916.01	3.09		6917.39	0.0057	9.42	862.24	216.59	0.83
Lower	10	PF 1	8323	6909.7	6915.73	3.30		6916.82	0.004153	8.36	995.24	236.09	0.72
Lower	9	PF 1	8527	6909.2	6915.29	3.34		6916.4	0.004186	8.46	1008.52	236.65	0.72
Lower	8	PF 1	8731	6908.7	6914.82	3.36		6915.97	0.00428	8.59	1016.73	236.99	0.73
Lower	7	PF 1	8935	6908.2	6914.33	3.35		6915.53	0.00445	8.77	1019.16	237.1	0.75
Lower	6	PF 1	9140	6907.7	6913.23	2.74	6913.05	6914.91	0.007367	10.41	878.11	231.07	0.94
Lower	5	PF 1	9344	6906.86	6912.4	2.73	6912.27	6914.15	0.007643	10.62	880.19	231.16	0.96
Lower	4	PF 1	9548	6906.02	6911.7	2.86	6911.48	6913.4	0.00714	10.46	912.41	232.51	0.93
Lower	3	PF 1	9752	6905.18	6911.2	3.19		6912.7	0.005722	9.81	994.03	235.99	0.84
Lower	2	PF 1	9752	6904.33	6910.67	3.49		6911.97	0.008048	9.13	1068.32	239.16	0.76
Lower	1	PF 1	9752	6903.49	6909.3	2.96	6909.03	6910.96	0.011833	10.34	943.04	233.86	0.91

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
HEC-RAS ROCK SIZE & SCOUR DEPTH

Station Location		Abt and Johnson Method $D_{50}=5.23(S)^{0.43}(q_d)^{0.56}$		Safety Factors Method (peak shear stress) D50=(t)/(a(Ws-Ww))=t/4.1 where a=0.04, Ws=165 pcf, Ww=62.4 pcf			USACE Equilibrium Scour Depth $h_e=(0.234*q_e^{8/9})/[g(Ss-1)]^{4/9}d_e^{1/3}$ (ft per unit width)				
River Station	Station Number	q_d = Design Unit Width Peak Discharge (cfs/ft)	$D_{50} = 5.23\ (S)^{0.43}\ (q_d)^{0.56}$ (inches)	y = Average Depth (ft)	Actual Shear Stress: $t = W_w yS$	$D_{50} = 12\ t / 4.1$ (inches)	he = Equilibrium Scour Depth (ft)	qe = equilibrium discharge (cfs/ft)	g (ft/sec2)	Ss = Sediment Specific Gravity	de = median grain-size diameter (mm)
	800.00										
49.00	1025.00	16.99	2.47	3.04	0.828	2.42	1.18	16.99	32.2	2.65	0.074
48.00	1100.00	17.28	2.50	3.07	0.839	2.46	1.20	17.28	32.2	2.65	0.074
47.00	1200.00	17.67	2.53	3.11	0.852	2.49	1.22	17.67	32.2	2.65	0.074
46.00	1300.00	18.06	2.56	3.15	0.863	2.53	1.25	18.06	32.2	2.65	0.074
45.00	1400.00	18.44	2.59	3.19	0.875	2.56	1.27	18.44	32.2	2.65	0.074
44.00	1500.00	18.82	2.62	3.23	0.885	2.59	1.30	18.82	32.2	2.65	0.074
43.00	1600.00	19.20	2.65	3.27	0.896	2.62	1.32	19.20	32.2	2.65	0.074
42.00	1700.00	19.57	2.67	3.31	0.903	2.64	1.34	19.57	32.2	2.65	0.074
41.00	1800.00	19.94	2.69	3.36	0.908	2.66	1.36	19.94	32.2	2.65	0.074
40.00	1900.00	20.28	2.70	3.41	0.905	2.65	1.38	20.28	32.2	2.65	0.074
39.00	2000.00	20.61	2.69	3.47	0.895	2.62	1.40	20.61	32.2	2.65	0.074
38.00	2100.00	20.90	2.65	3.55	0.874	2.56	1.42	20.90	32.2	2.65	0.074
37.00	2200.00	21.15	2.59	3.65	0.839	2.45	1.44	21.15	32.2	2.65	0.074
36.00	2300.00	21.34	2.50	3.78	0.789	2.31	1.45	21.34	32.2	2.65	0.074
35.00	2400.00	22.06	2.43	3.98	0.746	2.18	1.49	22.06	32.2	2.65	0.074
34.00	2500.00	24.68	2.82	4.02	0.913	2.67	1.65	24.68	32.2	2.65	0.074
33.00	2600.00	25.45	2.84	4.12	0.918	2.69	1.69	25.45	32.2	2.65	0.074
32.00	2700.00	28.26	3.37	4.06	1.174	3.43	1.86	28.26	32.2	2.65	0.074
31.00	2750.00	28.22	3.34	4.07	1.158	3.39	1.86	28.22	32.2	2.65	0.074
30.00	2800.00	28.44	3.36	4.09	1.168	3.42	1.87	28.44	32.2	2.65	0.074
29.00	2900.00	28.88	3.40	4.12	1.181	3.46	1.89	28.88	32.2	2.65	0.074
28.00	3000.00	29.31	3.43	4.15	1.196	3.50	1.92	29.31	32.2	2.65	0.074
27.00	3100.00	29.72	3.47	4.18	1.210	3.54	1.94	29.72	32.2	2.65	0.074

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
HEC-RAS ROCK SIZE & SCOUR DEPTH

Station Location		Abt and Johnson Method $D_{50}=5.23(S)^{0.43}(q_d)^{0.56}$		Safety Factors Method (peak shear stress) D50=(t)/(a(Ws-Ww))=t/4.1 where a=0.04, Ws=165 pcf, Ww=62.4 pcf			USACE Equilibrium Scour Depth $h_e=(0.234*q_e^{8/9})/[g(Ss-1)]^{4/9}d_e^{1/3}$ (ft per unit width)				
River Station	Station Number	q_d = Design Unit Width Peak Discharge (cfs/ft)	$D_{50} = 5.23 (S)^{0.43} (q_d)^{0.56}$ (inches)	y = Average Depth (ft)	Actual Shear Stress: $t = W_w yS$	$D_{50} = 12 t / 4.1$ (inches)	he = Equilibrium Scour Depth (ft)	qe = equilibrium discharge (cfs/ft)	g (ft/sec2)	Ss = Sediment Specific Gravity	de = median grain-size diameter (mm)
26.00	3200.00	30.12	3.49	4.21	1.219	3.57	1.97	30.12	32.2	2.65	0.074
25.00	3300.00	30.51	3.52	4.24	1.231	3.60	1.99	30.51	32.2	2.65	0.074
24.00	3400.00	30.89	3.55	4.27	1.243	3.64	2.01	30.89	32.2	2.65	0.074
23.00	3500.00	31.26	3.58	4.30	1.254	3.67	2.03	31.26	32.2	2.65	0.074
22.00	3600.00	31.62	3.61	4.32	1.266	3.70	2.05	31.62	32.2	2.65	0.074
21.00	3700.00	31.97	3.63	4.35	1.277	3.74	2.07	31.97	32.2	2.65	0.074
20.00	3800.00	32.32	3.66	4.37	1.289	3.77	2.09	32.32	32.2	2.65	0.074
19.00	3900.00	32.66	3.70	4.39	1.304	3.82	2.11	32.66	32.2	2.65	0.074
18.00	4000.00	33.02	3.75	4.39	1.330	3.89	2.13	33.02	32.2	2.65	0.074
17.00	4100.00	33.39	3.82	4.38	1.365	3.99	2.16	33.39	32.2	2.65	0.074
16.00	4200.00	33.82	3.94	4.35	1.429	4.18	2.18	33.82	32.2	2.65	0.074
15.00	4300.00	34.41	4.20	4.23	1.579	4.62	2.21	34.41	32.2	2.65	0.074
14.00	4400.00	35.25	3.74	4.14	1.144	3.35	2.26	35.25	32.2	2.65	0.074
13.00	4500.00	35.86	3.81	4.16	1.175	3.44	2.30	35.86	32.2	2.65	0.074
12.00	4600.00	36.51	3.91	4.16	1.217	3.56	2.33	36.51	32.2	2.65	0.074
11.00	4700.00	37.49	4.31	3.98	1.416	4.14	2.39	37.49	32.2	2.65	0.074
10.00	4800.00	35.25	3.64	4.22	1.092	3.20	2.26	35.25	32.2	2.65	0.074
9.00	4900.00	36.03	3.70	4.26	1.113	3.26	2.31	36.03	32.2	2.65	0.074
8.00	5000.00	36.84	3.78	4.29	1.146	3.35	2.35	36.84	32.2	2.65	0.074
7.00	5100.00	37.68	3.89	4.30	1.194	3.49	2.40	37.68	32.2	2.65	0.074
6.00	5200.00	39.56	4.96	3.80	1.747	5.11	2.51	39.56	32.2	2.65	0.074
5.00	5300.00	40.42	5.11	3.81	1.816	5.32	2.56	40.42	32.2	2.65	0.074
4.00	5400.00	41.06	5.00	3.92	1.748	5.12	2.59	41.06	32.2	2.65	0.074
3.00	5500.00	41.32	4.56	4.21	1.504	4.40	2.61	41.32	32.2	2.65	0.074
2.00	5600.00	40.78	5.25	4.47	2.243	6.57	2.58	40.78	32.2	2.65	0.074
1.00	5700.00	41.70	6.27	4.03	2.978	8.71	2.63	41.70	32.2	2.65	0.074

Appendix C.3 .....Calculation C.3 Interior Diversion Channel

Calculation C.3
Rio Algom Mining LLC
Ambrosia Lake, New Mexico
Arroyo Del Puerto
Interior Site Drainage Calculations
Interior Diversion Channel

1. The Interior Diversion Channel enters the Interior Drainage Channel at approximate Station 25+00 at an oblique angle. The slope of the interior diversion channel at the confluence is 0.9%. This channel is trapezoidal with a 28-foot channel bottom and carries 497 cfs. This channel varies in slope along its length. It has a hydraulic jump control section at Station 17+00 to 17+25. The slope levels out as it approaches the confluence with the interior drainage channel. The downstream slope of the interior drainage channel is 0.5% but the oblique angle of entrance combined with the channel bottom slope of the interior drainage channel creates a slope of 1.183%. The general concern for an entering flow is if the slope at the entrance decreases substantially causing a hydraulic jump to form going from supercritical flow to subcritical flow. Since the entrance slope of the interior diversion channel increases, the flow is not expected to have a hydraulic jump.
   200 feet of the interior diversion channel and 200 feet of the interior drainage channel are modeled with HEC-RAS assuming the interior diversion channel maintains its shape.
   River station 0 is considered to be the edge of the interior drainage channel as the interior diversion enters it.
   The model is run separately for subcritical flow, supercritical flow, and mixed flow. The results indicate that the flow remains in the subcritical flow regime.

2. Interior Drainage Channel Stationing and Elevation Summary -
   see Worksheet Tab "Elevation Summary" (calc Sheets 3 of 16)

3. HEC-RAS Output Results -
   see Worksheet Tab "HC-RAS Output" (calc. Sheet 4 of 16) for the HEC-RAS profile output results of the channel stationing for subcritical flow, supercritical flow, and mixed flow regimes.

   Interior Diversion Channel HEC-RAS Erosion Protection and Scour Depth specific calculations -
   see Worksheet Tab "HEC-RAS Rock Scour" (calc. Sheet 5 of 16) ffor the rock size and scour depth derived from channel parameters from the HEC-RAS modeling, rock size based on both the Abt and Johnson method and the Safety Factors Method (peak shear stress),
   and scour depth based on the USACE Equilibrium Scour Depth method.

4. Profile Plots
   see Worksheet Tab "Profile Plots" (calc Sheets 6 thru 7 of 16) for a visual illustration of the profiles of subcritical flow, supercritical flow, and mixed flow regimes.

5. PMF X-Sections
   see Worksheet Tab "PMF X-Sections" (calc Sheets 8 thru 16 of 16) for a visual illustration of the x-sections of the stationinf for subcritical flow, supercritical flow, and mixed flow regimes.

9 Conclusions:

a) The resulting rock size of D_{50}=7.8-inch used in the interior drainage channel is adequate protection from the entering flow of the interior diversion channel.
b) Scour depths at the confluence with the interior drainage channel due to the flow from the interior diversion channel are generally less than 1.0 feet.

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
INTERIOR DIVERSION CHANNEL STATIONING AND ELEVATION SUMMARY

Station Description	River Station #	Beginning Station (ft)	Reach Between Stations (ft)	Slope Between Stations(ft /ft)	Channel Elev Drop Downstream Reach (ft)	Fixed design Channel Bottom Widths (ft)	Channel Bottom Elev at Station (ft)	Bank Height (ft)	Bank Elevation (ft)
Diversion Channel Stationing	2.00	25+00	50	0.9%	0.45	28	6925.61	4	6929.61
Diversion Channel Stationing	1.50	25+50	50	0.9%	0.45	28	6925.16	4	6929.16
Diversion Channel Stationing	1.00	26+00	50	0.9%	0.45	28	6924.71	4	6928.71
Diversion Channel Stationing	0.50	26+50	50	0.9%	0.45	28	6924.26	4	6928.26
Confluence - Edge of Interior Channel	0.00	27+00	50	1.183%	0.59	28	6923.81	4	6927.81
Interior Channel Slope - Diversion Channel X-Sect	-0.50	27+50	50	1.183%	0.59	28	6923.22	4	6927.22
Interior Channel Slope - Diversion Channel X-Sect	-1.00	28+00	50	1.183%	0.59	28	6922.63	4	6926.63
Interior Channel Slope - Diversion Channel X-Sect	-1.50	28+50	50	1.183%	0.59	28	6922.04	4	6926.04
Interior Channel Slope - Diversion Channel X-Sect	-2.00	29+00	50			28	6921.45	4	6925.45

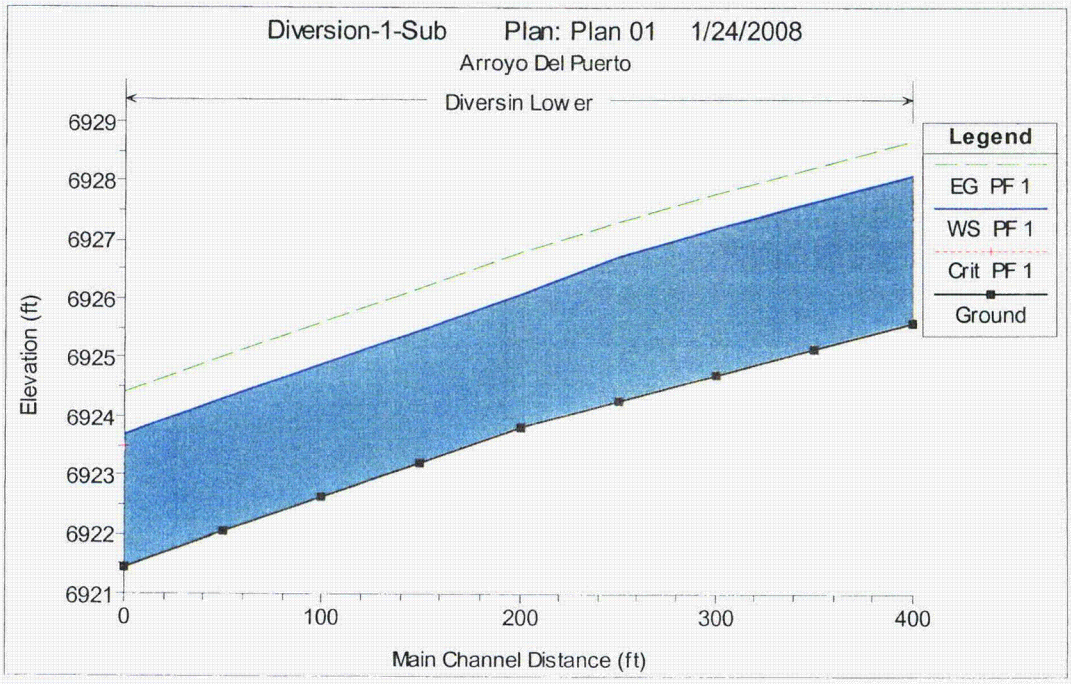
RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
HEC-RAS INTERIOR DIVERSION CHANNEL OUTPUT DATA

Reach	River Station	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/sec)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
Subcritical Flow Regime Results												
Lower	2.00	PF 1	497	6925.61	6928.09	0	6928.66	0.008996	6.09	81.59	37.9	0.73
Lower	1.50	PF 1	497	6925.16	6927.64	0	6928.21	0.008996	6.09	81.59	37.9	0.73
Lower	1.00	PF 1	497	6924.71	6927.19	0	6927.76	0.009015	6.1	81.54	37.9	0.73
Lower	0.50	PF 1	497	6924.26	6926.73	0	6927.31	0.009101	6.11	81.28	37.87	0.74
Lower	0.00	PF 1	497	6923.81	6926.06	0	6926.78	0.011801	6.78	73.25	37.01	0.85
Lower	-0.50	PF 1	497	6923.22	6925.47	0	6926.19	0.011801	6.78	73.25	37.01	0.85
Lower	-1.00	PF 1	497	6922.63	6924.88	0	6925.6	0.011792	6.78	73.27	37.02	0.85
Lower	-1.50	PF 1	497	6922.04	6924.29	0	6925.01	0.011801	6.78	73.25	37.01	0.85
Lower	-2.00	PF 1	497	6921.45	6923.7	6923.48	6924.42	0.011836	6.79	73.18	37.01	0.85
Supercrital Flow Regime Results												
Lower	2.00	PF 1	497	6925.61	6927.63	6927.63	6928.55	0.017993	7.66	64.88	36.1	1.01
Lower	1.50	PF 1	497	6925.16	6927.19	6927.19	6928.1	0.017801	7.63	65.11	36.12	1
Lower	1.00	PF 1	497	6924.71	6926.74	6926.74	6927.65	0.017904	7.65	64.99	36.11	1
Lower	0.50	PF 1	497	6924.26	6926.29	6926.29	6927.2	0.01783	7.64	65.07	36.12	1
Lower	0.00	PF 1	497	6923.81	6925.84	6925.84	6926.75	0.016997	7.65	64.95	36.11	1.01
Lower	-0.50	PF 1	497	6923.22	6925.25	6925.25	6926.16	0.016983	7.65	64.97	36.11	1.01
Lower	-1.00	PF 1	497	6922.63	6924.66	6924.66	6925.57	0.016969	7.65	64.99	36.11	1
Lower	-1.50	PF 1	497	6922.04	6924.07	6924.07	6924.98	0.016899	7.64	65.07	36.12	1
Lower	-2.00	PF 1	497	6921.45	6923.48	6923.48	6924.39	0.016997	7.65	64.95	36.11	1.01
Mixed Flow Regime Results												
Lower	2.00	PF 1	497	6925.61	6928.09	6927.64	6928.66	0.008996	6.09	81.59	37.9	0.73
Lower	1.50	PF 1	497	6925.16	6927.64	0	6928.21	0.008996	6.09	81.59	37.9	0.73
Lower	1.00	PF 1	497	6924.71	6927.19	0	6927.76	0.009015	6.1	81.54	37.9	0.73
Lower	0.50	PF 1	497	6924.26	6926.73	0	6927.31	0.009101	6.11	81.28	37.87	0.74
Lower	0.00	PF 1	497	6923.81	6926.06	0	6926.78	0.011801	6.78	73.25	37.01	0.85
Lower	-0.50	PF 1	497	6923.22	6925.47	0	6926.19	0.011801	6.78	73.25	37.01	0.85
Lower	-1.00	PF 1	497	6922.63	6924.88	0	6925.6	0.011792	6.78	73.27	37.02	0.85
Lower	-1.50	PF 1	497	6922.04	6924.29	0	6925.01	0.011801	6.78	73.25	37.01	0.85
Lower	-2.00	PF 1	497	6921.45	6923.7	6923.48	6924.42	0.011836	6.79	73.18	37.01	0.85

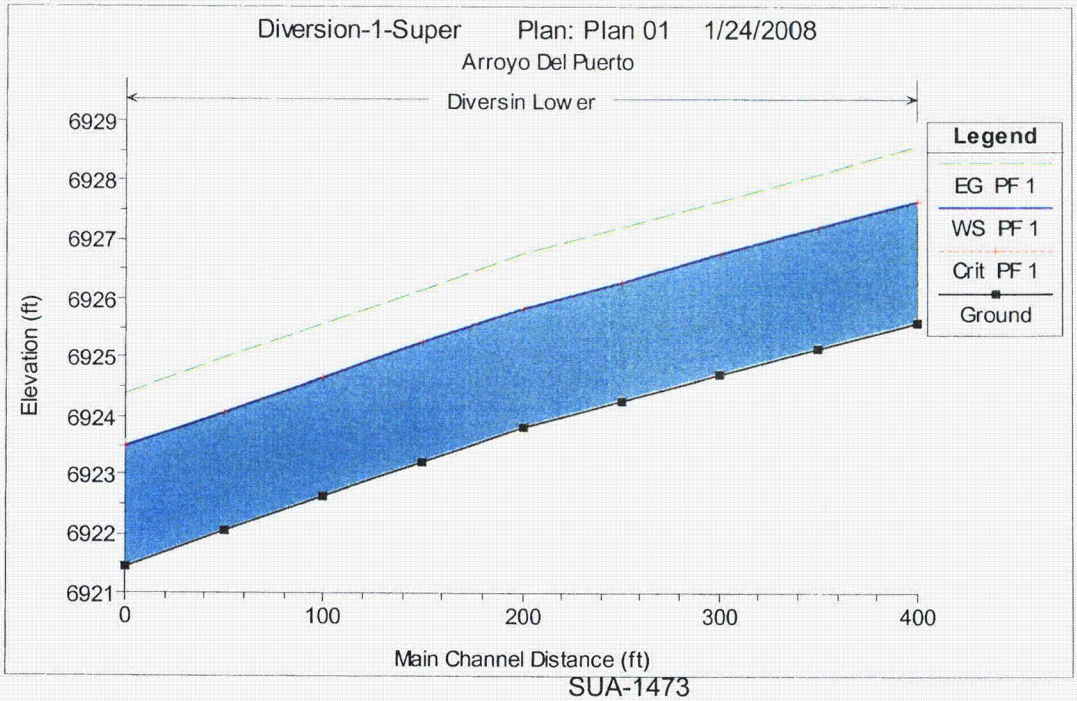
RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
INTERIOR DIVERSION CHANNEL - HEC-RAS ROCK SIZE & SCOUR DEPTH

Station Location		Abt and Johnson Method $D_{50}=5.23(S)^{0.43}(q_d)^{0.56}$		Safety Factors Method (peak shear stress) $D50=(t)/(a(Ws-Ww))=t/4.1$ where a=0.04, Ws=165 pcf, Ww=62.4 pcf			USACE Equilibrium Scour Depth $h_e=(0.234*q_e^{8/9})/[g(Ss-1)]^{4/9}d_e^{1/3}$ (ft per unit width)				
River Station	Station Number	q_d = Design Unit Width Peak Discharge (cfs/ft)	$D_{50} = 5.23 (S)^{0.43} (q_d)^{0.56}$ (inches)	y = Average Depth (ft)	Actual Shear Stress: $t = W_w yS$	$D_{50} = 12\ t / 4.1$ (inches)	he = Equilibrium Scour Depth (ft)	qe = equilibrium discharge (cfs/ft)	g (ft/sec2)	Ss = Sediment Specific Gravity	de = median grain-size diameter (mm)
Subcritical Flow Regime Results											
2.00	25+00	13.11	2.92	2.15	1.208	3.54	0.94	13.11	32.2	2.65	0.074
1.50	25+50	13.11	2.92	2.15	1.208	3.54	0.94	13.11	32.2	2.65	0.074
1.00	26+00	13.11	2.92	2.15	1.210	3.54	0.94	13.11	32.2	2.65	0.074
0.50	26+50	13.12	2.93	2.15	1.219	3.57	0.94	13.12	32.2	2.65	0.074
0.00	27+00	13.43	3.32	1.98	1.457	4.27	0.96	13.43	32.2	2.65	0.074
-0.50	27+50	13.43	3.32	1.98	1.457	4.27	0.96	13.43	32.2	2.65	0.074
-1.00	28+00	13.43	3.32	1.98	1.456	4.26	0.96	13.43	32.2	2.65	0.074
-1.50	28+50	13.43	3.32	1.98	1.457	4.27	0.96	13.43	32.2	2.65	0.074
-2.00	29+00	13.43	3.32	1.98	1.460	4.27	0.96	13.43	32.2	2.65	0.074
Supercrital Flow Regime Results											
2.00	25+00	13.77	4.04	1.80	2.018	5.91	0.98	13.77	32.2	2.65	0.074
1.50	25+50	13.76	4.02	1.80	2.002	5.86	0.98	13.76	32.2	2.65	0.074
1.00	26+00	13.76	4.03	1.80	2.011	5.89	0.98	13.76	32.2	2.65	0.074
0.50	26+50	13.76	4.02	1.80	2.004	5.87	0.98	13.76	32.2	2.65	0.074
0.00	27+00	13.76	3.94	1.80	1.908	5.58	0.98	13.76	32.2	2.65	0.074
-0.50	27+50	13.76	3.94	1.80	1.907	5.58	0.98	13.76	32.2	2.65	0.074
-1.00	28+00	13.76	3.93	1.80	1.906	5.58	0.98	13.76	32.2	2.65	0.074
-1.50	28+50	13.76	3.93	1.80	1.900	5.56	0.98	13.76	32.2	2.65	0.074
-2.00	29+00	13.76	3.94	1.80	1.908	5.58	0.98	13.76	32.2	2.65	0.074
Mixed Flow Regime Results											
2.00	25+00	13.11	2.92	2.15	1.208	3.54	0.94	13.11	32.2	2.65	0.074
1.50	25+50	13.11	2.92	2.15	1.208	3.54	0.94	13.11	32.2	2.65	0.074
1.00	26+00	13.11	2.92	2.15	1.210	3.54	0.94	13.11	32.2	2.65	0.074
0.50	26+50	13.12	2.93	2.15	1.219	3.57	0.94	13.12	32.2	2.65	0.074
0.00	27+00	13.43	3.32	1.98	1.457	4.27	0.96	13.43	32.2	2.65	0.074
-0.50	27+50	13.43	3.32	1.98	1.457	4.27	0.96	13.43	32.2	2.65	0.074
-1.00	28+00	13.43	3.32	1.98	1.456	4.26	0.96	13.43	32.2	2.65	0.074
-1.50	28+50	13.43	3.32	1.98	1.457	4.27	0.96	13.43	32.2	2.65	0.074
-2.00	29+00	13.43	3.32	1.98	1.460	4.27	0.96	13.43	32.2	2.65	0.074

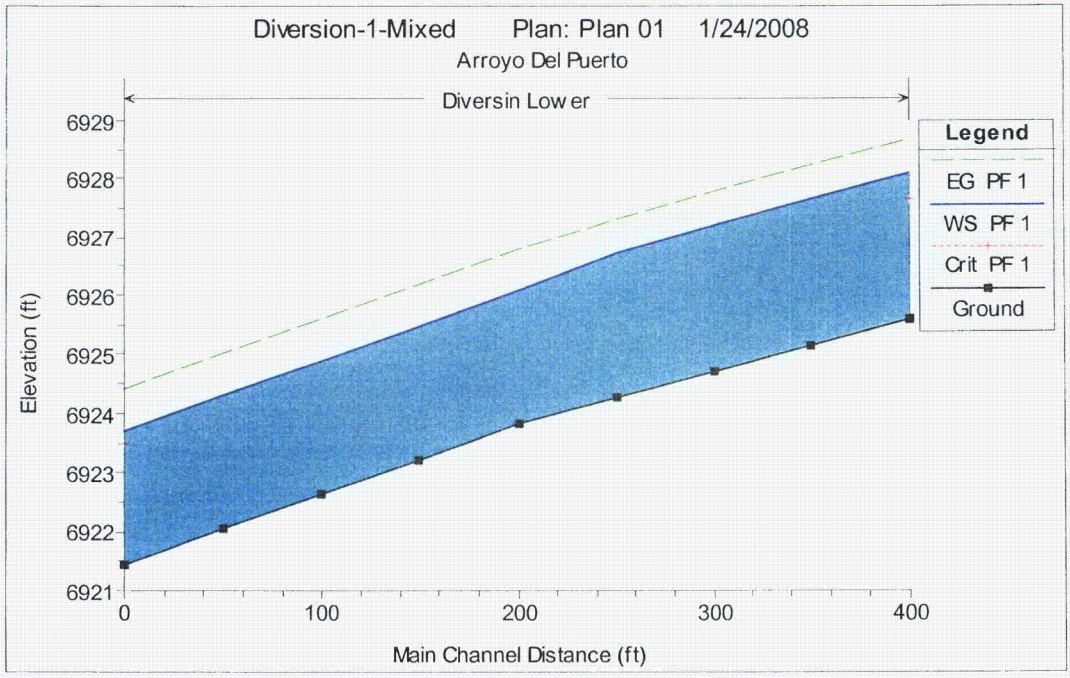
Diversion Channel - Subcritical Profile Plots



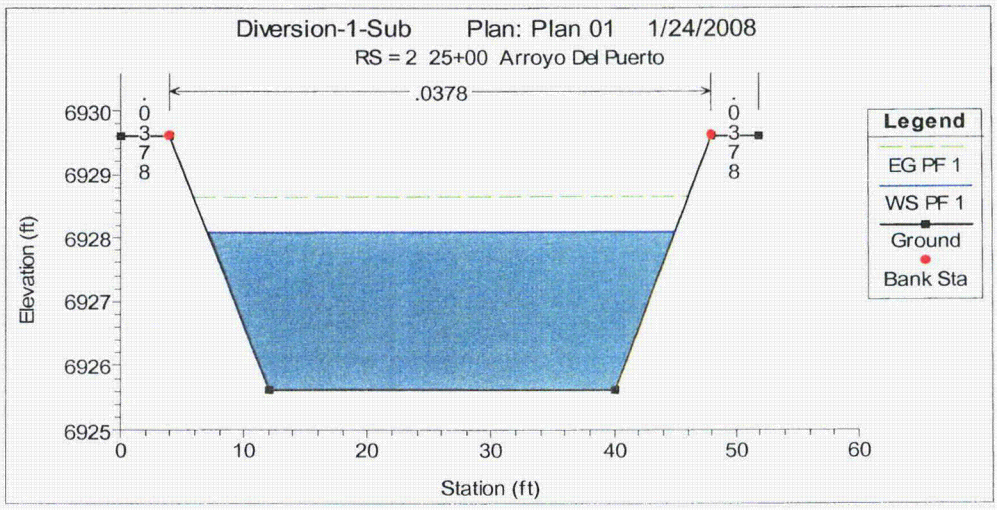
Diversion Channel - Supercritical Profile Plots

