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C. file

UMETCO MINERALS CORPORATION

WHITE MESA MILL

DRAINAGE REPORT
FOR
SUBMITTAL TO NRC

JANUARY, 1990

Umetco Minerals Corporation

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January 10, 1990

Mr. Ramon E. Hall, Director
U. S. Nuclear Regulatory Commission
Region IV
Uranium Recovery Field Office
Box 25325
Denver, CO 80225

Re: Umetco Minerals Corporation
SUA-1358: Docket No. 40-8681
White Mesa Mill, Utah
Cell 4A Request for Information

Dear Mr. Hall:

Attached are the calculations and data concerning hydrology at the White Mesa Mill, with special reference to Cell 4A. This submittal is the result of meetings and calls between your staff and that of Umetco. A summary of the history of hydrology issues at the mill is included, along with the specific requests for licensing action. This submittal should complete the information requirements so that unrestricted use of Cell 4A will be granted.

If I can answer any questions that you may have, please feel free to contact me.

Sincerely yours,

DK Sparling

D. K. Sparling
Site Superintendent dj

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INTRODUCTION

WHITE MESA MILL

Hydrologic Design Report

Background

During the license amendment process for the proposed Cell 4A, several questions were raised by NRC concerning the operating levels and freeboard requirements for all of the cells. Information has been provided in a piecemeal fashion to answer the specific NRC questions. NRC has responded to some of these submittals by asking for additional information. Recently Umetco and NRC have agreed upon the methods and procedures to be used to determine the freeboard and the operating levels. This document is intended to provide an in-depth analysis of each cell and drainage using the agreed upon methods. The use of these procedures has resulted in changes in the freeboard and operating levels found in recent correspondence between NRC and Umetco. Specifically some of the information found in Umetco letters dated September 6, 1989 and September 29, 1989 has been superseded by this submittal. This report also supercedes the "Drainage Report for Submittal to NRC", October 1989 that was not accepted by the NRC.

This report is divided into several sections to allow easy reference to a particular cell or calculation.

Design Standards

NRC requested that the hydrology, flood routing and freeboard requirements be determined using Staff Technical Position Paper WM 8201, January 1983, NUREG/CR4620, and the Army Corp. of Engineers Shore Protection Manual.

The PMP was determined using Hydrometeorological Report No. 49. The PMF and flood routing were derived from procedures found in NUREG/CR 4620.

Ditches 1, 2 and 3 were designed using SCS Technical Release 20 to determine the peak flows and for flood routing. Mannings equation was used to size the channels.

The freeboard above the PMF was determined using the Army Corp. of Engineers Shore Protection Manual. The design wind speed was set at 30 mph in accordance with conversations with NRC.

A new drainage map showing ditches 1, 2 and 3 is attached to this submittal.

A uniform soil retention loss equal to 0.24 in/hr was deducted from the total PMP volume requirement where applicable. In addition 4 depressions were assumed to store a portion of the runoff that would normally flow to Cell 1I. A map showing the depressions is included under the section labeled Cells 1 through 4.

Licensing Actions

The design of Cell 4A and 4B instigated a review of hydrology issues at the White Mesa Mill. The original freeboard limits, as calculated by D'Appolonia, were based on a PMF series that resulted in 15 inches of rainfall. The freeboard limits in all tailings cells were set at 5 feet, as measured from dike crest. Just prior to the start of mill operations, a request to change the freeboard limit in Cell 1-I was submitted to the NRC. The request was based on Hydrometeorlogical Report No. 49. The request assumed the construction of Diversion Ditches 2 and 3 and used a seasonal rainfall of less than 5 inches for the mill area contribution.

After mill acquisition by Umetco, it became known that Ditches 2 and 3 had not been built and this information was discussed with the NRC. At the time it was assumed that the 15 inch PMP was the basis for these ditches and therefor Ditches 2 and 3 were not required.

When these facts were brought together during Cell 4 review, immediate action was taken to provide 3 diversion ditches. Engineering data was then developed that showed construction activities had not yet conformed to design standards. Consultation with the NRC during this time also resulted in changes to design calculations. These changes required further construction activity in Ditch 1 and 2.

As of the date of this submittal, the design basis for diversion ditch construction has been agreed upon. The design indicates that some limited construction activity is necessary. All diversion ditches will be constructed to the minimum dike design cross-sections by March 31, 1990. This date was chosen to accommodate inclement weather conditions.

It became apparent in December 1989 that Umetco did not have the room in the tailings area to operate the mill without the availability of Cell 4A. As the PMP was a thunderstorm, the probability of the PMP event during the winter months was very low. On this basis temporary freeboard limits were agreed to based on one-half the PMP amount. The limits being applied for in this submittal are lower than those based on one-half the PMP. Current (January 11, 1990) solution levels are higher than the new limits being applied for, as shown by the following table.

	Current Levels	Temporary Limit	New Limit
Cell 1-I	5615.4	5616.1	5615.4
Cell 2	Dry	No liquid allowed	Liquids and tailings allowed when spillway is constructed
Cell 3	5604.8	5605.4	5603.0

Umetco requests that the following conditions be incorporated into the license amendment allowing use of Cell 4A.

1. Cell 4A use is approved for use as part of the mill tailings management system, with no restrictions on flow rates.

2. The restriction on liquids in Cell 2 is removed when the spillway is constructed. This will allow Cell 2 to be filled to capacity.

3. The new freeboard limits are made effective 60 days after Cell 4A use approval, so that liquid levels can be adjusted by pumping.

4. Diversion ditch completion date be set at March 31, 1990.

5. Maximum operating pool levels be listed as feet elevation from mean sea level, as determined by survey at the mill.

6. NRC review and approval of this submittal to be completed by March 1, 1990.

PMP/PMF SUMMARY

PMP/PMF Summary

The PMP was determined using Hydrometeorolgical Report No. 49. Both the 6 hour General Storm and the 6 hour Local Storm were determined. Due to the small drainage basin size the Local Storm produced the larger storm. The resultant 6 hour PMP used for design is 10 inches.

The PMP summary shows the depth requirements and maximum operating levels for each of the cells.

			WHITE MESA MILL			
			CELL 1I DRAINAGE REQUIREMENTS			
DRAINAGE AREA	AREA (ACRES)	PMP (INCHES)	DIRECT RUNOFF (AF)	SOIL RETENTION LOSS (AF)	MISC. STORAGE (AF)	TOTAL NET VOL. REQ. (AF
CELL 1I	52.9	10	44.2			
BASIN A2 W/O CELL1I	31.1	10	25.9	3.73		44.2
BASIN B3	64	10	53.3	7.68		22.17
DEPRESSION D	1.02					45.62
DEPRESSION A	0.88				4.4	-4.4
DEPRESSION B	1.45				1.8	-1.8
DEPRESSION C	0.42				2.3	-2.3
					0.5	-0.5
				TOTAL VOLUME REQUIRED		103 A
				PMP DEPTH =103AF/52.9AC		1.95FT
	SOIL RETENTION LOSS-HYDROLOGIC SOIL GROUP B =.24 IN/HR					
	* EXAMPLE	BASIN A2 31.1 AC*.24 IN/HR *6 HR/12 IN/FT= 3.73AF				
	FREEBOARD REQUIREMENTS		.9+1.95=2.85FT			
	MAX OPERATING LEVEL		**5618.2-2.6=5615.35**			

WHITE MESA MILL
CELL 3 DRAINAGE ANALYSIS

OPERATING LEVEL UNTIL 12/31/90 WITH CELL 2 APPROXIMATELY 25% RECLAIMED.

DRAINAGE AREA	AREA (ACRES)	PMP (INCHES)	DIRECT RUNOFF (AF)	SOIL RETENTION LOSS (AF) *	TOTAL NET VOL. REQ. (AF)			
CELL 2 RECLAIMED	20.2	10	16.8	2.4	14.4		WAVE RUNUP	.7FT
CELL 2 TAILINGS	60.5	10	50.4	3.6	46.8		PMP DEPTH REQ	4.27FT
SAND BEACH	49.4	10	41.2	3	38.2		TOTAL FREEBOARD	4.97FT
CELL 3 DIRECT	28.9	10	24		24		MAX OPERATING LEVEL	5608-4.97=5603.03
				TOTAL VOL REQ	123.4			
			DEPTH REQ. = 123.4/28.9=		4.27			

			WHITE MESA MILL						
			CELL 4 DRAINAGE ANALYSIS						
DRAINAGE									
AREA	AREA	PMP	DIRECT	SOIL RETENTION	TOTAL NET				
	(ACRES)	(INCHES)	RUNOFF (AF)	LOSS (AF) *	VOL. REQ. (AF)				
CELL 4A-DIRECT-NET							WAVE RUNUP	0.77FT	
	43.25	10	36		36		PMP DEPTH REQ	0.83FT	
				TOTAL VOL REQ.			TOTAL FREEBOARD	1.6FT	
				DEPTH REQ.=36/43.25=	.83FT				
							MAX OPERATING LEVEL	5598-1.6=5596.4	

Table 6.3A.—Local-storm PMP computation, Colorado River, Great Basin and California drainages. For drainage average depth PMP. Go to table 6.3B if areal variation is required.

Drainage White Mesa Cell 4 Area < 1.0 (mi^2) (km^2)
Latitude 37° 35' Longitude 109° 35' Minimum Elevation 5600 (ft) (m)

Steps correspond to those in sec. 6.3A.

1. Average 1-hr 1-mi^2 (2.6-km^2) PMP for drainage [fig. 4.5]. 8.5 in. (mm)

2. a. Reduction for elevation. [No adjustment for elevations up to 5,000 feet (1,524 m): 5% decrease per 1,000 feet (305 m) above 5,000 feet (1,524 m)]. 97 %

   b. Multiply step 1 by step 2a. 8.25 in. (mm)

3. Average 6/1-hr ratio for drainage [fig. 4.7]. 1.20

	Duration (hr)									
	1/4	1/2	3/4	1	2	3	4	5	6	
4. Durational variation for 6/1-hr ratio of step 3 [table 4.4].	74	89	95	100	110	115	118	119	120	%
5. 1-mi^2 (2.6-km^2) PMP for indicated durations [step 2b X step 4].				8.25					9.9	in. (mm)
6. Areal reduction [fig. 4.9].										%
7. Areal reduced PMP [steps 5 X 6].				8.25					9.9	in. (mm)
8. Incremental PMP [successive subtraction in step 7].										in. (mm)
	} 15-min. increments									

9. Time sequence of incremental PMP according to:

   Hourly increments [table 4.7]. in. (mm)

   Four largest 15-min. increments [table 4.8]. in. (mm)

CELLS 1 THRU 4

Cell 1I

Cell 1I is used primarily for sediment control and liquid management. The total volume requirement is 103 acre feet resulting in a depth requirement of 1.95 feet. The maintained crest elevation is 5618.2. The wave runup is .9 feet so the maximum operating level is 5615.4.

A uniform soil retention loss equal to .24 in/hr was deducted from the total volume requirement where applicable. In addition 4 depressions were assumed to store a portion of the runoff that would normally flow to Cell 1I. A map showing the depressions is attached to this section.

Note that no credits were taken for storage in the mill area sedimentation pond.

Cell 2 and 3

A four foot interim cover has been placed on over 25% of Cell 2. A soil retention loss of .24 in/hr was taken for the 25% of Cell 2 that has the interim cover and other areas not occupied by water. One half the soil retention loss, .12 in/hr, was taken for the remainder of Cell 2 since the tailings surface to a depth of approximately 4 inches is dry by the time the PMF would occur in late summer. The freeboard requirements for Cases 2 and 3 will change as more cover is placed on Cell 2. For the purpose of these calculations it was assumed all runoff from Cell 2 will flow to Cell 3.

The flood depth requirement for Cell 3 is constantly changing as new tailings are placed in the Cell. Three different cases were examined. The first case assumed Cell 3 was 35 percent full of tailings. Judging from the aerial topography and recent surveys Case 1 probably applies now. The depth to accommodate Case 1 is 2.44 feet. The storage requirement is 124.9 acre feet. Combined with a wave runup requirement of .92 feet the maximum operating level is elevation 5604.64.

Case 2 assumed that Cell 3 is 50 percent full of tailings. Based on current mill production this level will probably not be reached for at least 1 year. The depth required to accommodate the PMF under this scenario is 3.32 feet with a storage requirement of 124.0 acre feet. Since the fetch distance is less the wave runup decreases to .78 feet resulting in a maximum operating level elevation 5603.9.

Case 3 assumes Cell 3 is 75 percent full of tailings. The PMF depth requirement is 6.79 feet. The wave runup decreases to .62 feet resulting in a maximum operating level elevation 5600.59 By the time Case 3 would apply it is possible that Cell 2 will be completely reclaimed and the Cell 2 drainage will no longer flow into Cell 3. Other operational changes may also take place before this case would be valid.

Using all three (3) Cases, a graph labeled Figure 1 was developed showing the total freeboard requirements vs the cell surface area available for flood storage.

