William Paul Goranson, P.E.
Manager, Radiation Safety
Regulatory Compliance and Licensing

Rio Algom

Rio Algom Mining LLC
6305 Waterford Boulevard
Suite 400, Oklahoma City
Oklahoma 73118
405 858 4807 tel
405 848 1208 fax

September 26, 2002

ATTN: Document Control Desk
Dan Gillen, Chief
Fuel Cycle Licensing Branch, NMSS
US Nuclear Regulatory Commission
Washington, DC 20555

Subject: Responses to Staff Questions on Erosion Protection Design for Pond #3 and Additional Arroyo del Puerto Investigations License No: SUA-1473 Docket No: 40-8905

Dear Mr. Gillen:

As a follow-up to our August 28, 2002 public meeting, Rio Algom Mining LLC is submitting the attached report on the investigation of PMF considerations for the Arroyo del Puerto. The basis of this report is concerns raised by NRC staff regarding Rio Algom's determination of the PMF estimate and the potential impacts to the rock armor on the outslope of Pond #3. The original design report used the NRC approved PMF estimate of 78,000 cfs, but NRC staff stated that there was a chance that the PMF had a possibility of being 200% or more higher than the original approved PMF.

This report provides a design estimate for a 200,000 cfs PMF event, and the conclusion that is presented is that this scenario is unreasonably conservative and would be prohibitively expensive to implement. Secondly, the report provides a validation of the approved PMF estimate along with a sensitivity analysis of the variables used to develop that estimate. The conclusion of that analysis is that even removing all reasonable conditions, the proposed erosion protection design for the approved PMF, of 78,000 cfs, remains protective enough to meet the requirements of 10CFR§40 Appendix A.

If you have any questions, please call me at (405) 858-4807

Sincerely,

William Paul Goranson, P.E.
Manager, Radiation Safety, Regulatory Compliance and Licensing

Enclosures

CC: Jill Caverly, NRC
Bruce Law, RAM
Terry Fletcher, RAM
Peter Luthiger, RAM

NMSS01

bhpbilliton

MAXIM TECHNOLOGIES INC® **MEMORANDUM**

DATE: September 6, 2002

TO: Paul Goranson – Rio Algom

FROM: Bill Bucher – Maxim

SUBJECT: Ambrosia Lake Mill – Arroyo del Puerto Investigations

---

This memorandum summarizes work performed by Maxim Technologies in response to discussions about uncertainties associated with the probable maximum flood (PMF) that could occur at the Ambrosia Lake Mill, New Mexico. Calculations by previous consultants as well as Maxim found that the PMF for the Arroyo del Puerto is about 78,000 cfs. (Maxim, 2002). Ted Johnson at the Nuclear Regulatory Commission has suggested that the PMF could be as large as 200,000 cfs based on floods in Texas and New Mexico described in a publication of the Bureau of Reclamation *Comparison of Estimated Maximum Flood Peaks with Historic Floods* (USBOR, 1986). In this memorandum, I first describe the consequences for rock-sizing on Ponds 1 and 3 if a 200,000 cfs flood is used instead of 78,000 cfs. I then investigate the sensitivity of the PMF calculation to various factors and calculate what I consider to be an upper bound on the plausible magnitude of the PMF in Arroyo del Puerto. A final portion of this memorandum discusses the probable lateral migration rate of the Arroyo del Puerto.

Rock Sizing for a 200,000 cfs Flood

A HEC-RAS (US Army Corps of Engineers, 1998) calculation was executed for the Arroyo del Puerto during the design of Ponds 1 and 3 as presented the *Design Report, Pond 3 Erosion Protection and Erosion Protection for the Area North of Pond 1, Ambrosia Lake Mill, New Mexico* prepared by Maxim in April of this year. This model, which makes numerous assumptions about the future shape of the Arroyo del Puerto floodplain, was then used to size the rock needed to protect Ponds 1 and 3 on the west side of the Arroyo del Puerto. That same model was modified to calculate water surface elevations for a 200,000 cfs flood that were used as the basis for the rock sizes reported in this memorandum. No attempt was made to discover what hydrologic conditions could give rise to a 200,000 cfs flood.

Rock sizing was carried out using the method of the US Army Corps of Engineers in *Hydraulic Design of Flood Control Channels* (ASCE, 1995). This method is applicable to natural channels subject to flow depths greater than five feet. It calculates a D_{30} rock size which can be converted to a D_{50} rock sizing using a riprap gradation table in the same publication. Typically, the D_{50} rock size is about 30 percent larger than the D_{30} rock size in this gradation table. The thickness of riprap is the greater of the D_{100} rock size or 1.5 times the D_{50} rock size. These calculations were performed for the Pond 3 toe apron, embankment and surface as well as the Pond 1 embankment and toe apron.

The extent of the 200,000 cfs flood is found on a map in Figure 1 of Attachment A. Also included in attachment A are graphs of the profile and cross-section of the flood and output data sheets for the cross-sections that impact Ponds 1 and 3 (cross-sections 1 through 5). Five figures present details of the right overbank and indicate the areas requiring the calculated rock sizes. Attachment A contains supporting calculations on rock sizes and rock volumes as well.

The elevation of the water surface for the 78,000 cfs flood remains about one foot below the surface of Pond 3, however, the 200,000 cfs flood will cover a portion of Pond 3 and rise about four feet on the embankment of Pond 1. The increased area of impact on the Pond 3 surface and Pond 1 embankment should only be about 500 feet long, but its exact size is very sensitive to the final floodplain geometry and these calculations should not be taken as a final determination of that surface. The Pond 1 embankment will need somewhat larger rock (D_{50} = 9.4 inches) than is presently contemplated. The top surface of Pond 3 will also require a larger rock (D_{50} = 7.5 inches) over this limited area. However, the big difference between the 78,000 cfs flood and the 200,000 cfs flood is the rock required on the Pond 3 embankment. This rock will increase from the current design of 18-inches thickness of D_{50} = 12-inch rock to 32-inches thickness of D_{50} = 21-inch rock. We probably would not need to cover the entire south portion of the embankment with this larger rock because the water will not extend up the entire Pond 3 embankment in this area, but the entire north portion of the embankment will require this size rock. This larger rock size (D_{50} = 21 inch) will be needed in the Pond 3 embankment toe apron as well. The designed Pond 1 embankment toe has a larger rock size (D_{50}= 9.2 inches) than the size (D_{50}= 7.5 inches) required for the 200,000 cfs flood; therefore, no change is needed in the Pond 1 embankment toe design.

Sensitivity Analysis of the PMF Calculation

Calculation of a PMF from a probable maximum precipitation (PMP) event requires information on the type of storm, the geometry of the basin, the infiltration properties of the basin as well as assumptions about the behavior of the flood peak as it travels through the basin. The number and uncertainty of variables in the calculation can lead to greatly varying results in the magnitude of the PMF. I have performed a sensitivity analysis of some of these variables with the object of calculating what I call an upper bound to the PMF for the Arroyo del Puerto. The variables most likely to affect the magnitude of the flood peak are the infiltration rate, the lag time, and the precipitation distribution.

The sensitivity analysis was conducted using the HEC-1 model (US Army Corps of Engineers, 1990) which was originally used to calculate a 75,200 cfs PMF in Maxim's *Design Report, Pond 3 Erosion Protection and Erosion Protection for the Area North of Pond 1, Ambrosia Lake Mill, New Mexico* (Maxim, 2002). This PMF value was increased to 78,000 cfs for consistency with a value used by a previous consultant to the project. Each of the three variables to be tested was varied independently of the others to measure their individual effects on the PMF. Then those variables which significantly affect the PMF magnitude were given maximum probable values and an upper bound to the PMF magnitude was calculated.

Table 1 summarizes the sensitivity analysis. The curve number, as defined by the Soil Conservation Service (SCS, now the Natural Resources Conservation Service), is a measure of the ability of the basin to infiltrate rainfall. Two cases were examined for this parameter: 1) changing the curve number according to SCS procedures to account for an antecedent moisture condition that reflects previous wet conditions, and 2) assuming that the entire basin is impermeable. The first case corresponds to a curve number of 88 and the second case corresponds to a curve number of 100. Both cases significantly increase the PMF magnitude with the impermeable case resulting in a 108,600 cfs peak.

The lag time is defined as the time from the beginning of runoff at the measuring station to the peak runoff (Chow, 1964). A shorter lag time will increase the peak flow, other factors being equal. It is often related to the time of concentration, which is defined as the time it takes water to travel from the most distant point in the watershed to the measuring station. The time of concentration is typically calculated from the Kirpich equation, which is based on stream length and channel slope (Chow, 1964). SCS has determined that the lag time is typically 0.60 times the time of concentration (Waltemeyer, 2001), and this was the method used to determine the lag time in Maxim's original calculation. For this sensitivity analysis, I have used a lag time calculation method developed specifically for small basins in New Mexico by the US Geological Survey (Waltemeyer, 2001). This method calculates the lag time from basin length and basin shape based on regression equations developed from measurement of numerous flood hydrographs in New Mexico. This method reduces the lag time from the original 1.83 hours to 1.27 hours and increases the flood peak to 98,600 cfs.

TABLE 1
SUMMARY OF PMF SENSITIVITY ANALYSIS

Run No.	Description	PMF (cfs)
1	Base case for comparison	75,200
	Curve Number Sensitivity:	
2	Change Curve Number to 88 (from 73.4) corresponding to saturated conditions before the PMP begins.	96,000
3	Change Curve Number to 100 corresponding to zero infiltration.	108,600
	Lag Time Sensitivity:	
4	Change lag time to 1.27 hours (from 1.83 hours) based on New Mexico Method developed by USGS	98,600
	Rainfall Sequence Sensitivity:	
5	Change sequence from ACE sequence to HMR sequence.	69,000
6	Change sequence to most intense rainfall in fifth hour of storm.	78,000
	Upper Bound Calculation:	
7	Combine highest reasonable curve number (88) with shortest documented lag time (1.27 hr.)	126,000

Rainfall distributions can affect the flood peaks with later peak precipitation periods generally resulting in higher peak flows. The rainfall distribution used for the six-hour local storm was the US Army Corps of Engineers' distribution found in the *Hydrometeorological Report No. 55* (Hansen *et al*, 1988). This distribution places the peak precipitation period in the fourth hour of a six-hour storm. For the sensitivity analysis, I used the Hydrometeorological Report (HMR) distribution found in the same publication, which places the peak precipitation in the third hour, and a hypothetical distribution, which places the peak rainfall in the fifth hour. As expected, the HMR distribution decreased the peak flow and the hypothetical distribution increased the peak flow, but neither made significantly large changes to the PMF with only a 13 percent spread from the lowest to highest value.