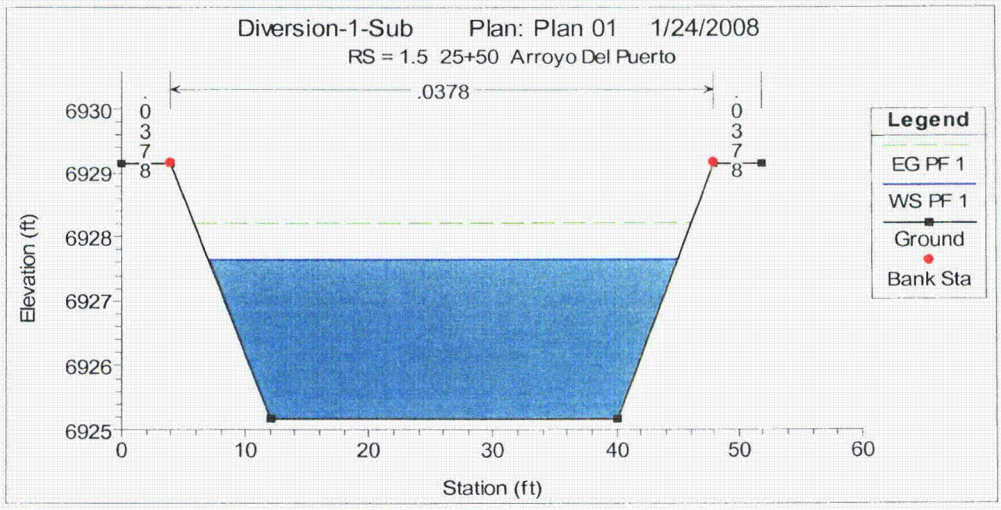


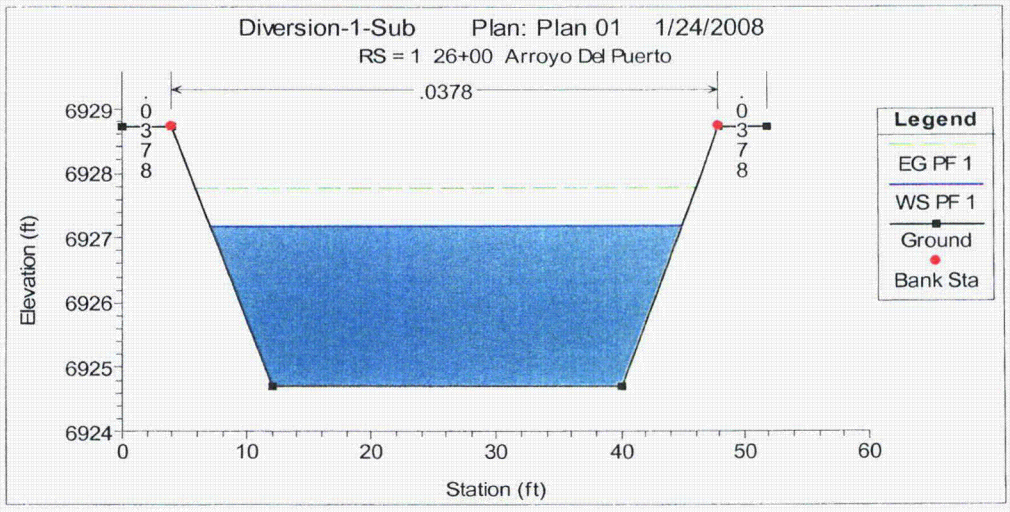
Diversion Channel - Mixed Profile Plots



Diversion Channel - Subcritical X-Section Plots







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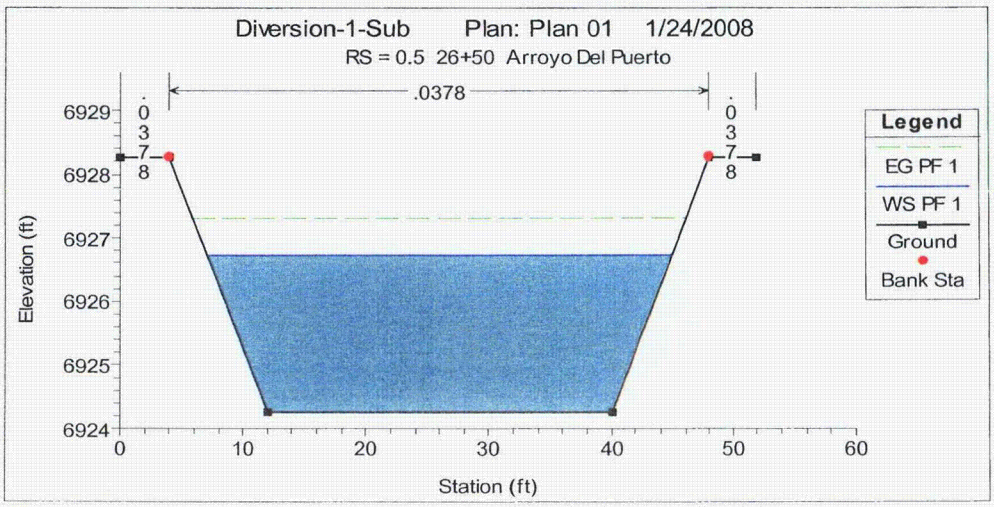
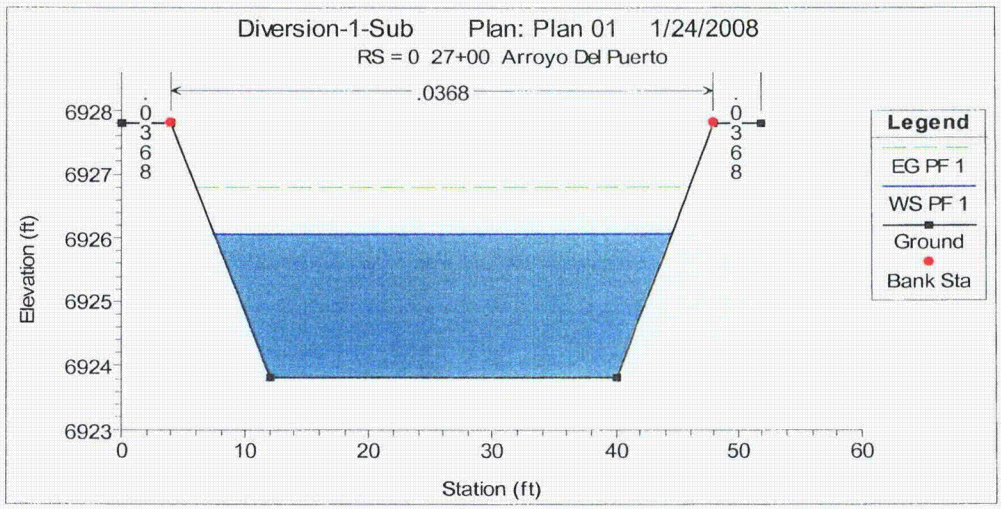
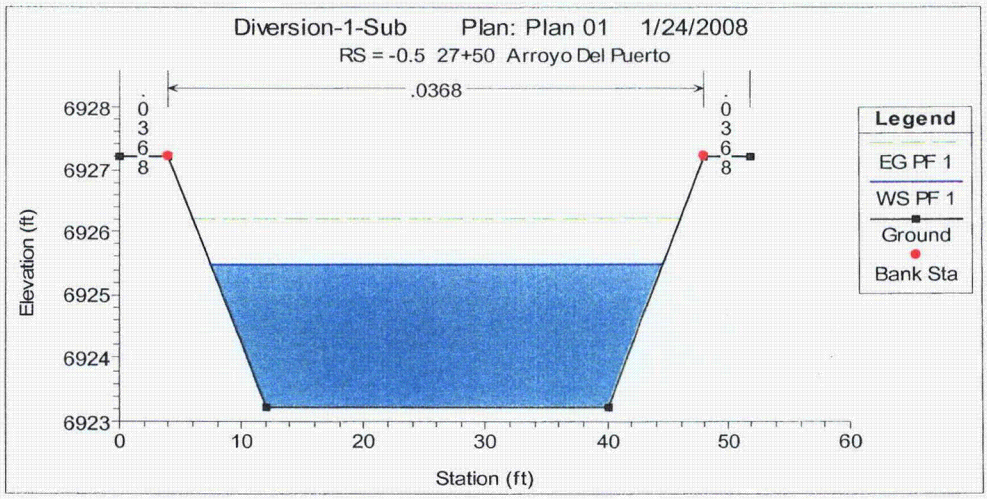
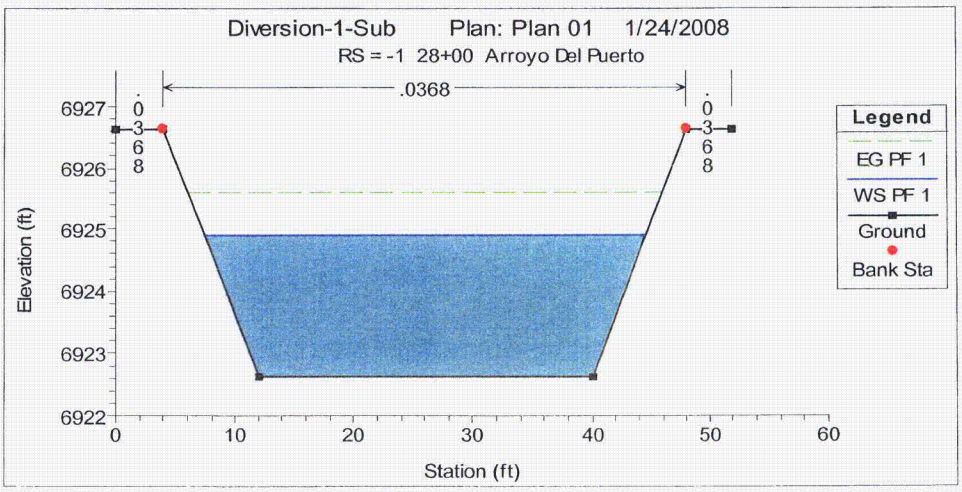
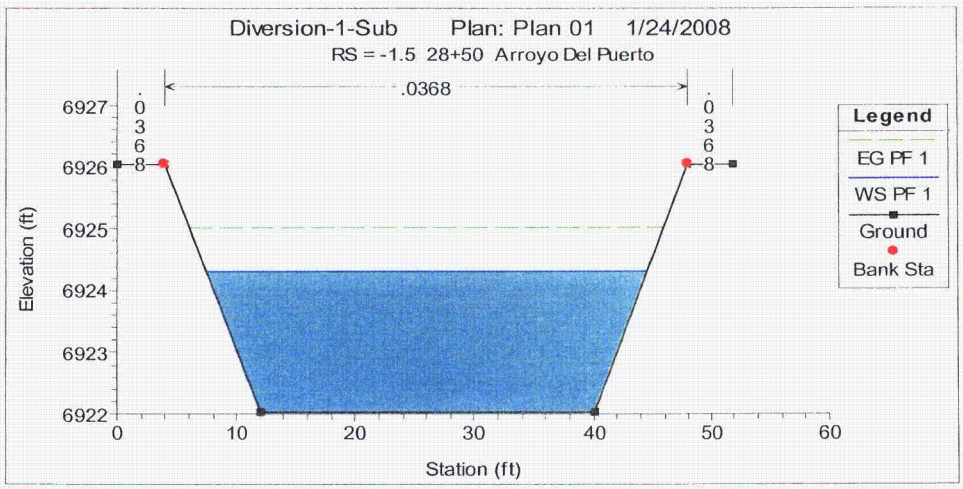


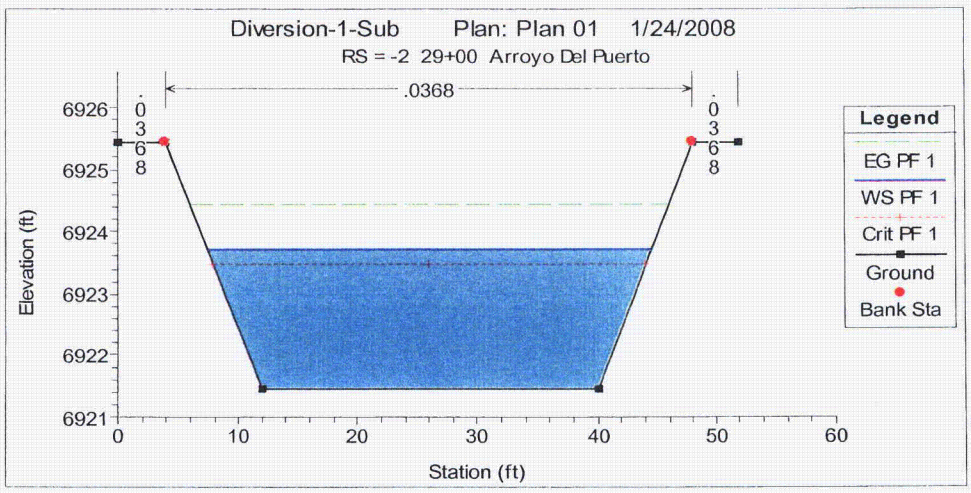




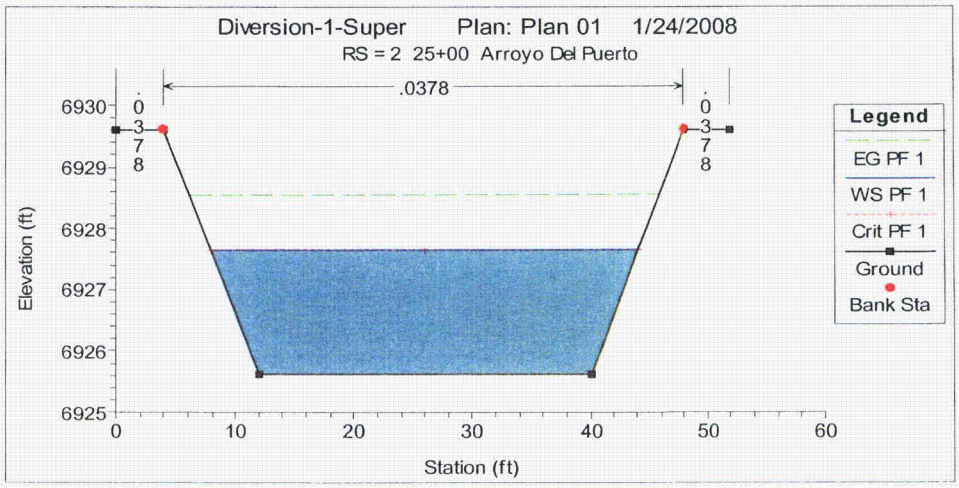
Diversion Channel - Subcritical X-Section Plots

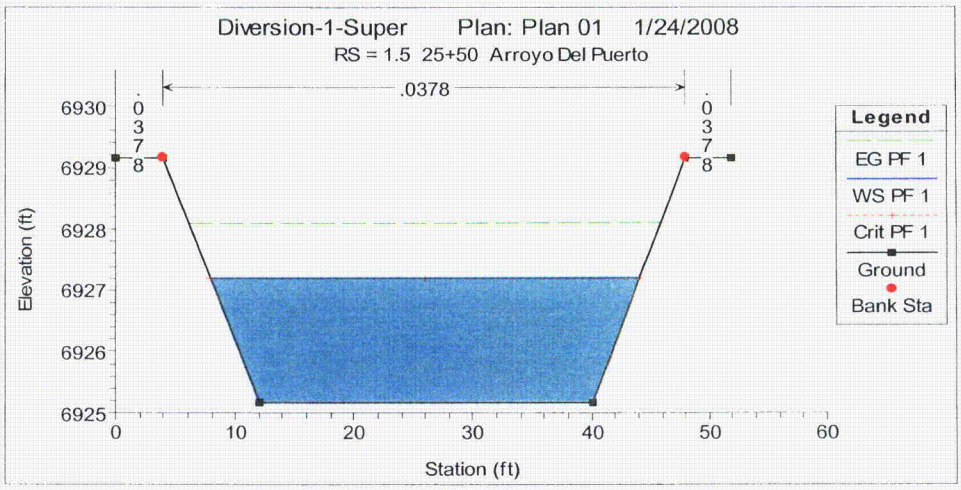


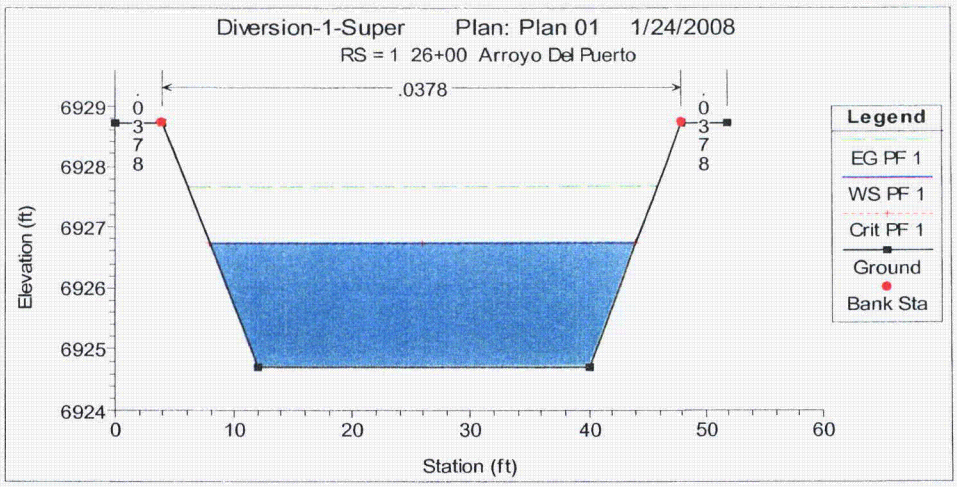




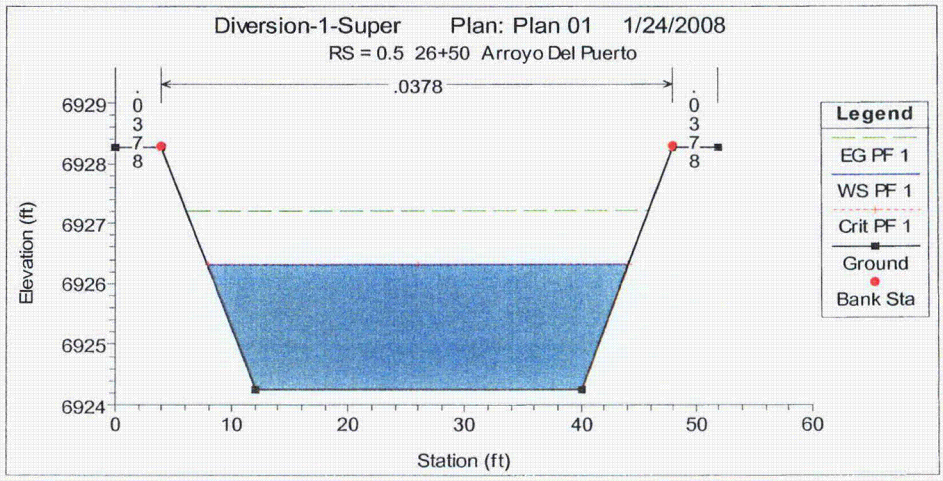
Diversion Channel - Supercritical X-Section Plots

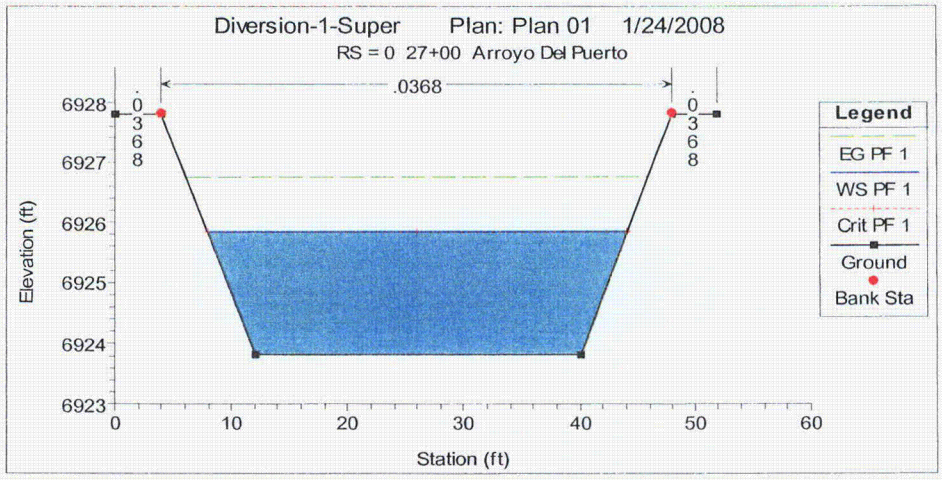


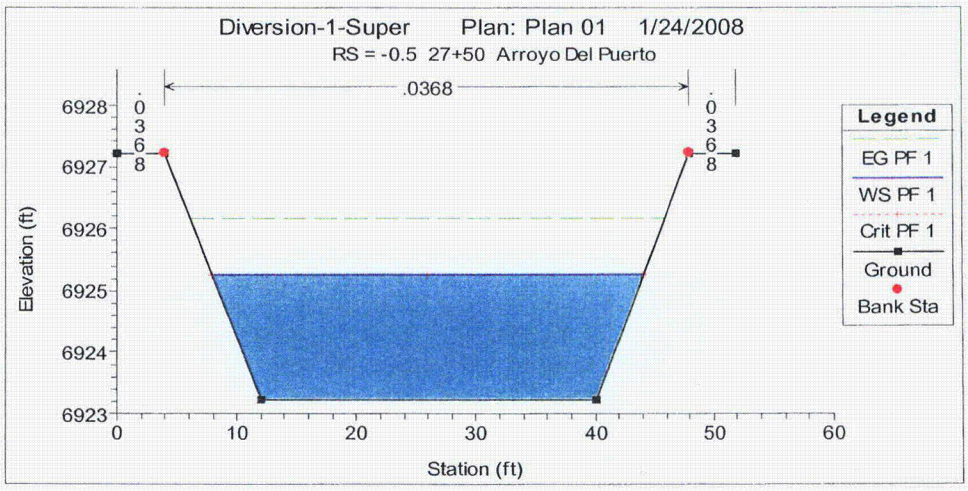




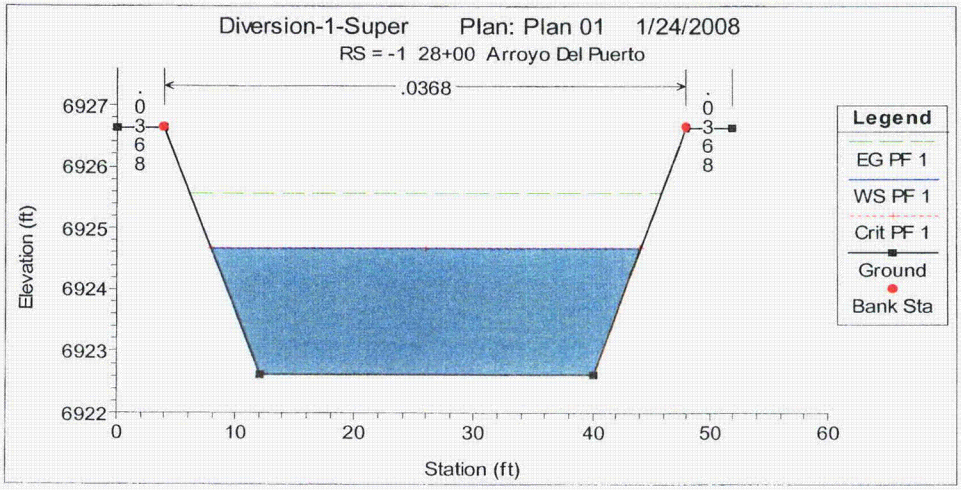
Diversion Channel - Supercritical X-Section Plots

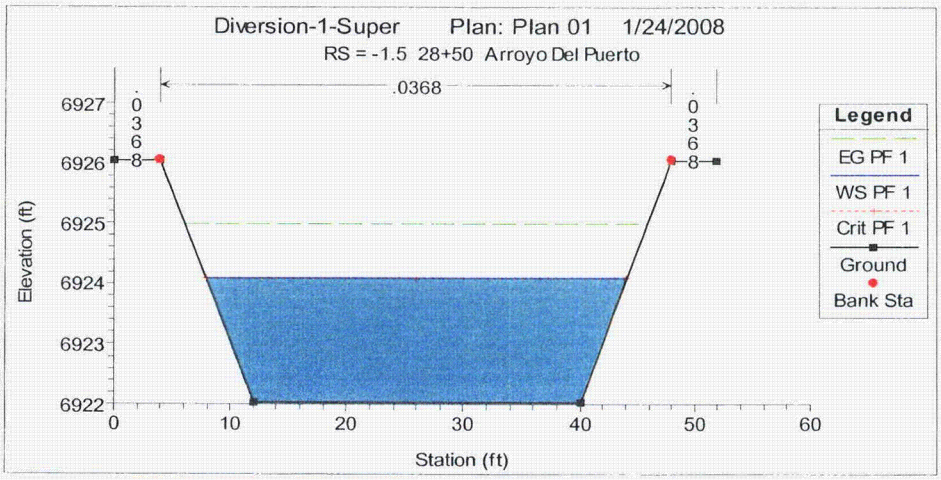


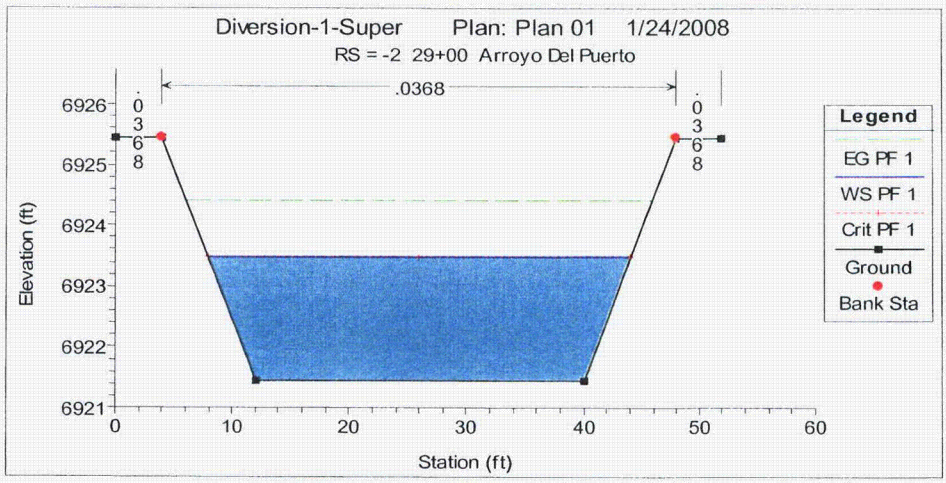




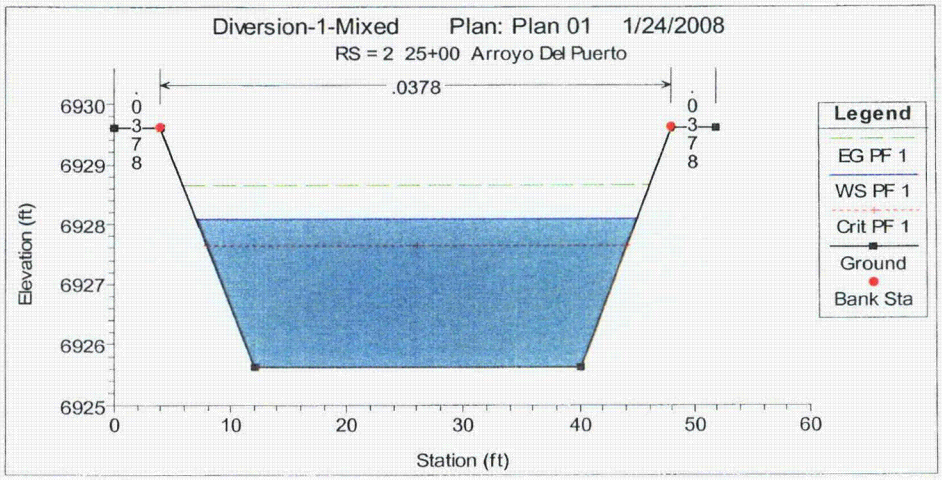
Diversion Channel - Supercritical X-Section Plots

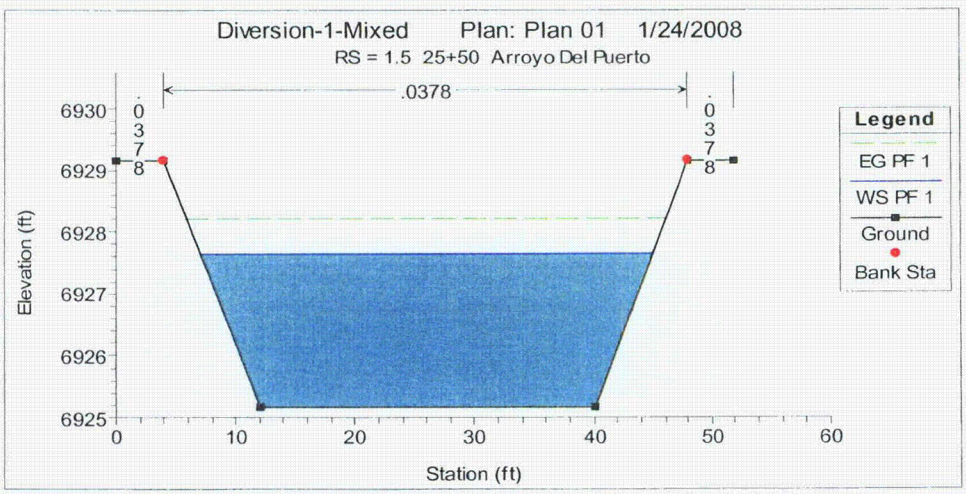


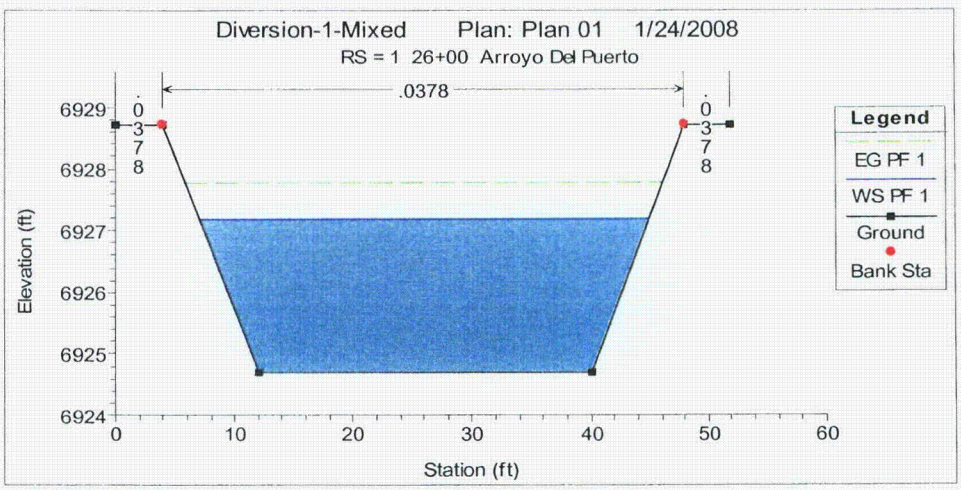




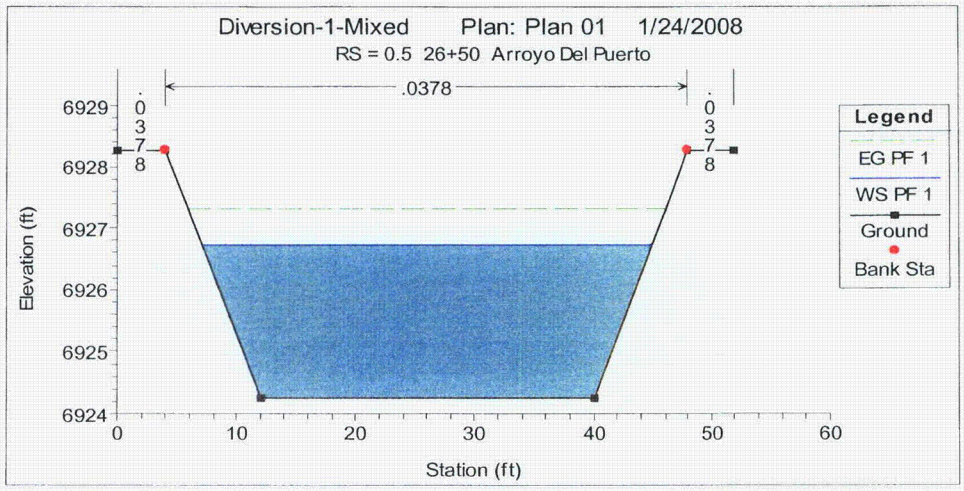
Diversion Channel - Mixed X-Section Plots