WHITE MESA MILL
CELL 3 DRAINAGE ANALYSIS

	CASE 1	CELL 2 25 % RECLAIMED/CELL 3 35% FULL OF TAILINGS						
DRAINAGE	AREA	PMP	DIRECT	SOIL RETENTION	TOTAL NET			
AREA	(ACRES)	(INCHES)	RUNOFF (AF)	LOSS (AF) *	VOL. REQ. (AF)			
CELL 2 RECLAIMED	20.2	10	16.8	2.4	14.4		WAVE RUNUP	.92FT
CELL 2 TAILINGS	60.5	10	50.4	3.6	46.8		PMP DEPTH REQ	2.44FT
35% SAND BEACH	27.1	10	22.6	1.6	21		TOTAL FREEBOARD	3.36FT
CELL 3 DIRECT	51.2	10	42.7		42.7			
				TOTAL VOL REQ	124.9 AF		MAX OPERATING LEVEL	5608-3.36=5604.64
			DEPTH REQ. = 124.9/51.2 =		2.44FT			

	CASE 2	CELL 2 25 % RECLAIMED/CELL 3 50% FULL OF TAILINGS						
DRAINAGE	AREA	PMP	DIRECT	SOIL RETENTION	TOTAL NET			
AREA	(ACRES)	(INCHES)	RUNOFF (AF)	LOSS (AF) *	VOL. REQ. (AF)			
CELL 2 RECLAIMED	20.2	10	16.8	2.4	14.4		WAVE RUNUP	.78FT
CELL 2 TAILINGS	60.5	10	50.4	3.6	46.8		PMP DEPTH REQ	3.32FT
50% SAND BEACH	40.9	10	34.1	2.5	31.6		TOTAL FREEBOARD	4.1FT
CELL 3 DIRECT	37.4	10	31.2		31.2			
				TOTAL VOL REQ	124 AF		MAX OPERATING LEVEL	5608-4.1=5603.9
			DEPTH REQ. = 124./37.4=		3.32 FT			

	CASE 3	CELL 2 25 % RECLAIMED/CELL 3 75 % FULL OF TAILINGS						
DRAINAGE	AREA	PMP	DIRECT	SOIL RETENTION	TOTAL NET			
AREA	(ACRES)	(INCHES)	RUNOFF (AF)	LOSS (AF) *	VOL. REQ. (AF)			
CELL 2 RECLAIMED	20.2	10	16.8	2.4	14.4		WAVE RUNUP	.62FT
CELL 2 TAILINGS	60.5	10	50.4	3.6	46.8		PMP DEPTH REQ	6.79FT
75% SAND BEACH	60.2	10	50.2	3.6	46.6		TOTAL FREEBOARD	7.41FT
CELL 3 DIRECT	18.1	10	15.1		15.1			
				TOTAL VOL REQ	122.9 AF		MAX OPERATING LEVEL	5608-7.41=5600.59
			DEPTH REQ. = 122.9/18.1=		6.79 FT			

*SOIL RETENTION FOR AREAS W/ GROUND COVER =.24 IN/HR
*SOIL RETENTION FOR AREAS W/ EXPOSED TAILINGS = .12 IN/HR

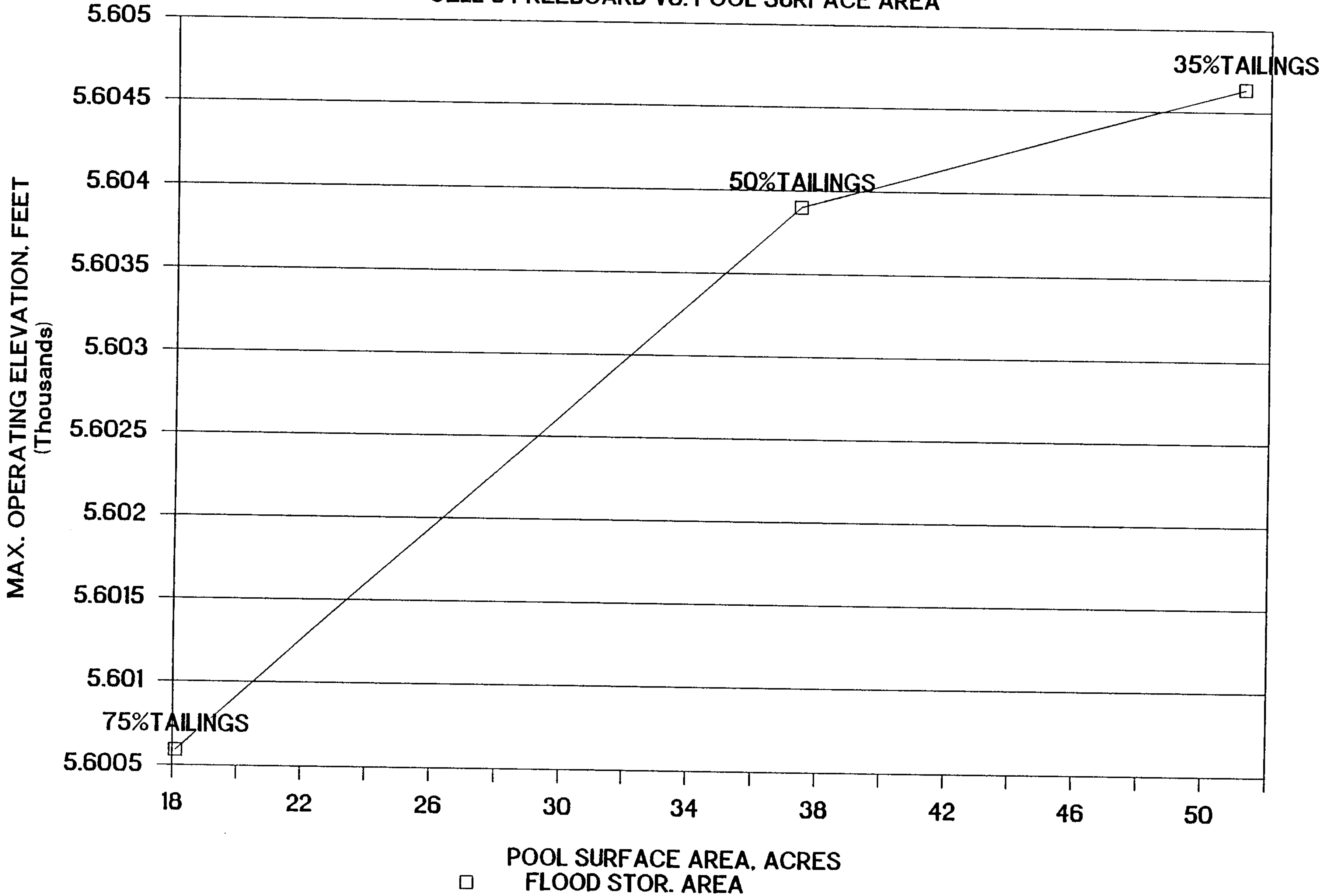
SINCE THE CELL 3 CAPACITY IS CONTINUALLY BEING DECREASED BY THE ADDITION OF TAILINGS, THE ABOVE 3 CASES WERE CONSIDERED. CELL 2 WAS ASSUMED TO BE 25% RECLAIMED FOR ALL CASES. THIS MEANS THAT AT LEAST 25% OF THE AREA IN DRAINAGE BASIN C HAS EITHER BEEN COVERED WITH THE INITIAL 4 FOOT OF COVER OR WAS NOT PART OF THE ORIGINAL CELL. SHEET C4-6 SHOWS THE 25% ESTIMATE IS CONSERVATIVE. REFERENCES TO THE 35%, 50%, AND 75% FULL OF TAILINGS REFERS TO THE AMOUNT OF SURFACE AREA AT ELEV. 5602.6 NOT AVAILABLE FOR STORING RUNOFF. ELEV. 5602.6 WAS ARBITRARILY CHOOSEN AS A CONVIENENT BENCHMARK TO MEASURE THE SURFACE AREA FROM THE AERIAL MAPS. THE CELL 3 DIRECT AREA REFERS TO THE SURFACE AREA AVAILABLE FOR LIQUID STORAGE. THE 3 CASES WERE USED TO DEVELOP THE GRAPH LABELED WHITE MESA MILL- CELL 3 FREEBOARD VS. POOL SURFACE AREA. THE TOTAL SURFACE AREA AVAILABLE FOR STORAGE ON 10/23/89, THE DATE OF THE LAST FLIGHT, WAS 46 ACRES INDICATING A MAX. OPERATING LEVEL AT ELEV. 5604.4 USING THE CELL 3 FREEBOARD GRAPH. THE VARIOUS AREAS ARE SHOWN ON THE DRAINAGE MAP, SHEET C4-6.

Cell 4

The tributary drainage area is 43.25 acres resulting in a PMF volume of 36 acre-feet. The wind runup procedure recommended by NRC, using a 30 mph wind speed, results in a freeboard above the flood pool of .77 feet. The freeboard coupled with the .83 foot flood depth results in a maximum operating level of elevation 5596.4.

WHITE MESA MILL

CELL 3 FREEBOARD VS. POOL SURFACE AREA



Procedure Freeboard Limits for Cell 3

The following procedure is intended to be used to determine the maximum operating pool level in Cell 3. This procedure is necessary because tailings sand deposition occupies some of the volume required to hold a PMP event. A summary of the procedure is as follows.

1. From a survey of the cell, the pool surface area will be determined.

2. From this area 17.3 acres will be subtracted. The wave run-up requirement is determined. The maximum operating pool level is then calculated from the dike crest elevation minus the flood volume requirements divided by the pool area minus the wave run-up requirements.

The basis of the procedure will now be discussed.

During the period of March 1988 through October 1989, 465,839 dry tons of tailings were added to Cell 3. From topographic maps generated from aerial photographs, see Figure 2, the pool surface area of Cell 3 was reduced by 11.9 acres. Total dry tons divided by change in pool area extent yields the number of tons required to reduce pool size by one acre, or 39,146 dry tons per acre.

The maximum amount of tailings that could be discharged in a one-year time period is 2,000 dry tons per day for 365 days with 93% mill availability, or 678,900 dry tons. The maximum tonnage divided by the number of tons required to reduce pool size by one acre yields the maximum expected pool area reduction, or 17.3 acres. This number is then subtracted from the pool surface area determined by survey or topographic means, yielding the reduced pool area.

The flood volume requirements are 123.4 acre-feet as per section titled Cells 1 through 4. The flood volume divided by the reduced pool area is the freeboard required for the flood. Wind-wave run-up is calculated as per the section titled Wave Run-up. The PMP freeboard requirement plus the wave run-up requirement yields the total required freeboard.

Note that this procedure overestimates the required freeboard and therefore is a conservative method. The overestimation is a result of not taking credit for the volume available for flood storage above the sand surface. In other words, this method assumes that the sand at the edge of the pool rises vertically to the top of the cell. The procedure also does not take credit for soil retention losses considered in the three cases used to develop the freeboard vs pool surface area graph.

The following example is calculated using the above procedure. On October 22, 1989 the pool area in Cell 3, determined by aerial photography, was 46.2 acres.

$$46.2 - 17.3 = 28.9 \text{ acres}$$

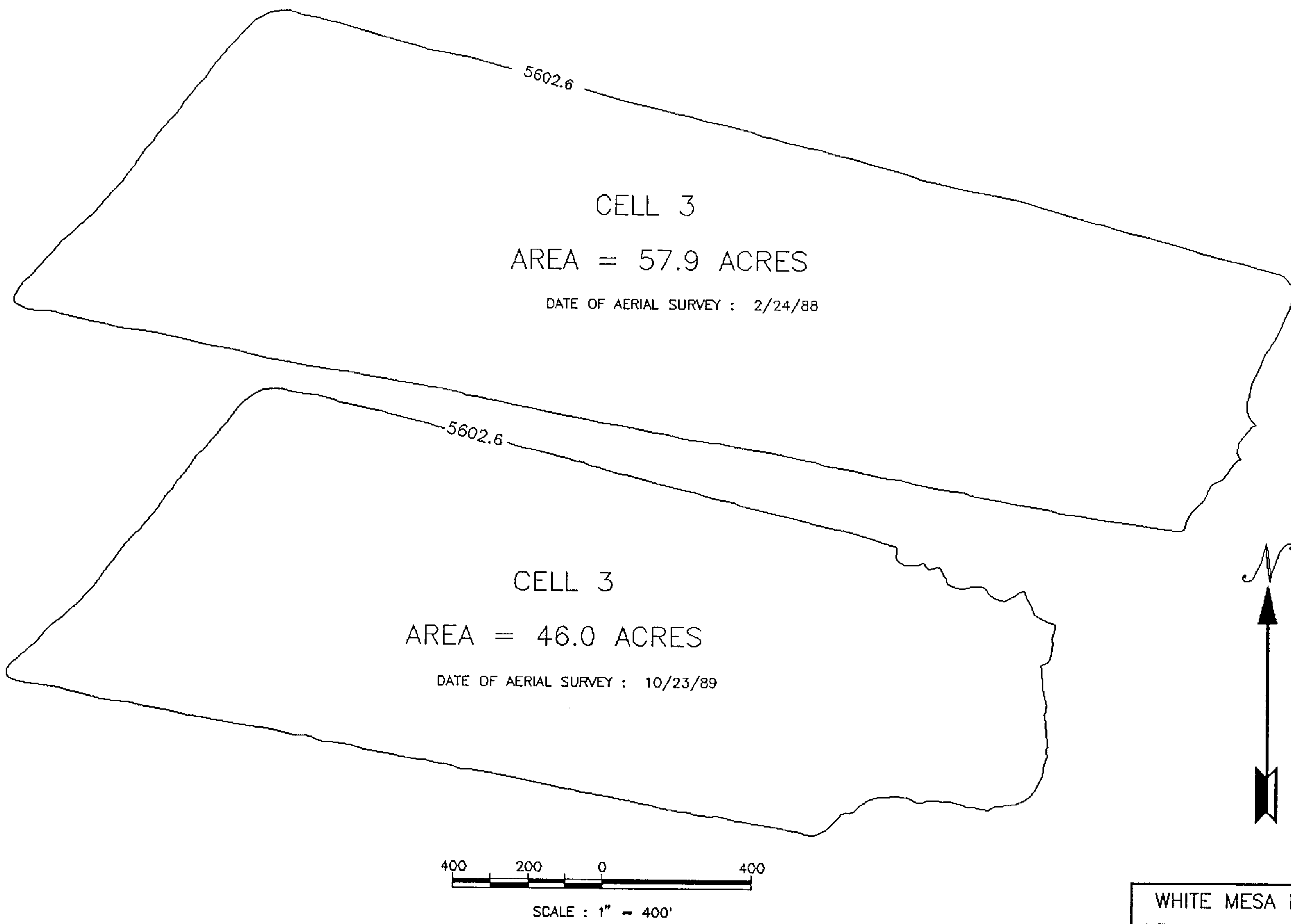
This is the area in which the storm volume must be stored. The storm volume was determined to be 123.4 acre-feet.