For infiltration in the Arroyo del Puerto, I believe the curve number of 88 is a maximum reasonable number because zero infiltration (curve number = 100) will not occur in a natural drainage with soils. I accepted the New Mexico method for lag time calculation as more site specific than the SCS method and used a lag time of 1.27 hours. It should be noted that the New Mexico method has not been verified on basins as large as the Arroyo del Puerto. I found that the PMF was not significantly increased by variations to the rainfall sequence. Therefore, my upper bound calculation uses the original rainfall sequence of the Army Corps of Engineers found in HMR-55. Based on these combined worst case conditions of curve number, lag time, and rainfall sequence, the value of 126,000 cfs represents a reasonable upper bound to a PMF calculation although the most probable value for the PMF is probably significantly less. A printout of the HEC-1 output for the upper bound calculation is found in Attachment B as well as a calculation of the lag time based on the New Mexico method.

Maxim proposes that the originally calculated value of 78,000 cfs be accepted as a reasonable value for the PMF in the Arroyo del Puerto and be used in design of Ponds 1 and 3 where applicable. This value is greater than half the value of the upper bound value of 126,000 cfs, assuring that a high degree of protection will be achieved even if the actual number for the PMF is in error.

Lateral Migration Rate

The Arroyo del Puerto originally consisted of a relatively narrow channel in a broad, alluvial floodplain. Such streams are subject to lateral migration, especially during flood events. The normal method of lateral migration is erosion of the outer banks on bends. It is possible that, over a sufficiently long period of time, the Arroyo del Puerto could migrate sufficiently to scour beneath the Pond 3 and Pond 1 embankments causing erosion of tailings materials. For that reason I have investigated the probable lateral migration rate of the reconstructed Arroyo del Puerto.

The Arroyo del Puerto flows through cohesive alluvial materials consisting typically of sandy clays. A literature search found almost no information on the migration of streams in cohesive, fine-grained materials. I therefore turned to information on the better studied coarse grained river systems and employed some conservative assumptions to estimate a lateral migration rate for the Arroyo del Puerto. Nanson and Hickin (1986) performed a statistical analysis of channel migration rates on streams in western Canada and developed regression equations that predict the migration rate base on flow and stream slope as well as other factors such as particle size and bank height. The particular equation that predicts linear (as opposed to volumetric) migration rates uses only the five-year recurrence interval flow and the stream slope as independent variables, and this equation was applied in this analysis.

To calculate the five-year recurrence flow, reference was made to USGS Water-Resources Investigations Report 96-4112, *Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico* (Waltemeyer 1996). This calculation is performed with a regional regression equation which relates peak discharge for various recurrence intervals including five years to basin area, basin elevation and the intensity of rainfall in the 10-year recurrence interval, 24-hour storm. Values for basin area and elevation were taken from topographic maps and the intensity of the 10-year, 24-hour storm was found in the *Precipitation-Frequency Atlas for the Western United States* (Miller *et al* 1973). The value for the five-year flow is 750 cfs and the typical channel slope in the vicinity of Ponds 1 and 3 is 0.005. These values result in a migration rate of about three feet per year. If the channel is reconstructed at least 300 feet from the toes of Ponds 1 and 3, it would take at least 100 years for the channel to migrate to the toes, assuming it is reconstructed as a natural channel. Calculations are summarized in Attachment C.

There are conservative assumptions built into this calculation. The migration rate calculated by this method is the migration rate at the outside of typical river bends. The remainder of the channel should migrate at a lesser rate. In addition, the equations are based on coarse sediments including sands, which could be much more mobile than cohesive, fine-grained sediments. To check the assumption that the migration rate in cohesive materials could be less than calculated, a paper was found giving the migration rate of a small Maryland stream in cohesive bank materials (Wolman 1959). This stream migrated a maximum of seven feet in five years, an average of 1.4 feet per year and a rate considerably less than the rate calculated above for the Arroyo del Puerto. Given the conservative assumptions involved in the above calculation, it is likely that the channel of the Arroyo del Puerto will take considerably longer than 100-years to migrate to the toes of the Pond 1 or Pond 3 embankments.

References

American Society of Civil Engineers (ASCE), 1995. *Hydraulic Design of Flood Control Channels.* U. S. Army Corps of Engineers engineer manual EM 1110-2-1601.

Chow, V.T., 1964. *Handbook of Applied Hydrology.* Chapter 21. McGraw-Hill, New York.

Hansen, E. M., D. D. Fenn, L. C. Schreiner, R. W. Stodt, and J. F. Miller, 1988. *Hydrometeorological Report No. 55A, Probable Maximum Precipitation Estimates – United States between the Continental Divide and the 103rd Meridian.* U. S. Department of Commerce, Silver Spring, Maryland.

Maxim Technologies, Inc., 2002. *Draft Design Report, Pond 3 Erosion Protection and Erosion Protection for the Area North of Pond 1, Ambrosia Lake Mill, New Mexico.*

Miller, J. F., R. H. Frederick, and R. J. Tracey, 1973. *Precipitation-Frequency Atlas for the Western United States.* National Oceanic and Atmospheric Administration, Silver Spring Maryland.

Nanson, G. C. and E. J. Hickin, 1986. *A Statistical Analysis of Bank Erosion and Channel Migration in Western Canada.* Geological Society of America Bulletin, v. 97, p. 497-504, April 1986.

United States Army Corps of Engineers, 1990. *HEC-1 Flood Hydrograph Package.* Version 4.0. Hydrologic Engineering Center, Davis, California.

United States Army Corps of Engineers, 1998. *HEC-RAS River Analysis System.* Version 2.2. Hydrologic Engineering Center, Davis, California.

United States Bureau of Reclamation, 1986. *Comparison of Estimated Maximum Flood Peaks with Historic Floods.* January, 1986.

Waltemeyer, S. D., 1996. *Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico.* USGS Water-Resources Investigations Report 96-4112.

Waltemeyer, S. D., 2001. *Relations for Estimating Unit-Hydrograph Parameters in New Mexico.* U.S. Geological Survey Water-Resources Investigation Report 01-4154.

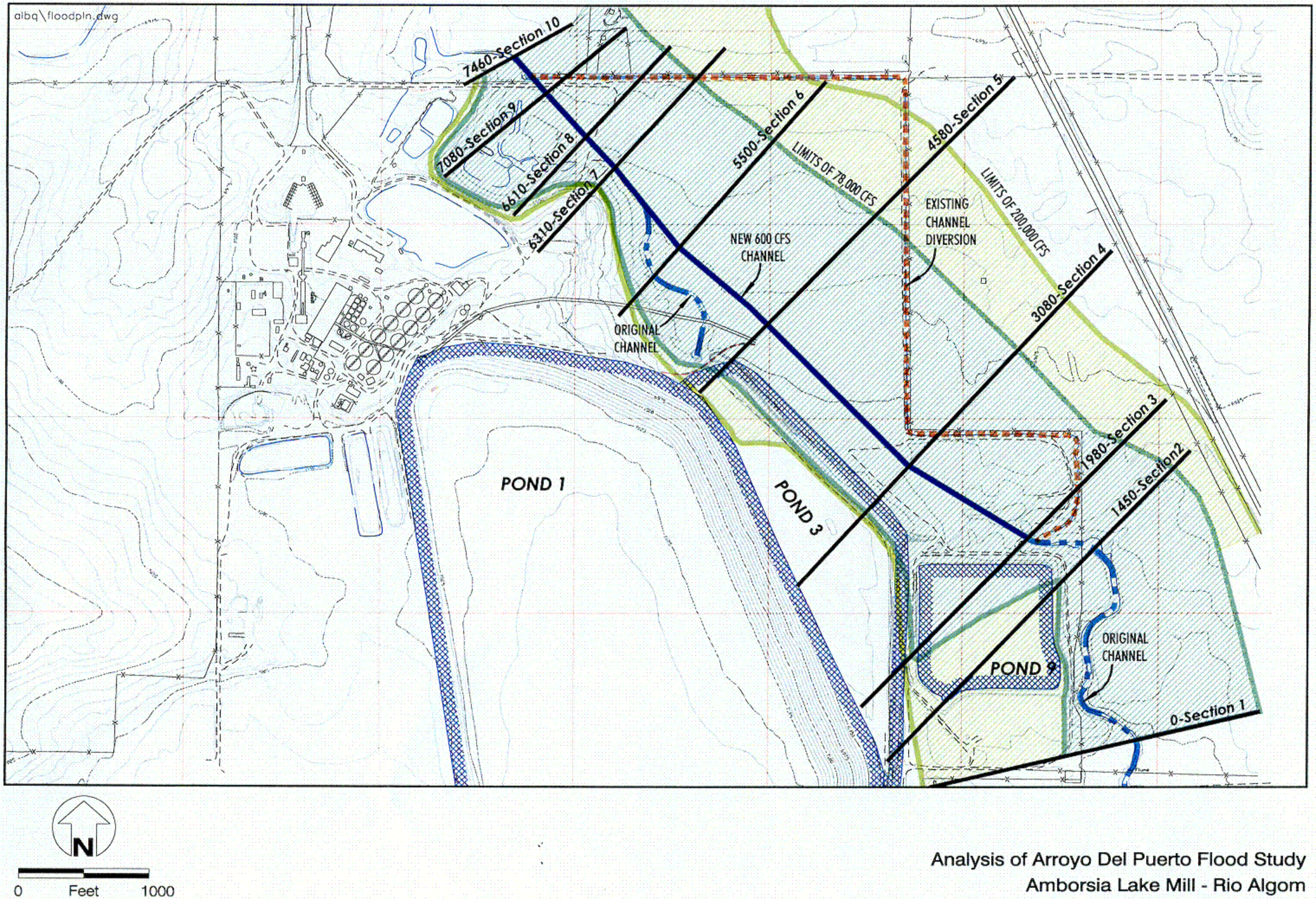
Wolman, M. G., 1959. *Factors Influencing Erosion of a Cohesive River Bank.* American Journal of Science, v. 257, pp. 204-216, March, 1959.