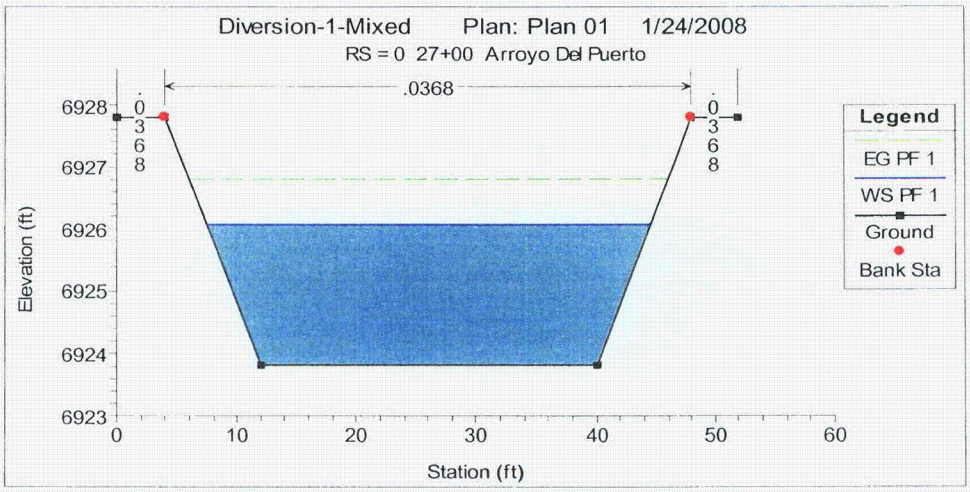


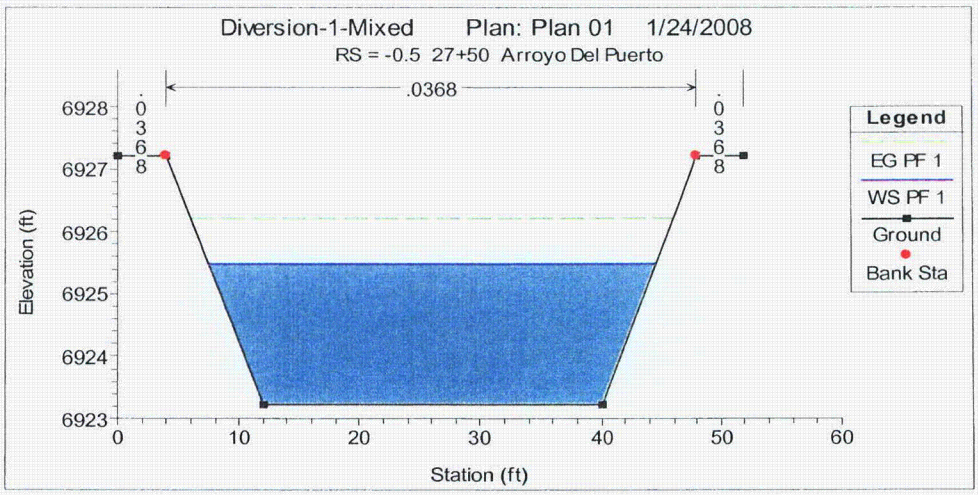




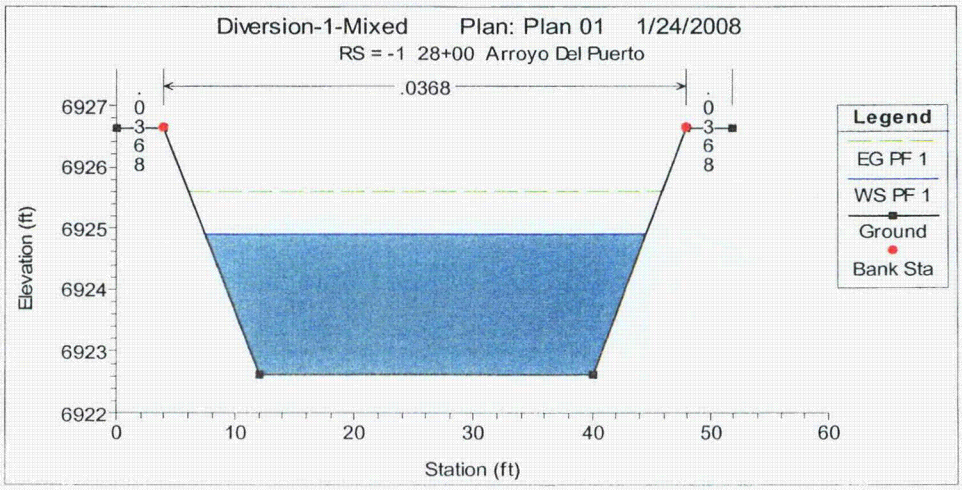
Diversion Channel - Mixed X-Section Plots

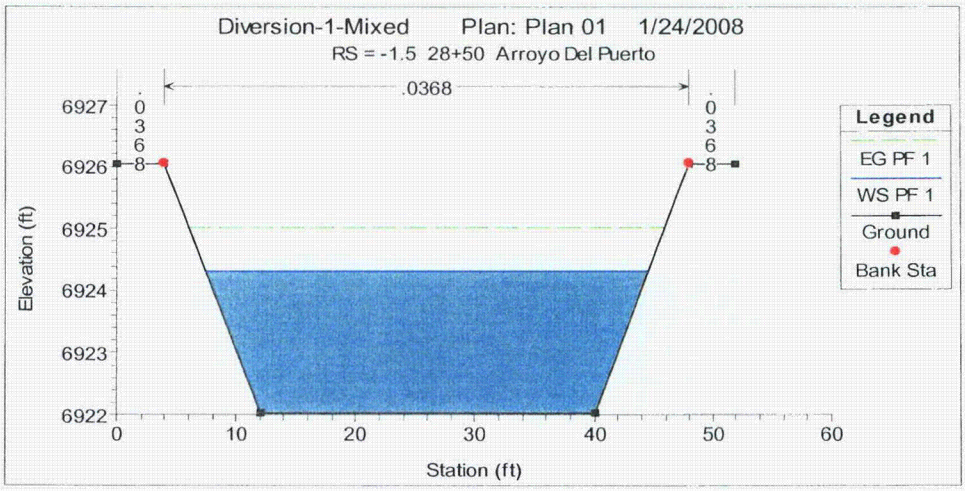


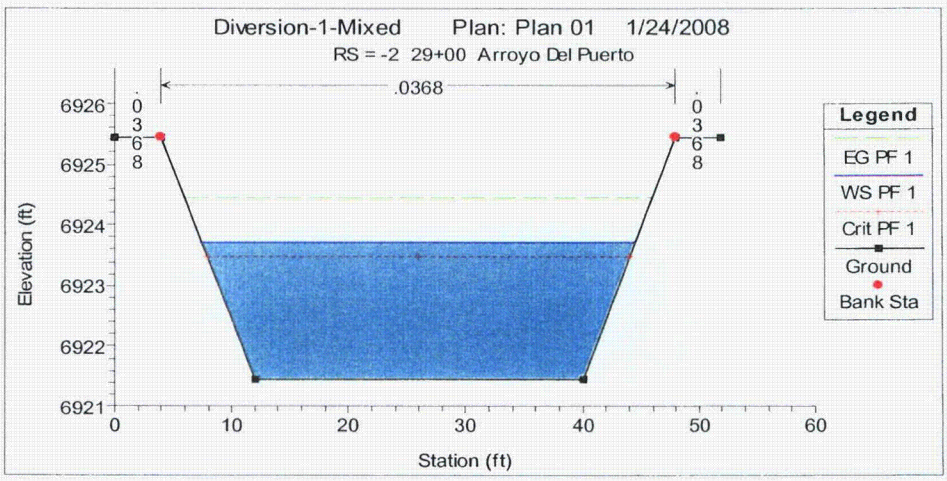




Diversion Channel - Mixed X-Section Plots







Appendix C.4 .....Calculation C.4 Interior Discharge Channel

Calculation C.4
Rio Algom Mining LLC
Ambrosia Lake, New Mexico
Arroyo Del Puerto
Interior Site Drainage Calculations
Interior Discharge Channel

1. The Interior Discharge Channel enters the Interior Drainage Channel at approximate Station 27+00 at an oblique angle. The slope of the interior discharge channel at the confluence is 1.75%. This channel is trapezoidal with a 25-foot channel bottom and carries 498 cfs. This channel varies in slope along its length. It has a hydraulic jump control section at Station 0+25 to 0+50. The slope remains constant at 1.75% as it approaches the confluence with the interior drainage channel. The downstream slope of the interior drainage channel is 0.5% but the oblique angle of entrance combined with the channel bottom slope of the interior drainage channel creates a slope of 1.183%. The general concern for an entering flow is if the slope at the entrance decreases substantially causing a hydraulic jump to form going from supercritical flow to subcritical flow. Since the entrance slope at the confluence reduce there is a concern that a hydraulic jump may occur.
   200 feet of the interior discharge channel and 200 feet of the interior drainage channel are modeled with HEC-RAS assuming the interior discharge channel maintains its shape.
   River station 0 is considers to be the edge of the interior drainage channel as the interior discharge enters it.
   The model is run separately for subcritical flow, supercritical flow, and mixed flow. The results indicate that the flow runs basically at critical until the slope decreases and the regime definitely becomes subcritical flow as shown by the mixed flow profile.

2. Interior Drainage Channel Stationing and Elevation Summary -
   see Worksheet Tab "Elevation Summary" (calc Sheets 3 of 16)

3. HEC-RAS Output Results -
   see Worksheet Tab "HC-RAS Output" (calc. Sheet 4 of 16) for the HEC-RAS profile output results of the channel stationing for subcritical flow, supercritical flow, and mixed flow regimes.

   Interior discharge Channel HEC-RAS Erosion Protection and Scour Depth specific calculations -
   see Worksheet Tab "HEC-RAS Rock Scour" (calc. Sheet 5 of 16) ffor the rock size and scour depth derived from channel parameters from the HEC-RAS modeling, rock size based on both the Abt and Johnson method and the Safety Factors Method (peak shear stress),
   and scour depth based on the USACE Equilibrium Scour Depth method.

4. Profile Plots
   see Worksheet Tab "Profile Plots" (calc Sheets 6 thru 7 of 16) for a visual illustration of the profiles of subcritical flow, supercritical flow, and mixed flow regimes.

5. PMF X-Sections
   see Worksheet Tab "PMF X-Sections" (calc Sheets 8 thru 16 of 16) for a visual illustration of the x-sections of the stationinf for subcritical flow, supercritical flow, and mixed flow regimes.

9 Conclusions:

a) The resulting rock size of D_{50}=7.8-inch used in the interior drainage channel is adequate protection from the entering flow of the interior discharge channel.
b) Scour depths at the confluence with the interior drainage channel due to the flow from the interior discharge channel are generally approximately 1.0 feet.

RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
INTERIOR DIVERSION CHANNEL STATIONING AND ELEVATION SUMMARY

Station Description	River Station #	Beginning Station (ft)	Reach Between Stations (ft)	Slope Between Stations(ft /ft)	Channel Elev Drop Downstream Reach (ft)	Fixed design Channel Bottom Widths (ft)	Channel Bottom Elev at Station (ft)	Bank Height (ft)	Bank Elevation (ft)
Diversion Channel Stationing	2.00	6+00	50	1.75%	0.875	28	6926.310	4	6930.31
Diversion Channel Stationing	1.50	6+50	50	1.75%	0.875	28	6925.435	4	6929.44
Diversion Channel Stationing	1.00	7+00	50	1.75%	0.875	28	6924.560	4	6928.56
Diversion Channel Stationing	0.50	7+50	50	1.75%	0.875	28	6923.685	4	6927.69
Confluence - Edge of Interior Channel	0.00	8+00	50	1.183%	0.592	28	6922.810	4	6926.81
Interior Channel Slope - Diversion Channel X-Sect	-0.50	8+50	50	1.183%	0.592	28	6922.218	4	6926.22
Interior Channel Slope - Diversion Channel X-Sect	-1.00	9+00	50	1.183%	0.592	28	6921.627	4	6925.63
Interior Channel Slope - Diversion Channel X-Sect	-1.50	9+50	50	1.183%	0.592	28	6921.035	4	6925.04
Interior Channel Slope - Diversion Channel X-Sect	-2.00	10+00	50			28	6920.444	4	6924.44

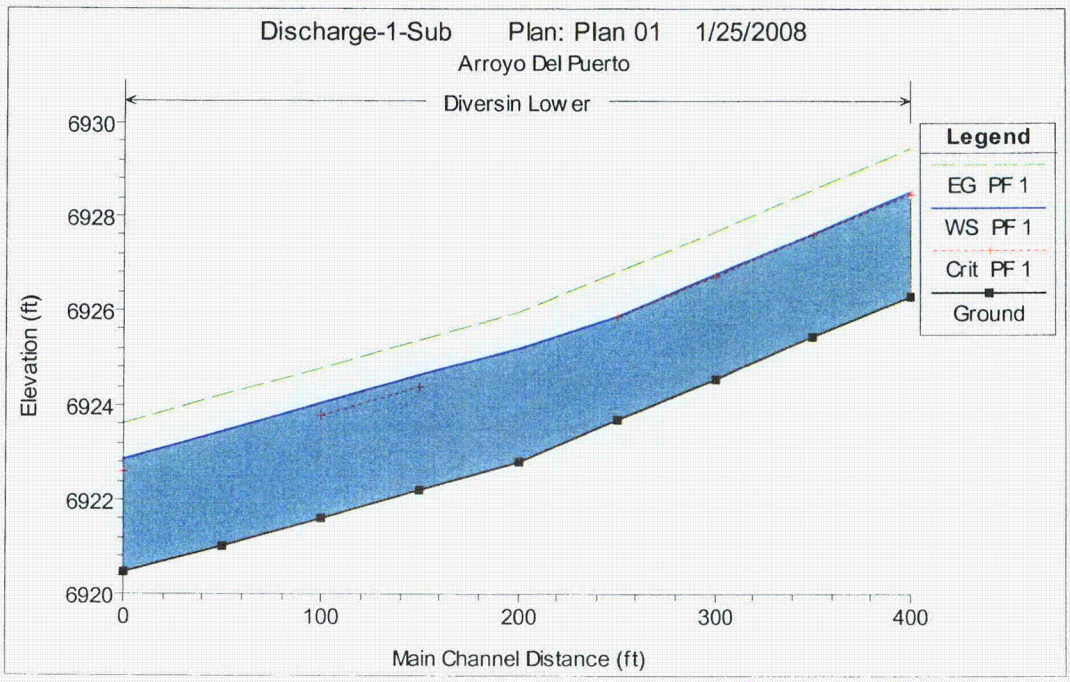
RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
HEC-RAS INTERIOR DIVERSION CHANNEL OUTPUT DATA

Reach	River Station	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/sec)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
Subcritical Flow Regime Results												
Lower	2.00	PF 1	498	6926.31	6928.5	6928.48	6929.43	0.017134	7.75	64.27	33.75	0.99
Lower	1.50	PF 1	498	6925.44	6927.61	6927.61	6928.56	0.017728	7.84	63.55	33.67	1.01
Lower	1.00	PF 1	498	6924.56	6926.75	6926.73	6927.68	0.017213	7.76	64.18	33.74	0.99
Lower	0.50	PF 1	498	6923.69	6925.86	6925.86	6926.81	0.017645	7.82	63.65	33.68	1
Lower	0.00	PF 1	498	6922.81	6925.21	0	6925.96	0.0118	6.97	71.5	34.6	0.85
Lower	-0.50	PF 1	498	6922.22	6924.62	6924.39	6925.37	0.011808	6.97	71.48	34.6	0.85
Lower	-1.00	PF 1	498	6921.63	6924.03	6923.8	6924.78	0.011791	6.96	71.52	34.6	0.85
Lower	-1.50	PF 1	498	6921.04	6923.44	0	6924.19	0.0118	6.97	71.5	34.6	0.85
Lower	-2.00	PF 1	498	6920.45	6922.85	6922.62	6923.6	0.011833	6.97	71.43	34.59	0.85
Supercrital Flow Regime Results												
Lower	2.00	PF 1	498	6926.31	6928.48	6928.48	6929.43	0.017742	7.84	63.53	33.66	1.01
Lower	1.50	PF 1	498	6925.44	6927.59	6927.61	6928.56	0.018205	7.91	62.99	33.6	1.02
Lower	1.00	PF 1	498	6924.56	6926.7	6926.73	6927.68	0.018377	7.93	62.79	33.58	1.02
Lower	0.50	PF 1	498	6923.69	6925.82	6925.86	6926.81	0.018728	7.98	62.4	33.53	1.03
Lower	0.00	PF 1	498	6922.81	6924.96	6924.98	6925.93	0.017254	7.91	62.99	33.6	1.02
Lower	-0.50	PF 1	498	6922.22	6924.39	6924.39	6925.34	0.016802	7.84	63.55	33.67	1.01
Lower	-1.00	PF 1	498	6921.63	6923.8	6923.8	6924.75	0.016789	7.83	63.57	33.67	1
Lower	-1.50	PF 1	498	6921.04	6923.21	6923.21	6924.16	0.016724	7.82	63.65	33.68	1
Lower	-2.00	PF 1	498	6920.45	6922.62	6922.62	6923.57	0.016815	7.84	63.53	33.66	1.01
Mixed Flow Regime Results												
Lower	2.00	PF 1	498	6926.31	6928.5	6928.48	6929.43	0.017134	7.75	64.27	33.75	0.99
Lower	1.50	PF 1	498	6925.44	6927.61	6927.61	6928.56	0.017728	7.84	63.55	33.67	1.01
Lower	1.00	PF 1	498	6924.56	6926.75	6926.73	6927.68	0.017213	7.76	64.18	33.74	0.99
Lower	0.50	PF 1	498	6923.69	6925.86	6925.86	6926.81	0.017645	7.82	63.65	33.68	1
Lower	0.00	PF 1	498	6922.81	6925.21	6924.98	6925.96	0.0118	6.97	71.5	34.6	0.85
Lower	-0.50	PF 1	498	6922.22	6924.62	6924.39	6925.37	0.011808	6.97	71.48	34.6	0.85
Lower	-1.00	PF 1	498	6921.63	6924.03	6923.8	6924.78	0.011791	6.96	71.52	34.6	0.85
Lower	-1.50	PF 1	498	6921.04	6923.44	0	6924.19	0.0118	6.97	71.5	34.6	0.85
Lower	-2.00	PF 1	498	6920.45	6922.85	6922.62	6923.6	0.011833	6.97	71.43	34.59	0.85

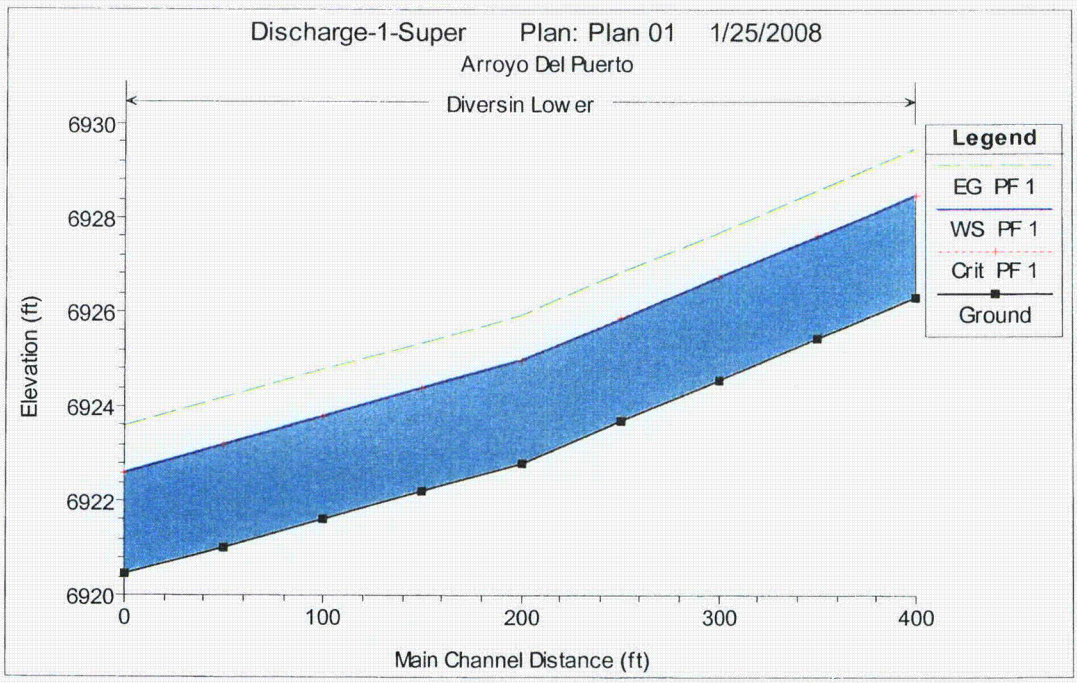
RIO ALGOM MINING , LLC
AMBROSIA LAKE, NEW MEXICO
ARROYO DEL PUERTO
INTERIOR SITE DRAINAGE CALCULATIONS
INTERIOR DIVERSION CHANNEL - HEC-RAS ROCK SIZE & SCOUR DEPTH

Station Location		Abt and Johnson Method $D_{50}=5.23(S)^{0.43}(q_d)^{0.56}$		Safety Factors Method (peak shear stress) $D50=(t)/(a(Ws-Ww))=t/4.1$ where a=0.04, Ws=165 pcf, Ww=62.4 pcf			USACE Equilibrium Scour Depth $h_e=(0.234*q_e^{8/9})/[g(Ss-1)]^{4/9}d_e^{1/3}$ (ft per unit width)				
River Station	Station Number	q_d = Design Unit Width Peak Discharge (cfs/ft)	$D_{50} = 5.23 (S)^{0.43} (q_d)^{0.56}$ (inches)	y = Average Depth (ft)	Actual Shear Stress: $t = W_w y S$	$D_{50} = 12 t / 4.1$ (inches)	he = Equilibrium Scour Depth (ft)	qe = equilibrium discharge (cfs/ft)	g (ft/sec2)	Ss = Sediment Specific Gravity	de = median grain-size diameter (mm)
Subcritical Flow Regime Results											
2.00	6+00	14.76	4.11	1.90	2.036	5.96	1.04	14.76	32.2	2.65	0.074
1.50	6+50	14.79	4.17	1.89	2.088	6.11	1.05	14.79	32.2	2.65	0.074
1.00	7+00	14.76	4.12	1.90	2.043	5.98	1.04	14.76	32.2	2.65	0.074
0.50	7+50	14.79	4.17	1.89	2.081	6.09	1.05	14.79	32.2	2.65	0.074
0.00	8+00	14.39	3.45	2.07	1.522	4.45	1.02	14.39	32.2	2.65	0.074
-0.50	8+50	14.39	3.45	2.07	1.522	4.46	1.02	14.39	32.2	2.65	0.074
-1.00	9+00	14.39	3.45	2.07	1.521	4.45	1.02	14.39	32.2	2.65	0.074
-1.50	9+50	14.39	3.45	2.07	1.522	4.45	1.02	14.39	32.2	2.65	0.074
-2.00	10+00	14.40	3.46	2.07	1.525	4.46	1.02	14.40	32.2	2.65	0.074
Supercrital Flow Regime Results											
2.00	6+00	14.80	4.18	1.89	2.090	6.12	1.05	14.80	32.2	2.65	0.074
1.50	6+50	14.82	4.23	1.87	2.130	6.23	1.05	14.82	32.2	2.65	0.074
1.00	7+00	14.83	4.25	1.87	2.144	6.28	1.05	14.83	32.2	2.65	0.074
0.50	7+50	14.85	4.28	1.86	2.175	6.37	1.05	14.85	32.2	2.65	0.074
0.00	8+00	14.82	4.13	1.87	2.018	5.91	1.05	14.82	32.2	2.65	0.074
-0.50	8+50	14.79	4.08	1.89	1.979	5.79	1.05	14.79	32.2	2.65	0.074
-1.00	9+00	14.79	4.08	1.89	1.978	5.79	1.05	14.79	32.2	2.65	0.074
-1.50	9+50	14.79	4.07	1.89	1.972	5.77	1.05	14.79	32.2	2.65	0.074
-2.00	10+00	14.80	4.08	1.89	1.980	5.80	1.05	14.80	32.2	2.65	0.074
Mixed Flow Regime Results											
2.00	6+00	14.76	4.11	1.90	2.036	5.96	1.04	14.76	32.2	2.65	0.074
1.50	6+50	14.79	4.17	1.89	2.088	6.11	1.05	14.79	32.2	2.65	0.074
1.00	7+00	14.76	4.12	1.90	2.043	5.98	1.04	14.76	32.2	2.65	0.074
0.50	7+50	14.79	4.17	1.89	2.081	6.09	1.05	14.79	32.2	2.65	0.074
0.00	8+00	14.39	3.45	2.07	1.522	4.45	1.02	14.39	32.2	2.65	0.074
-0.50	8+50	14.39	3.45	2.07	1.522	4.46	1.02	14.39	32.2	2.65	0.074
-1.00	9+00	14.39	3.45	2.07	1.521	4.45	1.02	14.39	32.2	2.65	0.074
-1.50	9+50	14.39	3.45	2.07	1.522	4.45	1.02	14.39	32.2	2.65	0.074
-2.00	10+00	14.40	3.46	2.07	1.525	4.46	1.02	14.40	32.2	2.65	0.074

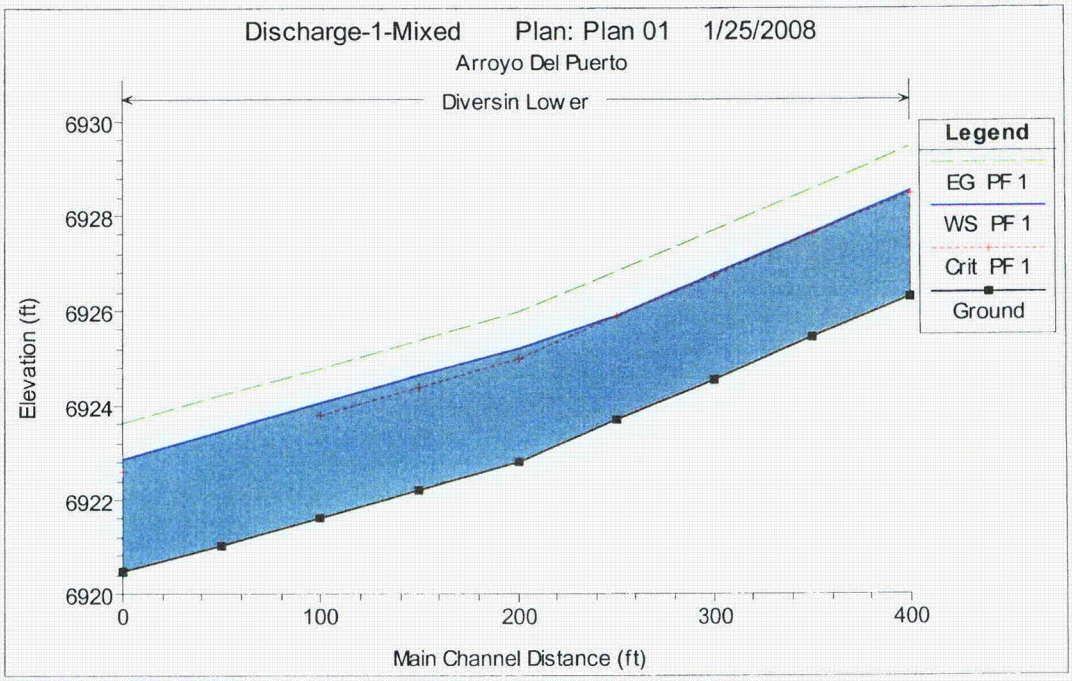
Diversion Channel - Subcritical Profile Plots



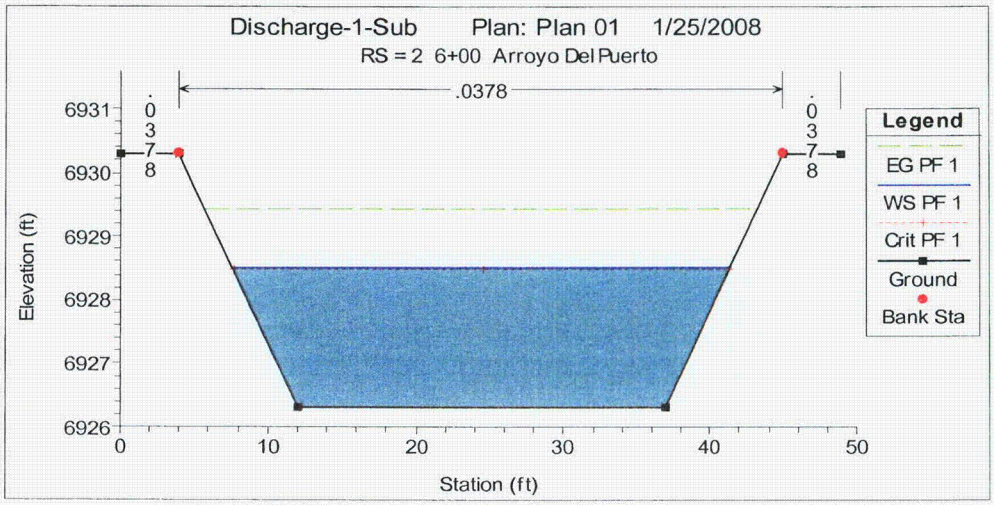
Diversion Channel - Supercritical Profile Plots

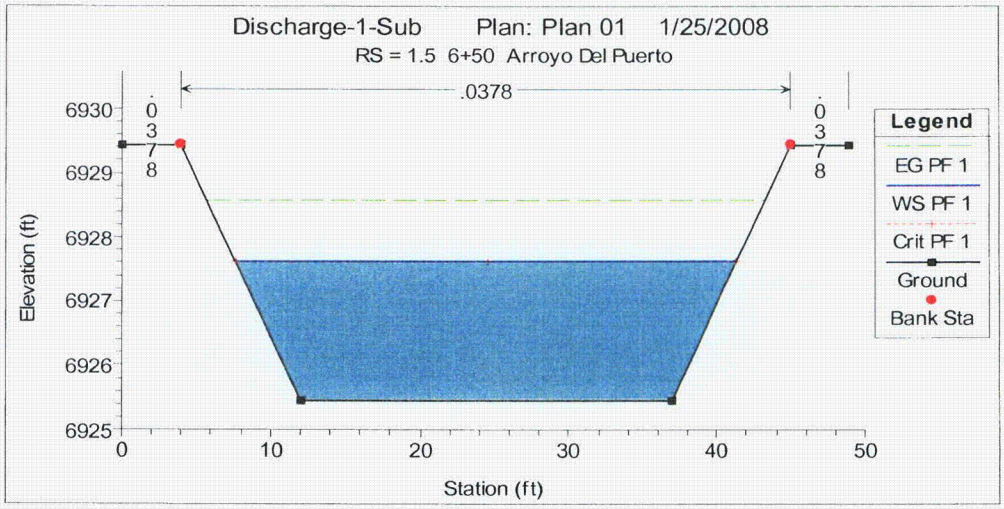


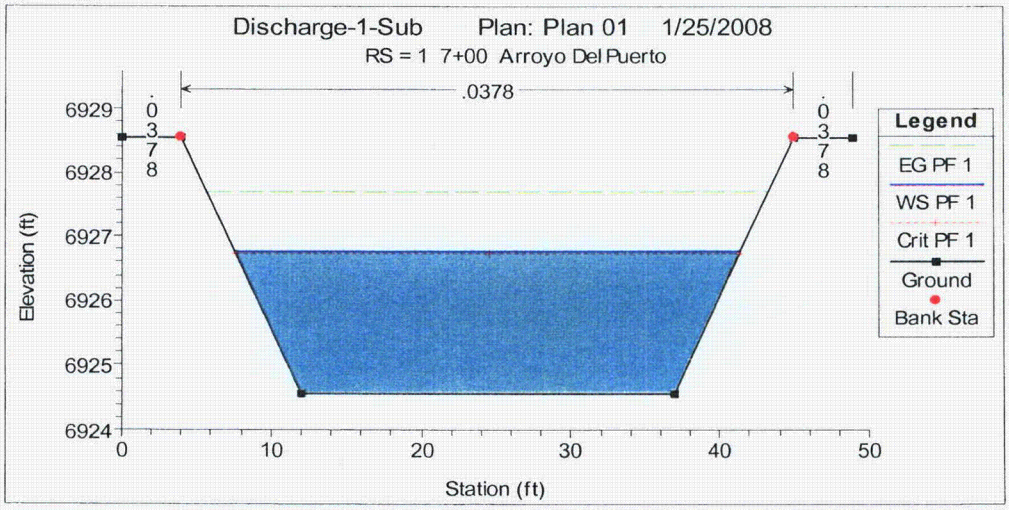
Diversion Channel - Mixed Profile Plots



Diversion Channel - Subcritical X-Section Plots







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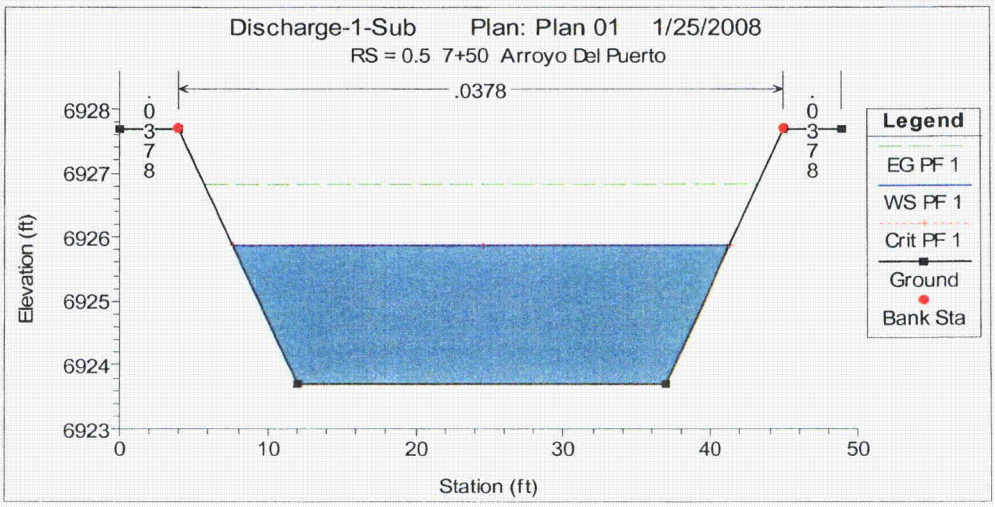