$$123.4 \text{ acre-feet} / 28.9 \text{ acres} = 4.3 \text{ feet}$$

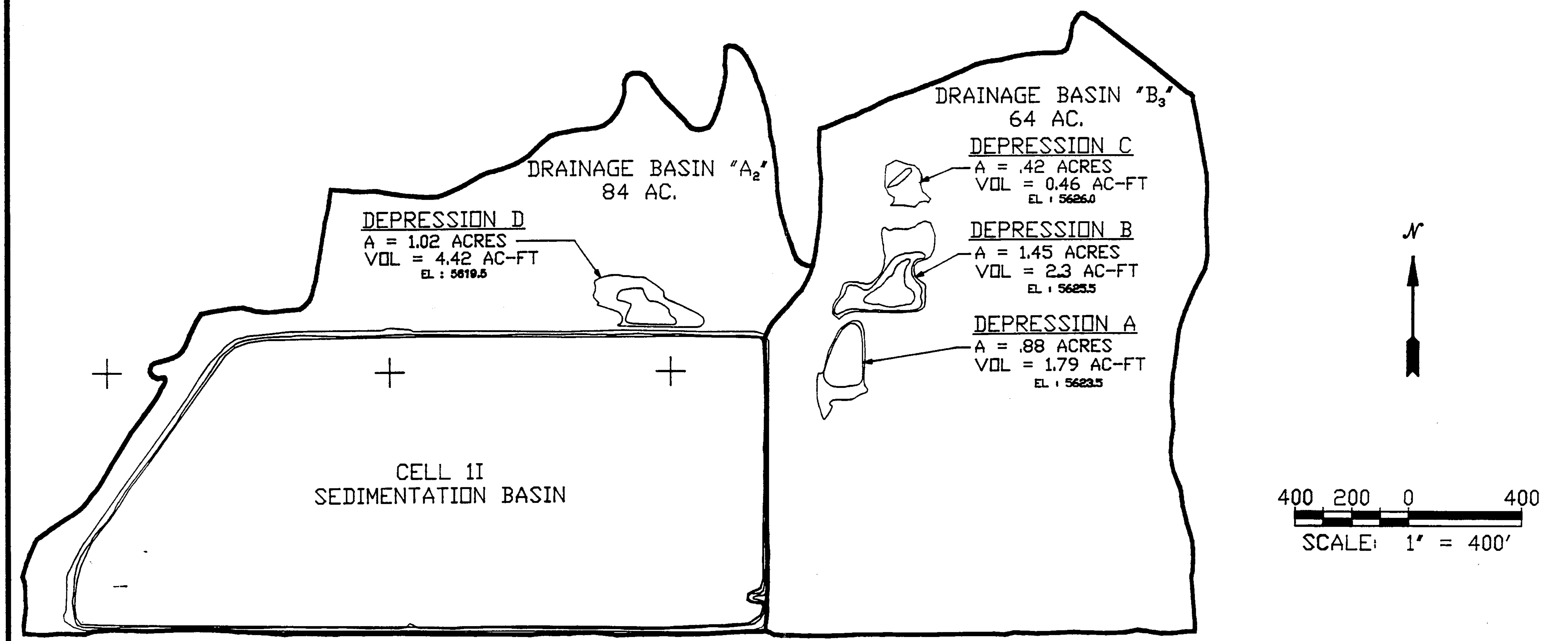
$$4.3 \text{ feet} + 0.7 \text{ feet} = 5.0 \text{ feet}$$

$$5608.0 \text{ feet} - 5.0 \text{ feet} = 5603.0 \text{ feet msl}$$

This procedure will be used yearly if the mill operates on a continuous basis. If the mill is shut down, then this procedure will be used and a submittal made to the NRC when 600,000 dry tons have been placed in Cell 3.



WHITE MESA MILL
AREA AVAILABLE FOR FLOOD STORAGE
DATE: 12/20/89 FIGURE 3



UMETCO MINERALS CORPORATION

MISCELLANEOUS
STORAGE DEPRESSIONS

DATE: 11/7/89

FIGURE 2

DITCH DRAINAGE

Ditch Drainage

1.0 Ditch 1

Ditch 1 is an existing ditch that was recently enlarged during routine maintenance. The ditch captures 23 acres of runoff from Drainage Basin A that would normally have flowed to Cell 1I. SCS TR20 procedures were used to determine the peak flow used for the channel design. A runoff curve number of 77 was selected from Table 2.2.d. of the SCS TR55 reference. The hydrologic soil group for all of the soils in the area is Type B. The peak discharge for the entire basin was determined to be 382 cfs. This discharge applies to the end of the ditch and the discharge decreases as you move up the channel. Manning's formula was used to size the channel resulting in channel dimensions near the end of the ditch consisting of a 10-foot wide bottom, 2 HORZ:1 VERT. Side slopes, average channel slope equal to .0011 ft/ft and a depth requirement of 4.6 feet. The AS-BUILT cross-sections verify for the most part that the channel meets these criteria. Areas with insufficient depth will have the dikes raised to achieve a minimum depth of 4.6 feet with .5 feet freeboard, by March 31, 1990. The stations requiring further work are shown on the cross-sections at the end of this report. The velocity at 4.6 feet depth is 4.4 fps.

2.0 Ditch 2

Ditch 2 and a flood detention pond were constructed to divert 48 acres of drainage from Basin B out of Cell 1I. The flood detention pond has a capacity of 20 acre feet. SCS TR20 procedures were used to determine the peak discharge requirement. The resultant inflow discharge was 1248 cfs, but the 20 AF storage attenuated the majority of the peak flow so the maximum discharge requirement from ditch 2 is 278 cfs. Mannings formula was used to size the channel. Channel dimensions include a 10-foot wide bottom, 2 HORZ: 1 VERT side slopes, .001 ft/ft slope and a depth requirements of 4.0 feet. In order to achieve the 4.0 foot depth, the channel bottom has to be lowered to elevation 5644.0 or to the same elevation as the toe of the pond dike. This work will be completed by March 31, 1990. The velocity at 4.0 feet depth is 3.9 fps.

3.0 Ditch 3

Ditch 3 drains approximately 2.6 acres away from Cell 1I. The peak discharge generated during a PMF is 97 cfs. Channel requirements are 10-foot wide bottom, 2 HORZ: 1 VERT. Side slopes, minimum slope equal to .003 ft/ft and a depth requirement equal to 1.7 feet. The procedures used were the same as the other ditches. The velocity is 4.3 feet per second.

4.0 Erosion Protection

Attached is a table showing the maximum permissible channel velocities taken from the Army Corp. of Engineers "Hydraulic Design of Flood Control Channels". The mean channel velocity for clay, where riprap would be required, is 6 feet per second. All of the above ditches have velocities below 5 feet per second so the channels were not riprapped. Also equipment will be available to provide maintenance at the ditches during operation. The ditches are not required once reclamation is complete.

velocity or shear that will erode the channel. The adoption of maximum permissible velocities that are used in the design of channels has been widely accepted since publication of a table of values by Fortier and Scobey.[51] The latest information on critical scour velocities is given in reference 50. The tabulation below gives a set of permissible velocities that can be used as a guide to design nonscouring flood control channels. Lane[46] presents curves showing permissible channel shear stress to be used for design, and

Suggested Maximum Permissible Mean Channel Velocities†

Channel Material	Mean Channel Velocity, fps
Fine sand	2.0
Coarse sand	4.0
Fine gravel††	6.0
Earth	
Sandy silt	2.0
Silt clay	3.5
Clay	6.0
Grass-lined earth (slopes less than 5%)‡	
Bermuda grass - sandy silt	6.0
- silt clay	8.0
Kentucky Blue Grass - sandy silt	5.0
- silt clay	7.0
Poor rock (usually sedimentary)	10.0
Soft sandstone	8.0
Soft shale	3.5
Good rock (usually igneous or hard metamorphic)	20.0

† Based on TM 5-886-4 and CE Hydraulic Design Conferences of 1958-1960.

†† For particles larger than fine gravel (about 20 mm = 3/4 in.), see plate 29.

‡ Keep velocities less than 5.0 fps unless good cover and proper maintenance can be obtained.

DITCH 1

WHITE MESA
DRAINAGE ABOVE DITCH 1

TRAPEZOID CHANNEL

	ENTER	OUTPUT	
BOTTOM WIDTH (B)-FT.	10	AREA (FT^2)	86.905
SIDE SLOPE (?:1)-(H:V)	2	HYD RADIUS -(FT.)	2.86
FLOW DEPTH (Y)-FT.	4.55	VELOCITY-(FPS)	4.42
ROUGHNESS COEF.(N)	0.0225	FLOW -(CFS)	384.01
CHANNEL GRADIENT (S)-FT/FT	0.0011	FROUDE #	0.21
		FLOW TYPE	SUBCRITICAL

DRAINAGE ABOVE DITCH I

BASIN AREA = .0359 Sq. Mi

L = 2800'

1hr PMP = 8.25"

Peak Q @ 382.4cfs @ Time = .37hrs

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******************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY******************

JOB TR-20
TITLE 001 WHITE MESA HYDROLOGY STUDY                                              10
TITLE     DRAINAGE ABOVE DITCH 1                                                  20
 5 RAINFL 1      1                0.1667                                          30
 8                      0.0        5.12       6.77       7.34       7.67         40
 8                     7.98        8.25       8.25       8.25       8.25
 9 ENDTBL
 6 RUNOFF 1   1         1         0.0359      77.        0.371 1    1 1 1       70
 6 REACH  3 002    1    2         2800.       0.224      1.501 1    1 1 1       80
   ENDATA                                                                         90
 7 INCREM 6                        0.01                                          100
 7 COMPUT 7 001      002           0.0         1.0        1.01 2      01         110
   ENDCMP 1                                                                      120
   ENDJOB 2                                                                      130

*******************************END OF 80-80 LIST*******************************
```

- Time Increment for Rainfall

Rainfall depths @ 10min increments

-A(sq mi)

-CN

tc

=x

Reach Length

C=m

Q=xA^m

- Time increment for hydrograph

Program is not sensitive to x & m
but you have to use a value to run
tried x = .4 & m = 1.33 w/same results

Channel Parameters

Q = 382 cfs

width = 10'

2:1 slopes

mannings n = .0225

Slope = .0011 ft/ft

∴ d = 4.55'

```
FILE NO.  1

                    COMPUTER PROGRAM FOR PROJECT FORMULATION - HYDROLOGY   USER NOTES

        THE USERS MANUAL FOR THIS PROGRAM IS THE MAY 1982 DRAFT OF TR-20.  CHANGES FROM THE 2/14/74 VERSION INCLUDE:

        REACH ROUTING - THE MODIFIED ATT-KIN ROUTING PROCEDURE REPLACES THE CONVEX METHOD.  INPUT DATA PREPARED FOR
           PREVIOUS PROGRAM VERSIONS USING CONVEX ROUTING COEFFICIENTS WILL NOT RUN ON THIS VERSION.

           THE PREFERRED TYPE OF DATA ENTRY IS CROSS SECTION DATA REPRESENTATIVE OF A REACH.  IT IS RECOMMENDED THAT
           THE OPTIONAL CROSS SECTION DISCHARGE-AREA PLOTS BE OBTAINED WHENEVER NEW CROSS SECTION DATA IS ENTERED.
           THE PLOTS SHOULD BE CHECKED FOR REASONABLENESS AND ADEQUACY OF INPUT DATA FOR THE COMPUTATION OF "M"
           VALUES USED IN THE ROUTING PROCEDURE.

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           MANUAL.  SUMMARY TABLE 2 DISPLAYS REACH ROUTING RESULTS AND ROUTING PARAMETERS FOR COMPARISON AND CHECKING.

        HYDROGRAPH GENERATION - THE PROCEDURE TO CALCULATE THE INTERNAL TIME INCREMENT AND PEAK TIME OF THE UNIT
           HYDROGRAPH HAVE BEEN IMPROVED.  PEAK DISCHARGES AND TIMES MAY DIFFER FROM THE PREVIOUS VERSION.  OUTPUT
           HYDROGRAPHS ARE STILL INTERPOLATED, PRINTED, AND ROUTED AT THE USER SELECTED MAIN TIME INCREMENT.

        INTERMEDIATE PEAKS - METHOD ADDED TO PROVIDE DISCHARGES AT INTERMEDIATE POINTS WITHIN REACHES WITHOUT ROUTING.