ATTACHMENT A

HEC-RAS OUTPUT AND

ROCK SIZING FOR 200,000 CFS FLOOD

SUPPORTING DATA



MAXIM TECHNOLOGIES INC® 1690030-300

Analysis of Arroyo Del Puerto Flood Study
Amborsia Lake Mill - Rio Algom
Grants, New Mexico
FIGURE 1

C01

Plan: ADP PMF   Arroyo del Puert   Ambrosla Mill RS: 4580   Profile: PF 1

E.G. Elev (ft)	6940.95	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.54	Wt. n-Val.	0.035	0.049	0.035
W.S. Elev (ft)	6939 41	Reach Len. (ft)	1500.00	1500.00	1500.00
Crit W.S. (ft)		Flow Area (sq ft)	14703.91	754.45	4800.84
E.G. Slope (ft/ft)	0.004028	Area (sq ft)	14703.91	754.45	4800.84
Q Total (cfs)	200000.00	Flow (cfs)	141681.40	9902.16	48416.44
Top Width (ft)	2876.71	Top Width (ft)	2174.05	40.00	662.66
Vel Total (ft/s)	9.87	Avg. Vel. (ft/s)	9.64	13.12	10 08
Max Chl Dpth (ft)	20.11	Hydr. Depth (ft)	6.76	18.86	7.24
Conv. Total (cfs)	3151453.0	Conv. (cfs)	2232511.0	156031.0	762910.6
Length Wtd. (ft)	1500.00	Wetted Per. (ft)	2174.11	42.36	662.94
Min Ch El (ft)	6919.30	Shear (lb/sq ft)	1.70	4.48	1.82
Alpha	1.02	Stream Power (lb/ft s)	16.39	58.78	18.36
Frctn Loss (ft)	8.14	Cum Volume (acre-ft)	1115.25	122.09	500.59
C & E Loss (ft)	0 14	Cum SA (acres)	171.82	6 93	78.56

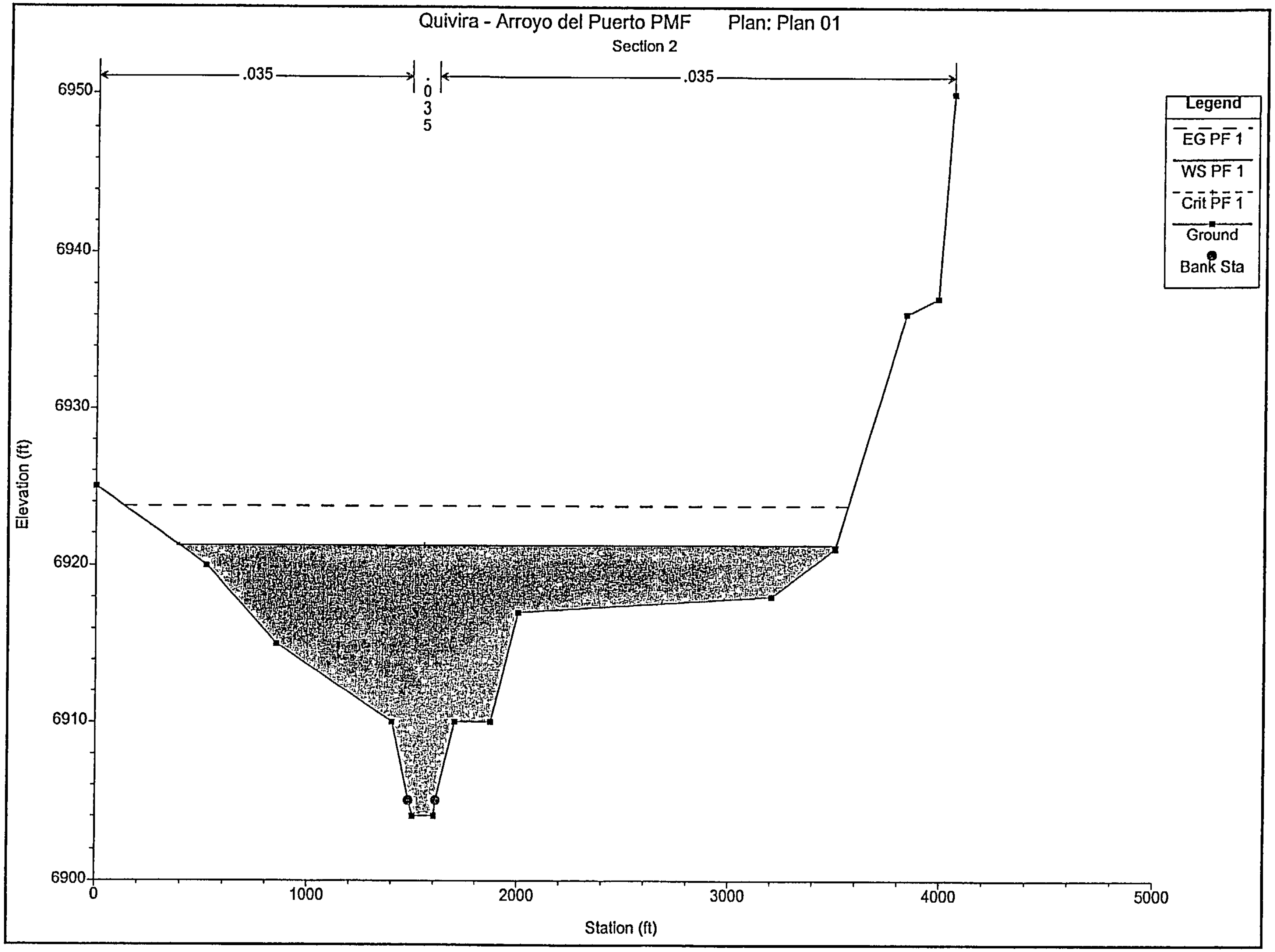
Plan: ADP PMF Arroyo del Puert Ambrosia Mill RS: 1980 Profile: PF 1

E.G. Elev (ft)	6925.91	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.80	Wt. n-Val.	0 035	0 049	0.035
W.S. Elev (ft)	6924.11	Reach Len. (ft)	400.00	530.00	400.00
Crit W.S. (ft)	6922.68	Flow Area (sq ft)	8816 69	714.24	9287.18
E.G. Slope (ft/ft)	0.004955	Area (sq ft)	8816 69	714.24	9287.18
Q Total (cfs)	200000.00	Flow (cfs)	86022.39	10025 03	103952.60
Top Width (ft)	2815.21	Top Width (ft)	1494.50	40.00	1280.71
Vel Total (ft/s)	10 63	Avg. Vel. (ft/s)	9.76	14.04	11.19
Max Chl Dpth (ft)	19.11	Hydr. Depth (ft)	5.90	17.86	7.25
Conv. Total (cfs)	2841233.0	Conv. (cfs)	1222048.0	142417.2	1476767.0
Length Wtd. (ft)	417.74	Wetted Per. (ft)	1494.59	42.36	1281.24
Min Ch El (ft)	6905.00	Shear (lb/sq ft)	1.82	5.22	2.24
Alpha	1.03	Stream Power (lb/ft s)	17.80	73.21	25.10
Frctn Loss (ft)	2.09	Cum Volume (acre-ft)	440.90	79.26	205.28
C & E Loss (ft)	0.07	Cum SA (acres)	64 72	4 55	40.13