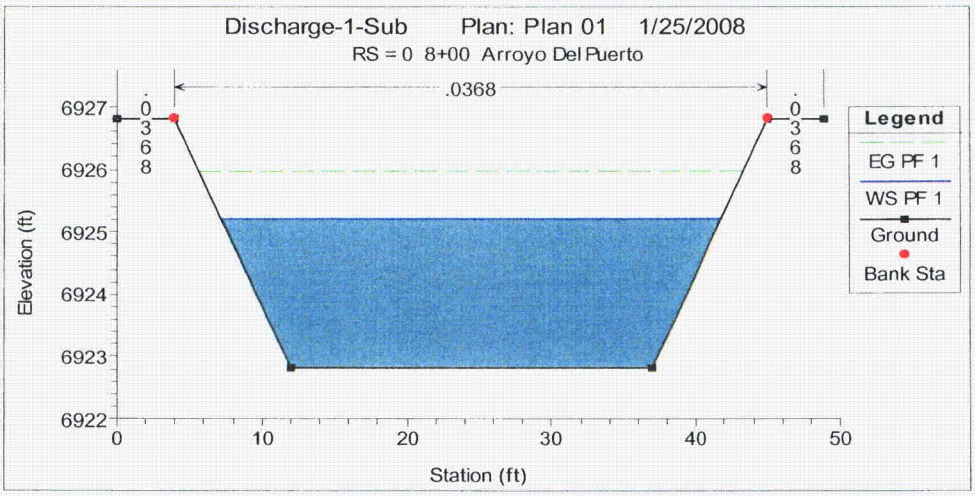

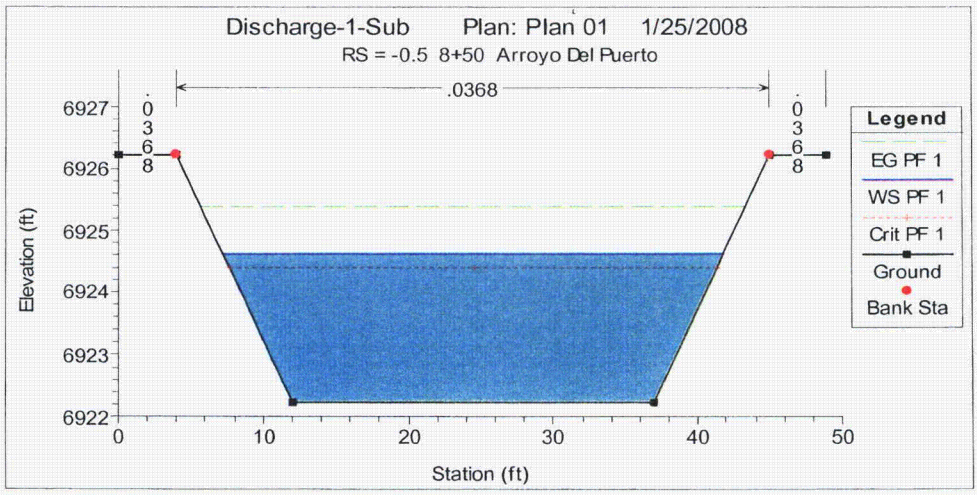

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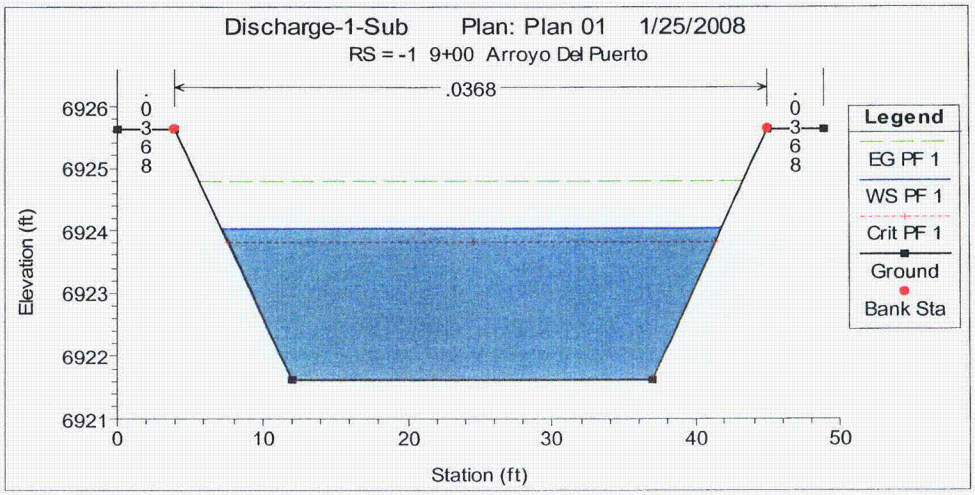

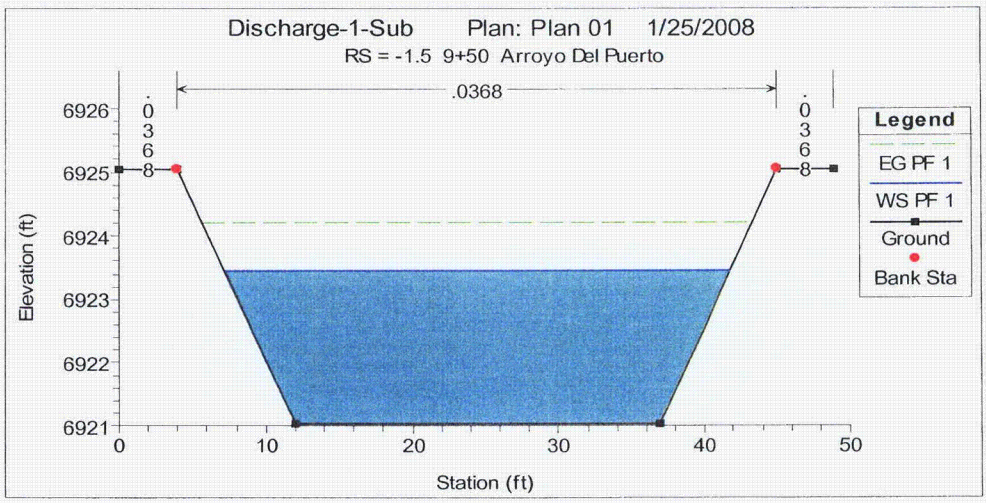

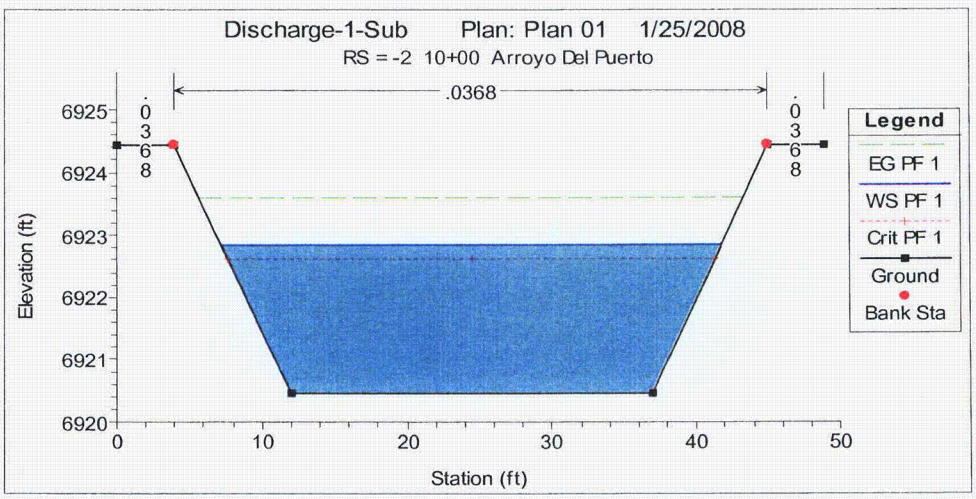

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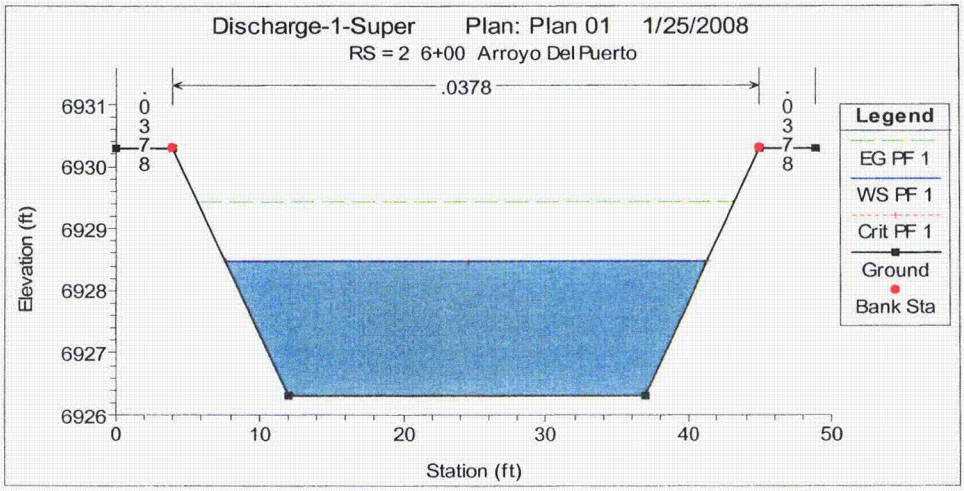

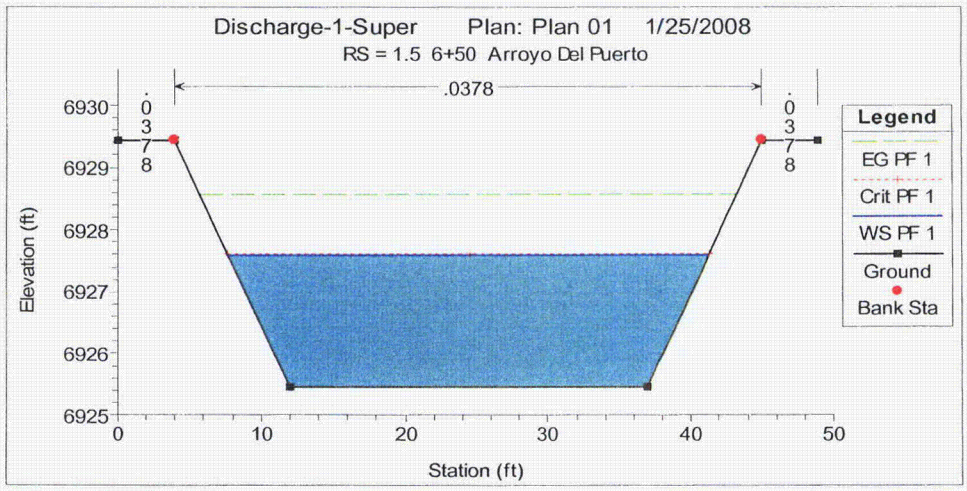

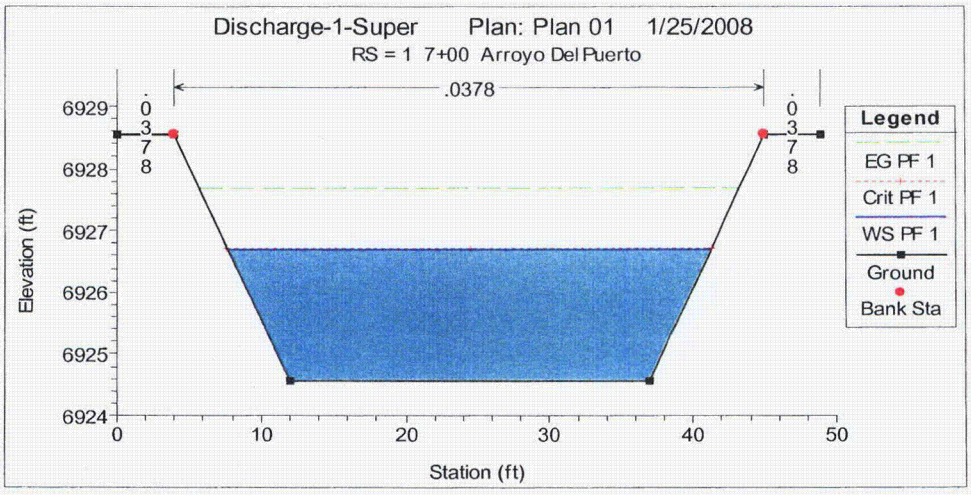

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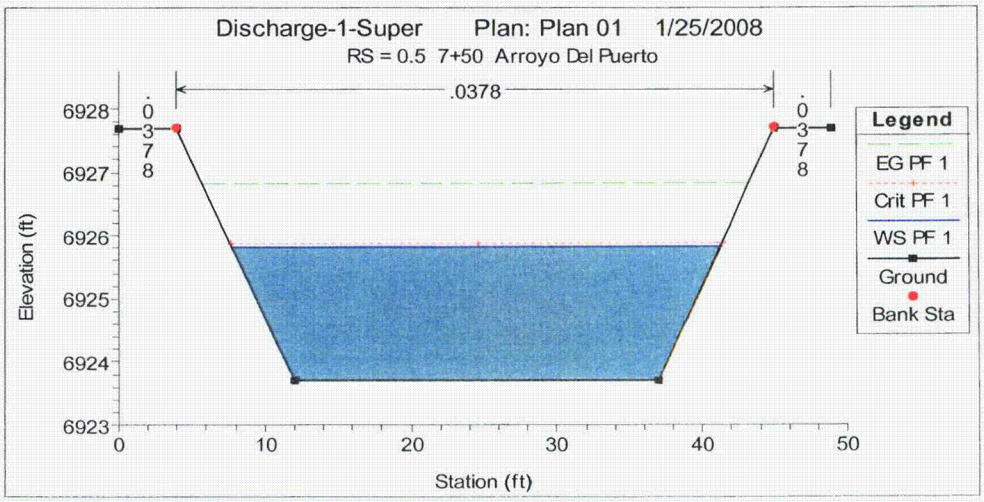

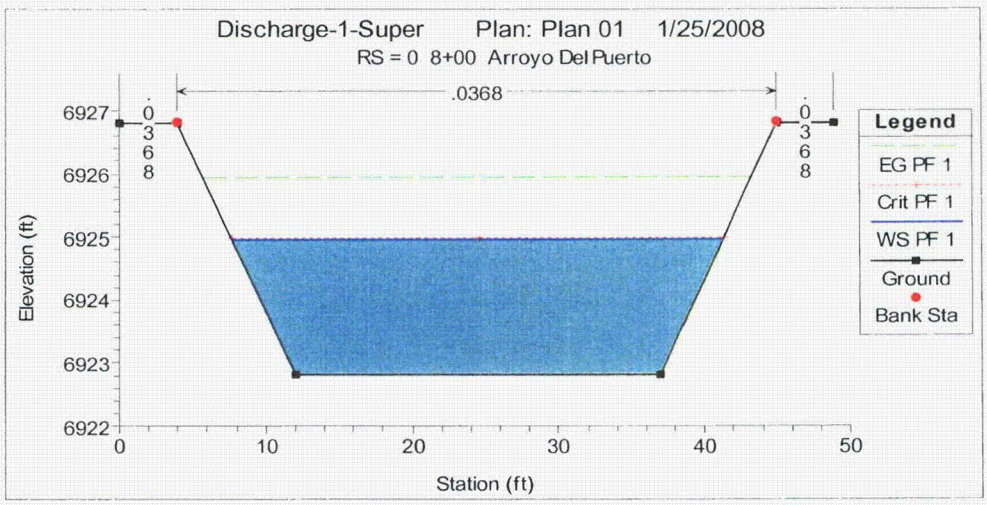

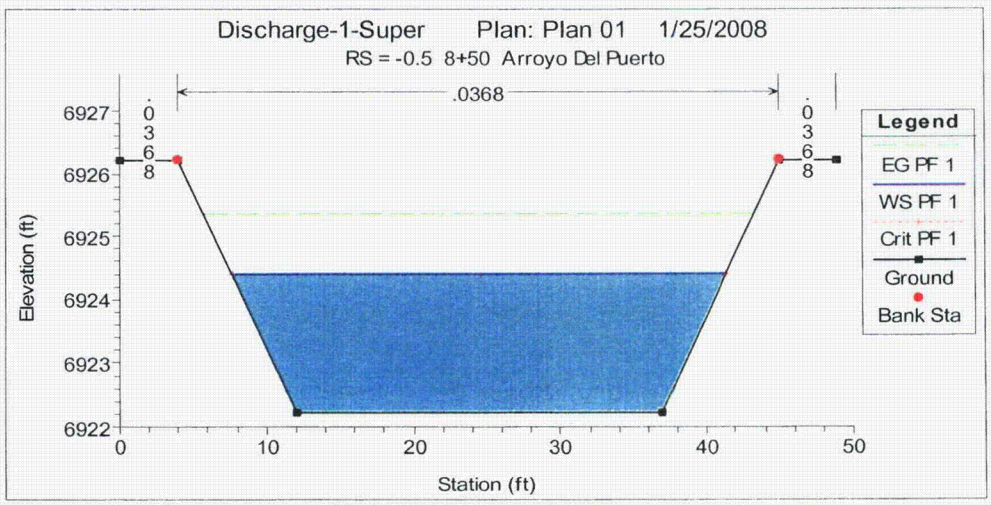

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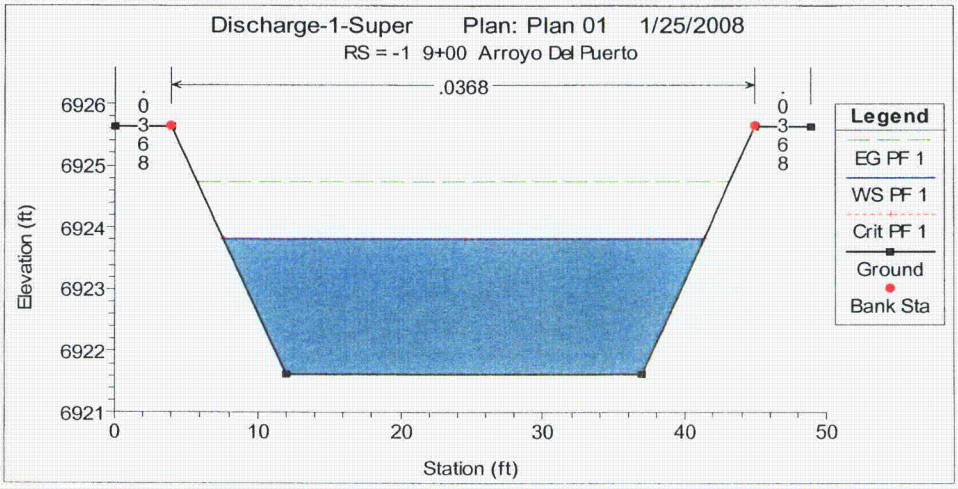

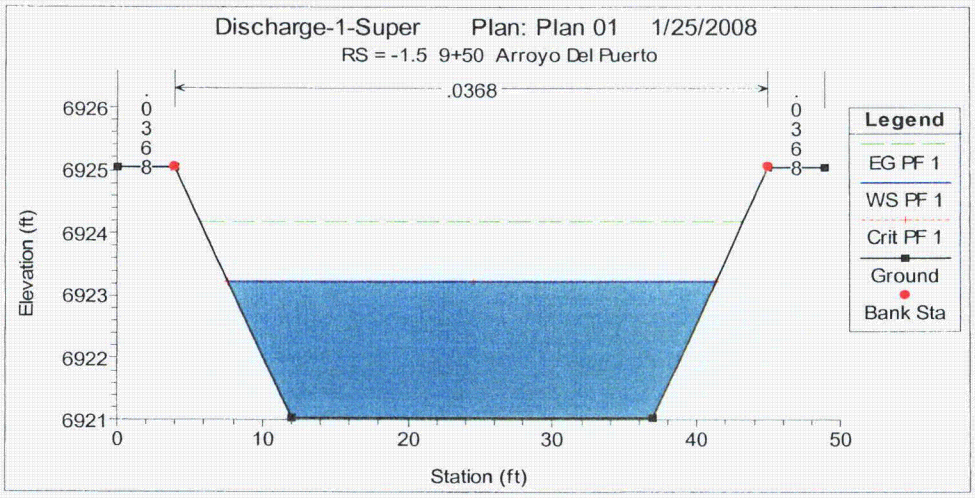

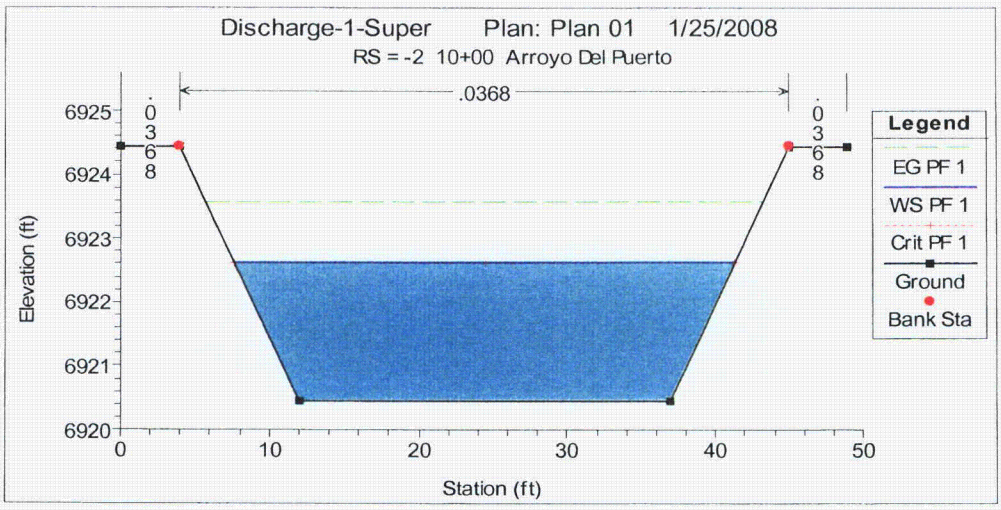

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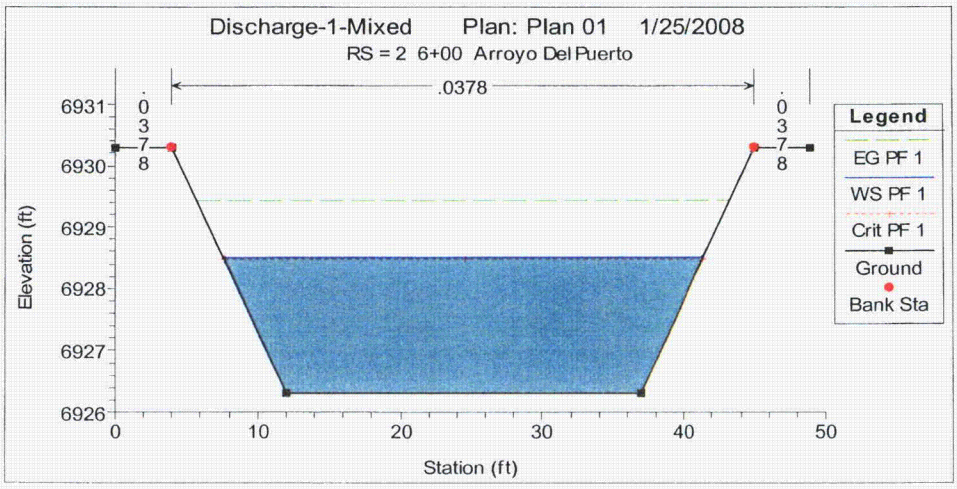

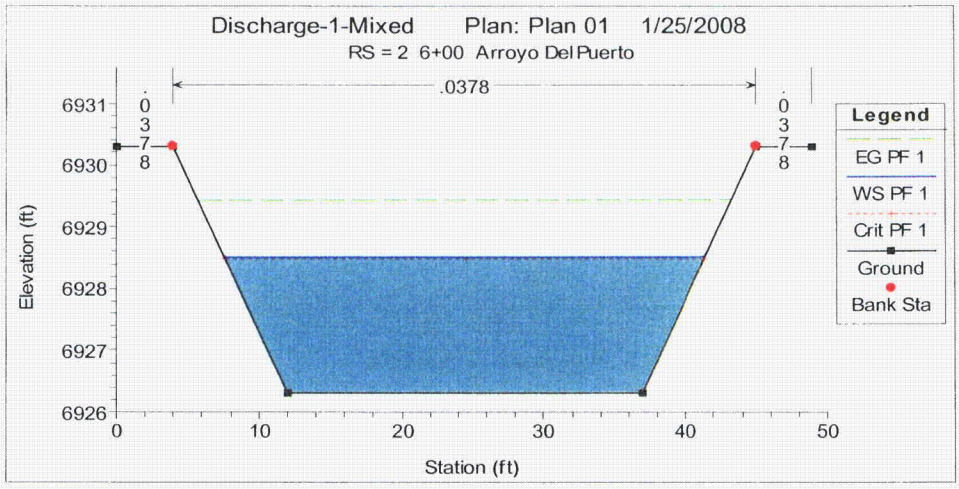

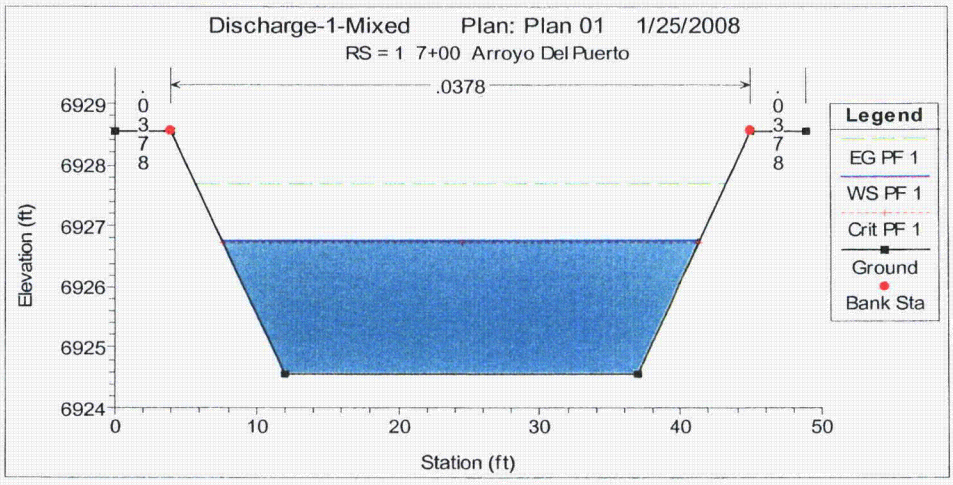

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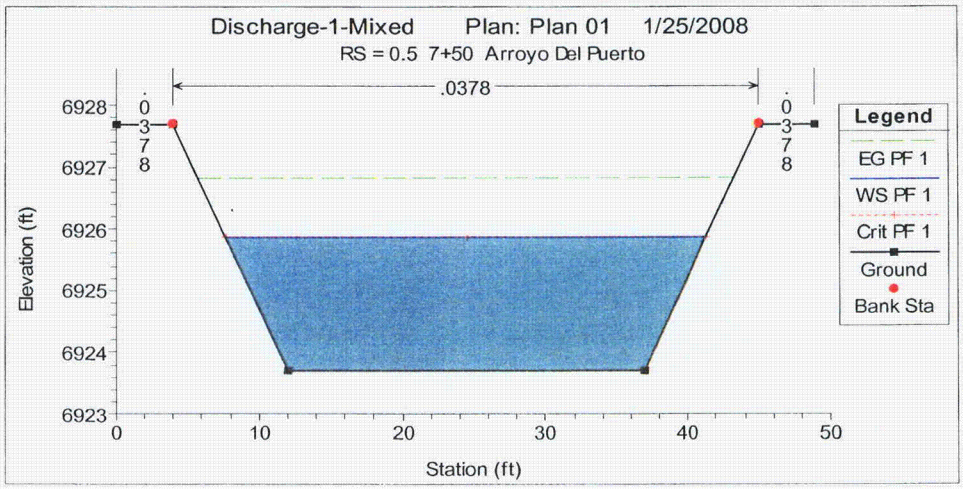

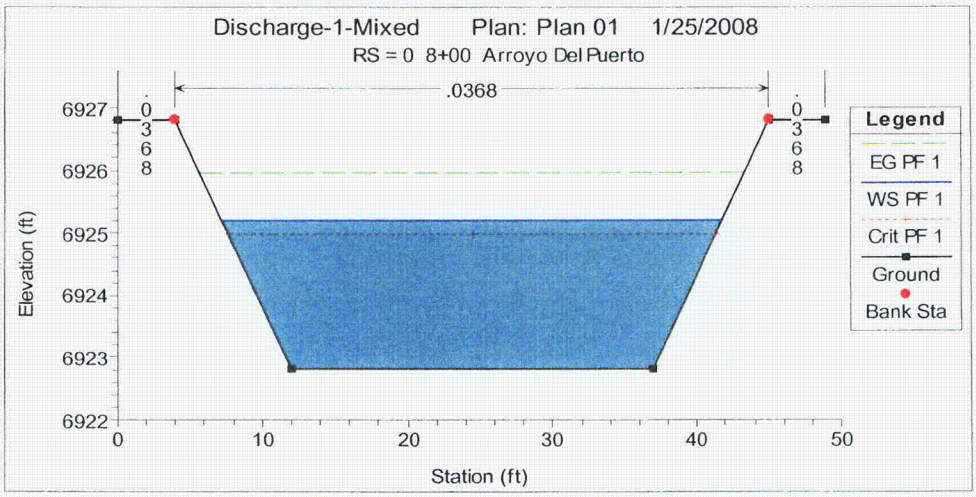

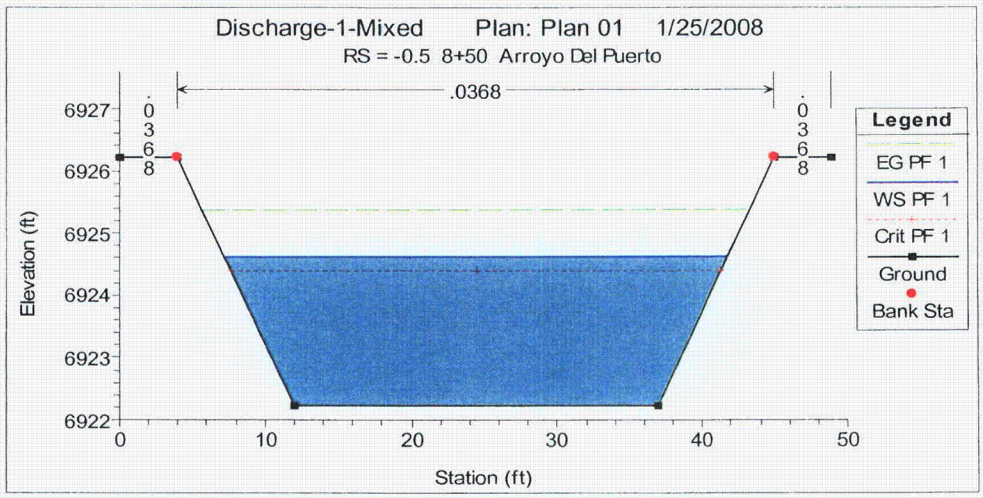

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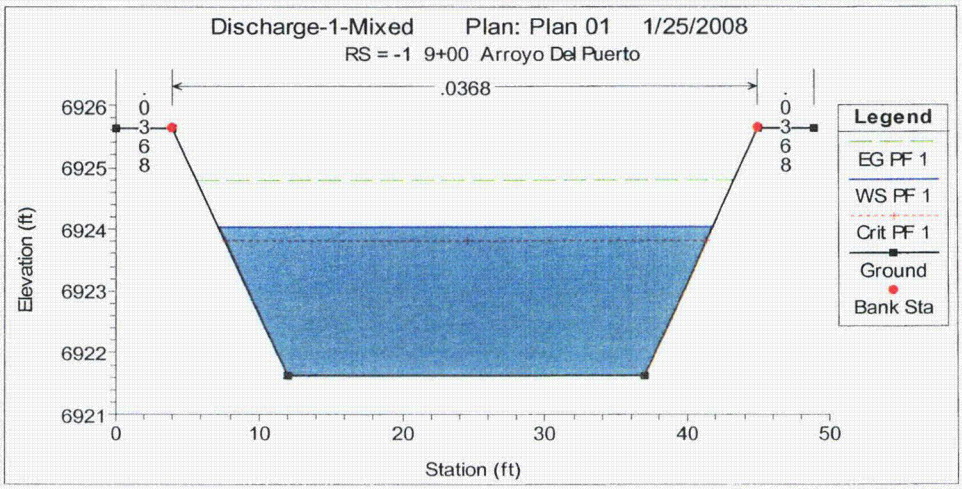

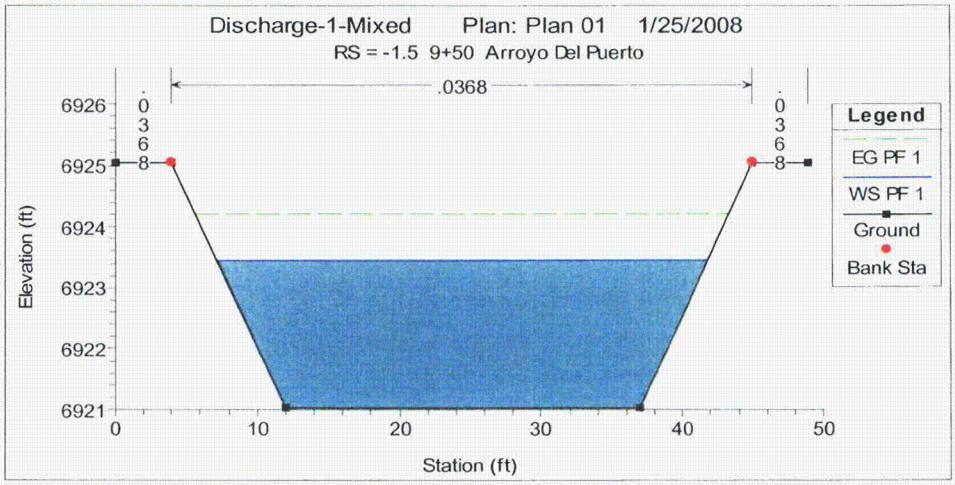

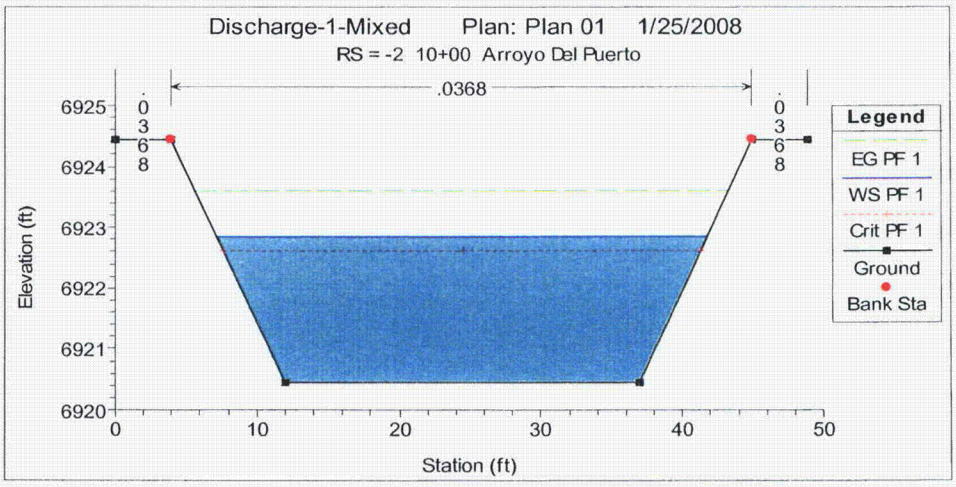

Appendix C.5 ..... Amendment to Pond 3 Erosion Protection & Erosion Protection for the Area North of Pond 1 Report

by Maxim Technologies
July 28, 2003

DESIGN REPORT
POND 3 EROSION PROTECTION
AND
EROSION PROTECTION FOR THE AREA NORTH OF POND 1
AMBROSIA LAKE MILL, NEW MEXICO

MAXIM TECHNOLOGIES INC®

10601 Lomas NE, Suite 106
Albuquerque, NM 87112
(505) 237-8440

July 28, 2003

Mr. William Paul Goranson
Rio Algom Mining Company, LLC.
Manager, Radiation Safety, Regulatory Compliance
6305 Waterford Boulevard, Suite 325
Oklahoma City, OK 73118

RE: Amendment To; Pond 3 Erosion Protection & Erosion Protection for the Area North of Pond 1 Design Report, Ambrosia Lake Mill, Grants, New Mexico Submitted April 22, 2002

Dear Mr. Goranson:

Maxim has completed reviewing the Task 3 Erosion Protection Design for potential rock size reduction. As part of the on-going review of erosion rock size for erosion protection at the Ambrosia Lake Mill, Grants, New Mexico, we have amended the erosion protection designs for two areas of the Task 3 design report. The **Pond 3 Erosion Protection And Erosion Protection For The Area North of Pond 1 Design report, dated April 22, 2002** used the available rock sizes for design of erosion protection. At your request, we have reviewed the original design and determined that the upper section of the diversion channel, approximately 900 ft and the Pond 1 embankment toe erosion protection apron, approximately 3,900 ft in length, may be constructed with the newly available rock size of D_{50} = 7.8 inches.