        OTHER - THIS VERSION CONTAINS SOME ADDITIONS TO THE INPUT AND NUMEROUS MODIFICATIONS TO THE OUTPUT.  USER
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     PROGRAM CHANGES SINCE MAY 1982:

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                   CORRECT REACH ROUTING PEAK TRAVEL TIME PRINTED WITH FULLPRINT OPTION
         5/02/83 - CORRECT COMPUTATIONS FOR ---
                           1. DIVISION OF BASEFLOW IN DIVERT OPERATION
                           2. HYDROGRAPH VOLUME SPLIT BETWEEN BASEFLOW AND ABOVE BASEFLOW
                           3. CROSS SECTION DATA PLOTTING POSITION
                           4. INTERMEDIATE PEAK WHEN "FROM" AREA IS LARGER THAN "THRU" AREA
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                           6. ORDERING "FLOW-FREQ" FILE FROM SUMMARY TABLE #3 DATA
                           7. BASEFLOW ENTERED WITH READHYD
                           8. LOW FLOW SPLIT DURING DIVERT PROCEDURE #2 WHEN SECTION RATINGS START AT DIFFERENT ELEVATIONS
                   ENHANCEMENTS ---
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                   CORRECT COMBINATION OF RATING TABLES FOR DIVERT
                   CHECK REACH ROUTING PARAMETERS FOR ACCEPTABLE LIMITS
                   ELIMINATE MINIMUM REACH TRAVEL TIME WHEN ATT-KIN COEFFICIENT EQUALS ONE
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EXECUTIVE CONTROL OPERATION INCREM MAIN TIME INCREMENT = 0.01 HOURS RECORD ID

EXECUTIVE CONTROL OPERATION COMPUT FROM XSECTION 1 TO XSECTION 2 RECORD ID 110
STARTING TIME = 0.00 RAIN DEPTH = 1.00 RAIN DURATION= 1.00 RAIN TABLE NO.= 1 ANT. MOIST. COND= 2
ALTERNATE NO.= 0 STORM NO.= 1 MAIN TIME INCREMENT = 0.01 HOURS

OPERATION RUNOFF CROSS SECTION 1

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
0.37	382.40	(RUNOFF)

FIRST HYDROGRAPH POINT = 0.00 HOURS TIME INCREMENT = 0.01 HOURS DRAINAGE AREA = 0.04 SQ.MI.

TIME(HRS)											
0.00	DISCHG	0.00	0.26	0.79	1.77	3.36	5.57	8.57	12.37	17.11	22.89
0.10	DISCHG	29.78	38.06	47.77	59.04	71.96	86.50	102.44	119.68	137.86	156.79
0.20	DISCHG	176.23	195.78	215.33	234.58	253.35	271.49	288.82	305.17	320.18	333.67
0.30	DISCHG	345.53	355.65	364.09	370.91	376.13	379.79	381.85	382.39	381.43	379.01
0.40	DISCHG	375.34	370.55	364.76	358.14	350.80	342.89	334.50	325.67	316.45	306.93
0.50	DISCHG	297.20	287.35	277.51	267.75	258.17	248.87	239.88	231.17	222.75	214.57
0.60	DISCHG	206.61	198.87	191.36	184.07	176.99	170.13	163.50	157.05	150.81	144.79
0.70	DISCHG	139.03	133.55	128.37	123.49	118.86	114.50	110.38	106.45	102.72	99.17
0.80	DISCHG	95.82	92.64	89.62	86.77	84.06	81.50	79.08	76.80	74.64	72.62
0.90	DISCHG	70.74	68.97	67.31	65.73	64.23	62.81	61.47	60.19	58.97	57.80
1.00	DISCHG	56.69	55.61	54.56	53.53	52.53	51.54	50.56	49.57	48.58	47.57
1.10	DISCHG	46.54	45.47	44.36	43.19	41.96	40.68	39.34	37.97	36.55	35.10
1.20	DISCHG	33.63	32.14	30.65	29.16	27.69	26.23	24.81	23.41	22.05	20.74
1.30	DISCHG	19.47	18.25	17.07	15.95	14.89	13.87	12.92	12.02	11.19	10.43
1.40	DISCHG	9.73	9.08	8.48	7.92	7.39	6.90	6.45	6.02	5.62	5.25
1.50	DISCHG	4.90	4.58	4.27	3.99	3.72	3.47	3.24	3.03	2.83	2.64
1.60	DISCHG	2.47	2.30	2.15	2.00	1.87	1.74	1.63	1.52	1.41	1.32
1.70	DISCHG	1.23	1.15	1.07	1.00	0.93	0.87	0.81	0.75	0.70	0.65
1.80	DISCHG	0.61	0.57	0.53	0.49	0.45	0.42	0.39	0.36	0.34	0.31
1.90	DISCHG	0.29	0.27	0.24	0.23	0.21	0.19	0.18	0.16	0.15	0.13
2.00	DISCHG	0.12	0.11	0.10	0.09	0.08	0.07	0.07	0.06	0.05	0.04
2.10	DISCHG	0.04	0.03	0.03	0.02	0.02	0.01	0.01	0.01	0.01	0.00

RUNOFF VOLUME ABOVE BASEFLOW = 8.25 WATERSHED INCHES, 191.11 CFS-HRS, 15.79 ACRE-FEET; BASEFLOW = 0.00 CFS

--- HYDROGRAPH FOR XSECTION 1, ALTERNATE 0, STORM 1, ADDED TO OUTPUT HYDROGRAPH FILE ---

OPERATION REACH CROSS SECTION 2

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
0.57	275.92	(NULL)

TIME(HRS) FIRST HYDROGRAPH POINT = 0.00 HOURS TIME INCREMENT = 0.01 HOURS DRAINAGE AREA = 0.04 SQ.MI.

0.00	DISCHG	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.05	0.14	0.30
0.10	DISCHG	0.56	0.97	1.54	2.32	3.36	4.69	6.37	8.45	11.00	14.06
0.20	DISCHG	17.71	21.97	26.89	32.47	38.73	45.65	53.21	61.37	70.08	79.31
0.30	DISCHG	88.98	99.04	109.41	120.02	130.77	141.58	152.35	163.01	173.47	183.67
0.40	DISCHG	193.54	203.02	212.04	220.57	228.54	235.93	242.71	248.85	254.35	259.20
0.50	DISCHG	263.42	266.99	269.95	272.29	274.03	275.20	275.81	275.89	275.48	274.61
0.60	DISCHG	273.31	271.63	269.59	267.24	264.58	261.67	258.51	255.12	251.55	247.79
0.70	DISCHG	243.89	239.84	235.67	231.40	227.04	222.61	218.13	213.61	209.07	204.53
0.80	DISCHG	200.00	195.49	191.01	186.56	182.16	177.82	173.53	169.31	165.15	161.07
0.90	DISCHG	157.06	153.14	149.30	145.54	141.87	138.29	134.80	131.40	128.10	124.88
1.00	DISCHG	121.76	118.72	115.78	112.92	110.14	107.45	104.84	102.31	99.85	97.47
1.10	DISCHG	95.16	92.92	90.73	88.61	86.54	84.53	82.57	80.64	78.75	76.90
1.20	DISCHG	75.08	73.28	71.50	69.74	68.00	66.27	64.55	62.84	61.15	59.46
1.30	DISCHG	57.79	56.13	54.48	52.85	51.23	49.63	48.05	46.49	44.96	43.44
1.40	DISCHG	41.95	40.49	39.06	37.65	36.28	34.95	33.64	32.38	31.15	29.95
1.50	DISCHG	28.79	27.66	26.57	25.52	24.50	23.51	22.56	21.64	20.75	19.89
1.60	DISCHG	19.06	18.27	17.50	16.76	16.05	15.36	14.71	14.07	13.47	12.88
1.70	DISCHG	12.32	11.78	11.26	10.77	10.29	9.84	9.40	8.98	8.57	8.19
1.80	DISCHG	7.82	7.47	7.13	6.81	6.50	6.20	5.91	5.64	5.38	5.13
1.90	DISCHG	4.90	4.67	4.45	4.24	4.05	3.86	3.67	3.50	3.34	3.18
2.00	DISCHG	3.03	2.88	2.75	2.62	2.49	2.37	2.26	2.15	2.04	1.94
2.10	DISCHG	1.85	1.76	1.67	1.59	1.51	1.44	1.37	1.30	1.23	1.17
2.20	DISCHG	1.11	1.06	1.00	0.95	0.90	0.86	0.81	0.77	0.73	0.70
2.30	DISCHG	0.66	0.63	0.60	0.56	0.54	0.51	0.48	0.46	0.43	0.41
2.40	DISCHG	0.39	0.37	0.35	0.33	0.32	0.30	0.28	0.27	0.26	0.24
2.50	DISCHG	0.23	0.22	0.21	0.20	0.19	0.18	0.17	0.16	0.15	0.14
2.60	DISCHG	0.13	0.13	0.12	0.11	0.11	0.10	0.09	0.09	0.09	0.08
2.70	DISCHG	0.08	0.07	0.07	0.06	0.06	0.06	0.05	0.05	0.05	0.04
2.80	DISCHG	0.04	0.04	0.04	0.03	0.03	0.03	0.03	0.03	0.02	0.02
2.90	DISCHG	0.02	0.02	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01

RUNOFF VOLUME ABOVE BASEFLOW = 8.25 WATERSHED INCHES, 191.08 CFS-HRS, 15.79 ACRE-FEET; BASEFLOW = 0.00 CFS

--- HYDROGRAPH FOR XSECTION 2, ALTERNATE 0, STORM 1, ADDED TO OUTPUT HYDROGRAPH FILE ---

EXECUTIVE CONTROL OPERATION ENDCMP COMPUTATIONS COMPLETED FOR PASS 1 RECORD ID 120

EXECUTIVE CONTROL OPERATION ENDJOB RECORD ID 130

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID		STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION			RUNOFF AMOUNT (IN)	PEAK DISCHARGE			
							BEGIN (HR)	AMOUNT (IN)	DURATION (HR)		ELEVATION (FT)	TIME (HR)	RATE (CFS)	RATE (CSM)
ALTERNATE 0 STORM 1														
XSECTION	1	RUNOFF	0.04	1	2	0.01	0.0	8.25	1.00	8.25	---	0.37	382.40	10651.9
XSECTION	2	REACH	0.04	1	2	0.01	0.0	8.25	1.00	8.25	---	0.57	275.92	7685.8

SUMMARY TABLE 2 - SELECTED MODIFIED ATT-KIN REACH ROUTINGS IN ORDER OF STANDARD EXECUTIVE CONTROL INSTRUCTIONS
(A STAR(*) AFTER VOLUME ABOVE BASE(IN) INDICATES A HYDROGRAPH TRUNCATED AT A VALUE EXCEEDING BASE + 10% OF PEAK
A QUESTION MARK(?) AFTER COEFF.(C) INDICATES PARAMETERS OUTSIDE ACCEPTABLE LIMITS, SEE PREVIOUS WARNINGS)

XSEC ID	REACH LENGTH (FT)	HYDROGRAPH INFORMATION: INFLOW PEAK (CFS)	INFLOW TIME (HR)	OUTFLOW PEAK (CFS)	OUTFLOW TIME (HR)	OUTFLOW+ INTERV.AREA PEAK (CFS)	OUTFLOW+ INTERV.AREA TIME (HR)	BASE-FLOW (CFS)	ROUTING PARAMETERS: VOLUME ABOVE BASE (IN)	MAIN TIME INCR (HR)	ITER-ATION #	Q AND A EQUATION COEFF (X)	Q AND A EQUATION POWER (M)	LENGTH FACTOR (K*)	PEAK RATIO O/I (Q*)	S/Q @PEAK (K) (SEC)	ATT-KIN COEFF (C)	PEAK TRAVEL TIME STOR-AGE (HR)	PEAK TRAVEL TIME KINE-MATIC (HR)
ALTERNATE		0	STORM	1															
2	2800	382	0.4	276	0.6	---	---	0	8.25	0.01	1	0.224	1.50	0.443	0.721	697	0.05	0.16	0.20

SUMMARY TABLE 3 - DISCHARGE (CFS) AT XSECTIONS AND STRUCTURES FOR ALL STORMS AND ALTERNATES

XSECTION/ STRUCTURE ID		DRAINAGE AREA (SQ MI)	STORM NUMBERS.......... 1
XSECTION	1	0.04	
ALTERNATE	0		382.40
XSECTION	2	0.04	
ALTERNATE	0		275.92

DITCH 2

WHITE MESA
DITCH 2 W\INVERT @ EL 5644

TRAPEZOID CHANNEL

	ENTER	OUTPUT	
BOTTOM WIDTH (B)-FT.	10	AREA (FT^2)	72
SIDE SLOPE (?:1)-(H:V)	2	HYD RADIUS -(FT.)	2.58
FLOW DEPTH (Y)-FT.	4	VELOCITY-(FPS)	3.93
ROUGHNESS COEF.(N)	0.0225	FLOW -(CFS)	283.08
CHANNEL GRADIENT (S)-FT/FT	0.001	FROUDE #	0.21
		FLOW TYPE	SUBCRITICAL

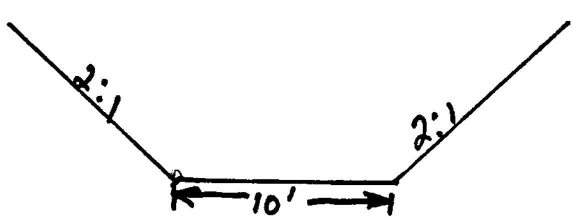
TR-20 S/N: 32001654

DATE: 12/18/1989 TIME: 09:51:35.70

DATA FILE: d:\haestad\wmdit2.t20

DITCH 2

This run represents lowering Ditch 2 until the invert is at 5644.0 but maintaining a channel width of 10', 2:1 side slopes and a channel slope equal to .001 ft/ft. Page 5 of the summary sheet shows the maximum flood stage elevation of 5647.99. Currently the top of the dike is at 5647.9 so at least another .10 ft. of fill has to be placed to contain the flood. We probably need another 0.5 ft of fill for freeboard so the dike should be raised to 5648.5.