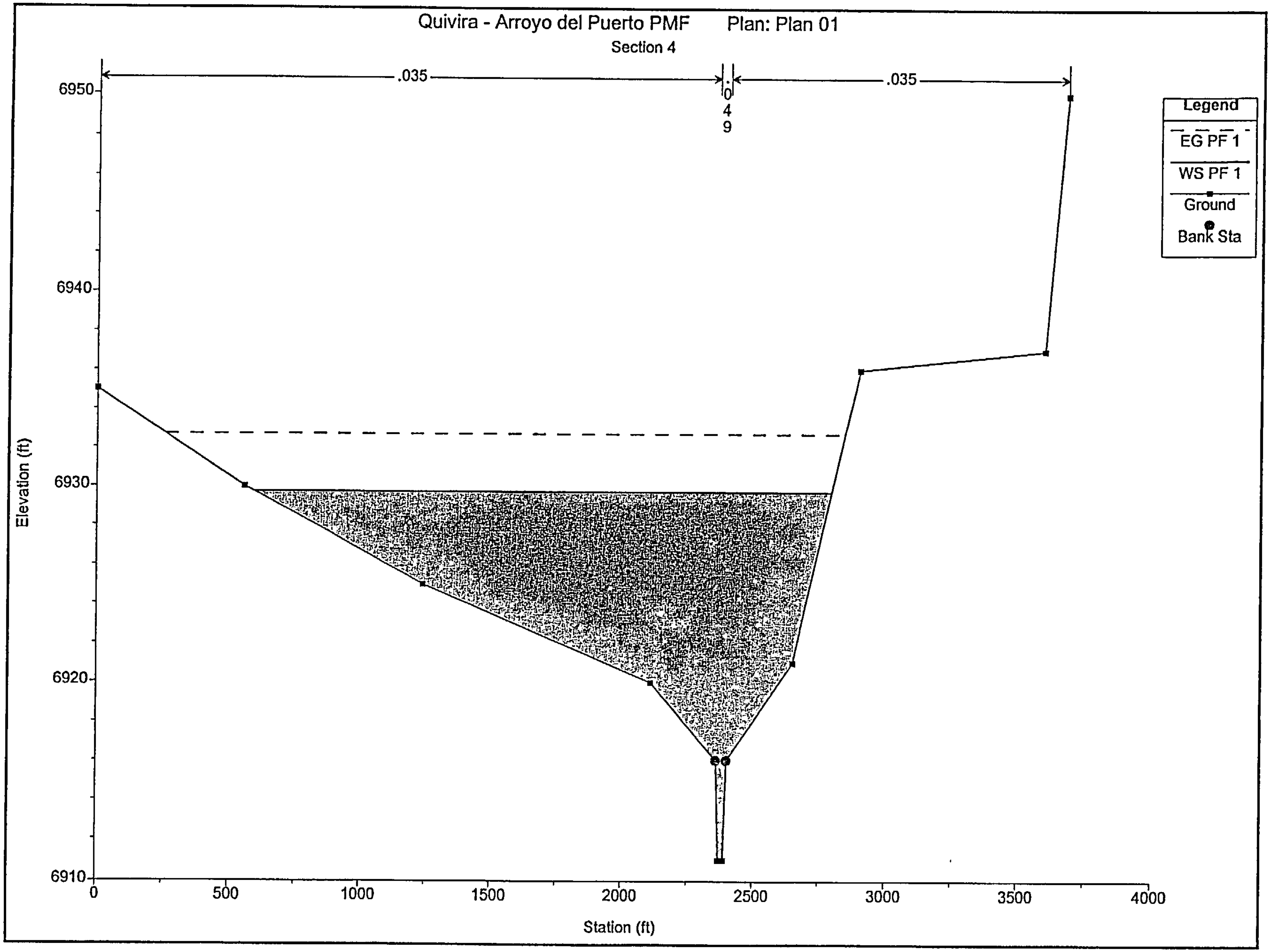
200,000 cfs

Plan: ADP PMF Arroyo del Puert Ambrosia Mill RS: 0 Profile: PF 1

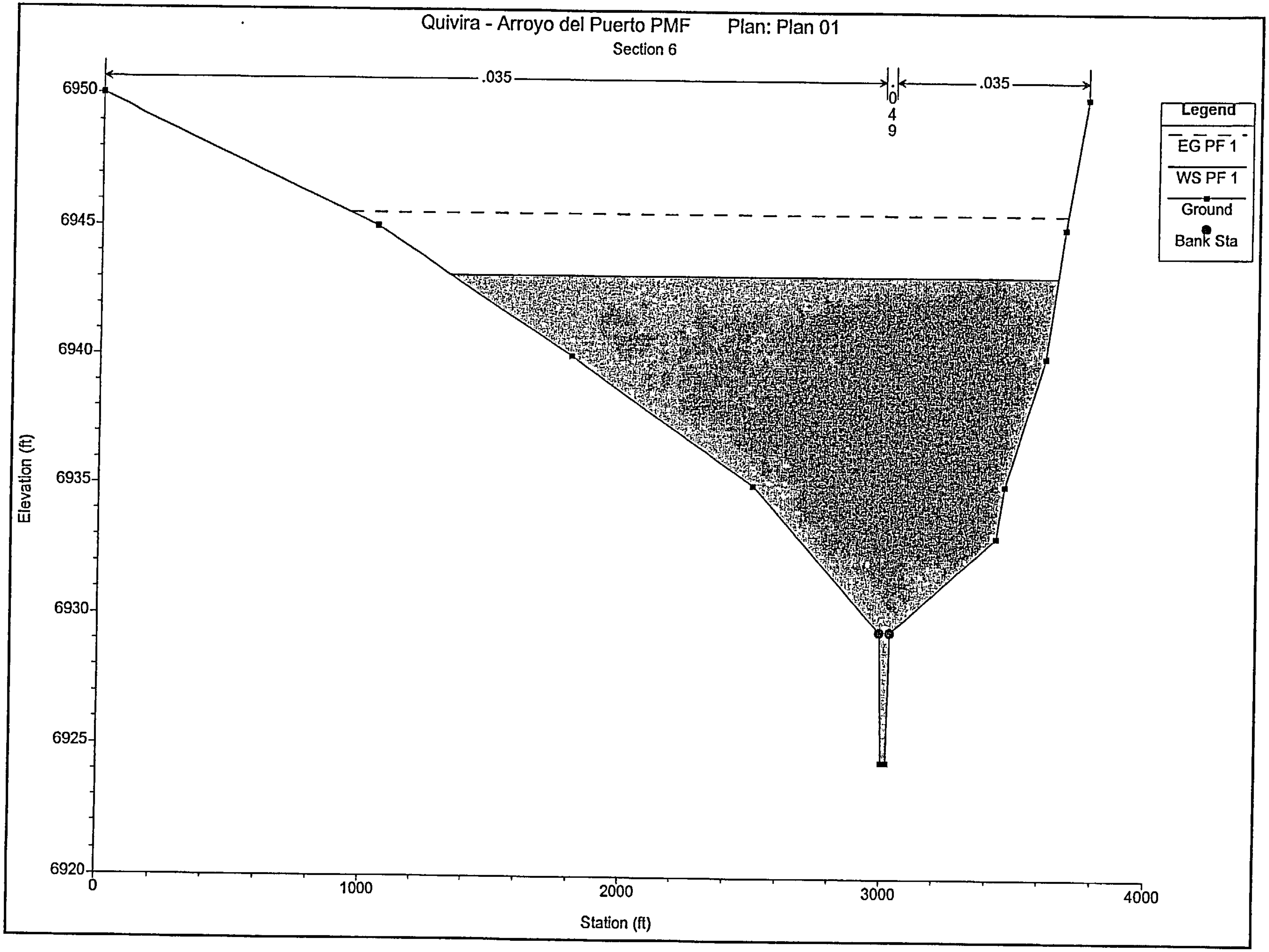
E.G. Elev (ft)	6914.72	Element	Left OB	Channel	Right OB
Vel Head (ft)	2.55	Wt. n-Val.	0.035	0.035	0.035
W.S. Elev (ft)	6912.17	Reach Len. (ft)			
Crit W.S. (ft)	6911.91	Flow Area (sq ft)	10573.23	746.68	5880.47
E.G. Slope (ft/ft)	0.006004	Area (sq ft)	10573.23	746.68	5880.47
Q Total (cfs)	200000.00	Flow (cfs)	129529 90	17171.04	53299.09
Top Width (ft)	2795.73	Top Width (ft)	1471.03	40 00	1284.69
Vel Total (ft/s)	11.63	Avg. Vel. (ft/s)	12.25	23 00	9 06
Max Chl Dpth (ft)	19 17	Hydr. Depth (ft)	7.19	18.67	4.58
Conv. Total (cfs)	2581220.0	Conv. (cfs)	1671725.0	221611.1	687883.3
Length Wtd. (ft)		Wetted Per. (ft)	1471.17	40.40	1285.73
Min Ch El (ft)	6893.00	Shear (lb/sq ft)	2 69	6 93	1.71
Alpha	1.22	Stream Power (lb/ft s)	33.00	159.32	15 54
Frctn Loss (ft)		Cum Volume (acre-ft)			
C & E Loss (ft)		Cum SA (acres)			



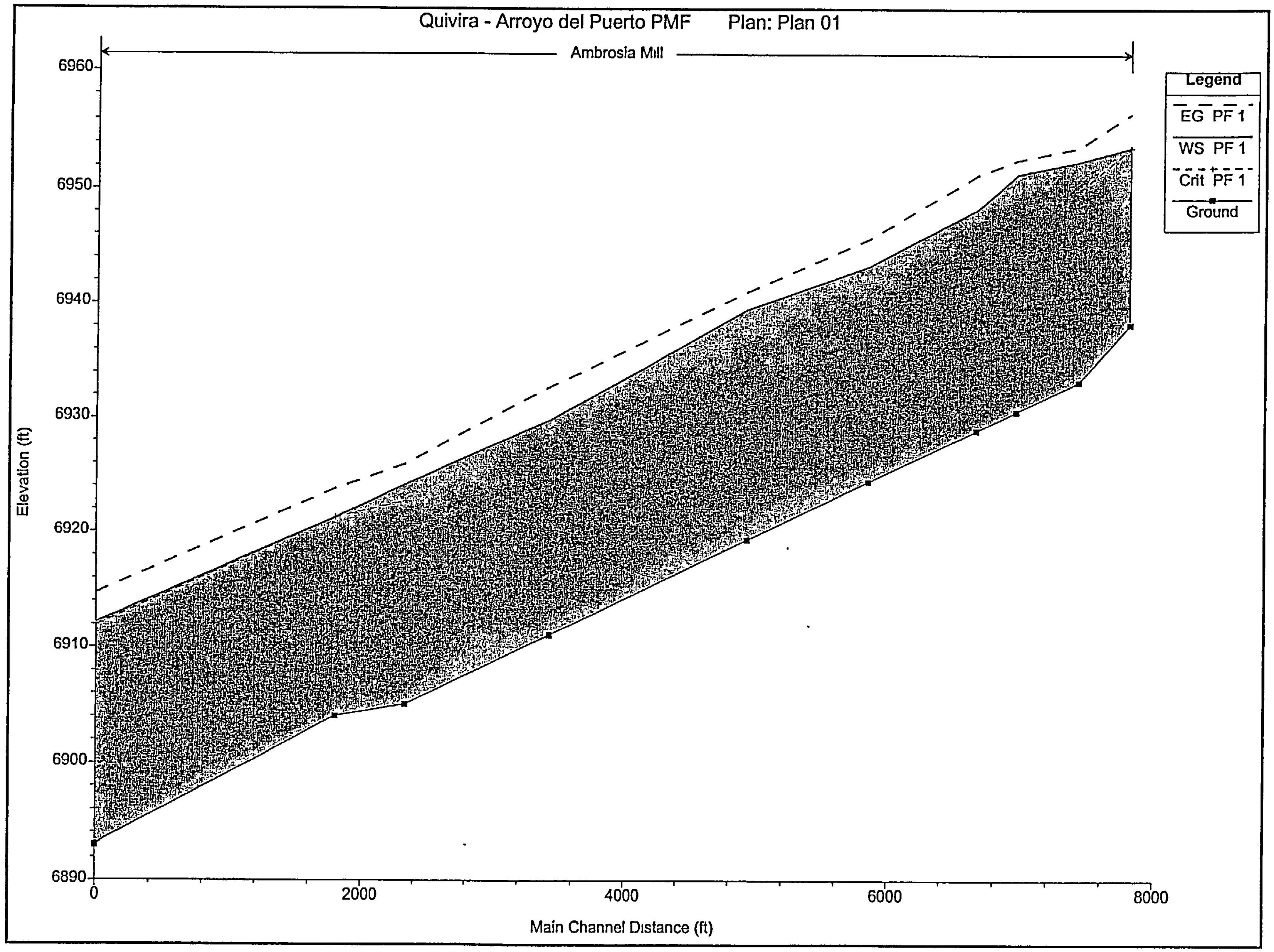
Quivira - Arroyo del Puerto PMF
Plan: Plan 01
Section 2
.035
.035
.035
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Bank Sta
Elevation (ft)
6950
6940
6930
6920
6910
6900
0
1000
2000
3000
4000
5000
Station (ft)