In the original design report the rock size used in these two areas were increased to a D_{50} = 9.2 inches to take advantage of the available rock sizes selected for erosion protection at the mill site. A rock size with a median rock diameter of 7.5 inches was determined for the Pond 1 apron and the diversion channel median rock size was determined to be 8.2 inches in the original calculations using the NUREG CR-1623 criteria. Both rock sizes were increased in the erosion protection designs to meet the available rock size. Through input by Rio Algom, review of the design calculations for this task, and availability of a new rock size recently used in revision of Task 1 for rock size reduction, we have redesigned the two areas to reduce rock size.

Because of the rock quality test results of four samples from the proposed rock source, submitted by Rio Algom, the calculated rock size of the Pond 1 apron was oversized in accordance to *Oversizing of Marginal-Quality Erosion Protection, Section 7 of Appendix D, NUREG CR-1623*. An oversizing factor of 1.04 was applied to the rock size to meet the NUREG requirements for rock having an average rock quality grade of 76%, increasing the required nominal rock size to 7.8-inches.

To reduce the rock size required in the upper section of the diversion channel, the channel bottom width was widened. By widening the channel bottom from 3 ft to 4ft, and in accordance with NUREG CR-1623 criteria, the nominal rock size was reduced from 8.2-inches to 7.0-inches. In applying the above noted oversizing factor, the nominal rock size was increased to 7.3 inches. The design configuration was based off the available rock size of a D_{50} = 7.8 inch rock.

The diversion channel and toe apron designs have been amended to reflect the D_{50} = 7.8 inch rock size. All other portions of this design task, Task 3, remain the same as originally submitted because of practical engineering constraints. The D_{50} = 7.8 inch rock size will permit the placement of the rock by mechanical means because the maximum rock size will be smaller than 12 inches.

Please do not hesitate to contact me if you have questions on this amendment.

Respectfully submitted,
Maxim Technologies, Inc.

Robert L. Harrington
Project Engineer

Rev: William H. Bucher
Cc: JM, TT

Enclosure: Amended Drawings (Sheet 3 and Sheet 8)

DESIGN REPORT
POND 3 EROSION PROTECTION
AND
EROSION PROTECTION FOR THE AREA NORTH OF POND 1
AMBROSIA LAKE MILL, NEW MEXICO

Prepared for:

Rio Algom Mining Company, LLC.
P.O. Box 218
Grants, New Mexico 87020

Prepared by:

Maxim Technologies, Inc.
10601 Lomas NE, Suite 106
Albuquerque, New Mexico 87112

WILLIAM H. BUCHER
NEW MEXICO
15469
REGISTERED PROFESSIONAL ENGINEER

7/23/03

April 22, 2002

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INTRODUCTION

This design report has been prepared by Maxim Technologies, Inc. (Maxim) for Rio Algom Mining Company, LLC. (Rio Algom) as part of an agreement for engineering services dated July 2001. Maxim has been retained to evaluate flood and long-term erosion conditions at the Ambrosia Lake Facility near Grants, New Mexico and to prepare designs for the long-term stability of tailings and evaporation ponds at this uranium mill tailings disposal site. This report provides the basis for the design required by the third of four tasks included in the agreement. Task three addresses potential erosion of the surface and embankment slope of Pond 3. As part of an addendum to Task 3, this report addresses potential erosion on approximately 20 acres north of the north embankment of Pond 1. Also included in this design report is the completion of the Pond 1 North Embankment Erosion Protection Channel/Apron, herein referred to as the Discharge Channel. Figure 1 is a schematic representation of the three (3) areas of concern in the current study.

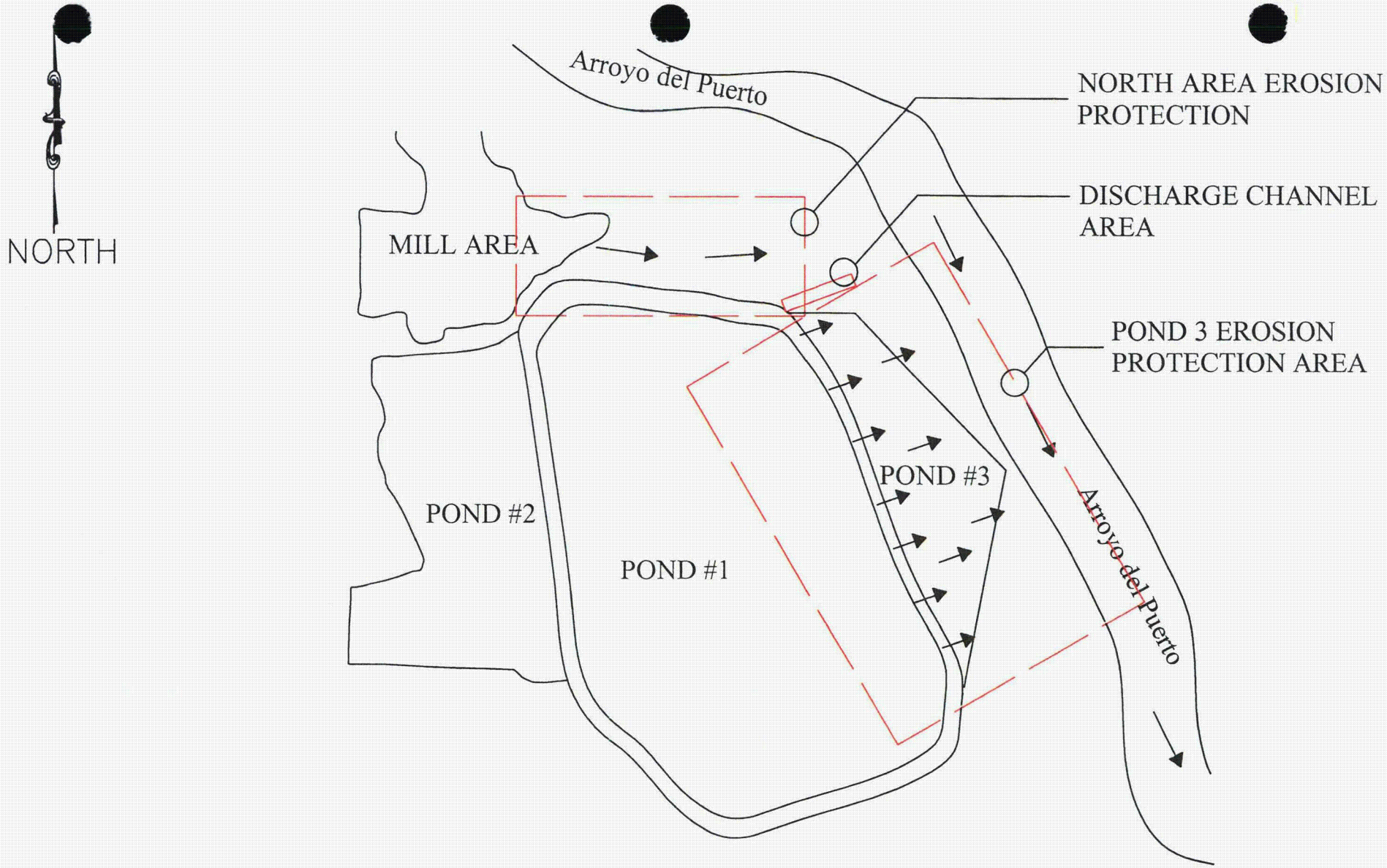
Pond 3 is a contaminated soils disposal site currently under reclamation by Rio Algom. The tailings disposal site was used in the past for burial of byproducts produced at the Ambrosia Lake Mill. The current reclamation activities for Pond 3 include but are not limited to placement of general site waste soils, re-grading of the pond surface contours, and contouring of the east embankment. The re-contouring activities at Pond 3 were not completed at the time of this design report submittal. This report provides a design to limit erosion after completion of the re-contouring activities of the following areas: the surface, the east embankment and the east embankment toe of Pond 3, and the interface between the Pond 1 embankment toe and the Pond 3 surface. Local topographic information supplied by Rio Algom indicates precipitation that falls on the Pond 1 embankment slope that interfaces with Pond 3 will run-off to the east and onto the surface of Pond 3. As part of this design, the surface of Pond 3 will be re-graded such that the precipitation from the Pond 1 embankment slope that flows onto the surface of Pond 3 and the precipitation that falls onto the Pond 3 surface will run across the surface of Pond 3 as sheet flow toward the east embankment of Pond 3, where it will then flow down the embankment and into the Arroyo del Puerto basin. Through the design process for erosion protection of this area, the east embankment of Pond 3 was determined to be within the Probable Maximum Flood (PMF) extent of the Arroyo del Puerto; therefore, the design for erosion protection of the east embankment was based on longitudinal flows created by the PMF in the Arroyo del Puerto.

Included in this report is a design for erosion protection of approximately 20 acres of generally undeveloped land adjacent to the north embankment of Pond 1. This area slopes away from the embankment to the northeast at an average grade of 7.5%. The Nuclear Regulatory Commission (NRC) has described the area as an area of concern for erosion caused by local storm events. The area is bound to the south by the Pond 1 embankment toe and a proposed erosion protection toe channel/apron for Pond 1, to the immediate west by a ridge that will act as a dividing line between this area and the mill facilities for runoff, to the east by the Arroyo del Puerto, and to the north by a proposed diversion channel placed appropriately through analysis of head-cutting directions depicted by the local topographic information provided by Rio Algom. Within this area are slopes in excess of 10%; however, these areas are limited in extent. Part of the erosion protection design for this area is the re-grading of the area and of slopes greater than 7.5%. The erosion protection design for this area is to re-contour the steeper slopes, to place erosion protection rock to limit local erosional head-cutting that could eventually impact the toe of Pond 1, and to capture flows from this area and discharge them to the Arroyo del Puerto via an engineered diversion channel.

The last erosion design included in this report is a discharge channel for the north embankment erosion protection channel/apron of Pond 1. The discharge channel will be used to direct flow captured in the channel/apron from the toe of the north embankment of Pond 1 around the north limits of Pond 3 where it will discharge through an energy dissipation outlet apron into native soils of the Arroyo del Puerto.

Erosion protection for the areas discussed above consists of seven items:

1. Toe erosion protection apron placed at the interface of Pond 1 and Pond 3,



Legend:
→ Local Flow Direction

AMBROSIA LAKE MILL RIO ALGOM MINING COMPANY, LLC. GRANTS, NEW MEXICO		MAXIM TECHNOLOGIES INC
GENERAL OVERVIEW OF TASK	DRAWING BY: RLH 2/21/02	
PROJECT No. 1690030-300 FILE NAME: 1690030Fig1.DWG	REVIEWED BY: WHB	FIGURE 1

2. Surface erosion protection for Pond 3,
3. Erosion protection for the east embankment of Pond 3,
4. Toe erosion protection apron placed at the toe of the east embankment of Pond 3
5. Surface run-off protection of the area north of Pond 1,
6. Diversion channel construction along the northern limit of the area north of Pond 1,
7. Discharge channel construction from the end of Pond 1 north embankment channel/apron to the Arroyo del Puerto basin.

This design report is limited to those items affecting design of the erosion protection areas outlined above. In the next section of this report, general methods of analysis are described including derivation of the PMP and calculation of the appropriate PMFs. In the following section, the design basis for each design item is presented. The report concludes with a discussion of other issues that affect the design.

1.0 METHODS

The analysis conducted for these designs are consistent with Nuclear Regulatory Commission (NRC) guidance, particularly, *Design of Erosion Protection for Long-Term Stabilization* (Johnson 1999). This guidance, referred to as NUREG-1623 in this report, requires, in most cases, that erosion protection be designed for a 1,000-year life to minimize future maintenance issues. Because flood events with a 1,000 year recurrence interval are difficult to quantify, the guidance recommends use of the probable maximum precipitation event (PMP) for design purposes. PMPs can be derived for various parts of the United States using appropriate hydrometeorological reports. The report that addresses New Mexico east of the continental divide is *Hydrometeorological Report No. 55A, Probable Maximum Precipitation Estimates – United States between the Continental Divide and the 103rd Meridian* (Hansen *et al.* 1988). Appropriate PMPs are used to develop runoff hydrographs and determine the probable maximum flood (PMF) for an area of concern. The final step in the design process is to apply the PMF to the appropriate erosion control design method. Guidance for design of riprap erosion protection is found in Appendix D of NUREG-1623.

Maxim verified the PMP calculation performed by others following the methods outlined in *Hydrometeorological Report No. 55A* (Hansen *et al.* 1988). The PMP rainfall depth calculated previously for the 1-hour local storm was 9.6 inches with no areal reduction. Maxim's calculation of the PMP depth arrived at a 9.5 inch value for the 1-hour, 1-square mile local storm, slightly less than the previously calculated value of 9.6 inches. Because the values are so similar, we used the slightly higher, previously determined values, in our calculations. Calculation sheets are attached in Appendix A.

For areas which are exposed to the PMF from the Arroyo del Puerto, the approach consisted of two tasks with the greater design requirements controlling the final design configurations.

1) Determining the erosion protection requirements based on run-off analysis for sheet flows down slopes as well as across pond surfaces in accordance with NUREG-1623, Appendix D, Section 2; and

2) Determining open channel and/or toe apron requirements to control the run-off and/or longitudinal flow from a PMF event by an open channel or apron in accordance with NUREG-1623, Appendix D, Sections 3, 4, and 6.

The study of longitudinal forces of the PMF for the Arroyo del Puerto drainage was conducted using HEC-1 (USACE 1990) and the PMF water-surface profile in the vicinity of Pond 3 was determined using HEC-RAS (USACE 1998). Previous correspondence from Quivira Mining Company to the Nuclear Regulatory Commission (QMC, 1990) indicated that the accepted PMF for Arroyo del Puerto at the Ambrosia Lake Millsite is 78,000 cfs. There are no indications how this PMF was calculated and we do not know the duration of the storm event used or if it was a local or general storm. Therefore, we reviewed several potential storm durations and types for the Arroyo del Puerto.

The Arroyo del Puerto basin contains 57.6 square miles upstream of the Ambrosia Lake Mill. Because no information was developed on the run-off characteristics of this basin, we used the same Soil Conservation Service curve number that was used on the Montanosa Mesa drainage, 73.4. The lag time was chosen to be 0.6 of the time of concentration or 1.83 hours. Areal reduction factors for the 57.6 square mile basin were used. We investigated the 1-hour and 6-hour local storms and the 6-hour, 24-hour, and 72 hour general storms with these basin characteristics and found that the highest value for the PMF was obtained with the 6-hour local storm which produced a peak flow of 75,200 cfs. This seemed to confirm the value of 78,000 cfs found in the QMC letter of 1990. We chose to use the higher value originally reported in all our subsequent calculations. Our calculations for the 6-hour local storm are included in Appendix A.

We constructed a hydraulic model of the Arroyo del Puerto in the vicinity of the Ambrosia Lake Mill and routed the 78,000 cfs peak flow through this area to determine the elevation of the flood water in the vicinity of Pond 3. We used existing topography from the Quivira Mining Company map with the following exceptions:

1. Arroyo del Puerto was rerouted to a new alignment east of its existing position in the vicinity of the mill. The new channel rejoins the original channel near the north-east corner of Pond 9.
2. The Pond 3 embankment was assumed to be constructed to a final elevation of 6936 feet at its eastern edge and to have a slope of 6 percent.
3. The existing groundwater dewatering trench was assumed to be backfilled.
4. Pond 9 was assumed to be removed and have a final elevation of 6917 feet.

With these assumptions, the elevation of the water surface at the northern edge of Pond 3 is about 6935 feet or one foot lower than the top of the embankment. The flood profile declines rapidly as it moves south along Pond 3 and attains an elevation of about 6917 feet at its southern extent. Note that these water surface elevations are very sensitive to assumptions about final topography in this area. Results of the HEC-RAS model for Arroyo del Puerto are found in Appendix A.

2.0 DESIGN BASIS

This section of the design report describes in detail the basis for the seven design items listed previously. The individual catchment areas and a description of the catchment surface type are briefly described for each design area. Visual observations during site visits were used to aid in fitting an appropriate design to limit erosional effects. The areas of erosion protection designs at the mill site are depicted on Sheet 1 of the design drawings.

As part of this erosion protection design, rock sizing was developed. The sizes were based on previous erosion protection design tasks at the Ambrosia Lake Mill to reduce the number of rock size gradations. Rock grading of D_{50} = 1.0, 3.2, and 9.2 inches have been previously selected for use in erosion protection systems at the mill site. These rock sizes have been used as the basis for rock sizing for this design task.

2.1 EROSION PROTECTION APRON – POND 1 AND POND 3 INTERFACE

This 31-acre catchment area consists of an embankment slope that is covered with a radon barrier and erosion protection rock. The embankment slopes downward to the east from a crest elevation of approximately 7,025 ft. Because of continuing re-contouring work being performed by Rio Algom at Pond 3, the embankment toe elevation is not yet determined; therefore, an elevation of 6,938 ft has been assumed for design purposes. The design grade of the embankment is 20%, according to Rio Algom. This value was verified to be between 17% and 20% by the topographic information supplied by Rio Algom. For the 31-acre catchment, a time of concentration was calculated for the longest slope length measured at the east embankment of Pond 1, 520 ft. With the noted slope length and the elevation change of the embankment, a time of concentration of 1.92 minutes was determined. This time of

concentration is near the smallest incremental rain duration provided by Nelson *et. al* (1986) for developing a PMP depth. Therefore the shortest incremental rainfall duration of 2.5 minutes was used for developing the PMP depth. The 2.5 minute PMP depth for a local storm of 9.6 inches is 2.64 inches. Using a runoff coefficient of 1.0 (rock covered surfaces with no infiltration), the Rational Method gives a unit peak flow of 0.76 cfs/ft for this catchment. Using a maximum embankment slope of 20% and a flow concentration factor of 2.5, the method of Abt *et al.* (1998) predicts a rock d_{50} of 7.5 inches for the erosion protection apron at the Pond 1 and Pond 3 interface. Calculations are included in Appendix B. The detailed erosion protection apron design and minimum apron configurations are shown on the design sheets included in Appendix E.

From the available rock sizes noted above, a rock d_{50} of 9.2 inches was selected for construction of the apron. The apron configuration was based on 15 times the d_{50} for the apron width and three times the d_{50} for the apron depth (NUREG 1623). For the Pond 1 embankment toe apron width, the predicted rock size of d_{50} = 7.5 inches was used for this calculation, making the channel width 9.4 ft. At the wishes of Rio Algom, the apron depth was based on a rock d_{50} of 9.2 inches for ease of placement of the larger rock into the apron, which exceeds the thickness requirement of NUREG 1623.

2.2 SURFACE EROSION PROTECTION – POND 3 SURFACE

The surface of Pond 3 at the time of this design study had not yet been regraded to its final configuration. Current reclamation activities at the mill site include the placement of waste site soils in Pond 3 where they will be covered and an erosion protection system placed. As part of the erosion protection system for the surface of Pond 3, Rio Algom has committed to placing a rock erosion protection layer. Whether the rock layer was required by run-off or not, Rio Algom has indicated that a rock layer will be placed to limit erosion caused by run-off as well as wind erosion of the soil barrier layer.

Although the finished pond surface elevation and slope conditions of the pond surface were not yet determined at the time of this study, it is anticipated that a 33 acre area will be covered. Based on conversations with Rio Algom, the pond surface was near a completion elevation; therefore, an assumed elevation near the current elevation was used to complete erosion protect designs. Included in this estimated elevation is the 12-inch thick cover layer, which results in an estimated elevation of 6,938 ft for the western limits of Pond 3 where the toe of Pond 1 intersects the Pond 3 surface. For design purposes, a pond surface grade of 0.3% for Pond 3 was established to prevent ponding or concentrated flows across the pond. With the start elevation of 6,938 ft at the toe of Pond 1 and a maximum horizontal surface length of 700 ft across Pond 3, the minimum ending elevation of the pond surface at a 0.3% grade will be 6,936 ft. The minimum elevation of the Pond 3 surface is approximately 1 ft above the highest elevation estimated for a PMF of the Arroyo del Puerto basin. This maximum PMF elevation will occur at the northeast end of Pond 3 and quickly lower in elevation as the PMF progresses south along the Pond 3 embankment. At the southeast limits of the Pond 3, the embankment will not be affected by a PMF event according to the calculations. Therefore, the PMF in the Arroyo del Puerto was not considered when designing protection for the surface of Pond 3.

For this 33-acre catchment area, a time of concentration was calculated for the longest surface slope length measured starting at the west limits of Pond 3 and ending at the furthest east point of the Pond 3 surface, assuming that the pond surface will be finish graded to drain perpendicular to the Pond 1 toe. With these limits, a measured slope length of 700 ft was established for design purposes. With the noted slope length of 700 ft and the 0.3% grade of the surface, a time of concentration of 11.6 minutes was determined. Because the Pond 1 embankment flows will be discharged onto the Pond 3 surface, the time of concentration for the precipitation that falls onto the embankment was added to the time of concentration calculated for the Pond 3 surface, making the total time of concentration equal to 13.52 minutes for the system. This time of concentration was then used to obtain the PMF depth using the incremental rain duration provided by Nelson *et. al* (1986). The incremental rainfall duration of 13.52 minutes was used for developing the PMP depth of 6.72 inches for a local storm of 9.6 inches. Using a runoff coefficient of 1.0 (used for rock covered surfaces and no infiltration) and the Rational Method, a unit peak flow of 0.84 cfs/ft for this catchment was calculated. With a maximum surface slope of 0.3%, the

method of Abt *et al.* (1998) predicts a rock d_{50} of 0.4 inches for the erosion protection layer of Pond 3 surface. Calculations are included in Appendix B. The detailed erosion protection design is shown on the design sheets included in Appendix E.

With an available rock size of d_{50} of 1.0 inch adequately meeting the required rock size, the surface erosion protection for Pond 3 will consist of d_{50} = 1.0 inch rock. The surface erosion protection rock will be placed three inches thick which meets the one times the d_{100} of the selected rock size for proper placement of the erosion protection layer (NUREG 1623).

2.3 SIDE SLOPE EROSION PROTECTION – POND 3 EAST EMBANKMENT SLOPE

For the side slope erosion protection design of the east embankment of Pond 3, we used the same assumptions of finish elevations for the Pond 3 surface. For design purposes we assumed an embankment crest elevation of 6,936 ft and a toe elevation of 6,923 ft. Because the Pond 3 embankment toe terminates in the Arroyo del Puerto basin, the toe elevations will vary, and the crest elevation will also vary due to the slope projection of 0.3% along varying horizontal surface lengths of Pond 3. For design purposes, we measured a slope length of 60 to 70 ft at the mid-point of the east embankment of Pond 3, which also corresponds with the longest horizontal flow surface length of Pond 3. This combination yields the greatest potential flow volume that the embankment slope should experience during a PMF event. Rio Algom indicated for design purposes that the embankment slope would be constructed at a 20% grade. For design calculations a slope length of 65 ft was used. The actual length may vary slightly; however, a slight length change should not affect design results.

For the design of the erosion protection system of the embankment slope, two different precipitation events were evaluated. The first was the local PMP that would fall on the slope of Pond 1 and run-off to the surface of Pond 3 combined with the run-off accumulated from rain that falls onto the Pond 3, which will run-off the pond surface, and onto the embankment slope. The second event was a PMF that could occur in the Arroyo del Puerto basin. After evaluation of the two separate storm events, it was found that the PMF that could occur in the Arroyo del Puerto would produce the larger erosional forces. The calculations for the rain event that would fall onto the surfaces noted above and run-off and down the embankment slope are included in Appendix B; however, because the controlling design criteria are developed by a PMF in the Arroyo del Puerto, only that flow is used for design purposes. The velocity and depth of flow determined from the hydraulic analysis were used to determine the riprap sizes on the embankment slope of Pond 3. Calculations for this analysis are found in Appendix B.

For rock sizing to prevent erosion caused by the longitudinal flows, the Army Corp of Engineers method for riprap sizing (ACE 1995) was used. With over-bank flow velocities in the Arroyo del Puerto of 11 fps at a depth of 10 ft, the ACE method predicts a d_{50} of 12 inches. The slope erosion protection rock layer will be placed at a thickness of one times the d_{100} of the selected rock size or 12 inches (NUREG 1623).

2.4 EROSION PROTECTION APRON – POND 3 EAST EMBANKMENT TOE

Again, assumptions were made as to the location of the east embankment toe of Pond 3 due to the on-going reclamation work at Pond 3. For design purposes the embankment toe elevation of 6,923 ft was assumed. The toe apron will dissipate energy from run-off from a 60 to 70 ft long slope constructed at a 20% grade that crests at an elevation of 6,936 ft. The measurement of the slope length is taken from the mid-point of the east embankment of Pond 3 which corresponds with the longest horizontal surface flow length of Pond 3. The largest flows from a PMP would be experienced at this point. For design calculations a slope length of 65 ft was used. The actual length may vary slightly; however, a slight length change should not affect the design results.

For the design of the erosion protection system of the embankment slope, two different precipitation events were evaluated. The first was a local PMP that would fall onto the slope of Pond 1, Pond 3 and run-off onto the Pond 3 east embankment slope where it will flow across the toe apron and into the Arroyo

del Puerto basin. The second event was a PMF that could occur in the Arroyo del Puerto basin. After evaluation of the two separate storm events, the PMF that could occur in the Arroyo del Puerto would produce the larger erosional forces. The calculations for the rain event that would fall onto the surfaces noted above and run-off to the embankment slope and across the apron are included in Appendix B; however, because the controlling design forces that the embankment could experience are developed by a PMF in the Arroyo del Puerto, only that flow is used for design purposes. The velocity and depth of flow determined from the hydraulic analysis were used to determine the need for riprap in the embankment toe apron of Pond 3. Calculations for this analysis are found in Appendix B.

For rock sizing to prevent erosion caused by the longitudinal flows at the toe of the embankment, the Army Corp of Engineers method for riprap sizing (ACE 1995) was used. With over-bank flow velocities in the Arroyo del Puerto of 11 fps at a depth of 10 ft, the ACE method predicts a d_{50} of 12 inches. However, for apron design, dimensions are based on slope runoff rock size, which is 8.5 inches. The apron width was based on 15 times this d_{50} (NUREG 1623) resulting in a channel width of 10.7 ft. For the apron depth, the d_{50} of 12 inch rock size was used. For ease of placement of the larger rock into the apron Algom, a thickness of three times d_{50} results in an apron depth of 36 inches which exceeds present guidance by NUREG 1623.

2.5 NORTH EROSION PROTECTION AREA – SURFACE PROTECTION

The 20 acres of undeveloped land adjacent to the north embankment of Pond 1 were investigated for the need of an erosion protection surface rock layer. The investigation was concentrated on the rainfall that would fall directly on this area and run-off potentially producing head-cutting that might undermine the north embankment channel/apron and embankment slope of Pond 1. Through analysis of the PMP on the area, it was determined that an erosion protection rock layer to be necessary to prevent potential head-cutting. The area is generally bound by topography that limits the area of flows into the system; therefore, only analysis of direct rainfall on this area was studied.

Minor head-cutting due to local rain events was visually observed in this area during site visits. The location of the head-cutting was used as the north boundary of the area to be rock covered for erosion protection. The head-cut area will be re-contoured and an engineered diversion channel constructed in its place to prevent future head-cutting. The diversion channel design is presented in the next subsection of this report, *North Erosion Protection Area – Diversion Channel*. The area south of the diversion channel, designated as the North Erosion Protection Area, will require a rock cover to prevent erosion during rainfall. To provide an effective rock cover, the slopes greater than 7.5% will need to be regraded to slopes equal to or less than 7.5%.

This catchment basin has a maximum elevation of 7,005 ft and a discharge elevation of 6,926. For the time of concentration analysis, a slope length of 1,700 ft was used over a drop in elevation of 79 ft. With the noted slope length and the elevation change of the area, a time of concentration of 7.82 minutes was determined. This time of concentration was then used to obtain the PMP depth using the incremental rain duration provided by Nelson *et. al* (1986). The incremental rainfall duration of 7.82 minutes was used to develop the PMP depth of 5.2 inches for a local storm of 9.6 inches. Using a runoff coefficient of 1.0 (rock covered surfaces with no infiltration) and the Rational Method, a unit peak flow of 1.6 cfs/ft for this catchment was calculated. With a maximum surface slope of 7.5%, the method of Abt *et al.* (1998) predicts a rock d_{50} of 2.2 inches for the erosion protection layer for the North Erosion Protection Area surface. Calculations are included in Appendix B. The detailed erosion protection design is shown on the design sheets included in Appendix E.

2.6 NORTH EROSION PROTECTION AREA – DIVERSION CHANNEL

In the north erosion protection area just described, minor head-cutting caused by past rainfall was visually observed during site visits. Rainfall that falls in the area flows toward the Arroyo del Puerto producing head-cut erosion that is directed west toward the mill site. An investigation into an appropriate erosion control method determined that head-cutting could be prevented by placing an engineered diversion

channel at this location. This approach places an open channel at the base of the slope that will catch the precipitation that falls on the north erosion protection area. The open channel design is based on NUREG-1623 channel design (Appendix D, Section 3) and the Army Corp of Engineers, *Hydraulic Design of Flood Control Channels,* design methods (ACE 1995). According to a phone discussion with Mr. Ted Johnson of the Nuclear Regulatory Commission (NRC), the parameters provided within the NUREG 1623 shall be adequate for design of open channels, using Sections 2 and 3 of Appendix D, the Abt and Johnson Method. The diversion channel terminates at an apron in the Arroyo del Puerto basin.