Typ. X-Section

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*****************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY*****************

JOB TR-20                                                                       
TITLE 001 WHITE MESA HYDROLOGY STUDY                                            10
TITLE     DRAINAGE ABOVE DITCH 2 USING RETENTION POND                           20
 5 RAINFL 1      1             0.1667                                           30
 8                      0.0      5.12         6.77         7.34         7.67    40   } Rainfall depths at 10min increments
 8                     7.98      8.25         8.25         8.25         8.25    
 9 ENDTBL                                                                       70
 6 RUNOFF 1  0101      1       0.0750          77.         0.161 1   1 1 1      80
 3 STRUCT     01                El.           Q(cfs)       Storage(AF)
 8                             5644.          0.0          0.0                     }
 8                             5645.8         62.7         5.0                     } Stage/storage/discharge curve
 8                             5646.0         75.90        5.2                     }
 8                             5647.9         269.26       20.0                    }
 9 ENDTBL
 6 REACH  3   01    1    2      1400. =L      0.224 =x     1.50 =m 1   1 1 1     90
 6 RESVOR 2    01 1      2     [5644.0         Q=xA^m        1 1 1 1 1 1
   ENDATA                      [starting elev.                                  100
 7 INCREM 6                      0.01 - Time Incr. for hydrograph
 7 COMPUT 7   01      01         0.0          1.0          1.01 2       01      110
   ENDCMP 1                                                                     120
   ENDJOB 2                                                                     130