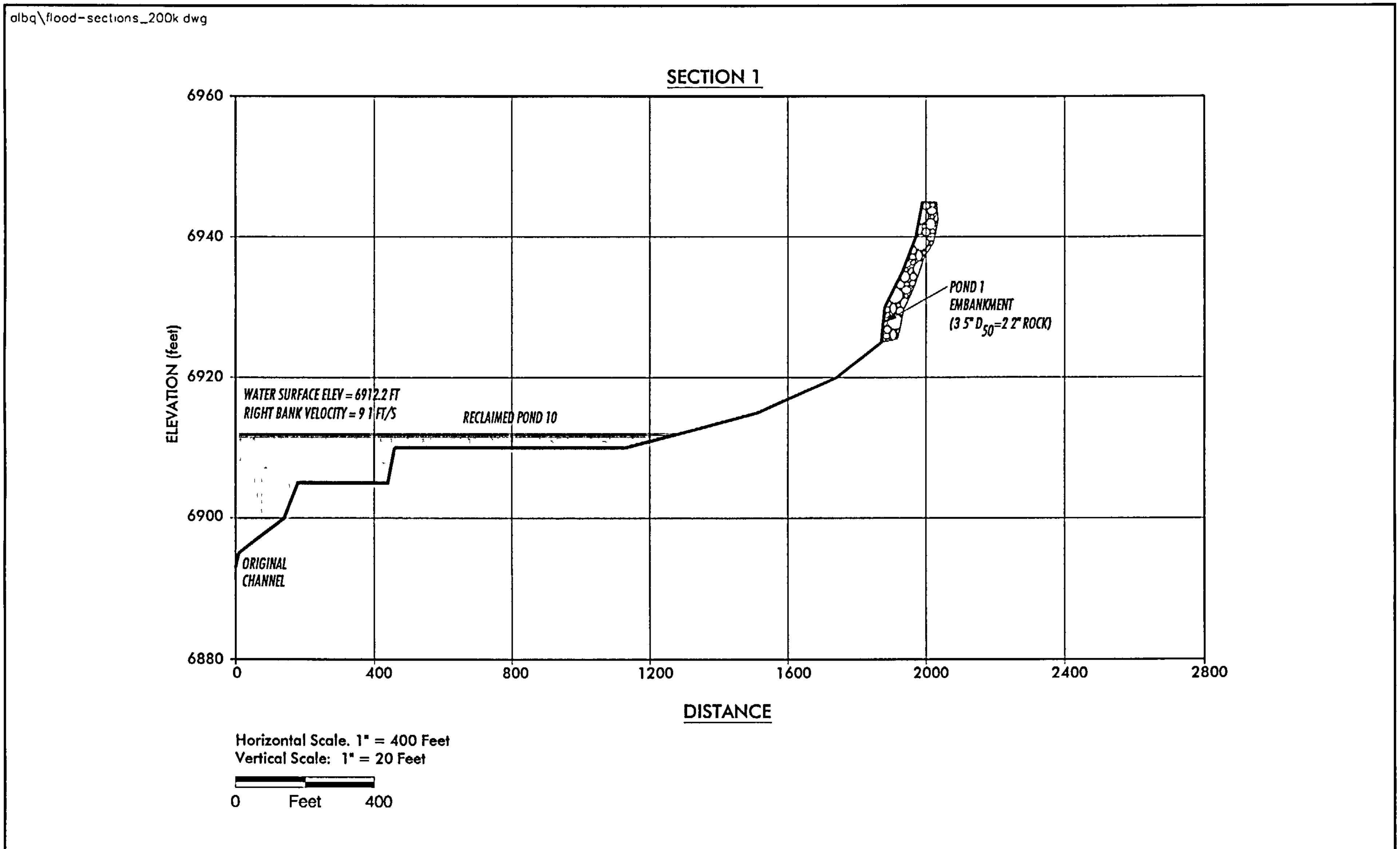
Quivira - Arroyo del Puerto PMF Plan: Plan 01
Section 4
.035
.
0
4
9
.035
Legend
EG PF 1
WS PF 1
Ground
Bank Sta
Elevation (ft)
6950
6940
6930
6920
6910
Station (ft)
0
500
1000
1500
2000
2500
3000
3500
4000



Quivira - Arroyo del Puerto PMF
Plan: Plan 01
Section 6
.035
0
4
9
.035
Legend
EG PF 1
WS PF 1
Ground
Bank Sta
Elevation (ft)
6950
6945
6940
6935
6930
6925
6920
0
1000
2000
3000
4000
Station (ft)



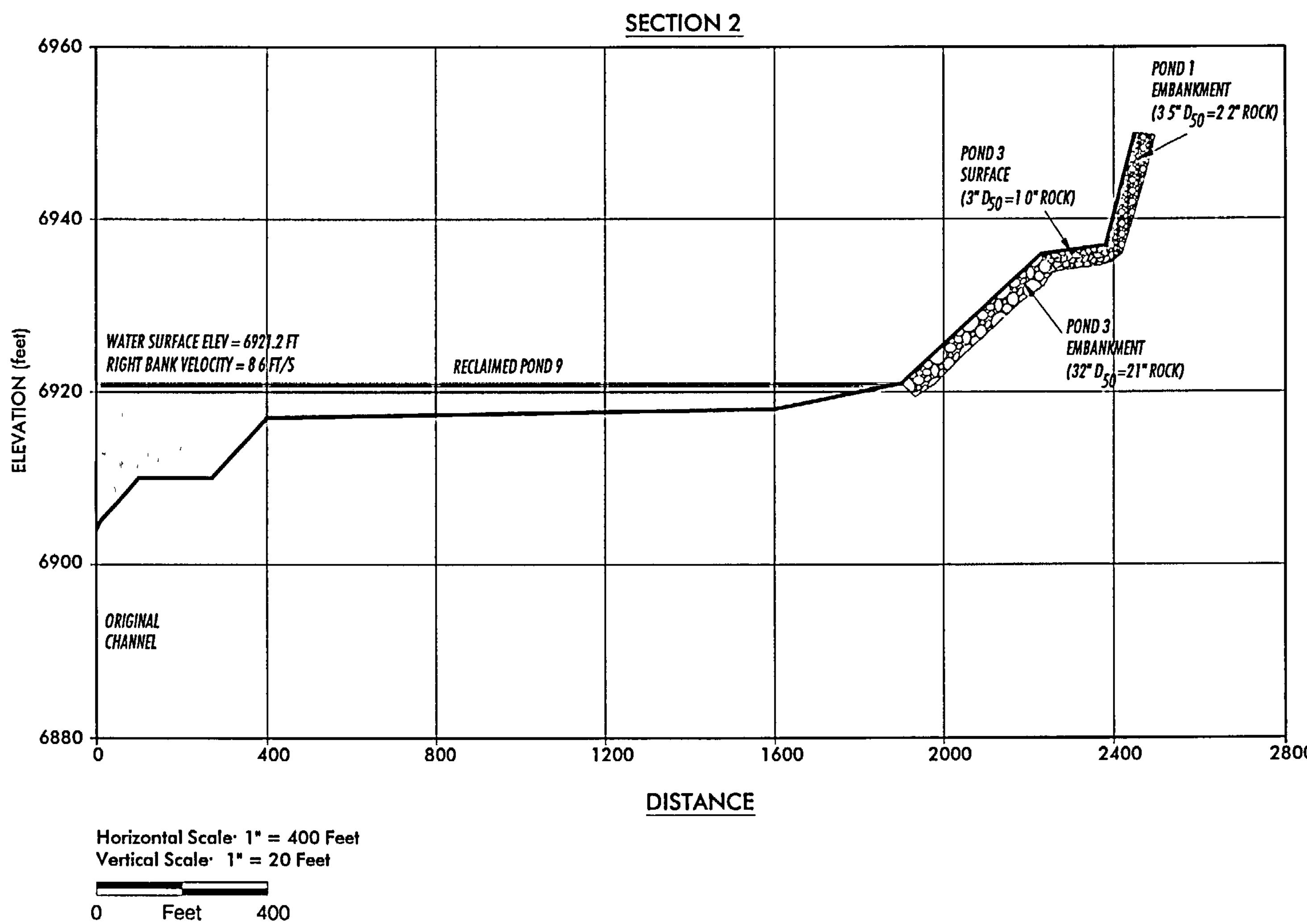
Quivira - Arroyo del Puerto PMF Plan: Plan 01
Ambrosia Mill
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Elevation (ft)
6960
6950
6940
6930
6920
6910
6900
6890
0
2000
4000
6000
8000
Main Channel Distance (ft)