For development of the erosion protection channel, the 20-acre north erosion protection area was divided into three segments because of the varying slopes and catchment areas along the diversion channel. The first segment extends from Station 0+00 to 9+00 where the channel is relatively gentle and the flows are relatively small. For design of the first segment, the time of concentration was developed using the maximum flow length for this area of 900 ft starting at elevation 7,007 ft and ending just above the increasing slope grades at elevation 6,989 ft. For this flow length, the time of concentration was calculated to be 6.6 minutes. From Nelson et al (1986), a 6.6 minute PMP depth for a local storm of 9.6 inches is 4.8 inches. The estimated catchment basin size is 2.2 acres. The Rational Method gives a peak flow of 97 cfs for the first channel segment. The channel was sized based on this flow and a two percent slope. The rock was sized using the method of Abt *et al* (1998) with a correction for side slope according to ASCE (1995). A rock size of D_{50} = 9.2 inches adequately protected a channel with a bottom width of three feet. Calculations are found in Appendix C.

The second segment runs from station 9+00 to 17+00 which is the steepest terrain traversed by the channel. Because the channel slope varies in this segment, the minimum slope of five percent was used to size the channel and the maximum slope of eight percent was used to size the riprap protection. The time of concentration was calculated based on a top elevation of 7,007 feet, a base elevation of 6,939 feet, and a channel length of 1,700 feet. The resulting time of concentration is 8.3 minutes; the corresponding incremental rainfall is 5.4 inches, and the rainfall intensity is 39 in/hr. Using a catchment basin size of 7.4 acres, the peak flow for the PMP is 288 cfs. A channel bottom width of 20 feet was used to contain this flow and a rock size of D_{50} = 9.2 inches adequately protected this channel on the steepest grades. Calculations are again found in Appendix C.

For the final segment of the channel, the slope is much flatter and the diversion channel will cross a low area that will need to be filled. A slope of 0.9 percent was selected for this segment beginning at station 17+00 and ending at 27+01.The time of concentration calculation was based on a start elevation of 7,007 feet, a base elevation of 6,926 feet and a channel length of 2,701 feet. The resulting time of concentration is 13.2 minutes; the incremental storm depth is 6.7 inches; and the rainfall intensity is 30.5 inches. Using the Rational Method with a catchment area of 16.3 acres, a peak flow of 497 cfs results. A channel with a bottom width of 28 feet will keep the water surface below three feet deep and allow protection of the channel with a rock size of D_{50} = 9.2 inches. Calculations are included in Appendix C.

A hydraulic jump develops between the steeper second segment of the diversion channel and the flatter third segment. In this reach, a hydraulic model was developed using HEC-RAS (USACE, 1998) to determine hydraulic conditions near the hydraulic jump. It was concluded that the channel needed additional height to contain the jump for a distance of about 25 feet. In addition, the transition area has been designed according to the apron design criteria found in NUREG-1623, Appendix D, Section 6 in order to dissipate the energy in this area. This requires placing rock with a D_{50} = 17 inches for a depth of 4.3 feet for the area of the expected hydraulic jump. Calculations are found in Appendix C.

2.7 POND 1 CHANNEL/APRON - DISCHARGE SECTION

Runoff from the north embankment of Pond 1 collects in a apron/channel designed in Task 2 that runs eastward towards the Arroyo del Puerto. Under this task, a discharge channel is designed to connect the apron/channel to the Arroyo del Puerto, circumventing Pond 3 on the north. The discharge channel begins with a transition section 25 feet long to convert the apron/channel to a normal channel with 2:1 side slopes. In the next 25 feet, the channel rock size is increased and thickened to accommodate the

expected hydraulic jump as the grade flattens. The channel then returns to the typical rock size and follows two large radius curves to join with the north erosion area diversion channel at the discharge apron. Curve radii were set about eight times the channel width to prevent increased erosion potential (ASCE, 1995).

The time of concentration calculation for the channel is based on a drop from 7,026 feet at the top of Pond 1 to 6,926 feet at the discharge apron. The length of the slope is 2,706 feet and the resulting time of concentration is 12.2 minutes. The incremental PMP depth for this period is 6.5 inches, and the rainfall intensity is 31.7 in/hr. Using a catchment basin of 15.7 acres, the expected discharge according to the Rational Method is 498 cfs. A channel with a 25-foot bottom width was designed to accommodate this flow on a 1.75 percent grade. The rock was sized according to Abt *et al* (1998) and a factor of 1.2 to was used to allow for the increased tractive force on the side slopes (ASCE, 1995). A D_{50} = 9.2 inch rock provides adequate protection of the channel under these conditions. Calculations are found in Appendix D and Appendix E contains drawing sheets for the discharge channel.

The discharge channel shares a common discharge apron with the north erosion protection area diversion channel. The intent of the discharge apron is to slow water velocities to 4 ft/sec. to prevent scour of the native silty clays found in the Arroyo del Puerto basin. The apron is built on a slope of 0.5 percent, and an apron width of 80 feet reduces the velocity of the combined flows of the two channels (995 cfs) to less than 4 ft/sec. A toe is constructed at the edge of the discharge apron to prevent scour beneath the apron. The toe depth is determined from by the equation presented in Abt *et al* (1996), which is recommended in Section 4 of Appendix D of NUREG-1623. For a one-hour PMP of 9.6 inches falling on the combined area of 32 acres, 1,115,000 cubic feet of water will runoff an impermeable surface. Assuming all water runs off in one hour (a worst-case assumption), the average flow rate is 310 cfs. Using these values in the scour equation, a scour depth of just under six feet results. The toe is designed to extend to a six-foot depth with a 2H:1V slope.

Wingwalls also extend for an additional 25 feet beyond the apron to protect the corners of the apron. The wing walls are constructed with a rock toe extending to a six-foot depth with 2H:1V side slopes. Before covering the rock toes with compacted native materials, the contractor should work filter rock into the upper portion of the rock to prevent the loss of native material in voids.

3.0 OTHER DESIGN/CONSTRUCTION CONSIDERATIONS

The erosion protection methods outlined in this design report are based on current engineering practices for erosion control at uranium mill sites. For these erosion protection methods to perform as intended, a "durable rock" should be used. The erosion protection methods outlined above have been developed around the use of "durable rock" as characterized in Section 7.2.1 *Procedures for Assessing Rock Quality*, (NUREG 1623 & 4620). If suitable rock meeting the criteria set forth by NUREG 1623 and 4620 is not available, oversizing of the rock is permissible per Section 7.2.2 *Oversizing Criteria* (NUREG 1623). If oversizing of the selected rock sizes in this design report is required due to the lack of "durable rock", the erosion protection methods provided in this report should be reviewed. Depending on the magnitude of the change, the change in rock size could affect channel capacity and apron capacity as well as the rock thickness. If a lighter rock such as sandstone is used, allowance may also need to be made for the lower density of the rock.

4.0 REFERENCES

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APPENDIX A

PMP/Local Storm Calculation

MAXIM TECHNOLOGIES INC

JOB NO. 1690030-200 JOB TITLE Quivira DATE 10/10/01 BY BD/RH (rairard)
SUBJECT PMP Calculation - Local Storm CHECKED _____ SHEET 1 OF 1

Reference: Hydrometeorological Report, No. 55A
U.S. Department of Commerce
NOAA, June 1988, Section 14.3

Stepwise Procedure, Local Storm PMP.

Index 1-hr 1-mi² PMP estimate at 5,000 ft elev

∴ From Plate IIIC 1-hr 1-mi² @ 5,000 ft

1.) 1-hr-1mi² PMP @ 5,000 ft el. = 10.5 ins.

2.) Adjustment For Mean Elevation of Site

Mean El. use 7,000 ft Site Elev.

Maximum 12-hr Persisting 1000 MB DP @ 76.6 °F From Figure 4.11

From 14.3 Elevation Adjustment = 0.90

3.) Index 1hr-1mi² PMP Estimate @ Mean El.

0.90 x 10.5 in = 9.5 in 1hr-mi²

4.) From Table 12.4, 6 hr storm is:

1.35 x 1hr = 12.83 in

5.) Areal Reduction Factors: Depend On Basin Location

— Summary: 1hr-1mi² PMP = 9.5" ≈ 9.6"

Previous work at The Site Indicated The PMP = 9.6". Because values we obtained compare to previously calculated values during past design work at Quivira by others, we will use the slightly higher value of 9.6" In our design calcs.

6hr Storm – 2"

HEC1 S/N: 1343001338 HMVersion: 6.33 Data File: n:\quivira\adpin1.txt

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*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *
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*              MAY   1991              *
*                                      *
*           VERSION 4.0.1E             *
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*                                      *
*  RUN DATE  09/07/2001  TIME  15:45:10 *
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*   U.S. ARMY CORPS OF ENGINEE
*   HYDROLOGIC ENGINEERING CEN
*        609 SECOND STREET
*      DAVIS, CALIFORNIA 95616
*          (916) 756-1104
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X     X  X        X               X
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:::                by                 :::
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37 Brookside Road * Waterbury, Connecticut 06708 * (203) 755-1666

THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTUR
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM


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LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

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   2          ID   6-HR. PMF, LOCAL STORM WITH AREAL REDUCTION,  - 9.2 IN.
   3          ID   SEPTEMBER 7, 2001
   4          ID   B. BUCHER, MAXIM TECHNOLOGIES, HELENA, MT
*** FREE ***
              *
              * *** TIME SPECIFICATION
   5          IT      15 01JUL01    0000      50
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              * Rainfall time increment
   6          IN      60
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                    * *** GLOBAL OUTPUT OPTIONS
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  8                 KK      IN1
  9                 KM    HYDROGRAPH FOR ARROYO DEL PUERTO DRAINAGE
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                    * Basin area
 10                 BA     57.6
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                    * Rainfall data
 11                 PB      9.2
 12                 PI      .02      .04      .12      .74      .05      .03
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                    * Basin Losses
 13                 LS        0     73.4        0
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                    * Unit hydrograph
 14                 UD     1.83
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*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                                        *    U.S. ARMY CORPS OF ENGINEE
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609 SECOND STREET

DAVIS, CALIFORNIA 95616

(916) 756-1104

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                    QUIVIRA - ARROYO DEL PUERTO FLOOD HYDROLOGY  FILE:ADPIN3.TXT
                    6-HR. PMF, LOCAL STORM WITH AREAL REDUCTION,  - 9.2 IN.
                    SEPTEMBER 7, 2001
                    B. BUCHER, MAXIM TECHNOLOGIES, HELENA, MT

  7 IO          OUTPUT CONTROL VARIABLES
                      IPRNT           2  PRINT CONTROL
                      IPLOT           0  PLOT CONTROL
                      QSCAL          0.  HYDROGRAPH PLOT SCALE

    IT          HYDROGRAPH TIME DATA
                       NMIN          15  MINUTES IN COMPUTATION INTERVAL
                      IDATE      1JUL 1  STARTING DATE
                      ITIME        0000  STARTING TIME
                         NQ          50  NUMBER OF HYDROGRAPH ORDINATES
                     NDDATE      1JUL 1  ENDING DATE
                     NDTIME        1215  ENDING TIME
                     ICENT          19  CENTURY MARK

                 COMPUTATION INTERVAL    0.25 HOURS
                      TOTAL TIME BASE   12.25 HOURS

        ENGLISH UNITS
            DRAINAGE AREA         SQUARE MILES
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PRECIPITATION DEPTH    INCHES
LENGTH, ELEVATION      FEET
FLOW                   CUBIC FEET PER SECOND
STORAGE VOLUME         ACRE-FEET
SURFACE AREA           ACRES
TEMPERATURE            DEGREES FAHRENHEIT


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          *            *
          **************
                    HYDROGRAPH FOR ARROYO DEL PUERTO DRAINAGE

  6 IN          TIME DATA FOR INPUT TIME SERIES
                      JXMIN          60  TIME INTERVAL IN MINUTES
                     JXDATE      1JUL 1  STARTING DATE
                     JXTIME           0  STARTING TIME

                SUBBASIN RUNOFF DATA

 10 BA           SUBBASIN CHARACTERISTICS
                      TAREA       57.60  SUBBASIN AREA

                 PRECIPITATION DATA

 11 PB                STORM        9.20  BASIN TOTAL PRECIPITATION

 12 PI             INCREMENTAL PRECIPITATION PATTERN
                    0.01      0.01      0.00      0.00      0.01      0.01      0.01      0.01      0.03      0.03
                    0.03      0.03      0.19      0.19      0.19      0.19      0.01      0.01      0.01      0.01
                    0.01      0.01      0.01      0.01
```

```
13 LS          SCS LOSS RATE
                    STRTL        0.72  INITIAL ABSTRACTION
                   CRVNBR       73.40  CURVE NUMBER
                    RTIMP        0.00  PERCENT IMPERVIOUS AREA