*****************************END OF 80-80 LIST*******************************
```

FILE NO. 1

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CORRECT REACH ROUTING PEAK TRAVEL TIME PRINTED WITH FULLPRINT OPTION
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2. HYDROGRAPH VOLUME SPLIT BETWEEN BASEFLOW AND ABOVE BASEFLOW
3. CROSS SECTION DATA PLOTTING POSITION
4. INTERMEDIATE PEAK WHEN "FROM" AREA IS LARGER THAN "THRU" AREA
5. STORAGE ROUTED REACH TRAVEL TIME FOR MULTIPEAK HYDROGRAPH
6. ORDERING "FLOW-FREQ" FILE FROM SUMMARY TABLE #3 DATA
7. BASEFLOW ENTERED WITH READHYD
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ENHANCEMENTS ---
1. REPLACE USER MANUAL ERROR CODES (PAGE 4-9 TO 4-11) WITH MESSAGES
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CHECK REACH ROUTING PARAMETERS FOR ACCEPTABLE LIMITS
ELIMINATE MINIMUM REACH TRAVEL TIME WHEN ATT-KIN COEFFICIENT EQUALS ONE

TR20 XEQ 12/18/1989 WHITE MESA HYDROLOGY STUDY 20 JOB 1 PASS 1
REV 09/01/83 DRAINAGE ABOVE DITCH 2 USING RETENTION POND 30 PAGE 2

EXECUTIVE CONTROL OPERATION INCREM MAIN TIME INCREMENT - 0.01 HOURS RECORD ID

EXECUTIVE CONTROL OPERATION COMPUT FROM XSECTION 1 TO STRUCTURE 1 RECORD ID 110
STARTING TIME - 0.00 RAIN DEPTH - 1.00 RAIN DURATION- 1.00 RAIN TABLE NO.- 1 ANT. MOIST. COND- 2
ALTERNATE NO.- 0 STORM NO.- 1 MAIN TIME INCREMENT - 0.01 HOURS

OPERATION RUNOFF CROSS SECTION 1

OPERATION RUNOFF STRUCTURE 1

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
0.21	1247.50	(RUNOFF)

TIME(HRS) FIRST HYDROGRAPH POINT - 0.00 HOURS TIME INCREMENT - 0.01 HOURS DRAINAGE AREA - 0.08 SQ.MI.

0.00	DISCHG	0.00	3.04	12.64	30.88	60.34	104.34	165.50	243.40	334.45	434.10
0.10	DISCHG	537.85	641.93	743.49	839.18	927.85	1008.35	1079.03	1138.04	1184.36	1218.26
0.20	DISCHG	1239.65	1247.49	1240.66	1218.73	1182.98	1136.36	1082.41	1024.59	965.11	906.78
0.30	DISCHG	851.30	799.66	752.81	712.37	678.16	648.38	621.13	595.21	569.38	542.43
0.40	DISCHG	513.96	484.35	454.26	424.33	395.20	367.44	341.57	317.84	296.41	277.48
0.50	DISCHG	261.15	247.19	235.01	224.05	213.97	204.44	195.13	185.95	176.96	168.23
0.60	DISCHG	159.89	152.01	144.65	137.92	131.83	126.44	121.76	117.80	114.48	111.68
0.70	DISCHG	109.28	107.16	105.26	103.53	101.94	100.49	99.16	97.96	96.87	95.88
0.80	DISCHG	95.00	94.23	93.56	92.98	92.49	92.04	91.60	91.14	90.61	89.99
0.90	DISCHG	89.25	88.43	87.55	86.64	85.74	84.85	84.02	83.24	82.54	81.92
1.00	DISCHG	81.37	80.76	79.88	78.61	76.79	74.25	70.84	66.60	61.68	56.33
1.10	DISCHG	50.77	45.20	39.77	34.65	29.91	25.61	21.83	18.63	15.95	13.68
1.20	DISCHG	11.74	10.08	8.64	7.40	6.33	5.42	4.63	3.96	3.38	2.88
1.30	DISCHG	2.46	2.09	1.78	1.52	1.29	1.09	0.92	0.78	0.65	0.55
1.40	DISCHG	0.46	0.38	0.31	0.25	0.20	0.16	0.12	0.09	0.06	0.04
1.50	DISCHG	0.02	0.01	0.00							

RUNOFF VOLUME ABOVE BASEFLOW - 8.25 WATERSHED INCHES, 399.24 CFS-HRS, 32.99 ACRE-FEET; BASEFLOW - 0.00 CFS

--- HYDROGRAPH FOR XSECTION 1, ALTERNATE 0, STORM 1, ADDED TO OUTPUT HYDROGRAPH FILE ---

--- HYDROGRAPH FOR STRUCTURE 1, ALTERNATE 0, STORM 1, ADDED TO OUTPUT HYDROGRAPH FILE ---

OPERATION REACH CROSS SECTION 1

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
0.28	1059.70	(NULL)

IME(HRS) FIRST HYDROGRAPH POINT - 0.00 HOURS TIME INCREMENT - 0.01 HOURS DRAINAGE AREA - 0.08 SQ.MI.

0.00	DISCHG	0.00	0.00	0.00	0.43	2.17	6.25	13.95	26.80	46.53	74.54
0.10	DISCHG	111.51	157.40	211.52	272.75	339.71	410.76	484.32	558.86	632.86	704.72
0.20	DISCHG	772.95	836.30	893.68	944.01	986.21	1019.28	1042.57	1055.91	1059.68	1054.69
0.30	DISCHG	1041.95	1022.72	998.33	970.07	939.16	906.90	874.36	842.21	810.76	780.10
0.40	DISCHG	750.12	720.58	691.18	661.76	632.24	602.66	573.15	543.89	515.11	487.04

0.50	DISCHG	459.92	433.97	409.39	386.31	364.79	344.77	326.16	308.84	292.67	277.49
0.60	DISCHG	263.18	249.68	236.90	224.83	213.42	202.68	192.60	183.19	174.45	166.39
0.70	DISCHG	159.01	152.27	146.16	140.61	135.58	131.02	126.88	123.13	119.72	116.62
0.80	DISCHG	113.81	111.26	108.95	106.85	104.96	103.26	101.72	100.35	99.10	97.97
0.90	DISCHG	96.92	95.93	94.98	94.05	93.13	92.20	91.28	90.37	89.46	88.58
1.00	DISCHG	87.72	86.89	86.11	85.35	84.57	83.72	82.73	81.53	80.01	78.10
1.10	DISCHG	75.76	73.00	69.83	66.33	62.55	58.58	54.50	50.39	46.33	42.39
1.20	DISCHG	38.63	35.08	31.76	28.67	25.82	23.20	20.80	18.61	16.62	14.82
1.30	DISCHG	13.19	11.72	10.41	9.22	8.16	7.22	6.37	5.62	4.95	4.36
1.40	DISCHG	3.83	3.36	2.95	2.58	2.26	1.97	1.72	1.50	1.30	1.13
1.50	DISCHG	0.98	0.84	0.73	0.62	0.54	0.46	0.39	0.34	0.29	0.25
1.60	DISCHG	0.21	0.18	0.15	0.13	0.11	0.10	0.08	0.07	0.06	0.05
1.70	DISCHG	0.04	0.04	0.03	0.03	0.02	0.02	0.01	0.01	0.01	0.01
1.80	DISCHG	0.01	0.00								

RUNOFF VOLUME ABOVE BASEFLOW = 8.25 WATERSHED INCHES, 399.23 CFS-HRS, 32.99 ACRE-FEET; BASEFLOW = 0.00 CFS

--- HYDROGRAPH FOR XSECTION 1, ALTERNATE 0, STORM 1, ADDED TO OUTPUT HYDROGRAPH FILE ---

OPERATION RESVOR STRUCTURE 1

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
0.49	278.37	5647.99

TIME(HRS) FIRST HYDROGRAPH POINT = 0.00 HOURS TIME INCREMENT = 0.01 HOURS DRAINAGE AREA = 0.08 SQ.MI.

0.00	DISCHG	0.00	0.02	0.10	0.32	0.79	1.63	3.00	5.08	8.01	11.88
0.00	ELEV	5644.00	5644.00	5644.00	5644.01	5644.02	5644.05	5644.09	5644.15	5644.23	5644.34
0.10	DISCHG	16.77	22.68	29.59	37.44	46.17	55.67	76.52	87.60	99.13	110.97
0.10	ELEV	5644.48	5644.65	5644.85	5645.07	5645.33	5645.60	5646.01	5646.12	5646.23	5646.34
0.20	DISCHG	122.98	135.01	146.92	158.55	169.74	180.38	190.35	199.62	208.16	215.98
0.20	ELEV	5646.46	5646.58	5646.70	5646.81	5646.92	5647.03	5647.12	5647.22	5647.30	5647.38
0.30	DISCHG	223.10	229.57	235.44	240.78	245.66	250.14	254.27	258.08	261.56	264.72
0.30	ELEV	5647.45	5647.51	5647.57	5647.62	5647.67	5647.71	5647.75	5647.79	5647.82	5647.86
0.40	DISCHG	267.55	270.04	272.18	273.97	275.43	276.57	277.40	277.96	278.28	278.37
0.40	ELEV	5647.88	5647.91	5647.93	5647.95	5647.96	5647.97	5647.98	5647.99	5647.99	5647.99
0.50	DISCHG	278.27	278.01	277.62	277.10	276.48	275.76	274.94	274.03	273.04	271.96
0.50	ELEV	5647.99	5647.99	5647.98	5647.98	5647.97	5647.96	5647.96	5647.95	5647.94	5647.93
0.60	DISCHG	270.80	269.57	268.27	266.90	265.48	264.02	262.52	260.98	259.43	257.86
0.60	ELEV	5647.92	5647.90	5647.89	5647.88	5647.86	5647.85	5647.83	5647.82	5647.80	5647.79
0.70	DISCHG	256.27	254.68	253.09	251.49	249.90	248.30	246.70	245.11	243.53	241.95
0.70	ELEV	5647.77	5647.76	5647.74	5647.73	5647.71	5647.69	5647.68	5647.66	5647.65	5647.63
0.80	DISCHG	240.37	238.81	237.25	235.71	234.17	232.65	231.13	229.63	228.14	226.66
0.80	ELEV	5647.62	5647.60	5647.59	5647.57	5647.56	5647.54	5647.53	5647.51	5647.50	5647.48
0.90	DISCHG	225.19	223.73	222.27	220.82	219.37	217.93	216.50	215.07	213.65	212.24
0.90	ELEV	5647.47	5647.45	5647.44	5647.42	5647.41	5647.40	5647.38	5647.37	5647.35	5647.34
1.00	DISCHG	210.84	209.44	208.06	206.67	205.29	203.89	202.48	201.05	199.58	198.07
1.00	ELEV	5647.33	5647.31	5647.30	5647.29	5647.27	5647.26	5647.24	5647.23	5647.22	5647.20
1.10	DISCHG	196.52	194.92	193.28	191.61	189.90	188.15	186.39	184.60	182.81	181.00

1.10	ELEV	5647.19	5647.17	5647.15	5647.14	5647.12	5647.10	5647.09	5647.07	5647.05	5647.03
1.20	DISCHG	179.20	177.39	175.58	173.78	171.99	170.21	168.43	166.67	164.92	163.18
1.20	ELEV	5647.02	5647.00	5646.98	5646.96	5646.94	5646.93	5646.91	5646.89	5646.87	5646.86
1.30	DISCHG	161.46	159.75	158.05	156.37	154.71	153.06	151.43	149.81	148.21	146.63
1.30	ELEV	5646.84	5646.82	5646.81	5646.79	5646.77	5646.76	5646.74	5646.73	5646.71	5646.69
1.40	DISCHG	145.06	143.50	141.97	140.44	138.94	137.45	135.97	134.51	133.07	131.64
1.40	ELEV	5646.68	5646.66	5646.65	5646.63	5646.62	5646.60	5646.59	5646.58	5646.56	5646.55
1.50	DISCHG	130.23	128.83	127.45	126.08	124.72	123.38	122.06	120.75	119.45	118.17
1.50	ELEV	5646.53	5646.52	5646.51	5646.49	5646.48	5646.47	5646.45	5646.44	5646.43	5646.42
1.60	DISCHG	116.90	115.64	114.40	113.17	111.96	110.76	109.57	108.39	107.23	106.07
1.60	ELEV	5646.40	5646.39	5646.38	5646.37	5646.35	5646.34	5646.33	5646.32	5646.31	5646.30
1.70	DISCHG	104.93	103.81	102.69	101.59	100.50	99.42	98.35	97.30	96.25	95.22
1.70	ELEV	5646.29	5646.27	5646.26	5646.25	5646.24	5646.23	5646.22	5646.21	5646.20	5646.19
1.80	DISCHG	94.19	93.18	92.18	91.19	90.21	89.24	88.29	87.34	86.40	85.47
1.80	ELEV	5646.18	5646.17	5646.16	5646.15	5646.14	5646.13	5646.12	5646.11	5646.10	5646.09
1.90	DISCHG	84.55	83.65	82.75	81.86	80.98	80.11	79.25	78.40	77.56	76.72
1.90	ELEV	5646.08	5646.08	5646.07	5646.06	5646.05	5646.04	5646.03	5646.02	5646.02	5646.01
2.00	DISCHG	75.90	71.87	68.05	64.44	62.37	61.73	61.09	60.46	59.84	59.22
2.00	ELEV	5646.00	5645.94	5645.88	5645.83	5645.79	5645.77	5645.75	5645.74	5645.72	5645.70
2.10	DISCHG	58.61	58.01	57.41	56.82	56.23	55.65	55.08	54.51	53.95	53.39
2.10	ELEV	5645.68	5645.67	5645.65	5645.63	5645.61	5645.60	5645.58	5645.56	5645.55	5645.53
2.20	DISCHG	52.84	52.30	51.76	51.23	50.70	50.17	49.66	49.14	48.64	48.14
2.20	ELEV	5645.52	5645.50	5645.49	5645.47	5645.46	5645.44	5645.43	5645.41	5645.40	5645.38
2.30	DISCHG	47.64	47.15	46.66	46.18	45.71	45.23	44.77	44.31	43.85	43.40
2.30	ELEV	5645.37	5645.35	5645.34	5645.33	5645.31	5645.30	5645.29	5645.27	5645.26	5645.25
2.40	DISCHG	42.95	42.51	42.07	41.64	41.21	40.78	40.36	39.94	39.53	39.13
2.40	ELEV	5645.23	5645.22	5645.21	5645.20	5645.18	5645.17	5645.16	5645.15	5645.13	5645.12
2.50	DISCHG	38.72	38.32	37.93	37.54	37.15	36.77	36.39	36.01	35.64	35.27
2.50	ELEV	5645.11	5645.10	5645.09	5645.08	5645.07	5645.06	5645.04	5645.03	5645.02	5645.01
2.60	DISCHG	34.91	34.55	34.19	33.84	33.49	33.15	32.81	32.47	32.13	31.80
2.60	ELEV	5645.00	5644.99	5644.98	5644.97	5644.96	5644.95	5644.94	5644.93	5644.92	5644.91
2.70	DISCHG	31.47	31.15	30.83	30.51	30.20	29.88	29.58	29.27	28.97	28.67
2.70	ELEV	5644.90	5644.89	5644.88	5644.88	5644.87	5644.86	5644.85	5644.84	5644.83	5644.82
2.80	DISCHG	28.37	28.08	27.79	27.51	27.22	26.94	26.66	26.39	26.12	25.85
2.80	ELEV	5644.81	5644.81	5644.80	5644.79	5644.78	5644.77	5644.77	5644.76	5644.75	5644.74
2.90	DISCHG	25.58	25.32	25.06	24.80	24.54	24.29	24.04	23.79	23.55	23.30
2.90	ELEV	5644.73	5644.73	5644.72	5644.71	5644.70	5644.70	5644.69	5644.68	5644.68	5644.67

RUNOFF VOLUME ABOVE BASEFLOW = 7.78 WATERSHED INCHES, 376.74 CFS-HRS, 31.13 ACRE-FEET; BASEFLOW = 0.00 CFS

--- HYDROGRAPH FOR STRUCTURE 1, ALTERNATE 0, STORM 1, ADDED TO OUTPUT HYDROGRAPH FILE ---

EXECUTIVE CONTROL OPERATION ENDCMP COMPUTATIONS COMPLETED FOR PASS 1 RECORD ID 120

XECUTIVE CONTROL OPERATION ENDJOB RECORD ID 130

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID		STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	PRECIPITATION AMOUNT (IN)	PRECIPITATION DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	PEAK DISCHARGE TIME (HR)	PEAK DISCHARGE RATE (CFS)	PEAK DISCHARGE RATE (CSM)
ALTERNATE 0 STORM 1														
XSECTION	1	RUNOFF	0.08	1	2	0.01	0.0	8.25	1.00	8.25	---	0.21	1247.50	16633.3
SECTION	1	REACH	0.08	1	2	0.01	0.0	8.25	1.00	8.25	---	0.28	1059.70	14129.4
TRUCTURE	1	RESVOR	0.08	1	2	0.01	0.0	8.25	1.00	7.78	$\boxed{5647.99}$	0.49	278.37	3711.6

SUMMARY TABLE 2 - SELECTED MODIFIED ATT-KIN REACH ROUTINGS IN ORDER OF STANDARD EXECUTIVE CONTROL INSTRUCTIONS
(A STAR(*) AFTER VOLUME ABOVE BASE(IN) INDICATES A HYDROGRAPH TRUNCATED AT A VALUE EXCEEDING BASE + 10% OF PEAK
A QUESTION MARK(?) AFTER COEFF.(C) INDICATES PARAMETERS OUTSIDE ACCEPTABLE LIMITS. SEE PREVIOUS WARNINGS)

XSEC ID	REACH LENGTH (FT)	HYDROGRAPH INFORMATION: INFLOW PEAK (CFS)	INFLOW TIME (HR)	OUTFLOW PEAK (CFS)	OUTFLOW TIME (HR)	OUTFLOW+ INTERV.AREA PEAK (CFS)	OUTFLOW+ INTERV.AREA TIME (HR)	BASE-FLOW (CFS)	ROUTING PARAMETERS: VOLUME ABOVE BASE (IN)	MAIN TIME INCR (HR)	ITER-ATION #	Q AND A EQUATION COEFF (X)	Q AND A EQUATION POWER (M)	LENGTH FACTOR (K*)	PEAK RATIO O/I (Q*)	S/Q @PEAK (K) (SEC)	ATT-KIN COEFF (C)	PEAK TRAVEL TIME STOR-AGE (HR)	PEAK TRAVEL TIME KINE-MATIC (HR)
ALTERNATE 0 STORM 1																			
1	1400	1247	0.2	1060	0.3	---	---	0	8.25	0.01	1	0.224	1.50	0.169	0.849	235	0.14	0.06	0.07

SUMMARY TABLE 3 - DISCHARGE (CFS) AT XSECTIONS AND STRUCTURES FOR ALL STORMS AND ALTERNATES

XSECTION/ STRUCTURE ID	DRAINAGE AREA (SQ MI)	STORM NUMBERS.......... 1
STRUCTURE 1	0.08	
ALTERNATE 0		278.37
XSECTION 1	0.08	
ALTERNATE 0		1059.70

DITCH 3

	WHITE MESA			
	DRAINAGE ABOVE DITCH 3			
TRAPEZOID CHANNEL				
	ENTER		**OUTPUT**	
BOTTOM WIDTH (B)-FT.	10		**AREA (FT^2)**	**22.78**
SIDE SLOPE (?:1)-(H:V)	2		**HYD RADIUS -(FT.)**	**1.29**
FLOW DEPTH (Y)-FT.	1.7		**VELOCITY-(FPS)**	**4.30**
ROUGHNESS COEF.(N)	0.0225		**FLOW -(CFS)**	**97.87**
CHANNEL GRADIENT (S)-FT/FT	0.003		**FROUDE #**	**0.50**
			FLOW TYPE	**SUBCRITICAL**

TR-20 S/N: 32001654

WHITE MESA

DITCH 3

DATE: 01/05/1990 TIME: 09:12:28.68

DATA FILE: d:\haestad\wmditch3.t20

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******************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY******************

 OB TR-20                                                                      10
TITLE 001 WHITE MESA HYDROLOGY STUDY                                           20
 ITLE     DRAINAGE ABOVE DITCH 3                                               30
 5 RAINFL 1     1            0.1667                                            40
 8                  0.0        6.11        7.34        7.67        7.98
 8                 8.25        8.25        8.25        8.25        8.25
 9 ENDTBL                                                                      70
 6 RUNOFF 1   1       1      0.0041         77.        0.061 1   1 1 1         80
 6 REACH  3 002   1   2        350.         0.4        1.331 1   1 1 1         90
   ENDATA                                                                     100
 7 INCREM 6                    0.01
 7 COMPUT 7 001   002           0.0         1.0         1.01 2      01        110
   ENDCMP 1                                                                   120
   ENDJOB 2                                                                   130

*******************************END OF 80-80 LIST*********************************
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ILE NO. 1

COMPUTER PROGRAM FOR PROJECT FORMULATION - HYDROLOGY USER NOTES

THE USERS MANUAL FOR THIS PROGRAM IS THE MAY 1982 DRAFT OF TR-20. CHANGES FROM THE 2/14/74 VERSION INCLUDE:

REACH ROUTING - THE MODIFIED ATT-KIN ROUTING PROCEDURE REPLACES THE CONVEX METHOD. INPUT DATA PREPARED FOR PREVIOUS PROGRAM VERSIONS USING CONVEX ROUTING COEFFICIENTS WILL NOT RUN ON THIS VERSION.

THE PREFERRED TYPE OF DATA ENTRY IS CROSS SECTION DATA REPRESENTATIVE OF A REACH. IT IS RECOMMENDED THAT THE OPTIONAL CROSS SECTION DISCHARGE-AREA PLOTS BE OBTAINED WHENEVER NEW CROSS SECTION DATA IS ENTERED. THE PLOTS SHOULD BE CHECKED FOR REASONABLENESS AND ADEQUACY OF INPUT DATA FOR THE COMPUTATION OF "M" VALUES USED IN THE ROUTING PROCEDURE.

GUIDELINES FOR DETERMINING OR ANALYZING REACH LENGTHS AND COEFFICIENTS (X,M) ARE AVAILABLE IN THE USERS MANUAL. SUMMARY TABLE 2 DISPLAYS REACH ROUTING RESULTS AND ROUTING PARAMETERS FOR COMPARISON AND CHECKING.

HYDROGRAPH GENERATION - THE PROCEDURE TO CALCULATE THE INTERNAL TIME INCREMENT AND PEAK TIME OF THE UNIT HYDROGRAPH HAVE BEEN IMPROVED. PEAK DISCHARGES AND TIMES MAY DIFFER FROM THE PREVIOUS VERSION. OUTPUT HYDROGRAPHS ARE STILL INTERPOLATED, PRINTED, AND ROUTED AT THE USER SELECTED MAIN TIME INCREMENT.

INTERMEDIATE PEAKS - METHOD ADDED TO PROVIDE DISCHARGES AT INTERMEDIATE POINTS WITHIN REACHES WITHOUT ROUTING.

OTHER - THIS VERSION CONTAINS SOME ADDITIONS TO THE INPUT AND NUMEROUS MODIFICATIONS TO THE OUTPUT. USER OPTIONS HAVE BEEN MODIFIED AND AUGMENTED ON THE JOB RECORD, RAINTABLES ADDED, ERROR AND WARNING MESSAGES EXPANDED, AND THE SUMMARY TABLES COMPLETELY REVISED. THE HOLDOUT OPTION IS NOT OPERATIONAL AT THIS TIME.

PROGRAM QUESTIONS OR PROBLEMS SHOULD BE DIRECTED TO HYDRAULIC ENGINEERS AT THE SCS NATIONAL TECHNICAL CENTERS:

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CHESTER, PA (NORTHEAST)  -- 215-499-3933,      FORT WORTH, TX (SOUTH) -- 334-5242 (FTS)
LINCOLN, NB (MIDWEST)    -- 541-5318 (FTS),    PORTLAND, OR (WEST)    -- 423-4099 (FTS)
OR HYDROLOGY UNIT, ENGINEERING DIVISION, LANHAM, MD -- 436-7383 (FTS).
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PROGRAM CHANGES SINCE MAY 1982:

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12/17/82 - CORRECT PEAK RATE FACTOR FOR USER ENTERED DIMHYD
           CORRECT REACH ROUTING PEAK TRAVEL TIME PRINTED WITH FULLPRINT OPTION
 5/02/83 - CORRECT COMPUTATIONS FOR ---
                1. DIVISION OF BASEFLOW IN DIVERT OPERATION
                2. HYDROGRAPH VOLUME SPLIT BETWEEN BASEFLOW AND ABOVE BASEFLOW
                3. CROSS SECTION DATA PLOTTING POSITION
                4. INTERMEDIATE PEAK WHEN "FROM" AREA IS LARGER THAN "THRU" AREA
                5. STORAGE ROUTED REACH TRAVEL TIME FOR MULTIPEAK HYDROGRAPH
                6. ORDERING "FLOW-FREQ" FILE FROM SUMMARY TABLE #3 DATA
                7. BASEFLOW ENTERED WITH READHYD
                8. LOW FLOW SPLIT DURING DIVERT PROCEDURE #2 WHEN SECTION RATINGS START AT DIFFERENT ELEVATIONS
           ENHANCEMENTS ---
                1. REPLACE USER MANUAL ERROR CODES (PAGE 4-9 TO 4-11) WITH MESSAGES
                2. LABEL OUTPUT HYDROGRAPH FILES WITH CROSS SECTION/STRUCTURE, ALTERNATE AND STORM NO'S
09/01/83 - CORRECT INPUT AND OUTPUT ERRORS FOR INTERMEDIATE PEAKS
           CORRECT COMBINATION OF RATING TABLES FOR DIVERT
```