MAXIM TECHNOLOGIES INC 1690030.600

Section 1 - 200,000 cfs Flood
Ambrosia Lake Mill
Grants, New Mexico
FIGURE 1

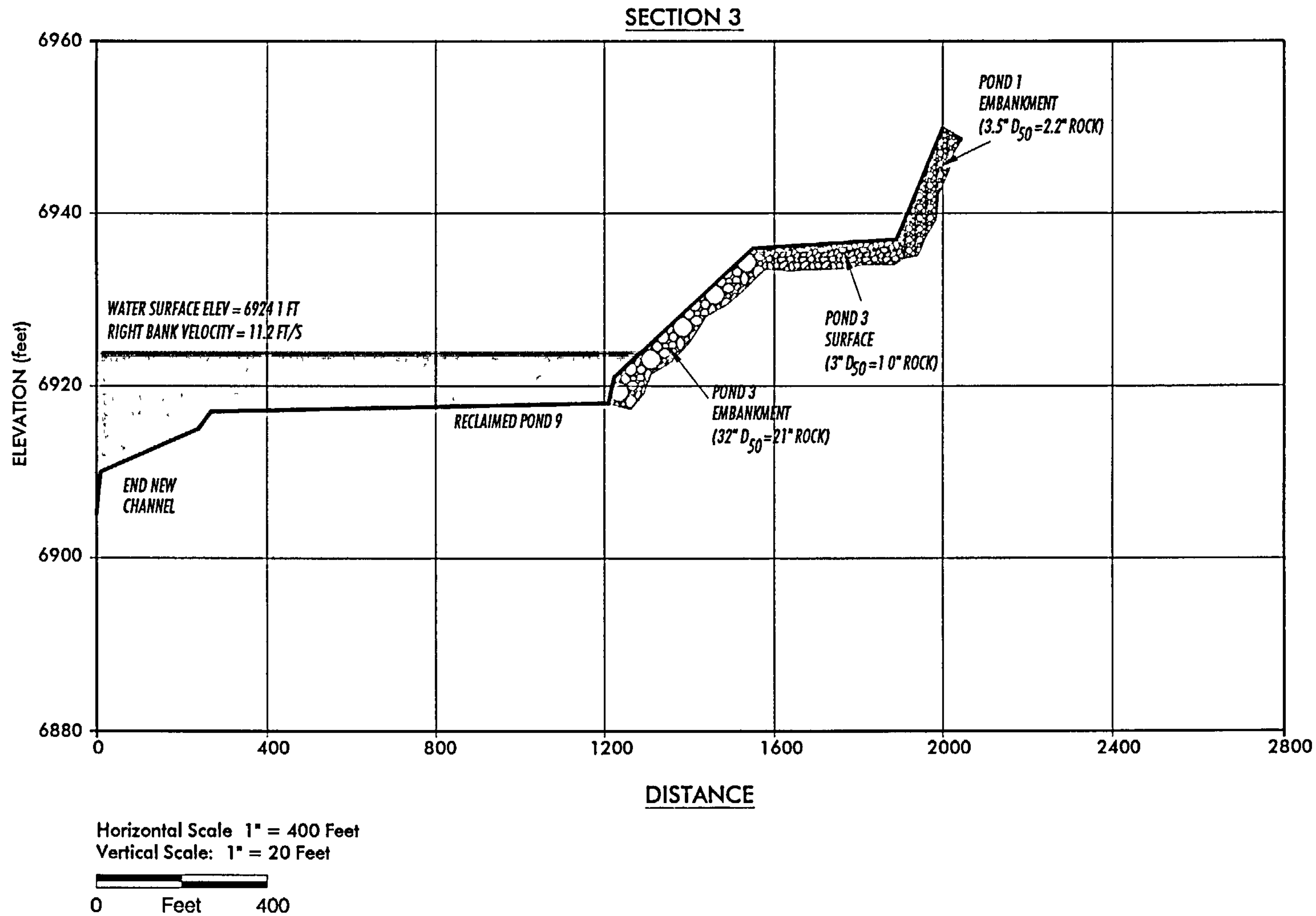
albq\flood-sections_200k dwg



Section 2 - 200,000 cfs Flood
Ambrosia Lake Mill
Grants, New Mexico
FIGURE 2

MAXIM TECHNOLOGIES INC® 1690030 600

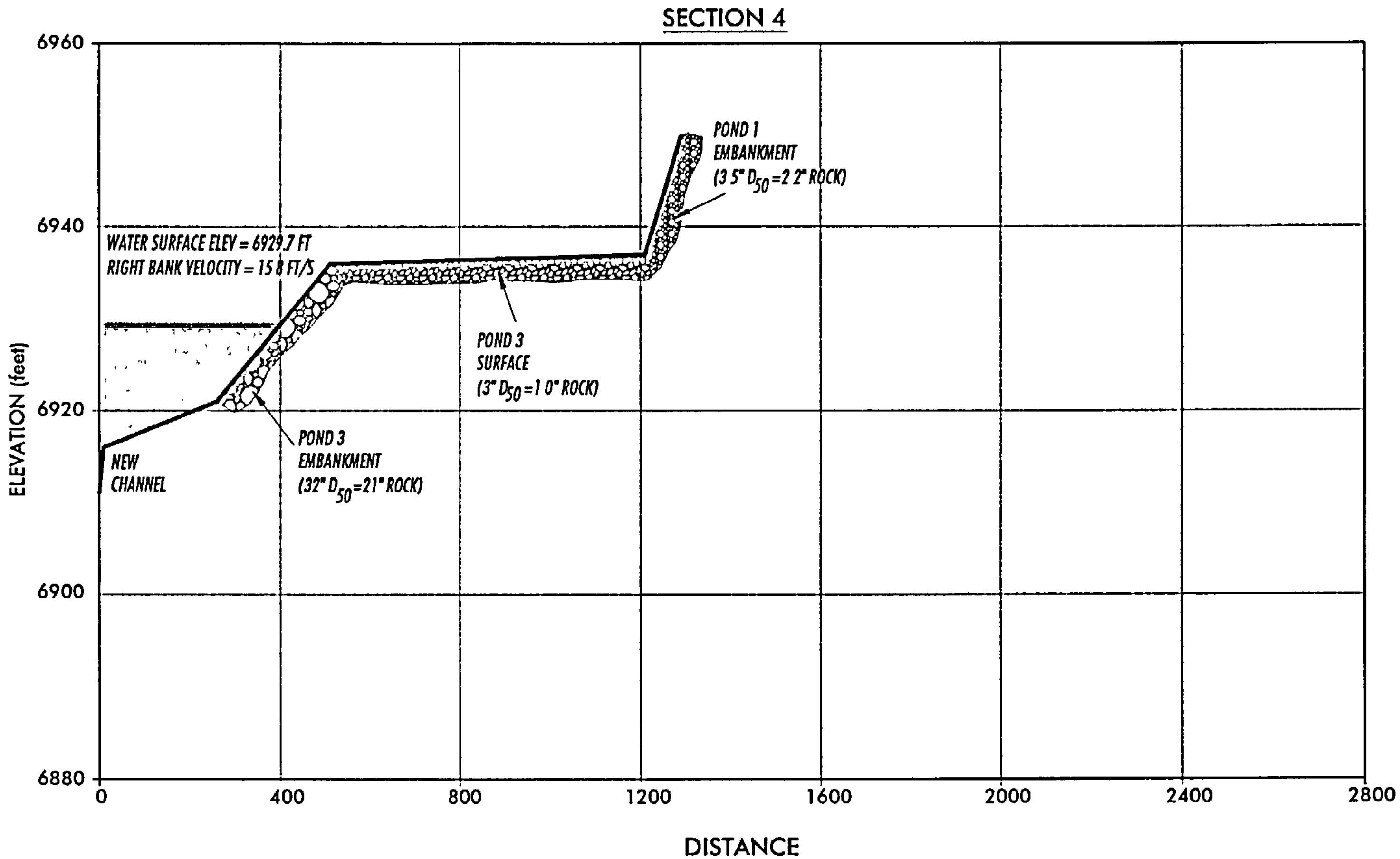
albq\flood-sections_200k dwg



Section 3 - 200,000 cfs Flood
Ambrosia Lake Mill
Grants, New Mexico
FIGURE 3

MAXIM TECHNOLOGIES INC® 1690030.600

albq\flood-sections_200k dwg



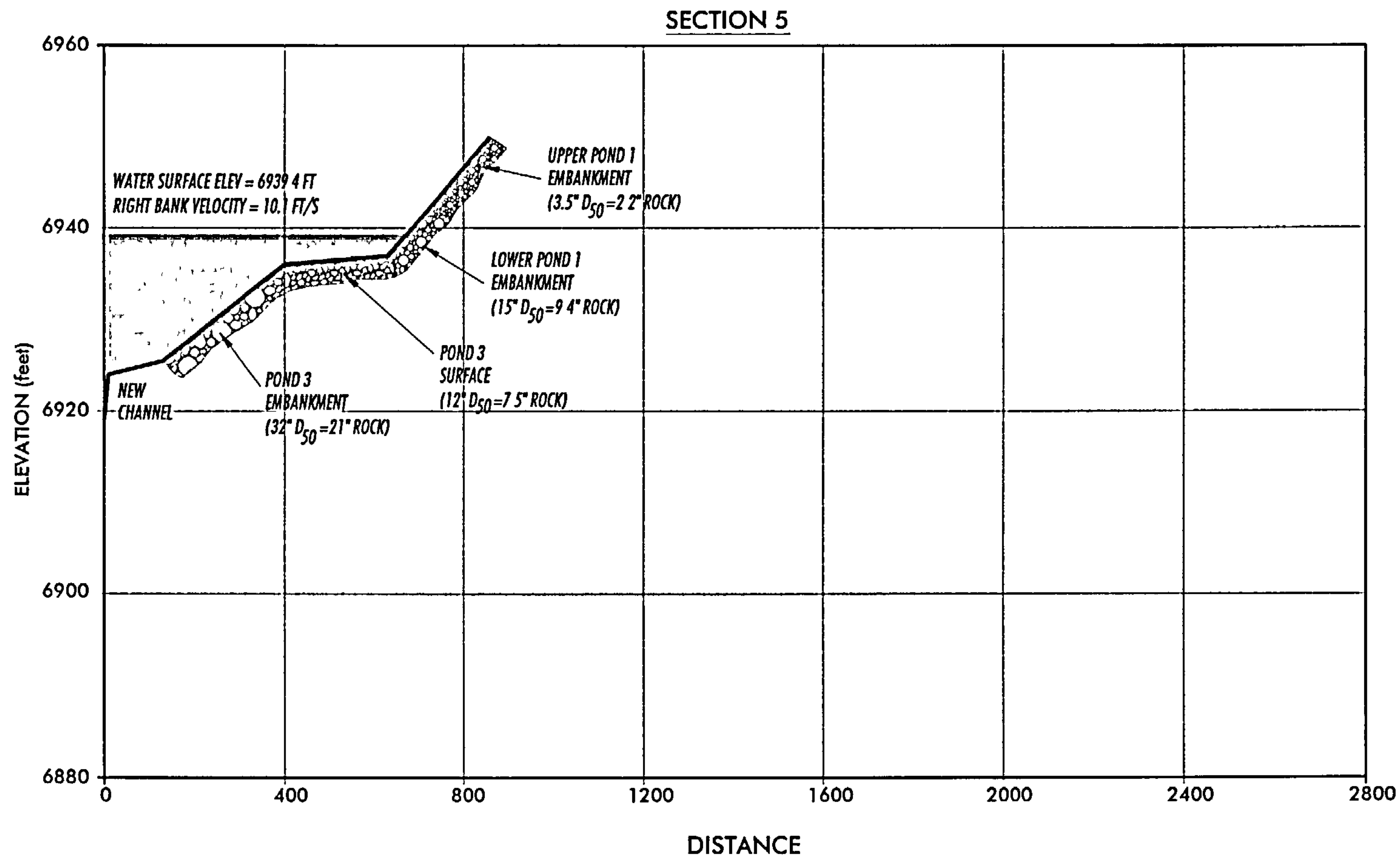
Horizontal Scale 1" = 400 Feet
Vertical Scale 1" = 20 Feet

0 Feet 400

Section 4 - 200,000 cfs Flood
Ambrosia Lake Mill
Grants, New Mexico
FIGURE 4

MAXIM TECHNOLOGIES INC 1690030 600

alba\flood-sections_200k dwg



Horizontal Scale. 1" = 400 Feet
Vertical Scale: 1" = 20 Feet

0 Feet 400

Section 5 - 200,000 cfs Flood
Ambrosia Lake Mill
Grants, New Mexico
FIGURE 5

MAXIM TECHNOLOGIES INC. 1690030 600

MAXIM TECHNOLOGIES INC® BY Bill Bucher DATE 8/15/02

JOB TITLE Rio Algom – Ambrosia Lake JOB NUMBER

SUBJECT 200,000 cfs rock sizing SHEET 1 of 2

Objective: Determine rock sizes required for Pond 1 and Pond 3 protection if a 200,000 cfs flood occurs in Arroyo del Puerto.