14 UD          SCS DIMENSIONLESS UNITGRAPH
                     TLAG        1.83  LAG
```

***

UNIT HYDROGRAPH
39 END-OF-PERIOD ORDINATES

705.	2138.	4134.	7003.	10294.	12728.	14053.	14204.	13660.	12456.
11010.	9091.	7084.	5649.	4569.	3747.	3083.	2512.	2008.	1644.
1341.	1075.	875.	709.	573.	472.	383.	310.	253.	203.
167.	141.	119.	97.	75.	56.	38.	20.	2.	

****
*****

HYDROGRAPH AT STATION IN1

****
*****

DA	MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q	*	DA	MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP
1	JUL	0000	1	0.00	0.00	0.00	0.	*	1	JUL	0615	26	0.00	0.00	0.00	60179
1	JUL	0015	2	0.05	0.05	0.00	0.	*	1	JUL	0630	27	0.00	0.00	0.00	51935
1	JUL	0030	3	0.05	0.05	0.00	0.	*	1	JUL	0645	28	0.00	0.00	0.00	43607
1	JUL	0045	4	0.05	0.05	0.00	0.	*	1	JUL	0700	29	0.00	0.00	0.00	36274
1	JUL	0100	5	0.05	0.05	0.00	0.	*	1	JUL	0715	30	0.00	0.00	0.00	30392
1	JUL	0115	6	0.09	0.09	0.00	0.	*	1	JUL	0730	31	0.00	0.00	0.00	25454
1	JUL	0130	7	0.09	0.09	0.00	0.	*	1	JUL	0745	32	0.00	0.00	0.00	21187
1	JUL	0145	8	0.09	0.09	0.00	0.	*	1	JUL	0800	33	0.00	0.00	0.00	17494
1	JUL	0200	9	0.09	0.09	0.00	0.	*	1	JUL	0815	34	0.00	0.00	0.00	14323
1	JUL	0215	10	0.28	0.27	0.00	2.	*	1	JUL	0830	35	0.00	0.00	0.00	11645
1	JUL	0230	11	0.28	0.24	0.03	29.	*	1	JUL	0845	36	0.00	0.00	0.00	9458
1	JUL	0245	12	0.28	0.21	0.06	128.	*	1	JUL	0900	37	0.00	0.00	0.00	7650

1 JUL 0300	13	0.28	0.19	0.09	358.	*	1 JUL 0915	38	0.00	0.00	0.00	6187
1 JUL 0315	14	1.70	0.78	0.92	1367.	*	1 JUL 0930	39	0.00	0.00	0.00	5029
1 JUL 0330	15	1.70	0.45	1.25	4046.	*	1 JUL 0945	40	0.00	0.00	0.00	4089
1 JUL 0345	16	1.70	0.29	1.41	9223.	*	1 JUL 1000	41	0.00	0.00	0.00	3325
1 JUL 0400	17	1.70	0.20	1.50	17933.	*	1 JUL 1015	42	0.00	0.00	0.00	2708
1 JUL 0415	18	0.11	0.01	0.10	29894.	*	1 JUL 1030	43	0.00	0.00	0.00	2196
1 JUL 0430	19	0.12	0.01	0.10	43620.	*	1 JUL 1045	44	0.00	0.00	0.00	1784
1 JUL 0445	20	0.11	0.01	0.10	57193.	*	1 JUL 1100	45	0.00	0.00	0.00	1455
1 JUL 0500	21	0.11	0.01	0.10	67905.	*	1 JUL 1115	46	0.00	0.00	0.00	1191
1 JUL 0515	22	0.07	0.01	0.06	73880.	*	1 JUL 1130	47	0.00	0.00	0.00	983
1 JUL 0530	23	0.07	0.01	0.06	75200.	*	1 JUL 1145	48	0.00	0.00	0.00	809
1 JUL 0545	24	0.07	0.01	0.06	72759.	*	1 JUL 1200	49	0.00	0.00	0.00	655
1 JUL 0600	25	0.07	0.01	0.06	67547.	*	1 JUL 1215	50	0.00	0.00	0.00	514
						*						

****
*****

TOTAL RAINFALL = 9.20, TOTAL LOSS = 3.26, TOTAL EXCESS = 5.94

PEAK FLOW	TIME	MAXIMUM AVERAGE FLOW					
		6-HR	24-HR	72-HR	12.25-HR	(CFS)	(HR)
(CFS)	75200.	5.50	35645.	17987.	17987.	17987.	
(INCHES)	5.754	5.928	5.928	5.928			
(AC-FT)	17675.	18210.	18210.	18210.			

CUMULATIVE AREA = 57.60 SQ MI

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAG
HYDROGRAPH AT	IN1	75200.	5.50	35645.	17987.	17987.	57.60		

HEC-RAS Plan: ADP PMF River: Arroyo del Puert Reach: Ambrosia Mill Profile: PF 1

Reach	River Sta	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
		(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Ambrosia Mill	7480	78000.00	6938.00	6950.23	6950.23	6952.03	0.008301	19.47	7828.17	1990.25	1.00
Ambrosia Mill	7080	78000.00	6933.06	6947.21	6946.20	6948.42	0.005969	12.40	9063.01	2156.39	0.61
Ambrosia Mill	6810	78000.00	6930.47	6945.79		6946.66	0.002788	8.98	10649.12	1819.70	0.42
Ambrosia Mill	6310	78000.00	6928.82	6943.34		6945.36	0.007495	14.16	6895.15	1257.27	0.69
Ambrosia Mill	5500	78000.00	6924.36	6939.21		6940.49	0.004540	11.20	8856.19	1674.95	0.54
Ambrosia Mill	4580	78000.00	6919.30	6934.77		6936.09	0.005033	12.15	8746.61	1747.90	0.57
Ambrosia Mill	3080	78000.00	6911.05	6926.21		6927.77	0.006100	13.18	8072.49	1660.94	0.62
Ambrosia Mill	1980	78000.00	6905.00	6920.54	6919.80	6921.62	0.005331	12.55	9668.55	2324.85	0.59
Ambrosia Mill	1450	78000.00	6904.00	6916.59	6916.59	6919.12	0.005642	17.14	7020.86	1247.15	0.86
Ambrosia Mill	0	78000.00	6893.00	6907.57	6907.33	6909.55	0.006007	19.05	7643.78	1482.35	0.90

APPENDIX B

CALCULATIONS FOR POND 1 PROTECTION APRON

MAXIM TECHNOLOGIES INC

JOB NO. 1690030-300 JOB TITLE Quivira DATE 12/10/01 BY RLH

SUBJECT Pond 1 Run-off to Pond 3 CHECKED _____ SHEET 1 OF 2

Apron Calculations / Method

Using Method of NUREG CR-1623 Appendix D Section 6 - Riprap Sizing at Toe

1.) Slope Length = 520 ft (From Quivira Fly Over Survey)

Note: Two Slopes Measured For Length Both Similar In Length. Longest Length of 460 ft chosen for calcs.

2.) Maximum Slope:

Length = 520 ft

Δ elev. = 7025 - 6938' = 87 ft

Slope = 17% (From Fly Over Survey)

Embank. Slope Designed At 20% (From Quivira)

Max. Slope = 20% For calculations.

3.) Time of Concentration:

$$t_c = \left(11.9\, L^3 / H\right)^{0.385}$$

$$= \left(11.9 \left(\frac{520}{5280}\right)^3 / 87\right)^{0.385}$$

t_c = 0.032 hrs = 1.92 minutes

4.) * Use 25 min. rainfall From Nelson et al (1986) - Shortest Increment Available

25 min PMP = 0.275 x 9.6" = 2.64"

* - Using 9.6" PMP For Local Storm

(continued)

MAXIM TECHNOLOGIES INC

JOB NO. 1690030-300 JOB TITLE Omnisa DATE 12/10/01 BY RLH

SUBJECT Pond 1 Run-OFF to Pond 3 CHECKED _____ SHEET 2 OF 2

5.) Use Rational Method For Unit Flow Width

L = 460 Area Unith = 520 x 1 ft = 520 ft² = 0.012 acres

$i = 60 \times \frac{2.64''}{2.5} = 63$ in/hr

Flow, q_u @ Unit Width =

$q_u = Cia = 1.0 \times 63 \times 0.012 = 0.76 \frac{cfs}{ft}$

C = 1.0 (Rock Covered Slope)

6.) Using Equation D-13 From NUREG 1623 Appendix D - Rock Sizing

Concentration Factor $C_f = 2.5$

$D_{50} = 10.46 \cdot S^{0.43} (C_f \times q_u)^{0.56}$

$D_{50} = 10.46 \cdot (.2)^{0.43} (2.5 \times 0.76)^{0.56}$

$D_{50} = 7.5''$

Note: Assuming Pond 3 Is Level North to South.

7.) Minimum Apron Configurations:

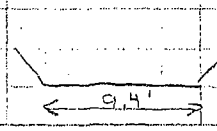
Per NUREG 1623

1) Minimum Depth of Rock @ 7.5" = 3 x D_{50}

Apron Placement Depth = 23"

2) Minimum Width of Rock Apron @ 7.5"

= D_{50} x 15 = 7.5" x 15 = 112.5"



Pavement Width = 9.4 ft

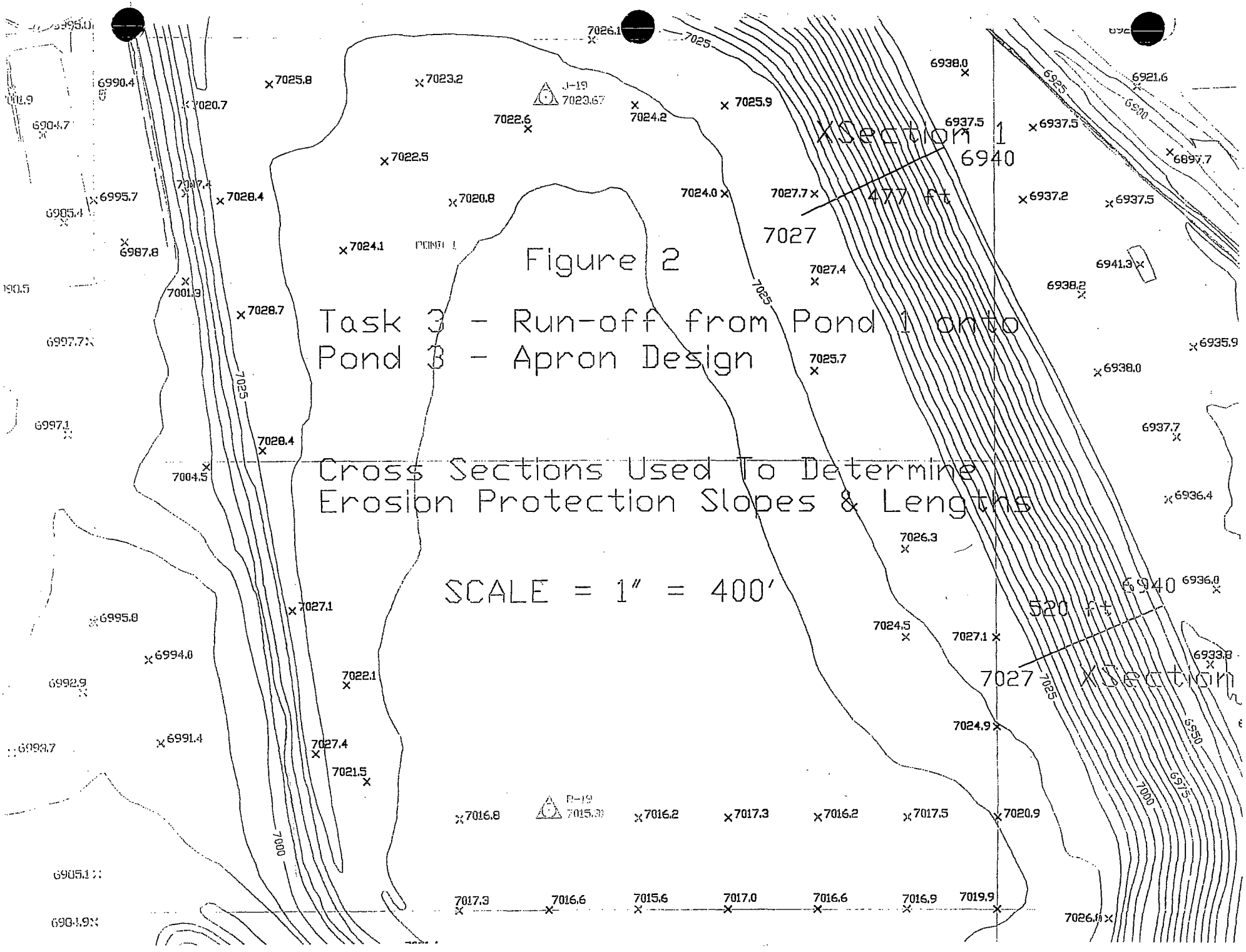


Figure 2
Task 3 - Run-off from Pond 1 onto Pond 3 - Apron Design
Cross Sections Used To Determine Erosion Protection Slopes & Lengths
SCALE = 1" = 400'
XSection 1
6940
477 ft
7027
520 ft
6940
7027
XSection
J-19
7023.67
P-19
7015.39
POND 1
7025
7000
6950
6975

MAXIM TECHNOLOGIES INC

JOB NO. 169003-300 JOB TITLE Quivira DATE 12/10/01 BY RLH

SUBJECT Pond 3 Top Rocked Slope CHECKED _____ SHEET 1 OF 2

Top Slope of Pond 3 Erosion Control Calculations:

Using Method of NUREG CR-1623 Appendix D Section 2 - Rip Rap Sizing on Slopes.

1.) Slope Length = 700 ft (From Quivira Fly Over Survey) (Pond 3 Surface)
Note: Used longest length obtainable on surface of Pond 3.

2.) Maximum Slope:

Length = 700 ft

(*) Set Top of Pond 3 Slope to 0.3%

Assumptions:
Pond 3 Surface will be graded such that concentrated flows are not present; that the Pond 3 Surface will all be constructed at 0.5% with positive drainage into the Arroyo del Puerto.

3.) Time of Concentrations: (For Pond 3 Surface)

$$t_c = (11.9 \times L^3/H)^{0.385}$$

$$= (11.9 \times \frac{700^3}{5.50}/2)^{0.385}$$

$$t_c = 0.193 \text{ hours} = 11.6 \text{ minutes}$$

4.) From Nelson et al (1986) - Percent of Probable Maximum Precipitation For Rainfall Durations. (1 hr = 1 mi²) 9.6" PMP

Total Time of Concent. = t_c + t_c of Pond 1 Slope

$$T_c = 11.6 \text{ min} + 1.92 \text{ min} = 13.52 \text{ min}$$

Rainfall Increment from From Nelson et al. For 13.52 min = % of 1 hour PMP = 70%

∴ 13.52 min PMP = 0.70 × 9.6" = 6.72"

(continued)

MAXIM TECHNOLOGIES INC

JOB NO. 16900030-300 JOB TITLE Quivira DATE 12/10/01 BY RLH

SUBJECT Pond 3 Top Rocked Slope CHECKED _____ SHEET 2 OF 2

5.) Using Rational Method For Unit Flow Width

$L_{P_2 side} = 500 ft$

$L_{P_3 surface} = 700 ft$ Unit Area = 700 ft x 1 ft = 700 ft^2

$\rightarrow L_{System} = L_{P_2 side} + L_{P_3 surface} = 500 + 700 = 1200 ft = 1200 ft^2$

$= \underline{0.028 \text{ acres}}$

$i_{intensity} = 60 \times \frac{6.72}{13.52} = 29.8$ in/hr

$q_{unit flow} = Cia = 1.0 \times 29.8 \times 0.028$

$q_u = \underline{0.84}$ cfs/ft

$C = 1.0$ (Rock Covered Slope)

6.) Using Equation in Section 2.4.1, the Abt and Johnson Method for Top Slopes.

$D_{50} = 5.23 (Slope)^{0.43} (q_u)^{0.56}$

q_u: Unit Flow Across Pond 3 includes Flow (Run Off) From The Pond 1 Embankments. The Unit Flow Was previously calculated For the Runoff Area at The Pond 1 and Pond 3 Interface. Unit Flow Used For The calculating the D_{50} For the Pond 3 Surface was developed using the sum of the t and unit area of both Pond 1 Side Slope and Pond 3 Surface.

$D_{50} = 5.23 (S_{slope})^{0.43} (q_{u_{sum}})^{0.56}$

$D_{50} = 5.23 (0.003)^{0.43} (0.84)^{0.56}$

$D_{50} = \underline{0.4 \text{ inches}}$

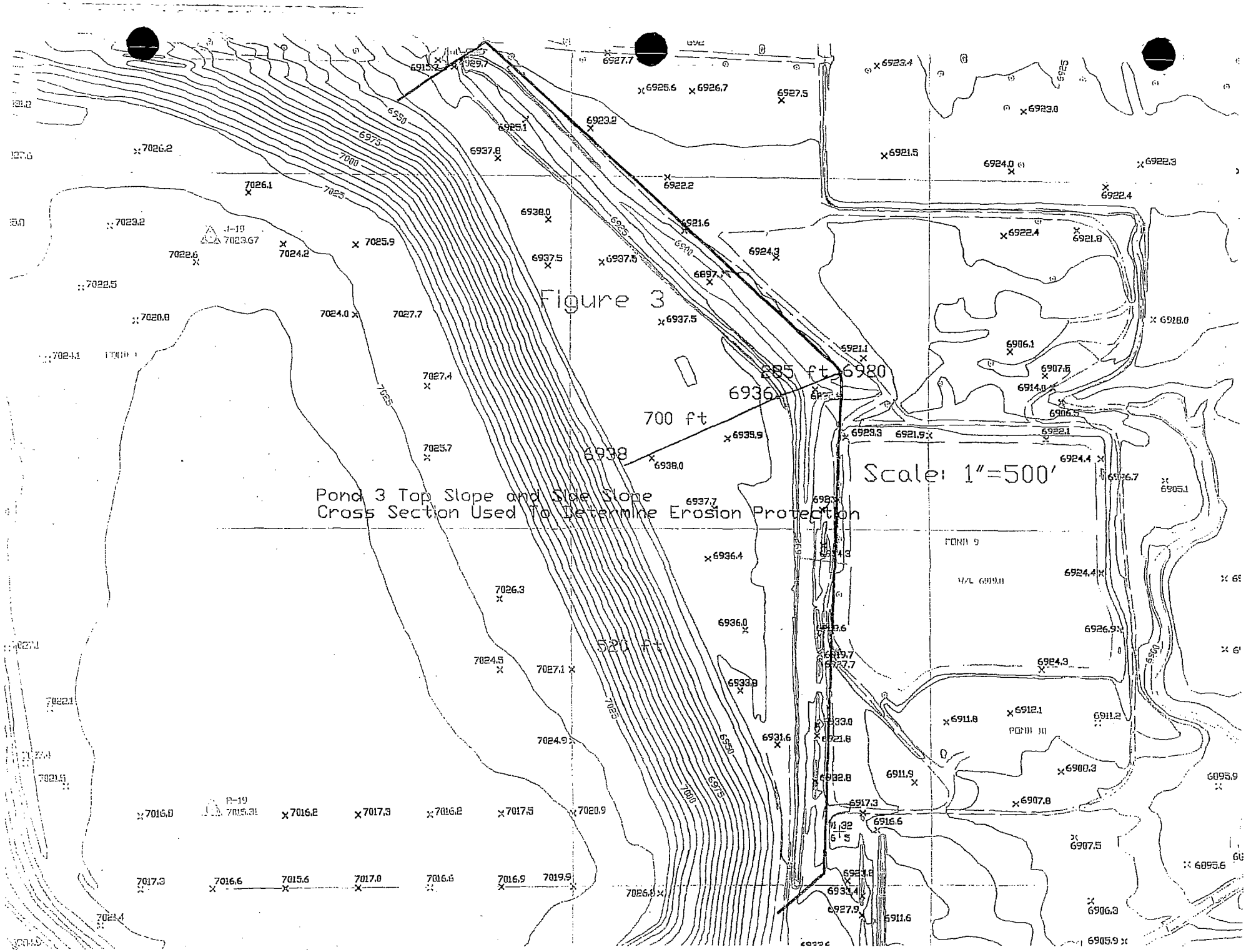


Figure 3
Pond 3 Top Slope and Side Slope
Cross Section Used To Determine Erosion Protection
Scale: 1"=500'
285 ft
700 ft
520 ft
6980
6936
6938
POND 9
POND 10
W/L 6919.0

MAXIM TECHNOLOGIES INC

JOB NO. 169030-300 JOB TITLE Quivira DATE 12/10/01 BY RLH

SUBJECT Pond 3 Side Rocked Slope And Apron Calcs. CHECKED _____ SHEET 1 OF 3

Side Slope of Pond 3 Erosion Control Calculations

Using Method of NUREG CR-1623 Appendix D Section 2: Rip Rap Sizing on Slopes

1.) Slope Length = 65' Ft (From Quivira Fly Over Survey)

Note: Used Average to Long Slope Lengths From Projected Slope on an Estimated Pond 3 Crest Elevation of 6936 ft. And Approximate Toe Elevation of 6923 ft. was used in these Calculations. $\Delta el = 13'$

2.) Maximum Slope:

Length = 65 ft
$\Delta elev = 6936 - 6923 = 13$ ft > 20% Slope 5H:1V

* Assumptions: Crest Elevation of Pond 3 Assumed to be 6936 ft and Toe Elevation to be 6923 ft for Design Calculations. With these Parameters, the Average Slope Length is approximately 65 ft.

3.) Time of Concentrations: (For Pond 3 Embankment Surface)

$$t_c = (11.9 \times L^3/H)^{0.385}$$

$$= \left(11.9 \times \left(\frac{65'}{5280'}\right)^3 / 13'\right)^{0.385}$$

$$t_c = 0.006 \text{ hrs} = 0.36 \text{ minutes}$$

4.) Total Time of Concentration, t_c equal to Σt_c:

$\therefore \Sigma t_c = t_c$ Pond 1 Emb + t_c Pond 3 Surface + t_c Pond 3 Emb.

Total Time of Concentration = t_c + t_c of Pond 1 Side + t_c of Pond 3 Surface

$T_c = 0.36 + 1.92 + 11.6$ min = 13.88 min

Rainfall Increment From Nelson et al For 13.88 min = % 1 hour PMP

$\therefore$ 13.88 min PMP = 0.71 x 9.6" = 6.8"

MAXIM TECHNOLOGIES INC

JOB NO. 1690030-300 JOB TITLE Quivira DATE 12/10/01 BY RLH

SUBJECT Pond 3 Side Packed Slope And Apron Calcs. CHECKED _____ SHEET 2 OF 3

5.) Using Rational Method For Unit Flow Width:

L_{sum} = Pond 1 Side + Pond 3 Surface = 1220 ft

L = 65 ft Area = 65 ft × 1 ft = 65 ft²

L = 1220 + 65 = 1285 ft = 1285 ft² Area = 0.030 acres

$i_{intensity} = 60 \times \frac{6.8}{13.88 \text{ min}} = 29.4 \text{ in/hr}$

$q_{unit\,flow} = Cia = 1.0 \times 29.4 \times 0.030 = \underline{\underline{0.88}} \frac{cfs}{ft}$

C = 1.0 (Rock Covered Slopes)

6.) Using Equation in Section 2.4.1 the Abt and Johnson Method For Side Slopes.

$D_{50} = 5.23 (Slope)^{0.43} (q_u)^{0.56}$

q_u = Unit Flow Across Pond 3 Side Slope includes Flow from Pond 1 side slope plus Pond 3 top slope and Pond 3 side Slope. Unit Flow used to For calculating the D_{50} For Pond 3 Side Slope was developed using the sum of the t_c and unit width area of Pond 1 side Slope, Pond 3 surface and Pond 3 Side Slope

$q_u = 0.88$ cfs/ft

$D_{50} = 5.23 \times (20\%)^{0.43} \times (0.882)^{0.56}$

0.932

$D_{50} = \underline{\underline{2.43 \text{ inches}}}$

7.) Pond 3 Side Slope Rock Apron Calculations: Using Same Parameters and Results Above To calculate Apron Rock sizing with The use of equation D-18 of Appendix D.

$D_{50} = 10.46 \times S^{0.43} \times (q \times C_f)^{0.56}$

(Continued)

MAXIM TECHNOLOGIES INC

JOB NO. 1690030-300 JOB TITLE Quivira DATE 12/10/01 BY RLH

SUBJECT Pond 3 Side Slope/Apron CHECKED _____ SHEET 3 OF 3

~~Rock Calculations.~~

7.) Continued. (Apron Calc. For Rock Size)

$$D_{50} = 10.46 \times (0.20)^{0.43} \times (0.982 \times 3.0)^{0.56}$$

Note: Flow Concentration Factor = 3.0

D_{50} = 8.5 inches

8.) Minimum Apron Configurations

Per NUREG 1623

1.) Minimum Depth of Rock @ Apron 8.5 inches

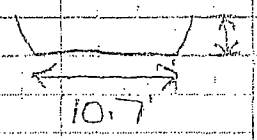
Depth = $3 \times D_{50}$

Depth = 25.5 inches

2.) Minimum Width of Rock Apron @ 5.7 inches

Width = $15 \times D_{50}$

Width = 128 inches = 10.7 ft



MAXIM TECHNOLOGIES INC

JOB NO. 16900030-300 JOB TITLE Quivira DATE 12/10/01 BY RLH

SUBJECT Task 3 - North Arroyo Rock CHECKED _____ SHEET 1 OF 3

- Erosion Protection For Slopes North of Pond 1, Near Pond 1 North Embankment and South of The Arroyo De Puerto

Using Method of NUREG 1623 Appendix D Section 2 - Rip Rap Sizing on Slopes

1.) Slope Length:

Longest Possible Slope Length, 1,700 ft

Starting at Elevation 7005
Ending at Elevation 6926

* Route Chosen From Maximum Possible Length and Along Steeper Sloped Areas.

2.) Average Slope:

Slopes for Calculation of Erosion Protection were obtained from Topographic information provided by Quivira.

- Station 0+00 to Station 6+50

Average Slope is approximately 30

- Station 6+50 to Station 13+00

Average Slope is approximately 15

- Station 13+00 to Station 17+00

Average Slope is approximately 25

3.) Time of Concentration:

Time of Concentration was calculated below using the Entire Drainage Path Length of 1,700 ft.

(continued)

MAXIM TECHNOLOGIES INC

JOB NO. 16907030-300 JOB TITLE Quivira DATE 12/10/01 BY RLH

SUBJECT Task 3- North Area Rock CHECKED _____ SHEET 2 OF 3

3.) Continued.

$$t_c = (11.9 \times L^3/H)^{0.385}$$

$$t_c = \left(11.9 \times \frac{\left(\frac{1700}{5280}\right)^3}{79}\right)^{0.385}$$

$$t_c = 0.13 \text{ hrs} = 7.82 \text{ minutes}$$

4.) From Nelson et al (1986) - Percent of Probable Maximum Precipitation For Rainfall Duration (1 hr - 1 mi²)

@ t_c = minutes; % of 1 hr - PMP = 54%

Using PMP for Local Storm of 9.6"

∴ 7.82 min PMP ⇒ 0.54 × 9.6" = 5.2"

5.) Using Rational Method For Unit Flow Width

L = 1,700 ft Area = 1,700 ft² = 0.04 acres,

$$i_{intensity} = 60 \times \frac{5.2}{7.82} = 39.9 \text{ in/hr}$$

$$q_{unit\ flow} = Cia = 1.0 \times 39.9 \times 0.04$$

$$q_u = 1.6 \text{ cfs/ft}$$

C = 1.0 (Rock Surfaced Slope)

(a.) Using Equation in Section 3.4.1, the Abt and Johnson Method For Slopes,

$$D_{50} = 5.23 (\text{Slope})^{0.43} (q_u)^{0.56}$$

$$D_{50} = 5.23 (7.5\%)^{0.43} (1.6)^{0.56}$$

$$D_{50} = 2.2 \text{ inches}$$ For Entire Slope.

MAXIM TECHNOLOGIES INC

JOB NO. 1690030-300 JOB TITLE Quivira DATE 12/10/09 BY RLH

SUBJECT _____ CHECKED _____ SHEET 3 OF 3

6. a.) Rock Sizing by breaking North Area into two(2) sections: The flatter top slope Area and the steeper longer sections of the lower slopes.

1.) Upper Section: STA 0+00 to STA 6+50

Iterative

- Slope Length L = 650 ft
- Average Slope = 3.0%
- Change In Elev. = 7005 to 6985 = 20 ft
- t_c = 0.073 hrs. = 4.4 min.
- @ t_c = 4.4 min the 4.4 min PMP for a 1hr storm is 8 From Nelson et al (1986)

4.4 min PMP = 9.6" x 0.44 = 4.2 in

- Unit Area - Rational Method Area = 650 ft² = 0.015 acres.
- Intensity $i = 60 \times \frac{4.2}{4.4 \text{ min}} = 57.6$ in/hr
- $q_u = CiA = 1.0 \times 57.6 \times 0.015 = 0.865$
- $D_{50} = 5.23(0.03)^{0.43}(0.865)^{0.56}$

.22

D_{50} = 1.1 inch (Upper 650 ft.)

2.) Lower Section: STA 6+50 to STA 17+00

Rock Size same as calculated in Step 6.

D_{50} = 2.2 inches.

MAXIM TECHNOLOGIES INC

JOB NO. 1690030-3002 JOB TITLE Rio Algom DATE 01/08/03 BY RLH

SUBJECT North Area Erosion/Diversion Channel CHECKED _____ SHEET ____ OF ____

I. Station 0+00 to Station 9+00 (Diversion Channel)

Note: Calculations to reduce rock size used in channel section from 8.2 inches to 7.5 inches. Original design report was submitted using the 8.2 inch value and then a rock size of 9.2 inches was used for the channel configuration to reduce the number of rock gradations required to complete the erosion protection at the mill site, the 9.2" rock is used in several other designs. Now with a D_{50} = 7.8" rock size available for erosion protection, this section was reevaluated for the smaller rock size for cost benefits.

1) Parameters

Area = 2.2 acres

Channel Length = 900 ft

Start Elev. = 7,007 ft

End Elev. = 6,989 ft

Change in Elev. = 18 ft

Average Slope = 2.0%

* See Original Design Report, Appendix G, for development of the above parameters for channel development.

2) Time of Concentration, t_c:

$$t_c = \left(11.9 \cdot \frac{\left(\frac{900}{5280}\right)^3}{18\,ft}\right)^{0.385}$$

t_c = 0.11 hrs = 6.63 minutes

3) Intensity, i:

6.63 min PMP = 51% (9.6 in) = 4.9"

$i = 60 \times 4.9 / 6.63 = 43.9$ in/hr

MAXIM TECHNOLOGIES INC

JOB NO. 690030-300 **JOB TITLE** Rio Algom **DATE** 01/08/03 **BY** RLH

SUBJECT North Area Erosion/Diversion Channel **CHECKED** _____ **SHEET** 2 **OF** 2

4.) Flow, Q:

$Q = CiA = 1.0 \times 43.9 \frac{in}{hr} \times 2.2$ acres

$Q = 96.6$ cfs

5.) Channel Configuration:

Parameters:

- See Attached Spreadsheet, Flow-Pro Output
- 4 ft wide Trapezoidal channel
- 2H:1V side slopes
- Average slope = 2.0% (From Rio Algom Survey)
- 2.2 acre Area
- PMP = 9.6" 1 hr - 1 mi²
- Flow = 96.6 cfs

6.) Sizing of Rock:

$D_{50} = 5.23 \left(\frac{96.6 \text{ cfs}}{4 \text{ ft}}\right)^{0.56} \cdot (2\%)^{0.43}$ 0.10597

$D_{50} \approx 5.8"$ in channel bottom

$D_{50} = 5.8" \times 1.2 = 7.0"$ in channel side slopes

$D_{50} = \underline{7.0"} \approx$ Will Use Available 7.8" Rock For Channel Design per Rio Algom.

$\boxed{D_{50} = 7.8"}$ For Side Slopes & Bottom.

(*) See Original Design Report For Full Development of the above noted values, Appendix C.

Task 3 Channel Design - Channel - Inputs to get channel Depth

DISCHARGE CHANNEL

1. MANNING OUTPUT SECTION 1 STA 0+00 to STA 9+00

OPEN CHANNEL DESIGN STRICKLER'S EQUATION

ASSUMPTIONS & EQUATIONS Where K=Ave Flume Data (ACE 1994)

Mannings Value, n ~K(D90)^0.16667

D90 of D50 = 9.2 Inch Rock Particle Distribution = 12" (See Design Report For Gradation).

Note: Army Corp. of Engineers - Hydraulic Design of Flood Control Channels (1991)

CALCULATION: Using Army Corp. Of Engineers (ACE) Method (ACE, 1991)

Input	Value	Units
D50	9.2	Inches

Output	Value	Units
n	0.0507	

2. CHANNEL DEPTH OUTPUT

Depth, Flowrate, Slope, and Roughness

HEC-2 will compute the depth, flowrate, slope, roughness for the channel type of your choice. It also computes the velocity, area, wetted perimeter, and hydraulic radius.

Depth | Flowrate | Slope | Roughness

Select channel type

Trapezoidal | Circular | U-shaped | Elongated circular

Flowrate (ft^3/s)	96.6	Depth (ft)	2.242
Width	4	Velocity (ft/s)	5.078
Mannings N	.0507	Area (ft^2)	19.023
Bottom slope	.02	Wetted perimeter (ft)	12.027
Side slope	2	Hydraulic radius (ft)	1.58

Compute | Close

APPENDIX C

CHANNEL FLOW CALCULATIONS

MAXIM TECHNOLOGIES INC

JOB NO. 1690030-300 JOB TITLE Quivira DATE 1/10/02 BY RLH

SUBJECT North Area Erosion / Diversion Channel CHECKED _____ SHEET 1 OF 10

Station 0+00 To Station 9+00 (Diversion Channel)

Method: Calculate Flow Rate For Channel Used To Control Flows From South of This Channel And North of Pond 1 & To Prevent Headcutting Using NUREG CR-1623

Location: Area North of Pond 1 Where Possible Head Cutting May Occur. STA 0+00 to 9+00.

- Channel Length = 900 ft
- Start Elev. = 7,007 ft
  End Elev. = 6,989 ft $\Delta El = 18$ ft
- Average Slope $= 100 \frac{\Delta El}{Length} = 100\left(\frac{18 ft}{900 ft}\right) = 2.0\%$

A) PMP For Local Storm = 9.6" (hr - 1m, 2)

Calculate PMF Using Rational Method

1.) Area = 2.9 Acres

- Area Between Diversion Channel & Erosion Protection Channel / Approx. of Pond 1 That May Drain Into The Diversion Channel. Most Run-Off Will Run North East On Area Noted, Without Entering The Channel As Sheet Flow, Draining Into The Discharge Channel Or Beyond Station 9+00. Assuming 75% Of Run Off To Enter Channel From This Area and Will Do So Above Station 9+00.
- Area North of Diversion Channel Shall Flow Away From Channel As Determined From Topographic Information Provided by Rio Algom.

(Continued) Actual Area = 75% of 2.9 Acres = 2.2 acres

MAXIM TECHNOLOGIES INC

JOB NO. 1690030-300 JOB TITLE Quivira DATE 1/10/02 BY RLH

SUBJECT North Area Erosion Diversion Channel CHECKED _____ SHEET 2 OF 10

Continued STA 0+00 to STA 9+00 (Diversion Channel)

2.) Time of Concentration, t_c:

$$t_c = (11.9 \cdot L^3/H)^{0.385}$$

$$t_c = (11.9 \cdot (900)^3/18)^{0.385}$$

$t_c = 0.110$ hrs $= \underline{6.63}$ minutes

3.) Intensity, i:

Incremented Rainfall From Table 2.1 (Nelson, 1986)

6.63 min PMP = 51% (9.6 in) = 4.9"

$i = 60 \times 4.9/6.63 = 43.9$ in/hr

4.) Flow, Q:

$Q = CiA$ $C = 1.0$ (Rock Covered Surface)

$Q = 1.0 \times 43.9 \times 2.2$ acres

$Q = 96.6$ CFS

5.) Channel Configurations:

Parameters:

- See Attached Spreadsheet No. 1
- 3 ft wide - Trapezoidal
- 2H:1V Side Slopes
- Average Slope = 20% (From Quivira)
- 2.2 acre Area (95% of Total Area)
- PMP = 9.6" /hr - 1 in.²
- Flow = 96.6 cfs

(Continued)

MAXIM TECHNOLOGIES INC

JOB NO. 1670.20.200 JOB TITLE Quivira DATE 1/10/02 BY BLH

SUBJECT North Area Erosion Diversion Channel CHECKED _____ SHEET 3 OF 10

Continued STA 0+00 to STA 9+00 (Diversion Channel)

6.) Sizing of Rock

Using Johnson & Abt et al (1998)

$$D_{50} = 5.23\, q_u^{0.56} * S^{0.43}$$

$$q_u = \frac{Q}{Width} = \text{Unit Flow}$$

$$q_u = 96.6/3\ ft = 32.2\ cfs/ft$$

$$D_{50} = 5.23(32.2)^{0.56} * (0.02)^{0.43}$$

$$D_{50} = 6.8''$$ For Bottom of Channel

Side Slope Factor: Per ACE Hydraulic Design of Flood Control Channels Sec. 3.7 Stone Size (1995)

Side Slope Factor = 1.2 X Bottom Size D_{50}

$$D_{50} = 1.2 \times 6.8 = 8.2''$$

Side Slopes

$$\boxed{D_{50} = 8.2''}$$ Use For Side & Bottom of Channel

II Station 9+00 to Station 17+00 (Diversion Channel)

- Channel Length = 800 ft (t_c Length = 1700 ft)
- Start EL = 7,007 ΔEl = 68 ft For t_c
  End EL = 6,939
- Average Slopes = 8% For Rock Sizing & 5% For Channel Sizing
- PMP = 9.6"

1.) Area = 6.5 Acres + Area of Sta 0+00 to 9+00
Area = 6.5 + 2.9 Acres = 9.4 acres

Notes: Assume 70% Flows Into Channel From 6.5 Acre Area
Assume 100% Flows Into Channel From 2.9 Acre Area
A = (.7)6.5 + 2.9 = 7.4 Acres

(continued)

MAXIM TECHNOLOGIES INC

JOB NO. 1690030-300 JOB TITLE Quivira DATE 1/10/03 BY RLH

SUBJECT North Area Diversion Channel CHECKED _____ SHEET 4 OF 10

Continued STA 9+00 to 17+00 (Diversion Channel)

2.) $t_c = \left(11.9 \times \left(\frac{1700}{5280}\right)^3 / 68\ ft\right)^{0.385}$

$t_c = 0.138$ hrs = 83 minutes

3) Intensity, C:

Incremental Rainfall From Table 21 (Nelson 1986)

8.3 min PMP = 0.56 x 9.6" = 5.4"

$i = 60 \times \frac{5.4}{8.3} = 39.0$ in/hr

4.) Flow, Q:

Q = 1.0 x 39.0 x 7.4

C = 1.0 Rock (paved surface)

Q = 288 cfs

5) Channel Configurations:

Parameters:
- Depth - See Attached Spreadsheet No. 2
- 20 ft Wide - Trapezoidal
- 2H:1V Side Slopes
- Average Slope = 5% (Channel Sizing)
- PMP = 9.6" 1hr - Mi²
- Flow = 288 cfs

6.) Sizing of Rock
- Average Slope = 8% (Rock Sizing)

Using Isbash Stability Chart (1936)