CHECK REACH ROUTING PARAMETERS FOR ACCEPTABLE LIMITS
ELIMINATE MINIMUM REACH TRAVEL TIME WHEN ATT-KIN COEFFICIENT EQUALS ONE

EXECUTIVE CONTROL OPERATION INCREM   MAIN TIME INCREMENT = 0.01 HOURS   RECORD ID

EXECUTIVE CONTROL OPERATION COMPUT   FROM XSECTION 1 TO XSECTION 2   RECORD ID 110
STARTING TIME = 0.00   RAIN DEPTH = 1.00   RAIN DURATION= 1.00   RAIN TABLE NO.= 1   ANT. MOIST. COND= 2
ALTERNATE NO.= 0   STORM NO.= 1   MAIN TIME INCREMENT = 0.01 HOURS

OPERATION RUNOFF   CROSS SECTION 1

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
0.16	96.62	(RUNOFF)

IME(HRS)   FIRST HYDROGRAPH POINT = 0.00 HOURS   TIME INCREMENT = 0.01 HOURS   DRAINAGE AREA = 0.00 SQ.MI.

0.00	DISCHG	0.00	2.58	10.75	25.68	43.59	59.94	72.50	80.82	86.31	89.85
0.10	DISCHG	92.23	93.84	94.92	95.61	96.08	96.39	96.61	96.17	92.97	84.76
0.20	DISCHG	71.80	57.70	45.50	36.55	30.69	27.00	24.47	22.77	21.62	20.90
0.30	DISCHG	20.41	20.07	19.85	19.68	19.35	18.43	16.57	14.01	11.50	9.42
0.40	DISCHG	7.96	7.06	6.45	6.03	5.75	5.58	5.46	5.38	5.32	5.29
0.50	DISCHG	5.26	5.24	5.20	5.15	5.09	5.04	5.00	4.97	4.95	4.94
0.60	DISCHG	4.93	4.93	4.92	4.92	4.92	4.92	4.92	4.91	4.89	4.82
0.70	DISCHG	4.71	4.60	4.49	4.42	4.38	4.34	4.32	4.31	4.30	4.29
0.80	DISCHG	4.29	4.29	4.28	4.28	4.23	3.96	3.39	2.63	1.87	1.26
0.90	DISCHG	0.83	0.55	0.36	0.24	0.16	0.10	0.07	0.04	0.03	0.02
1.00	DISCHG	0.01	0.00								

RUNOFF VOLUME ABOVE BASEFLOW = 8.24 WATERSHED INCHES,   21.81 CFS-HRS,   1.80 ACRE-FEET;   BASEFLOW = 0.00 CFS

--- HYDROGRAPH FOR XSECTION 1, ALTERNATE 0, STORM 1, ADDED TO OUTPUT HYDROGRAPH FILE ---

OPERATION REACH   CROSS SECTION 2

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
0.21	89.73	(NULL)

IME(HRS)   FIRST HYDROGRAPH POINT = 0.00 HOURS   TIME INCREMENT = 0.01 HOURS   DRAINAGE AREA = 0.00 SQ.MI.

0.00	DISCHG	0.00	0.00	0.00	0.00	0.50	2.47	6.95	14.02	22.88	32.45
0.10	DISCHG	41.78	50.37	57.98	64.59	70.23	75.00	78.97	82.27	84.99	87.23
0.20	DISCHG	88.96	89.73	88.77	85.50	80.13	73.45	66.33	59.46	53.19	47.65
0.30	DISCHG	42.85	38.76	35.31	32.43	30.05	28.08	26.46	25.09	23.80	22.41
0.40	DISCHG	20.78	18.99	17.15	15.37	13.77	12.36	11.14	10.10	9.23	8.50
0.50	DISCHG	7.89	7.40	6.99	6.66	6.38	6.15	5.96	5.79	5.65	5.52
0.60	DISCHG	5.41	5.32	5.25	5.19	5.14	5.09	5.06	5.03	5.01	4.99
0.70	DISCHG	4.98	4.96	4.93	4.89	4.83	4.76	4.70	4.64	4.58	4.53
0.80	DISCHG	4.49	4.45	4.42	4.39	4.37	4.36	4.34	4.32	4.25	4.08
0.90	DISCHG	3.80	3.43	3.01	2.59	2.19	1.84	1.53	1.27	1.04	0.85

1.00	DISCHG	0.70	0.57	0.46	0.37	0.30	0.24	0.19	0.16	0.13	0.10
1.10	DISCHG	0.08	0.06	0.05	0.04	0.03	0.03	0.02	0.02	0.01	0.01
1.20	DISCHG	0.01	0.00								

RUNOFF VOLUME ABOVE BASEFLOW = 8.24 WATERSHED INCHES, 21.81 CFS-HRS, 1.80 ACRE-FEET; BASEFLOW = 0.00 CFS

--- HYDROGRAPH FOR XSECTION 2, ALTERNATE 0, STORM 1, ADDED TO OUTPUT HYDROGRAPH FILE ---

EXECUTIVE CONTROL OPERATION ENDCMP COMPUTATIONS COMPLETED FOR PASS 1 RECORD ID 120

XECUTIVE CONTROL OPERATION ENDJOB RECORD ID 130

JMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	AMOUNT (IN)	DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	TIME (HR)	RATE (CFS)	RATE (CSM)
ALTERNATE 0 STORM 1													
XSECTION 1	RUNOFF	0.00	1	2	0.01	0.0	8.25	0.83	8.24	---	0.16	96.62	23565.2
XSECTION 2	REACH	0.00	1	2	0.01	0.0	8.25	0.83	8.24	---	0.21	89.73	21886.0

UMMARY TABLE 2 - SELECTED MODIFIED ATT-KIN REACH ROUTINGS IN ORDER OF STANDARD EXECUTIVE CONTROL INSTRUCTIONS
(A STAR(*) AFTER VOLUME ABOVE BASE(IN) INDICATES A HYDROGRAPH TRUNCATED AT A VALUE EXCEEDING BASE + 10% OF PEAK
A QUESTION MARK(?) AFTER COEFF.(C) INDICATES PARAMETERS OUTSIDE ACCEPTABLE LIMITS, SEE PREVIOUS WARNINGS)

		HYDROGRAPH INFORMATION							ROUTING PARAMETERS									PEAK	
						OUTFLOW+			VOLUME	MAIN	ITER-	Q AND A			PEAK	S/Q	ATT-	TRAVEL TIME	
SEC	REACH	INFLOW		OUTFLOW		INTERV.AREA		BASE-	ABOVE	TIME	ATION	EQUATION		LENGTH	RATIO	@PEAK	KIN	STOR-	KINE-
ID	LENGTH	PEAK	TIME	PEAK	TIME	PEAK	TIME	FLOW	BASE	INCR	#	COEFF	POWER	FACTOR	O/I	(K)	COEFF	AGE	MATIC
	(FT)	(CFS)	(HR)	(CFS)	(HR)	(CFS)	(HR)	(CFS)	(IN)	(HR)		(X)	(M)	(K*)	(Q*)	(SEC)	(C)	(HR)	(HR)
ALTERNATE 0 STORM 1																			
2	350	97	0.2	90	0.2	---	---	0	8.24	0.01	1	0.400	1.33	0.180	0.929	169	0.19	0.03	0.05

:UMMARY TABLE 3 - DISCHARGE (CFS) AT XSECTIONS AND STRUCTURES FOR ALL STORMS AND ALTERNATES

XSECTION/ TRUCTURE ID		DRAINAGE AREA (SQ MI)	STORM NUMBERS.......... 1
XSECTION	1	0.00	
ALTERNATE	0		96.62
XSECTION	2	0.00	
ALTERNATE	0		89.73

WAVE RUNUP

WESTERN ENGINEERS, INC. SINCE 1952 CONSULTING ENGINEERS / LAND SURVEYORS
2150 Hwy. 6 & 50, Grand Junction, CO 81505-9422 • 303/242-5202 • FAX 242-1672

Frank Webber, PE
Umetco Minerals Corporation
1600 Ute Avenue
Grand Junction, CO 81501

October 27, 1989

RE: WAVE RUNUP - White Mesa Mill Tailings Cells

Dear Frank:

This letter report presents the findings of a wave runup study at the referenced site. Should you have any questions about the results or procedure please give me a call.

AUTHORIZATION AND SCOPE

The wave runup study was authorized by Frank Webber of Umetco Minerals Corp. on Friday, Oct. 20, 1989. The purpose of the study was to determine wave runup potential in the tailings cells at the White Mesa Mill Site in a manner acceptable to the Nuclear Regulatory Commission.

The scope of work included runup calculations for Cells 1I, 3, and 4A. It was requested that several operational scenarios be evaluated for Cell 3. No evaluation was requested for Cell 2.

The required procedure for determining wave runup is presented in the U.S. Army Corp of Engineers Shore Protection Manual (SPM). This two volume manual presents detailed methodology for determining wave characteristics and runup. It was requested that the wave characteristics in the tailings cells be determined using the procedures for wave forecasting in shallow water. During the course of the study it was determined that deep water procedures were applicable and they, therefore, were used in lieu of the shallow water procedures. The rational behind this change is covered in the in the ensuing discussion.

DISCUSSION

Fetch Distance

Fetch distance for each cell was determined as outlined in SPM Chap 3, Sec. V.1. The longest possible fetch distance in each cell was used. For cell 3, three scenarios were developed. The first scenario is the present condition where the east 1/3 of the cell is full of tailings and the western 2/3's is full of water. The second scenario is for the east 1/2 of the cell to be full of tailings and the west 1/2 to be full of water. The third scenario calls for the east 3/4 of the cell to be full of tailings while the west 1/4 is full of water.

Water Depth

Water depths corresponding to the maximum possible were chosen for the cells. This was done because higher waves are generated in deep water than in shallow water, and is was desired to evaluate each cell in the most conservative manner possible.

Shallow Water -vs- Deep Water Methodology

It was requested that the runup evaluation use the shallow water methodology. During analysis it was determined that deep water methods were applicable rather than the shallow water methods. The waves generated on the tailings cells have small amplitudes and periods. The relationship between the water depth in the cells and the wave period showed, in all cases, that the deep water methodology applied. The deep water methodology is the more conservative as wave energy dissipation in the cell floor is not considered.

Wind Speed

A sustained wind speed of 30 mph was used for all runup calculations. This wind speed was used at the direction of the NRC.

RESULTS

The findings are summarized in the table below.

CELL #	WIND SPEED (mph)	DEPTH (feet)	FETCH (feet)	WAVE HEIGHT (feet)	RUNUP (feet)
1I	30	15	2500	0.62	0.90
3-W 2/3	30	15	2610	0.63	0.92
3-W 1/2	30	15	1930	0.55	0.78
3-W 1/4	30	15	1340	0.45	0.62
4A	30	36	1928	0.54	0.77

SENSITIVITY ANALYSIS

Some analyses were done to evaluate the sensitivity of runup to fetch distance and runup to water depth. For a constant wind speed of 30 mph runup increases approximately 0.1 foot for every 500 foot increase in fetch. In deep water (as in this case) runup is independent from water depth. In very shallow water, a small increase in depth results in relatively large increase in runup. As water depth continues to increase runup becomes less sensitive to the depth. Graphs of these analyses are included in the attachments.

Sincerely;
WESTERN ENGINEERS, INC.

John M. Currier, PE

JMC/jmc

attach: 1) calculations
2) fetch distance maps
3) sensitivity analysis graphs
4) Shore Protection Manual excerpts

UMETCO

BLANDING WAVE RUN UP STUDY. 10/24/97

1 of

GIVEN

1) USE Shore PROTECTION MANUAL
Shallow WATER WAVE CHARACTERISTICS
2) 30 mph sustained WIND SPEED.

CELL 1 I

1) FETCH DISTANCE
REF. Shore PROTECTION MANUAL Chap 3 SEC V.1
pages 3-39 thru 3-42

ANGLE	FETCH
0°	2800'
3	2550
3	2750
6	2250
6	2700
9	2100
9	2650
12	1900
12	2650

Avg 2483.3 SAY 2500 FT

2) DEPTH OF WATER IN CELL FOR RUNUP CALCS.
$d = 15'$

12/24/?? [illegible]
2 f

3) WAVE HEIGHT PREDICTION

A. SHALLOW WATER METHODOLOGY.

REF: <u>SHORE PROTECTION MANUAL</u> Chap 3, Sec VII.1.
pages 3-55 thru 3-66

FETCH = 2500 FT
WIND STRESS FACTOR (U_A) = 30 mph
d = 15 ft.