Method: Use USACE method (ASCE 1995) because of large depth of flow and non-trapezoidal shape. This method requires velocity of flow and depth of flow which are taken from a HEC-RAS run for 200,000 cfs. The right over-bank velocity is used since it abuts Ponds 1 and 3.

Pond 3 Embankment:

At Section 4: $d = 6929.7 - 6924 = 5.7'$

$V_{rob} = 15.8$ ft/s

Using Figure 3-7 (ASCE, 1995) $D_{30} = 1.4'$

Using Table 3-1 (ASCE, 1995) $D_{50} = \underline{21 \text{ in.}}$ (average)

Thickness = $1.5 \times D_{50} \approx \underline{32 \text{ in}}$

APRON: Because rock required for longitudinal flow from Arroyo del Puerto is greater than that required for precipitation runoff (8.5 in), use rock size for longitudinal flow for apron

$D_{50} = 21$ in. However, thickness of 32 inches is more than adequate for precipitation runoff ($3 \times 8.5" = 25.5"$).

POND 3 SURFACE:

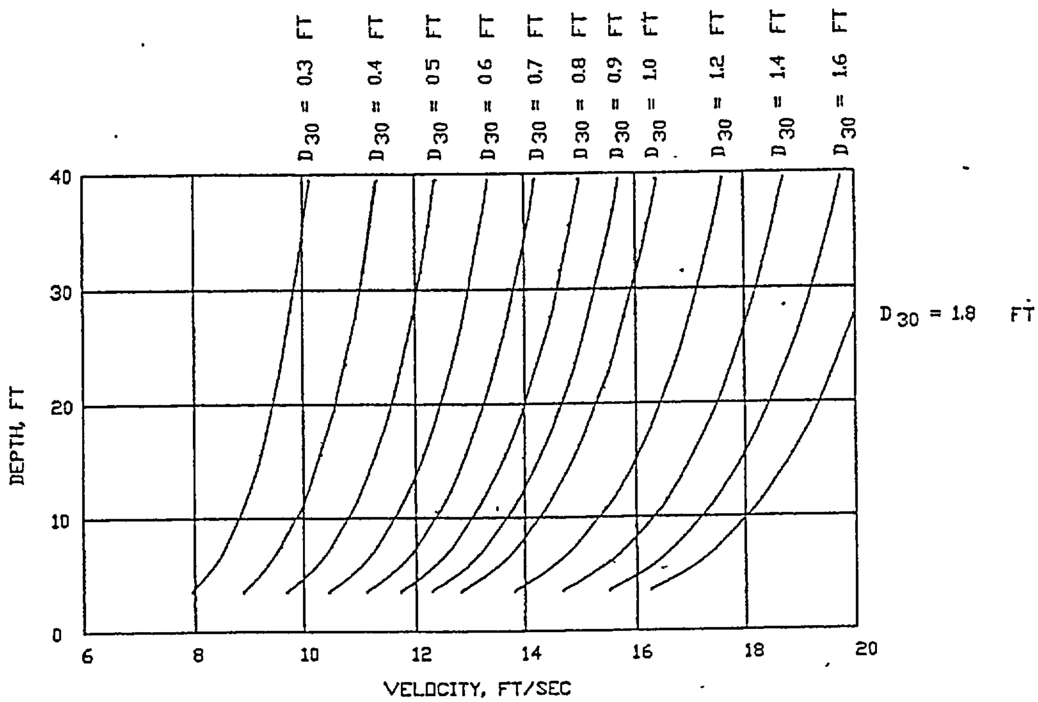
At Section 5, $d = 6941.0' - 6936' = 5'$

$V = 10.1$ ft/sec

From Figure 3-7, $D_{30} = 0.5$ in

From Table 3-1 $D_{50} = \underline{7.5}$ in. (average)

Thickness = $D_{100} = \underline{12"}$ from Table 3-1



NOTE: APPLICABLE TO THICKNESS $1D_{100}$ (max) AND CHANNEL BOTTOMS OR SIDE SLOPES FLATTER THAN OR EQUAL TO 1V ON 4H. STONE WEIGHT 165 pcf, C_S= 0.30, C_V= C_T= 1.0 S_f= 1.1 BASED ON EQUATION 3-3.

FIG. 3-7. Depth-Averaged Velocity Versus D_{30} and Depth

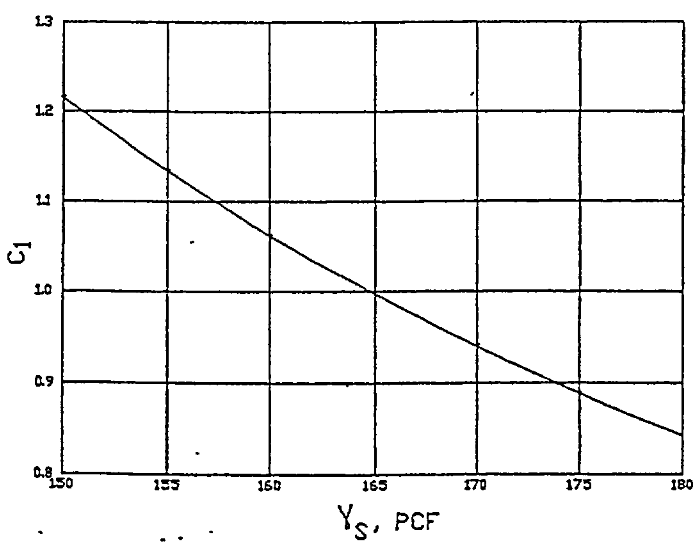


FIG. 3-8. Correction for Unit Stone Weight [D_{30} = C_1·(D_{30} from Figure 3-7) Where C_1 = Correction for Unit Stone Weight; Note: Do Not Make This Correction if D_{30} Computed from Equation 3-3]

Correction for the vertical velocity distribution in bends is shown in Figure 3-10. Limited testing has been conducted to determine the effects of blanket thickness greater than $1D_{100}$(max) on the stability of riprap. Results are shown in Figure 3-10.

(2) The basic procedure to determine riprap size using this method is as follows:

1. Determine average channel velocity (HEC-2 or other uniform flow computational methods, or measurement)

2. Find V_{SS} using Figure 3-3

3. Find D_{30} using Figure 3-7

4. Correct for other unit weights, side slopes, vertical velocity distribution, or thicknesses using Figures 3-8 through 3-10

5. Find gradation having D_{30}(min) ≥ computed D_{30}.

MAXIM TECHNOLOGIES INC® BY Bill Bucher DATE 8/15/02

JOB TITLE Rio Algom-Ambrosia Lake JOB NUMBER

SUBJECT Pond 3 Embankment - 200,000 cfs SHEET 1 of 1

Rock for Pond 3 Embankment - 200,000 cfs.

Assumptions: For 2000 ft of embankment at north end, cover entire slope with 32" of $D_{50}=21"$

For south 1800' of embankment, cover lower half of slope with $D_{50}=21"$ rock. On remainder just protect from precipitation runoff with 35" of $D_{50}=24"$ rock

For Apron, use same thickness (36") as original design.

Slope is 5:1

Use filter rock and sand under all surfaces

North Portion: 5:1 from 6936 to 6924' = 61 ft.

$2000 \times 61 \times \frac{32}{12} = 325000$ ft³

Filter gravel $2000 \times 61 \times .5 = 61,000$ ft³

South Portion

5:1 from 6936' to 6921' = $15' \times \sqrt{26} = 76.5$ ft.

$\frac{1}{2} \times 1800 \times 76.5 \times \frac{32}{12} = 184,000$ ft³ $D_{50}=21"$ rock

$\frac{1}{2} \times 1800 \times 76.5 \times \frac{35}{12} = 20,000$ ft³ $D_{50}=24"$ rock

Filter gravel $1800 \times 76.5 \times \frac{6}{12} = 69,000$ ft³

Filter Sand " 69,000 ft³

Apron: 378 ft² × 3800 ft = 144,000 ft³ $D_{50}=21"$ rock

Filter gravel = 18.9 ft × 3800 ft × 0.5 ft = 36,000 ft³

Filter sand = 18.9 × 3800 × 0.5 = 36,000 ft³

SUMMARY:

	ft³	CY
$D_{50}=21"$	653,000	24,000
$D_{50}=24"$	20,000	700
Filter Gravel	166,000	6,000
Filter Sand	166,000	6,000

ATTACHMENT B

SENSITIVITY ANALYSIS FOR PMF CALCULATION

SUPPORTING DATA

CN=88 (AMC = III)
Lag Time = 1.1 hr. 1.27
(New Mexico Method)

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

****************************************
***************************************
*                                      *                                                *
*
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                                                *   U.S. ARMY CORPS OF ENGINEERS
*
*              MAY   1991              *                                                *   HYDROLOGIC ENGINEERING CENTER
*
*           VERSION 4.0.1E             *                                                *        609 SECOND STREET
*
*                                      *                                                *     DAVIS, CALIFORNIA 95616
*
*  RUN DATE  08/15/2002  TIME  13:32:15 *                                               *         (916) 756-1104
*
*                                      *                                                *
*
****************************************
***************************************