$q_w = 288/20 = 14.4$ cfs/ft

$D_{50} = 5.83 (14.4)^{0.56} \times (0.08)^{0.43}$

continued

MAXIM TECHNOLOGIES INC

JOB NO. 1690030-300 JOB TITLE Quivira DATE 1/10/02 BY RLA

SUBJECT North Area Diversion Channel CHECKED _____ SHEET 5 OF 10

Continued STA 9+00 to 17+00 (Diversion Channel)

6.) Continued

$D_{50} = 7.8$ For Bottom of Channel

$D_{50_{SS}} = 1.2 \times D_{50_{Bottom}}$

$D_{50_{SS}} = 9.3''$ For Side Slopes.

$\boxed{D_{50} = 9.3''}$ Use For Side & Bottom of Channel

I. Station 17+00 to Station 27+01 (End) (Diversion Channel)

- Channel Length = 1,001 ft (t_c length = 2,701 ft)
- Start El = 7,007 ft ΔEl = 81 ft for t_c
  End El = 6,926 ft
- Average Slope = 0.9% For Rock Sizing 0.9% For Channel Sizing
- PMP = 9.6" /hr - 1 mi²

1.) Area = 9.2 Acres (75% Into Div. Channel, 20% Into Discharge Channel)
+ 6.5 Acres Area (100%)
+ 2.9 Acres Area (100%)
$A = (0.75 \times 9.2) + 6.5 + 2.9 = \underline{16.3}$ acres

2.) $t_c = \left(11.9 \times \left(\frac{2701}{2380}\right)^3 / 8\right)^{0.385} \times 60 = \underline{13.2}$ min.

3.) Intensity, i:

Incremental Rainfall From Table 2.1 (Nelson 1986)

13.2 min PMP = 0.70 × 9.6 = 6.72"

$i = 60 \times \frac{6.72}{13.2} = 30.5$ in/hr

4.) Flow, Q:

$Q = 1.0\ (30.5)(16.3) = \underline{497\ cfs}$

C = 1.0 (Rock lined surface)

Continued STA 17+00 to Station 27+01 (End) (Diversion Channel)

5.) Channel Configurations:

Parameters:
- Depth - see Attached Spreadsheet, No. 3
- 28' ft wide - Trapezoidal
- 2H:1V Side Slopes
- Average Slope = 0.9% for Channel Sizing
- PMP = 9.6" 1hr - 1mi²
- Flow = 497 cfs

6.) Sizing of Rock
- Average Slope = 0.9% (Rock Sizing)

Using Johnson & Abt et. al (1998)

$$q_u = \frac{497}{28} = 17.75 \text{ cfs/ft}$$

$$D_{50} = 5.23 (17.75)^{0.56} (0.009)^{0.43}$$

$$D_{50} = 7.2 \text{ inches for Bottom of Channel}$$

$$D_{50} = 7.2(1.2) = 8.7 \text{ inches Side Slopes}$$

$$\boxed{D_{50} = 8.7 \text{ inches}} \text{ for Side \& Bottom of Channel}$$

IV Channel Summary: (Diversion Channel)

Location	D_{50}	Width	Depth
STA 0+00 to STA 09+00	8.2*	3 ft	3.5 min**
STA 09+00 to STA 17+00	9.3*	20 ft	2.5 min**
STA 17+00 to STA 27+01	8.7*	28 ft	3.5 min**

* Per Rio Algom - A D_{50} = 9.2" Rock Size has been used in other design tasks @ Ambrosia Lake Mill. To Reduce Rock Gradings For Erosion Protection @ the Mill Site - D_{50} = 9.2 in. Will Be Used Throughout

(Continued)

MAXIM TECHNOLOGIES INC

JOB NO. 1690030-300 JOB TITLE Quivira DATE 1/10/02 BY RLA

SUBJECT North Diversion Channel CHECKED _____ SHEET 7 OF 10

** Depth: The Channel Depth Was Determined Using a Channel Hydraulics Modeling Program, Flow Pro 2.0. Summary Sheets Are Attached.

Task 3 Channel Design - Channel - Inputs to get channel Depth

No.1

1. MANNING OUTPUT

OPEN CHANNEL DESIGN - STRICKLER'S EQUATION

ASSUMPTIONS & EQUATIONS — Where K=Ave Flume Data (ACE 1994)

Mannings Value, n ~ K(D90)^0.16667

D90 of D50 = 9.2 inch Rock Particle Distribution = 12" (See Design Report For Gradation)

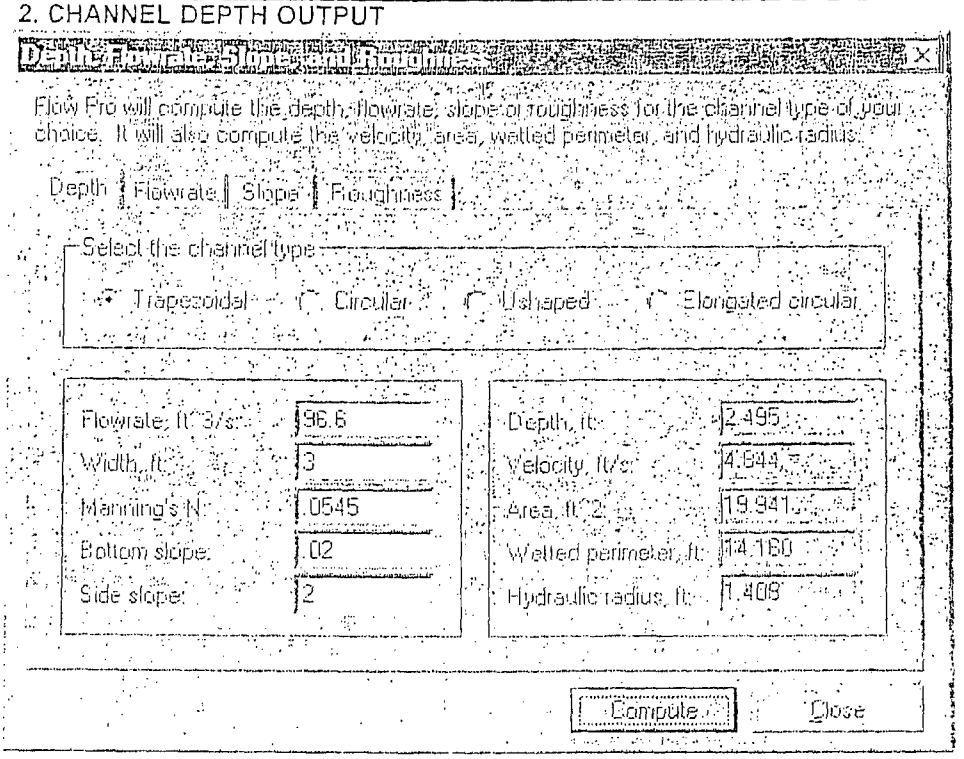
Note: Army Corp. of Engineers - Hydraulic Design of Flood Control Channels (1991)

CALCULATION: Using Army Corp. Of Engineers (ACE) Method (ACE, 1991)

Inputs	Value	Units
D50	12	inches

Output	Value	Units
n	0.0545	

2. CHANNEL DEPTH OUTPUT



Task 3 Channel Design - Channel - Inputs to get channel Depth

No. 2

1. MANNING OUTPUT

OPEN CHANNEL DESIGN - STRICKLER'S EQUATION

ASSUMPTIONS & EQUATIONS — Where K=Ave Flume Data (ACE 1994)

Mannings Value, n ~ K(D90)^0.16667

D90 of D50 = 9.2 inch Rock Particle Distribution = 12" (See Design Report For Gradation)

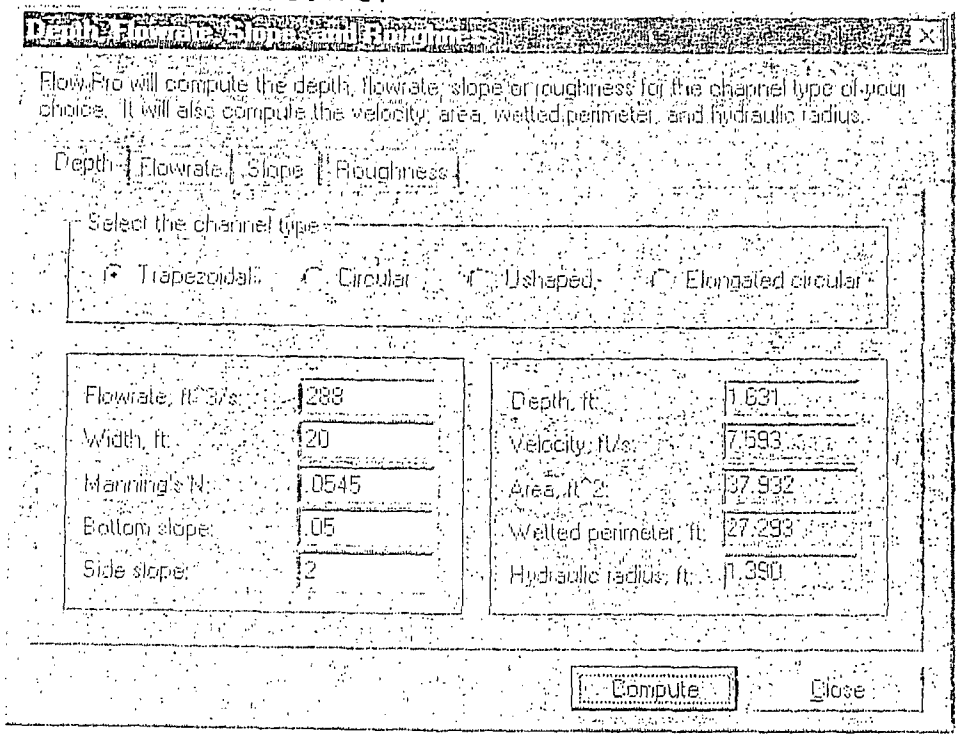
Note: Army Corp. of Engineers - Hydraulic Design of Flood Control Channels (1991)

CALCULATION: Using Army Corp. Of Engineers (ACE) Method (ACE, 1991)

Inputs	Value	Units
D50	12	inches

Output	Value	Units
n	0.0545	

2. CHANNEL DEPTH OUTPUT



10/10

Task 3 Channel Design - Channel - Inputs to get channel Depth

No.3

1. MANNING OUTPUT

OPEN CHANNEL DESIGN - STRICKLER'S EQUATION

ASSUMPTIONS & EQUATIONS — Where K=Ave Flume Data (ACE 1994)

Mannings Value, n ~ K(D90)^0.16667

D90 of D50 = 9.2 inch Rock Particle Distribution = 12" (See Design Report For Gradation)

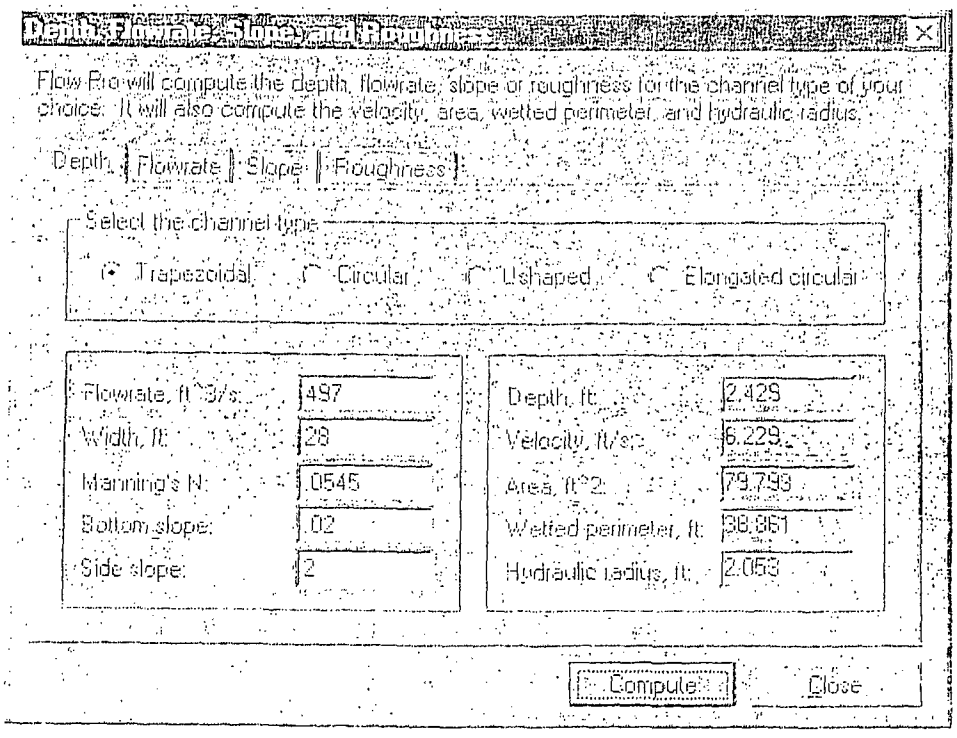
Note: Army Corp. of Engineers - Hydraulic Design of Flood Control Channels (1991)

CALCULATION: Using Army Corp. Of Engineers (ACE) Method (ACE, 1991)

Inputs	Value	Units
D50'	12	inches

Output	Value	Units
n	0.0545	

2. CHANNEL DEPTH OUTPUT



MAXIM TECHNOLOGIES INC

JOB NO. 1690030-300 JOB TITLE Rio Algom DATE 3-11-02 BY RLH

SUBJECT D_{50} = 17" Rock Gradation CHECKED _____ SHEET 1 OF 1

— D_{50} Gradation Determination:

Method: Hydraulic Design of Flood Control Channels Section 3-2 Chapt 3, Army Corp of Eng. 1995

Hydraulic Jump Section On Channels Require Rock Size of D_{50} = 17".

1.) D_{50} = 17", Per Figure 3-2 ⇒ 250 lbs

2.) From Table 3-1 - Gradations For RipRap

With D_{50} ⇒ W_{50} = 250 lbs.

Using D_{100} = 27" Gradation Max Size

Size	Weight Range (Gradation) lbs Max	Min
D_{100}	984 (27")	394 (20")
D_{50}	292 (19")	197 (15")
D_{15}	146 (14")	62 (10")

∴ D_{100} = 27"

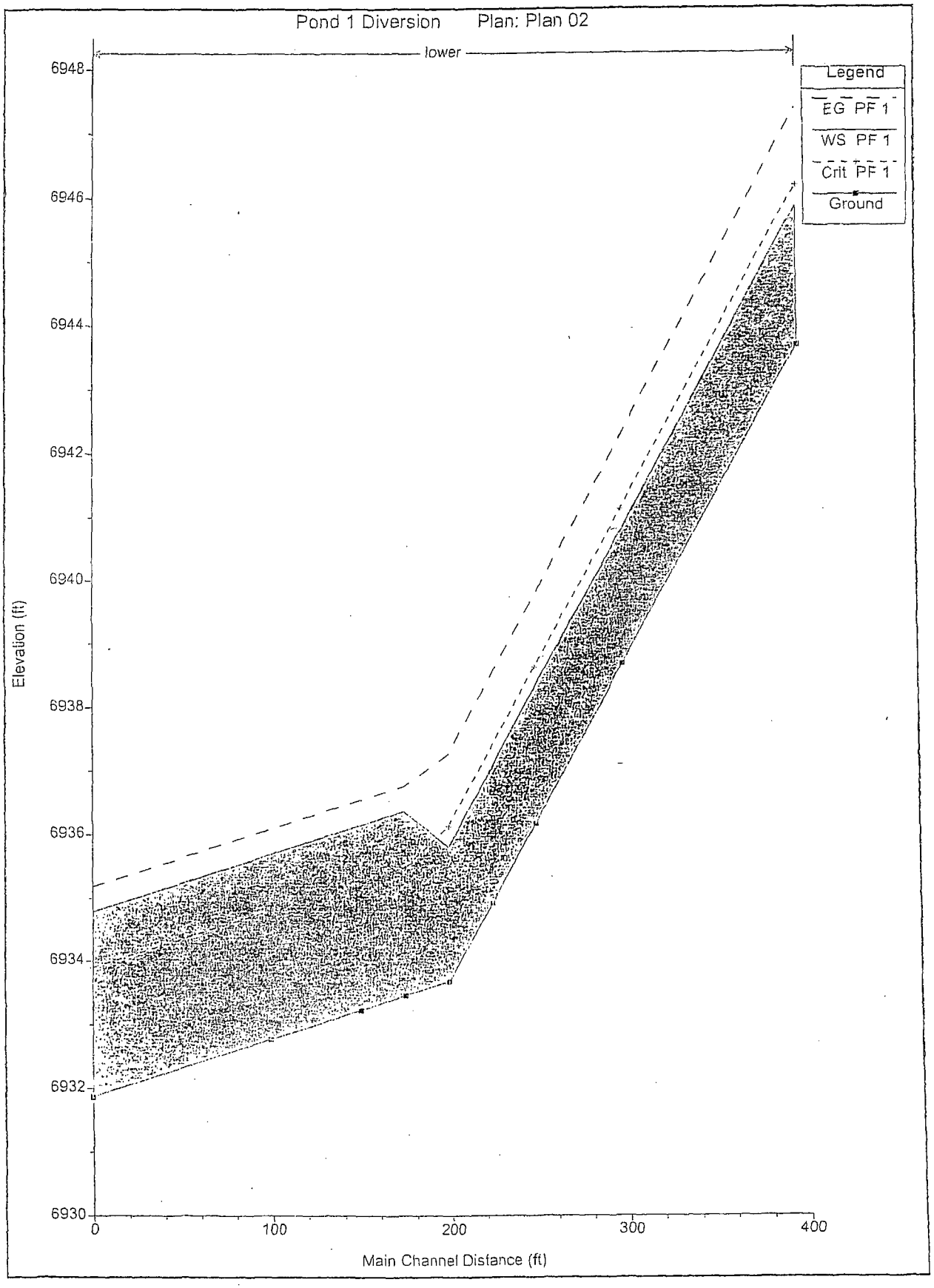
D_{50} = 17"

3.) Gradation:

Sieve Designation	% Passing
27	100%
15"	50-85%
14"	15-50%
10"	0-15%

HEC-RAS Plan: Plan 02 River: Pond1diversion Reach: lower Profile: PF 1

Reach	River Sta	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
		(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq.ft)	(ft)	
lower	400	498.00	6943.67	6945.85	6946.18	6947.38	0.050035	9.93	50.16	26.09	1.26
lower	300	498.00	6938.67	6940.77	6941.12	6942.26	0.052044	9.78	50.93	28.42	1.29
lower	250	498.00	6936.17	6938.32	6938.62	6939.73	0.048539	9.55	52.14	28.59	1.25
lower	225	498.00	6934.92	6937.05	6937.37	6938.49	0.049952	9.64	51.64	28.52	1.26
lower	200	498.00	6933.67	6935.80	6936.12	6937.24	0.049952	9.64	51.64	28.52	1.26
lower	175	498.00	6933.45	6936.36	6935.47	6936.76	0.008986	5.05	98.56	39.65	0.56
lower	150	498.00	6933.22	6936.14		6936.53	0.008939	5.04	98.73	39.67	0.56
lower	100	498.00	6932.77	6935.68	6934.80	6936.08	0.008986	5.05	98.56	39.65	0.56
lower	0	498.00	6931.87	6934.78	6933.90	6935.18	0.009007	5.06	98.48	39.65	0.57


Pond 1 Diversion Plan: Plan 02
lower
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Elevation (ft)
6948
6946
6944
6942
6940
6938
6936
6934
6932
6930
0
100
200
300
400
Main Channel Distance (ft)

Appendix C.6 ..... Amendment to Erosion Protection Drawings for Pond 3 Area Task 3 Design Report

by Maxim Technologies
October 13, 2006

MAXIM
TECHNOLOGIES
A DIVISION OF TETRA TECH, INC.

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Office 505.237.8440
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October 13, 2006

Mr. Terry Fletcher
Rio Algom Mining, LLC
Highway 605 & 509
Grants, NM 87020

Reference: Amendments to Erosion Protection Drawing for Pond 3 area
Task 3 Design Report
Ambrosia Lake Mill, Grants, NM

Dear Mr. Fletcher,

As requested by Rio Algom Mining, LLC, Maxim Technologies has conducted a review of the erosion protection design for the Pond 3 area. Specifically, this office reviewed the erosion protection rock sizing and the need for a secondary sand filter beneath the primary filter material being placed under the riprap erosion protection material.

The following is a summary of our findings and drawing revisions on the above issues

- Rock sizing – we reviewed the rock size requirements at the Pond 1-Pond 3 interface. The correct rock size is D_{50}=7.8 inches. Drawing 3 of 13 has been corrected to reflect this value.

- Filter Layers (Bedding Material) – The use of multi-layer filters below the rock protection layer for the Pond 3 area was analyzed. The result of the analysis is presented in a design memorandum attached to this letter. The analysis shows that a primary filter having the gradation as shown on the plans will meet the required criteria for protection and that a secondary filter (sand layer) is not needed. Drawings 3, 4, and 5 representing the channel for the Pond 3 area have been revised with the secondary sand layer removed. Please note that slight adjustments to dimensions of the channels and other design features have been made due to the removal of the secondary sand layer.

- An additional detail on Drawing 4 of 13 has been provided to illustrate the crest of Pond 3 and the blending of the side slope erosion protection (D_{50}=12") with the Pond 3 surface erosion protection (D_{50}=1.0").

- Drawing 5 of 13 has been revised to extend the D_{50}=1.0" rock along the northeast and east sides of the Pond 3 embankment at the toe of the erosion protection apron. The extended rock shall be blended to match the natural grade surface within an approximate 10-foot distance.

MAXIM

After reviewing this letter, drawings and design memorandum, if you have any questions, please do not hesitate to contact this office, 505-237-8440.

Sincerely,

Curtis O. Sealy, P. E.
Maxim Technologies

CURTIS O. SEALY
NEW MEXICO
10084
LICENSED PROFESSIONAL ENGINEER

Amendment to the Pond 3 area

Erosion Protection

Submitted April 22, 2005 and July 28, 2003

Rio Algom Mining, LLC

Ambrosia Lake Mill, Grants, New Mexico

By
Maxim Technologies
Albuquerque, New Mexico

October 13, 2005

October 13, 2006

Design Memorandum for Placement of Filter Layers (Bedding Media) Below Riprap – Pond 3 Erosion Protection and Erosion Protection for the Area North of Pond 1

Rio Algom Mining Company

Ambrosia Lake Mill, New Mexico

Where riprap is placed on hydraulic structures for erosion protection it is well understood that a filter layer (bedding material) is necessary between the native fine grained materials (protected soil) and the coarse riprap. The locations where filter layers are generally required beneath riprap include dams, tailings impoundment covers including side slopes, toes of slopes, and transition areas. Flow impact areas, stilling areas and channels also require the use of filter layers underneath riprap. The primary purpose of the filter layer is to bed the riprap and prevent the loss of the protected soil into the coarse stone. The filter material also prevents penetration of rock into the soil and prevents soil erosion due to flow at the soil/rock interface. The layer also serves to inhibit pooling of precipitation and runoff from infiltrating into the impoundment cover. For the case of tailings pile covers and open channels, the situation differs from seepage through dams because of the lack of a significant hydraulic gradient from the native protected soil into the rock Water tends to move along the interface between the coarse riprap layer and the protected fine grain soil layer. In this case the need for a fine bedding layer below the primary filter (multiple filters) is seldom justified. However, the need for an additional bedding layer below the primary filter can be analyzed by methods presented in the USNRC regulatory guidance documents.

The issue becomes the potential for loss of fine grained material at the interface of the filter material and native soil. If the interstitial flow velocity at the primary filter/soil interface is insufficient to move soil particles then a secondary filter material is not required. USNRC guidance documents NUREG-4620 and NUREG-1623 recommends an analysis proposed by Leps (1973)[1] be used to evaluate the interstitial velocity at the filter/soil interface.

---

[1] Leps, J. M., "Flow through Rockfill," **Embankment Dam Engineering**, John Wiley and Sons, pp 87-107 1973

Leps investigated the flow through rock and determined the following relationship based on properties of the porous media.

$$V_v = Wm^{0.5} I^{0.54} \quad (1.1)$$

Where Vv is the average velocity of water (inches/sec) in the voids of the rockfill, W is an empirical constant for a specific rock material, m is the hydraulic mean radius and I is the hydraulic gradient. Table 1 presents a series of empirically derived values for the hydraulic mean radius, m, and the $Wm^{0.5}$ parameter as developed by Leps and presented in NUREG-4620.

Table 1
Empirically derived values for equation 1.1

Rock Size (Inch)	m (Inch)	$M^{0.5}$ ($Inch^{1/2}$)	$Wm^{0.5}$ (Inch/sec.)
3/4	0.09	0.30	10
2	0.24	0.49	16
6	0.75	0.87	28
8	0.96	0.98	32
24	3.11	1.76	58
48	6.43	2.54	84

Using equation 1.1 the allowable velocity was calculated for the following parameters.

Slope = 20 percent

D_{50} rock size = 1 inch

$Wm^{0.5}$ = 12 interpolated from Table 1.

$$\text{Then } V_v = \frac{12(0.2)^{.54}}{12 \text{ (Unit Conversion)}} = 0.42 \text{ ft/sec}$$

Generally sizes of rock in the filter layer range from 3/16 inch to an upper limit of 3 to 3 ½ inches, depending on the gradation of the riprap. Typically, the filter blanket thickness is one-half the riprap layer thickness.

A series of tests were conducted at Colorado State University by Abt and Ruff[2] (1985) and Equation 1.1 was found to be accurate within ± 15%.

The USNRC guidance document NUREG-1623 suggests that when the computed interstitial velocity is less than 0.5 feet/sec a secondary filter below the primary rock filter may not be needed. When velocities are between 0.5 and 1.0 feet/sec the need for a filter

[2] Abt, S. R. and Ruff, J. R., 1985 Preliminary Results of Interstitial flow and Overtopping flow tests on riprap. Presented to the Nuclear Regulatory Commission, Silver Spring, Maryland, November 21, 1985.

layer will be dependent upon the type of soil material placed at the interface. The guidance further recommends that a filter be provided for interstitial velocities that exceed 1.0 feet/sec.

Using equation 1.1 with a primary filter blanket gradation below the riprap shown in Table 2, the interstitial velocity at the filter /soil interface for a 20 percent slope will be 0.4 feet/second. Consequently a secondary filter layer (sand) between the primary filter blanket and soil interface will not be required for slopes less than this value.

Table 2
Primary Filter Material Gradation
Ambrosia Lake, Facility
Rio Algom Mining Company, LLC

Sieve Designation	Percent Passing
3"	100
2"	80-100
¾"	20-70
3/8"	10-30
No 4	0-10

CURTIS O. SEALY
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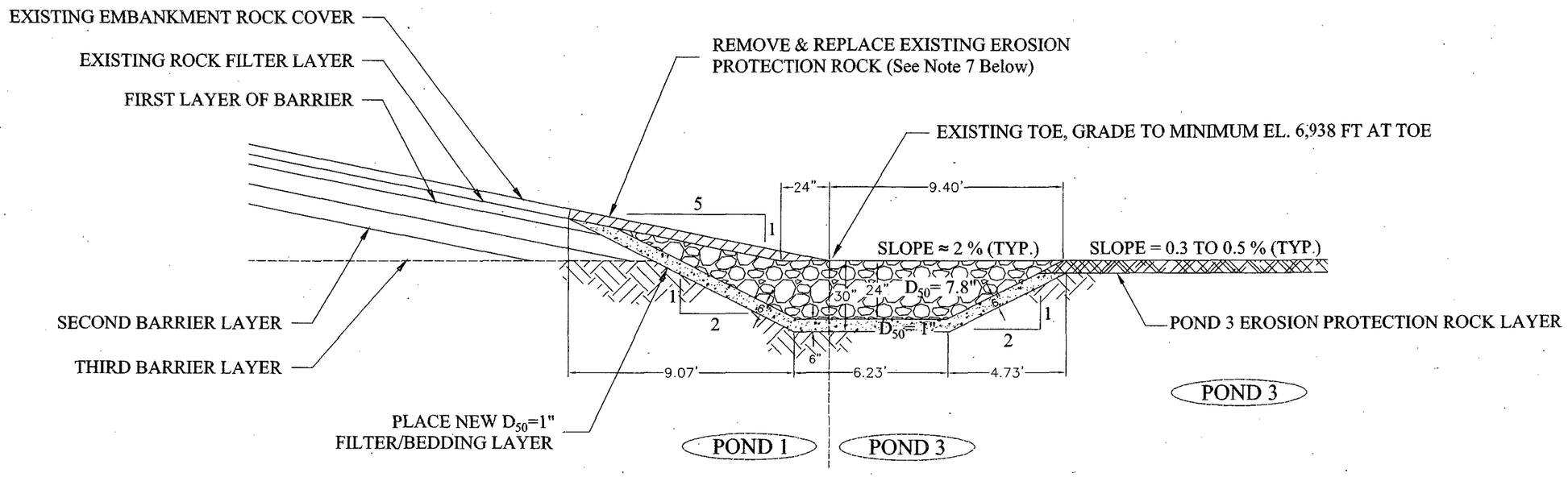
LICENSED PROFESSIONAL ENGINEER

Curtis O. Sealy, P.E.

POND 1 EROSION PROTECTION APRON
FOR RUN-OFF FROM POND 1 EMBANKMENT SLOPE ONTO POND 3 SURFACE
Typical Section



NOTES:

1. Rock Riprap for erosion protection aprons shall be placed in conformance with Appendix F of *Design of Erosion Protection for Long-Term Stabilization,* U.S. Nuclear Regulatory Commission's NUREG - 1623 Draft Report.
2. Erosion protection aprons shall be sloped to the downstream edge with a minimum slope of two percent or at a slope that matches the slope of the natural ground, should it be steeper than two percent.
3. Erosion protection apron excavations shall be constructed with 2H:1V slopes to permit placement of the filter materials as shown above. No tailings shall be excavated. The toe of tailings determines the alignment of erosion protection apron.
4. The erosion protection apron shall be constructed of a rock diameter D_{50} = 7.8" conforming to the following radation:

Sieve Designation	Percent Passing
12"	100
9"	60 - 85
6"	5 - 30
4"	0 - 5

5. The erosion protection filter/bedding layer shall be placed at a minimum thickness of 6" along the length of the apron. The filter/bedding layer shall extend up the 2H:1V sides and end below the Pond 3 erosion protection rock. Filter/bedding material shall be spread and compacted in one layer.
6. The erosion protection apron filter/bedding layer shall be constructed of a rock diameter D_{50}=1", conforming to the following gradation:

Filter/Bedding Gravel

Sieve Designation	Percent Passing
3"	100
2"	80 - 100
3/4"	20 - 70
3/8"	10 - 30
No.4	0 - 10

7. Existing erosion protection disturbed during construction of the erosion protection aprons shall be replaced in a manner that maintains existing slopes and riprap conditions as approved previously by the U.S. Nuclear Regulatory Commission. Care shall be taken in removing and stockpiling the riprap such that the material is not degraded or otherwise damaged. Rock degraded or otherwise out of conformance with the NUREG 1623 due to removal and replacement methods shall be replaced with similar approved materials. Care should be taken so that existing tailings are not disturbed during erosion protection apron construction.
8. The erosion protection apron shall be constructed continuously from the north embankment erosion protection channel/apron (north end of Pond 3) to the south embankment erosion protection channel/apron (south end of Pond 3).
9. The erosion protection apron shall be constructed level from the north end to the south end to prevent longitudinal flows within the apron. The erosion protection apron shall be constructed such that flow from the embankment will flow perpendicular to the apron and onto the Pond 3 surface where it will drain away from the erosion protection apron.

1 in = 5 ft

CURTIS O. SEALY
NEW MEXICO
10084
LICENSED PROFESSIONAL ENGINEER

CURTIS O. SEALY, PE

10/13/06
DATE

AMBROSIA LAKE MILL
RIO ALGOM MINING, LLC.
GRANTS, NEW MEXICO
TASK 3 EROSION PROTECTION
POND 1 TOE EROSION PROTECTION APRON

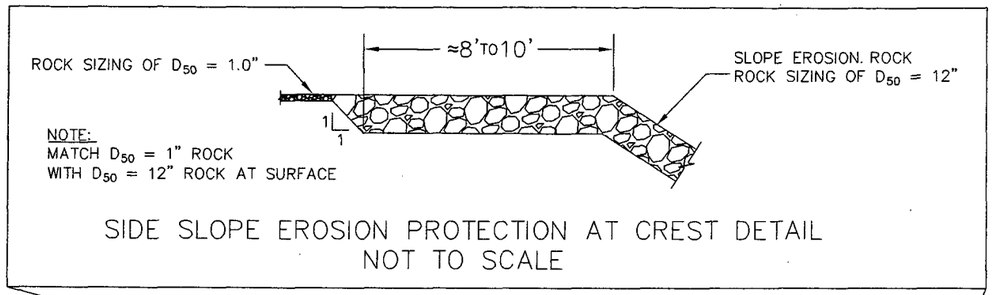
MAXIM TECHNOLOGIES INC

PROJECT No. 1690030-300
FILE NAME: 1690030S3-R2.DWG
DRAWING BY: RLH 2/28/02
MODIFIED BY: EJS 10/4/06
REVIEWED BY: COS

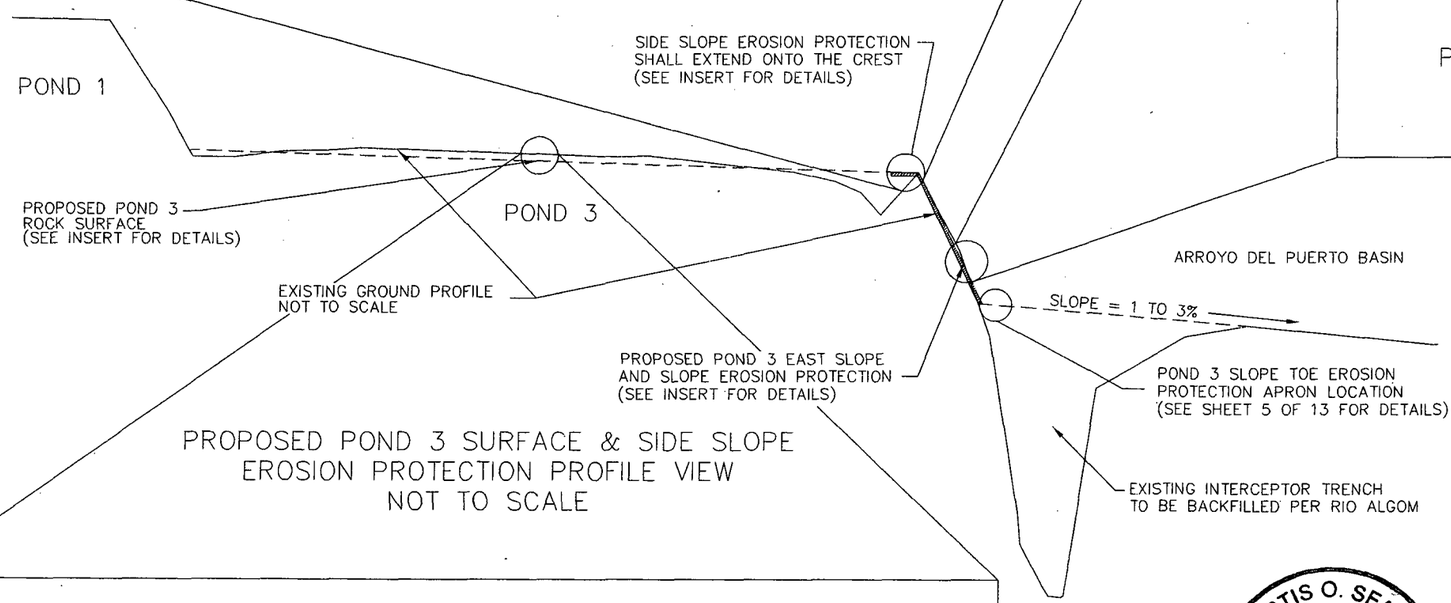
SHEET 3 of 13

POND 3 - POND SURFACE AND EAST EMBANKMENT EROSION PROTECTION

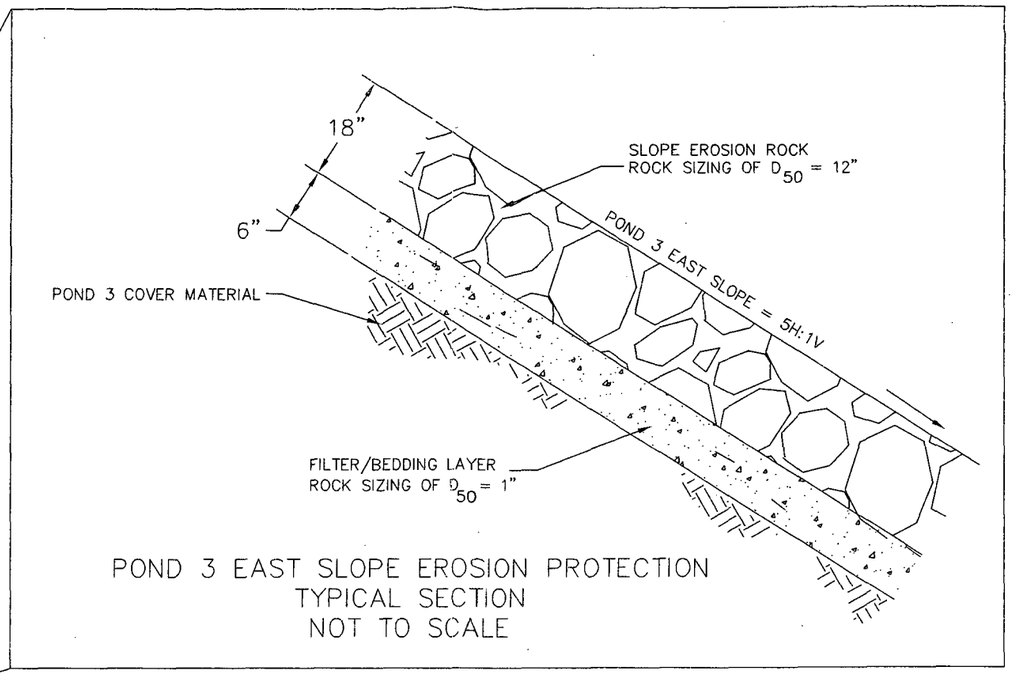
Typical Sections



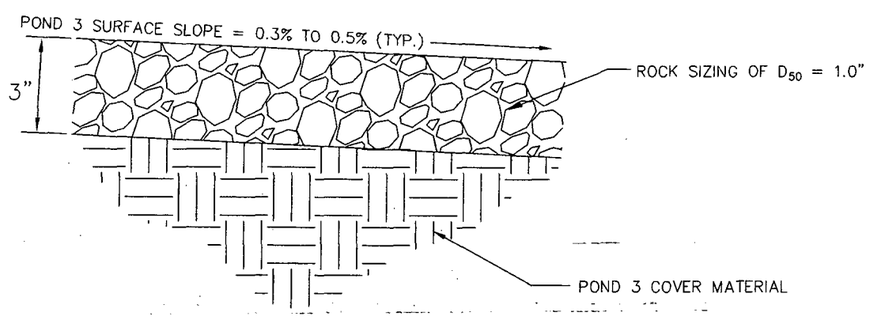
SIDE SLOPE EROSION PROTECTION AT CREST DETAIL
NOT TO SCALE



PROPOSED POND 3 SURFACE & SIDE SLOPE
EROSION PROTECTION PROFILE VIEW
NOT TO SCALE



POND 3 EAST SLOPE EROSION PROTECTION
TYPICAL SECTION
NOT TO SCALE



POND 3 SURFACE EROSION PROTECTION TYPICAL SECTION
NOT TO SCALE

NOTES:

1. Rock Riprap for erosion protection shall be placed in conformance with Appendix F of *Design of Erosion Protection for Long-Term Stabilization*, U.S. Nuclear Regulatory Commission's NUREG - 1623 Draft Report.
2. The erosion protection for the surface of Pond 3 shall be constructed of a rock diameter D_{50} = 1.0" conforming to the following gradation:

Erosion Protection Rock (D_{50} = 1.0")

Sieve Designation	Percent Passing
3"	100
2"	80 - 100
3/4"	20 - 70
3/8"	10 - 30
No.4	0 - 10

3. Erosion protection on the east embankment slope of Pond 3 shall be constructed of a rock diameter D_{50} = 12.0" and Dhall be placed on a minimum 6" thick filter/bedding layer, and conform to the following gradation:

Erosion Protection Rock (D_{50} = 12")

Sieve Designation	Percent Passing
18"	100
14"	60 - 90
12"	25 - 50
10"	10 - 30
6"	0 - 10

4. The erosion protection filter/bedding layer shall be placed at a minimum thickness of 6" along the length of the slope. The filter/bedding layer shall extend the full slope length. Filter/bedding material shall be spread and compacted in one layer, and be constructed of a rock diameter of D_{50} = 1" conforming to the following gradation:

Filter/Bedding Gravel (D_{50} = 1.0")

Sieve Designation	Percent Passing
3"	100
2"	80 - 100
3/4"	20 - 70
3/8"	10 - 30
No.4	0 - 10

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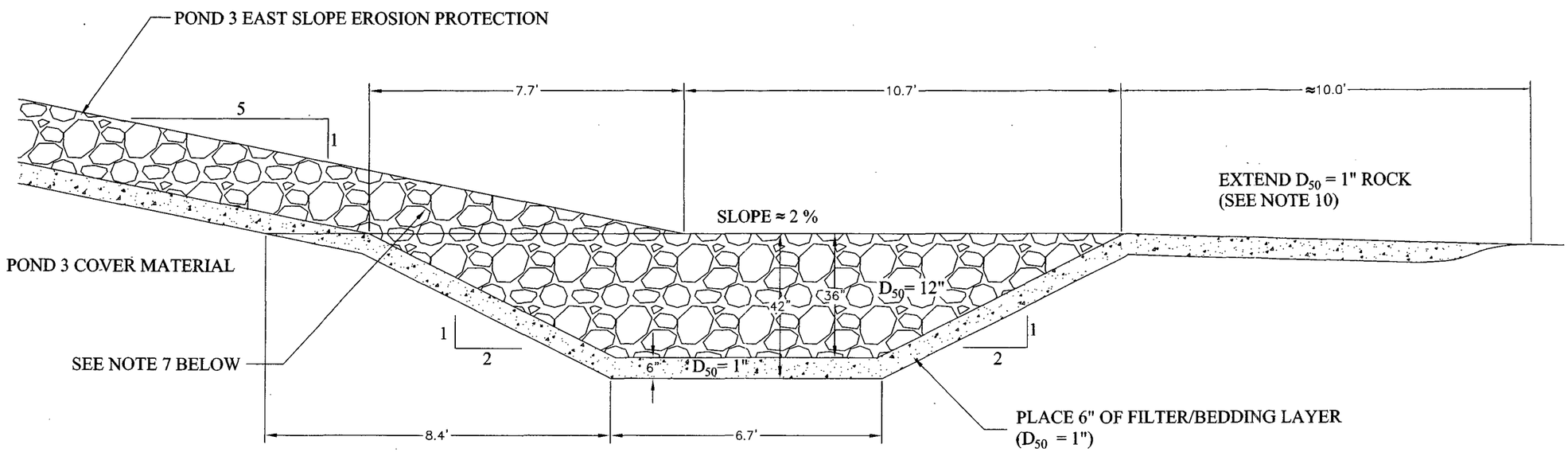
AMBROSIA LAKE MILL
RIO ALGOM MINING, LLC.
GRANTS, NEW MEXICO
TASK 3 EROSION PROTECTION
POND 3 TOE EROSION PROTECTION DETAILS

MAXIM TECHNOLOGIES INC

PROJECT No. 1690030-300
FILE NAME: 1690030S4-R1.DWG
DRAWING BY: RLH 2/25/02
MODIFIED BY: EJS 10/4/06
REVIEWED BY: COS

POND 3 - TOE EROSION PROTECTION APRON - EAST EMBANKMENT
FOR RUN-OFF FROM POND 3 EMBANKMENT SLOPE
Typical Section



NOTES:

1. Rock Riprap for erosion protection aprons shall be placed in conformance with Appendix F of *Design of Erosion Protection for Long-Term Stabilization,* U.S. Nuclear Regulatory Commission's NUREG - 1623 Draft Report.
2. Erosion protection aprons shall be sloped to the downstream edge with a minimum slope of two percent or at a slope that matches the slope of the natural ground, should it be steeper than two percent.
3. Erosion protection apron excavations shall be constructed with 2H:1V slopes to permit placement of the filter materials as shown above.
4. The erosion protection apron shall be constructed of a rock diameter D_{50} = 12" conforming to the following gradation:

Sieve Designation	Percent Passing
18"	100
14"	60 - 90
12"	25 - 50
10"	10 - 30
6"	0 - 10

5. The erosion protection filter/bedding layer shall be placed at a minimum thickness of 6" along the length of the apron. The filter/bedding layer shall extend up the 2H:1V sides and end below the Pond 3 erosion protection rock. Filter/bedding material shall be spread and compacted in one layer.
6. The erosion protection apron filter/bedding layer shall be constructed of a rock diameter D_{50} = 1", and conform to the following gradation:

Filter/bedding Gravel

Sieve Designation	Percent Passing
3"	100
2"	80 - 100
3/4"	20 - 70
3/8"	10 - 30
No.4	0 - 10

7. Existing erosion protection disturbed during construction of the erosion protection aprons shall be replaced in a manner that maintains existing slopes and riprap conditions as approved previously by the U.S. Nuclear Regulatory Commission. Care shall be taken in removing and stockpiling the riprap such that the material is not degraded or otherwise damaged. Rock degraded or otherwise out of conformance with the NUREG 1623 due to removal and replacement methods shall be replaced with similar approved materials. Care should be taken so that existing tailings are not disturbed during erosion protection apron construction.
8. The erosion protection apron shall be constructed continuously from the north embankment erosion protection channel/apron (north end of Pond 3) to the south embankment erosion protection channel/apron (south end of Pond 3).
9. The erosion protection apron shall be constructed level from the north end to the south end to prevent longitudinal flows within the apron. The erosion protection apron shall be constructed such that flow from the embankment will flow perpendicular to the apron and onto the Pond 3 surface where it will drain away from the erosion protection apron.
10. Extend the D_{50}=1" rock along the Northeast and East sides of the Pond 3 embankment at the toe erosion protection apron. The extended rock shall be blended to match the natural grade surface within an approximate 10-foot distance.

1 in = 3 ft

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GRANTS, NEW MEXICO
TASK 3 EROSION PROTECTION
POND 3 TOE EROSION PROTECTION APRON

MAXIM TECHNOLOGIES INC

PROJECT No. 1690030-300
FILE NAME: 1690030S5-R1.DWG
DRAWING BY: RLH 1/19/02
MODIFIED BY: EJS 10/4/06
REVIEWED BY: COS

SHEET 5 of 13