                                          X     X  XXXXXXX   XXXXX        X
                                          X     X  X        X     X      XX
                                          X     X  X        X             X
                                          XXXXXXX  XXXX     X       XXXXX X
                                          X     X  X        X             X
                                          X     X  X        X     X       X
                                          X     X  XXXXXXX   XXXXX       XXX


                                       :::::::::::::::::::::::::::::::::::::::::::
                                       :::::::::::::::::::::::::::::::::::::::::::
                                       :::                                     :::
                                       :::  Full Microcomputer Implementation  :::
                                       :::                 by                  :::
```

```
:::          Haestad Methods, Inc.          :::
:::                                         :::
:::::::::::::::::::::::::::::::::::::::::::::::
:::::::::::::::::::::::::::::::::::::::::::::::
```

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 STRUCTURE.
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

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

  1           ID   QUIVIRA - ARROYO DEL PUERTO FLOOD HYDROLOGY  FILE:ADPIN6.TXT
  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
              *
              * Rainfall time increment
  6           IN      60
              *
              * *** GLOBAL OUTPUT OPTIONS
  7           IO       2       0
              *
              * ***
              *
              *

  8           KK     IN1
  9           KM   HYDROGRAPH FOR ARROYO DEL PUERTO DRAINAGE
              *
              * Basin area
 10           BA    57.6
              *
              * Rainfall data
 11           PB     9.2
 12           PI     .02     .04     .12     .74     .05     .03
              * Basin Losses
 13           LS       0      88       0
              *
              * Unit hydrograph
 14           UD    1.27
              *
              * ***
              *
              *
 15           ZZ
```

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

*****************************************
****************************************
*                                       *                                                 *
*
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                                 *   U.S. ARMY CORPS OF ENGINEERS
*
*              MAY   1991               *                                                 *   HYDROLOGIC ENGINEERING CENTER
*
*           VERSION 4.0.1E              *                                                 *         609 SECOND STREET
*
*                                       *                                                 *      DAVIS, CALIFORNIA 95616
*
*  RUN DATE  08/15/2002  TIME  13:32:15 *                                                 *          (916) 756-1104
*
*                                       *                                                 *
*
*****************************************
****************************************



                              QUIVIRA - ARROYO DEL PUERTO FLOOD HYDROLOGY  FILE:ADPIN6.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
             PRECIPITATION DEPTH     INCHES
             LENGTH, ELEVATION       FEET
             FLOW                    CUBIC FEET PER SECOND
             STORAGE VOLUME          ACRE-FEET
             SURFACE AREA            ACRES
             TEMPERATURE             DEGREES FAHRENHEIT


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***
***


            **************
            *            *
   8 KK     *       IN1  *
            *            *
            **************
                         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	0.27	INITIAL ABSTRACTION
CRVNBR	88.00	CURVE NUMBER
RTIMP	0.00	PERCENT IMPERVIOUS AREA

14 UD SCS DIMENSIONLESS UNITGRAPH

TLAG	1.27	LAG

***

UNIT HYDROGRAPH
27 END-OF-PERIOD ORDINATES

1703.	5185.	10795.	16716.	19689.	19785.	17779.	14878.	10906.	7885.
5868.	4486.	3351.	2496.	1869.	1387.	1027.	769.	574.	431.
321.	242.	190.	147.	104.	68.	32.			

****
****

HYDROGRAPH AT STATION IN1

****
****

DA MON HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q	*	DA MON HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q
1 JUL 0000	1	0.00	0.00	0.00	0.	*	1 JUL 0615	26	0.00	0.00	0.00	46535.
1 JUL 0015	2	0.05	0.05	0.00	0.	*	1 JUL 0630	27	0.00	0.00	0.00	37234.
1 JUL 0030	3	0.05	0.05	0.00	0.	*	1 JUL 0645	28	0.00	0.00	0.00	29723.
1 JUL 0045	4	0.05	0.05	0.00	0.	*	1 JUL 0700	29	0.00	0.00	0.00	23284.
1 JUL 0100	5	0.05	0.05	0.00	0.	*	1 JUL 0715	30	0.00	0.00	0.00	17983.
1 JUL 0115	6	0.09	0.09	0.00	0.	*	1 JUL 0730	31	0.00	0.00	0.00	13703.
1 JUL 0130	7	0.09	0.09	0.01	11.	*	1 JUL 0745	32	0.00	0.00	0.00	10296.
1 JUL 0145	8	0.09	0.08	0.02	60.	*	1 JUL 0800	33	0.00	0.00	0.00	7656.
1 JUL 0200	9	0.09	0.07	0.02	195.	*	1 JUL 0815	34	0.00	0.00	0.00	5710.
1 JUL 0215	10	0.28	0.16	0.11	603.	*	1 JUL 0830	35	0.00	0.00	0.00	4274.
1 JUL 0230	11	0.28	0.12	0.15	1514.	*	1 JUL 0845	36	0.00	0.00	0.00	3213.

1 JUL 0245	12	0.28	0.09	0.18	3192.	*	1 JUL 0900	37	0.00	0.00	0.00	2414.
1 JUL 0300	13	0.28	0.08	0.20	5763.	*	1 JUL 0915	38	0.00	0.00	0.00	1812.
1 JUL 0315	14	1.70	0.26	1.44	11136.	*	1 JUL 0930	39	0.00	0.00	0.00	1346.
1 JUL 0330	15	1.70	0.12	1.59	21408.	*	1 JUL 0945	40	0.00	0.00	0.00	958.
1 JUL 0345	16	1.70	0.07	1.64	39171.	*	1 JUL 1000	41	0.00	0.00	0.00	627.
1 JUL 0400	17	1.70	0.04	1.66	64955.	*	1 JUL 1015	42	0.00	0.00	0.00	384.
1 JUL 0415	18	0.11	0.00	0.11	92430.	*	1 JUL 1030	43	0.00	0.00	0.00	217.
1 JUL 0430	19	0.12	0.00	0.11	114908.	*	1 JUL 1045	44	0.00	0.00	0.00	120.
1 JUL 0445	20	0.11	0.00	0.11	126022.	*	1 JUL 1100	45	0.00	0.00	0.00	84.
1 JUL 0500	21	0.11	0.00	0.11	123772.	*	1 JUL 1115	46	0.00	0.00	0.00	58.
1 JUL 0515	22	0.07	0.00	0.07	111112.	*	1 JUL 1130	47	0.00	0.00	0.00	38.
1 JUL 0530	23	0.07	0.00	0.07	93420.	*	1 JUL 1145	48	0.00	0.00	0.00	24.
1 JUL 0545	24	0.07	0.00	0.07	75215.	*	1 JUL 1200	49	0.00	0.00	0.00	14.
1 JUL 0600	25	0.07	0.00	0.07	58864.	*	1 JUL 1215	50	0.00	0.00	0.00	7.
						*						

```
*****************************************************************************************************
****

   TOTAL RAINFALL =    9.20, TOTAL LOSS =    1.46, TOTAL EXCESS =    7.74

 PEAK FLOW     TIME                          MAXIMUM AVERAGE FLOW
                                 6-HR       24-HR        72-HR      12.25-HR   (CFS)        (HR)
                         (CFS)  126022.       4.75                 47407.      23499.      23499.      23499.
                      (INCHES)      7.652       7.744       7.744       7.744
                       (AC-FT)     23508.      23790.      23790.      23790.

                      CUMULATIVE AREA =   57.60 SQ MI
```

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

```
                                                  PEAK   TIME OF      AVERAGE FLOW FOR MAXIMUM PERIOD          BASIN        MAXIMUM       TIME OF
            OPERATION           STATION           FLOW    PEAK                                                  AREA         STAGE       MAX STAGE
6-HOUR        24-HOUR        72-HOUR

            HYDROGRAPH AT                                         IN1    126022.     4.75         47407.      23499.       23499.        57.60
```

*** NORMAL END OF HEC-1 ***

MAXIM TECHNOLOGIES INC® BY B. Bucher DATE 8/15/02

JOB TITLE Rio Algom - Ambrosia Lake JOB NUMBER

SUBJECT Lag Time for Arroyo del Puerto SHEET

Using USGS WRI Report 01-4154 (Relations for Estimating Unit-Hydrograph Parameters in New Mexico)

lag time = $t_L = 0.040\ L^{0.606}\ Sh^{0.255}$

L = Stream length (miles)

Sh = Shape or basin width per stream length (ft per mile)

Basin Area = 57.6 mi^2

Basin Length = 12.4 mi

Ave width = $\frac{57.6}{12.4}$ = 4.65 mi → 24,500 ft

$Sh = \frac{24,500}{12.4} = 1980\ \frac{ft}{mi}$

$t_L = 0.040 \times 12.4^{0.606} (1980)^{0.255}$

$= 1.27$ hr

Compared to 1.83 hr. from SCS method.

ATTACHMENT C

LATERAL MIGRATION RATE CALCULATION

WHB
9/6/02

Arroyo del Puerto Lateral Migration Rate

Use equation 1 of Nanson + Hickin (1986)

$$\bar{M}^* = 1.663\, Q_5^{0.482} S^{0.368}$$

Q_5 = 5 yr peak flow in m^3/s

Use Waltemeyer 1996 for Q_5.

For Central Mtn. Valley region of New Mexico, $Q_5 = 2.57 \times 10^5 A^{0.47} \left(\frac{E_c}{1000}\right)^{-4.49} I_{24,10}^{1.76}$

A = basin area = 57.6 mi²

E_c = Avr. of 10% and 85% points on stream length (12.4 miles total)

10% point at 1.24 miles, EL = 6970'

85% point at 10.5 miles, EL = 7450'

$E_c = (6970 + 7450)/2 = 7210'$

$I_{24,10}$ = Intensity of 24hr - 10yr storm = 1.9" (Precipitation - Frequency Atlas for Western US, 1973)

$$Q_5 = 2.57 \times 10^5 (57.6)^{0.47} \left(\frac{7210}{1000}\right)^{-4.49} (1.9)^{1.76}$$

= **750 cfs** or 21.2 m^3/s

S = 0.005 from mapping = 5 m/km

D_{50} = assume 0.18mm, the low end of the range for which the equation is valid