Fig 3-29 pg 3-58

H = 0.65 FT (wave height)
T = 1.34 SEC (wave period)
t = 12 min (neccessary wind duration

NOTE: Waves in a water depth of 15' with wave periods less than 2.4 secs. are considered to be deep water waves.

B. Deep Water METHODOLOGY

REF: SHORE PROTECTION MANUAL Chap 3, Sec V. 3
pgs 3-44 thru 3-51

Fig 3-24 pg 3-50
NA due to limits on FETCH DISTANCE

USE EQNS OUTLINED ON pg. 3-48

H (ft) T (s) U_A (FT/s) F (ft) t (s)

$$H = 2.82 \times 10^{-4}\ U_A F^{1/2}$$

$$T_m = 2.825 \times 10^{-2} (U_A F)^{1/3}$$

$$t = 2.16 \times 10^{1} \left(\frac{F^2}{U_A}\right)^{1/3}$$

$U_A = 44$ ft/sec
$F = 2500$ ft

$H = 0.62$ FT (WAVE HEIGHT)
$T = 1.35$ SEC. (WAVE PERIOD)
$t = 1127$ SEC (18.8 min) (REC. WIND duration

The Deep water methodology answers are insignificantly different than answers from the Shallow water methodology.

4) WAVE RUNUP

REF: SHORE PROTECTION MANUAL: CHAP VII, SEC 2.1
PGS 7-16 THRU 7-39

$d_s = 15$ ft (water depth)
$H_o' = 0.62$ ft (wave height)
$T = 1.35$ SEC (wave period)

$$\frac{d_s}{H_o} = \frac{15 \text{ ft}}{0.62 \text{ ft}} = 24.2$$

THEREFORE USE Fig 7-12, pg. 7-23

WAVE FORECASTING FOR SHALLOW WATER
SHORE PROTECTION MANUAL, CHAP. 3, SEC.VI.1, pg. 3-55

Ua = Wind speed in ft/sec
F = Fetch distance in feet
d = water depth in feet
t = neccessary wind duration to establish waves
d/T^2>= 2.56 then deep water conditions govern

Ua (fps)	depth (feet)	Fetch (feet)	H'a*c Height (feet)	T'a*c T (period) (seconds)	t (minutes)	d/T^2
44	15	2500.00	0.61	1.32	8.21	8.65

WAVE FORECASTING FOR DEEP WATER
SHORE PROTECTION MANUAL, CHAP. 3, SEC.V.3, pg. 3-44

Ua = Wind speed in ft/sec
F = Fetch distance in feet
d = water depth in feet
t = neccessary wind duration to establish waves
d/T^2>= 2.56 then deep water conditions govern

Ua (fps)	depth (feet)	Fetch (feet)	H'a*c Height (feet)	T'a*c T (period (seconds)	t (minutes)
44	15	2500	0.62	1.35	18.78

$$\frac{H_o}{gT^2} = \frac{0.62\ FT}{\left(32.2\frac{ft}{sec^2}\right)(1.35\ sec)^2} = 0.0106$$

EMBANKMENT SLOPE

FROM FIG 7-12 Where:

$\cot \Theta = 3.0,$
$H_o / gT^2 = 0.0106;$

$$\frac{R}{H_o} = 1.30$$

$R = 1.30\ H_o = (1.30)(0.62\ FT) = 0.806\ FT$

USE FIG 7-13 PG 7-24 TO ADJUST FOR SCALE EFFECTS

$\tan \Theta = 0.333$
USE CURVE FOR $H = 1.5' - 4.5'$

runup correction factor, $K = 1.12$

RUNUP, $R = (1.12)(0.806\ FT) = 0.90\ FT.$

CELL 3

TWO RUNUP CALCULATIONS WILL BE DONE FOR CELL 3. ONE FOR THE WEST 2/3 of the cell and one for the west 1/3 of the cell

CELL 3 – WEST 2/3

1) FETCH

Angle	Fetch
0	3000
3	2600
3	2950
6	2300
6	2900
9	2100
9	2880
12	1900
12	2850

Avg: 2608.8 say 2610 FT

2) DEPTH OF WATER IN CELL FOR CALCS

$d = 15'$

3) WAVE HEIGHT PREDICTION

60f

WAVE FORECASTING FOR SHALLOW WATER
SHORE PROTECTION MANUAL, CHAP. 3, SEC.VI.1, pg. 3-55

Ua = Wind speed in ft/sec
F = Fetch distance in feet
d = water depth in feet
t = neccessary wind duration to establish waves
d/T^2>= 2.56 then deep water conditions govern

Ua (fps)	depth (feet)	Fetch (feet)	H'a*c Height (feet)	T'a*c T (period) (seconds)	t (minutes)	d/T^2
44	15	2610.00	0.62	1.33	8.48	8.42

WAVE FORECASTING FOR DEEP WATER
SHORE PROTECTION MANUAL, CHAP. 3, SEC.V.3, pg. 3-44

Ua = Wind speed in ft/sec
F = Fetch distance in feet
d = water depth in feet
t = neccessary wind duration to establish waves
d/T^2>= 2.56 then deep water conditions govern

Ua (fps)	depth (feet)	Fetch (feet)	H'a*c Height (feet)	T'a*c T (period (seconds)	t (minutes)
44	15	2610	0.63	1.37	19.33

4. WAVE RUNUP

$$\frac{d_s}{H_o} = \frac{15}{.63} = 23.8$$

Fig 7-12

$$\frac{H}{gT^2} = \frac{.63\ ft}{32.2\,(1.37\ sec)^2} = 0.0104$$

$$\frac{R}{H_o} = 1.30$$

$$R = (1.30)(.63) = 0.819'$$

Correction Factor 1.12

Runup = 1.12 x 0.819 = 0.917' Say 0.92'

10/24/
80°F
UNITED

CELL 3 W 1/3

1) FETCH

L	FETCH
0	1950
3	1800
3	1900
6	1650
6	1850
9	1550
9	1880
12	1450
12	1775

AVG: 1756.1' SAY 1760 FT

2) DEPTH

15'

3) WAVE HEIGTH PREDICTION

H = 0.52 FT
T = 1.20 SEC
t = 14.9 MIN

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WAVE FORECASTING FOR SHALLOW WATER
SHORE PROTECTION MANUAL, CHAP. 3, SEC.VI.1, pg. 3-55

Ua = Wind speed in ft/sec
F = Fetch distance in feet
d = water depth in feet
t = neccessary wind duration to establish waves
d/T^2>= 2.56 then deep water conditions govern

Ua (fps)	depth (feet)	Fetch (feet)	H'a*c Height (feet)	T'a*c T (period) (seconds)	t (minutes)	d/T^2
44	15	1760.00	0.52	1.18	6.33	10.81

WAVE FORECASTING FOR DEEP WATER
SHORE PROTECTION MANUAL, CHAP. 3, SEC.V.3, pg. 3-44

Ua = Wind speed in ft/sec
F = Fetch distance in feet
d = water depth in feet
t = neccessary wind duration to establish waves
d/T^2>= 2.56 then deep water conditions govern

Ua (fps)	depth (feet)	Fetch (feet)	H'a*c Height (feet)	T'a*c T (period (seconds)	t (minutes)
44	15	1760	0.52	1.20	14.86

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24 OCT 87
10-?

4. Wave Run Up

$$\frac{d_s}{H_o} = \frac{15}{.52} = 28.5$$

$$\frac{H}{gT^2} = \frac{.52}{32.2(1.20)^2} = 0.0112$$

Fig 7-12

$$\frac{R}{H_o} = 1.25$$

$R = (1.25)(0.52) = 0.65$ FT.

Scale Correction Factor Fig 7-13

$K = 1.12$

Runup = $(0.65 \text{ FT})(1.12) = 0.728$ FT say 0.73'

UNITED
24 OCT 91
[illegible]

CELL 4 A.

1) FETCH

∠	F
0	2250
3	2100
3	2050
6	1950
6	1950
9	1800
9	1860
12	1700
12	1700

Avg: 1927.8 FT SAY 1928 FT

2) DEPTH

36'

3) WAVE HEIGHT PREDICTION

H = .54 FT

T = 1.24 SEC

t = 15.8 min

SHORE PROTECTION MANUAL, CHAP. 3, SEC.VI.1, pg. 3-55

Ua = Wind speed in ft/sec
F = Fetch distance in feet
d = water depth in feet
t = neccessary wind duration to establish waves
d/T^2>= 2.56 then deep water conditions govern

Ua (fps)	depth (feet)	Fetch (feet)	H'a*c Height (feet)	T'a*c T (period) (seconds)	t (minutes)	d/T^2
44	36	1928.00	0.54	1.22	6.92	24.03

WAVE FORECASTING FOR DEEP WATER
SHORE PROTECTION MANUAL, CHAP. 3, SEC.V.3, pg. 3-44

Ua = Wind speed in ft/sec
F = Fetch distance in feet
d = water depth in feet
t = neccessary wind duration to establish waves
d/T^2>= 2.56 then deep water conditions govern

Ua (fps)	depth (feet)	Fetch (feet)	H'a*c Height (feet)	T'a*c T (period (seconds)	t (minutes)
44	36	1928	0.54	1.24	15.80

4) WAVE RUNUP

$$\frac{d_s}{H_0} = \frac{36}{.54} = 66.7 \qquad \text{Fig } 7\text{-}12$$

$$\frac{H}{gT^2} = \frac{.54}{32.2(1.24)^2} = 0.0109$$

$$\frac{R}{H_0} = 1.275$$

$$R = (1.275)(.54) = 0.689 \text{ FT.}$$

SCALE CORRECTION Fig 7-13

$$K = 1.12$$

$$\text{RUNUP} = (1.12)(.689 \text{ FT}) = 0.772 \text{ FT}$$

SUMMARY

CELL #	WIND SPEED (mph)	DEPTH (feet)	FETCH (feet)	WAVE HEIGHT (feet)	RUNUP (feet)
1 I	30	15	2500	0.62	0.7[illegible]
3 - W. 2/3	↓	15	2610	0.63	0.72
3 - W 1/3		15	1760	0.52	0.75
3 - W 1/2		15	1930	0.55	0.73
3 - W 1/4		15	1340	0.45	0.62
4A		36	1928	0.54	0.77

UTTETE

10/26/8
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CELL 3 – W 1/2

1) FETCH

L	FETCH
0°	2150
3	1950
3	2100
6	1850
6	2050
9	1675
9	2000
12	1575
12	2000

Avg: 1927.8

Say 1930 FT

2) depth

15'

3) WAVE height

H = 0.55 FT

T = 1.24 sec

t = 15.81 min

CELL 3 – W 1/4

1) FETCH

L	FETCH
0	1475
3	1400
3	1425
6	1325
6	1375
9	1250
9	1325
12	1200
12	1275

Avg = 1338.7

Say 1340

2) depth

15'

3) WAVE height

H = 0.45 FT

T = 1.10 sec

t = 12.39 sec

UMETCO - Blanding, Cell 3 - W 1/2 & W 1/4 W.E.I. 26-Oct-89

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WAVE FORECASTING FOR SHALLOW WATER
SHORE PROTECTION MANUAL, CHAP. 3, SEC.VI.1, pg. 3-55

Ua = Wind speed in ft/sec
F = Fetch distance in feet
d = water depth in feet
t = neccessary wind duration to establish waves
d/T^2>= 2.56 then deep water conditions govern

Ua (fps)	depth (feet)	Fetch (feet)	H'a*c Height (feet)	T'a*c T (period) (seconds)	t (minutes)	d/T^2	
44	15	1930.00	0.54	1.21	6.78	10.19	- W 1/2
44	15	1340.00	0.45	1.08	5.16	12.87	- W 1/4

WAVE FO
SHORE P
WAVE FORECASTING FOR DEEP WATER
SHORE PROTECTION MANUAL, CHAP. 3, SEC.V.3, pg. 3-44

Ua = Wind speed in ft/sec
F = Fetch distance in feet
d = water depth in feet
t = neccessary wind duration to establish waves
d/T^2>= 2.56 then deep water conditions govern

Ua (fps)	depth (feet)	Fetch (feet)	H'a*c Height (feet)	T'a*c T (period (seconds)	t (minutes)	
44	36	1930	0.55	1.24	15.81	- W 1/2
44	36	1340	0.45	1.10	12.39	- W 1/4

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10/26/07
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CELL 3 - W 1/2

4] RUNUP

$$\frac{d_s}{H_o} = \frac{15}{.55} > 3$$

$$\frac{H}{gT^2} = \frac{.55}{(32.2)(1.24)^2} = 0.0111$$

Fig 7-12

$$\frac{R}{H_o} = 1.26$$

$R = 1.26(0.55) = 0.693$

F 7.13 CORRECTION FACTOR

$K = 1.12$

RUNUP = 1.12(.693) = 0.776

SAY 0.78'

CELL 3 - W 1/4

4] RUNUP

$$\frac{d_s}{H_o} = \frac{15}{.45} > 3$$

$$\frac{H}{gT^2} = \frac{.45}{(32.2)(1.10)^2} = 0.0116$$

FROM Fig 7-12

$$\frac{R}{H_o} = 1.225$$

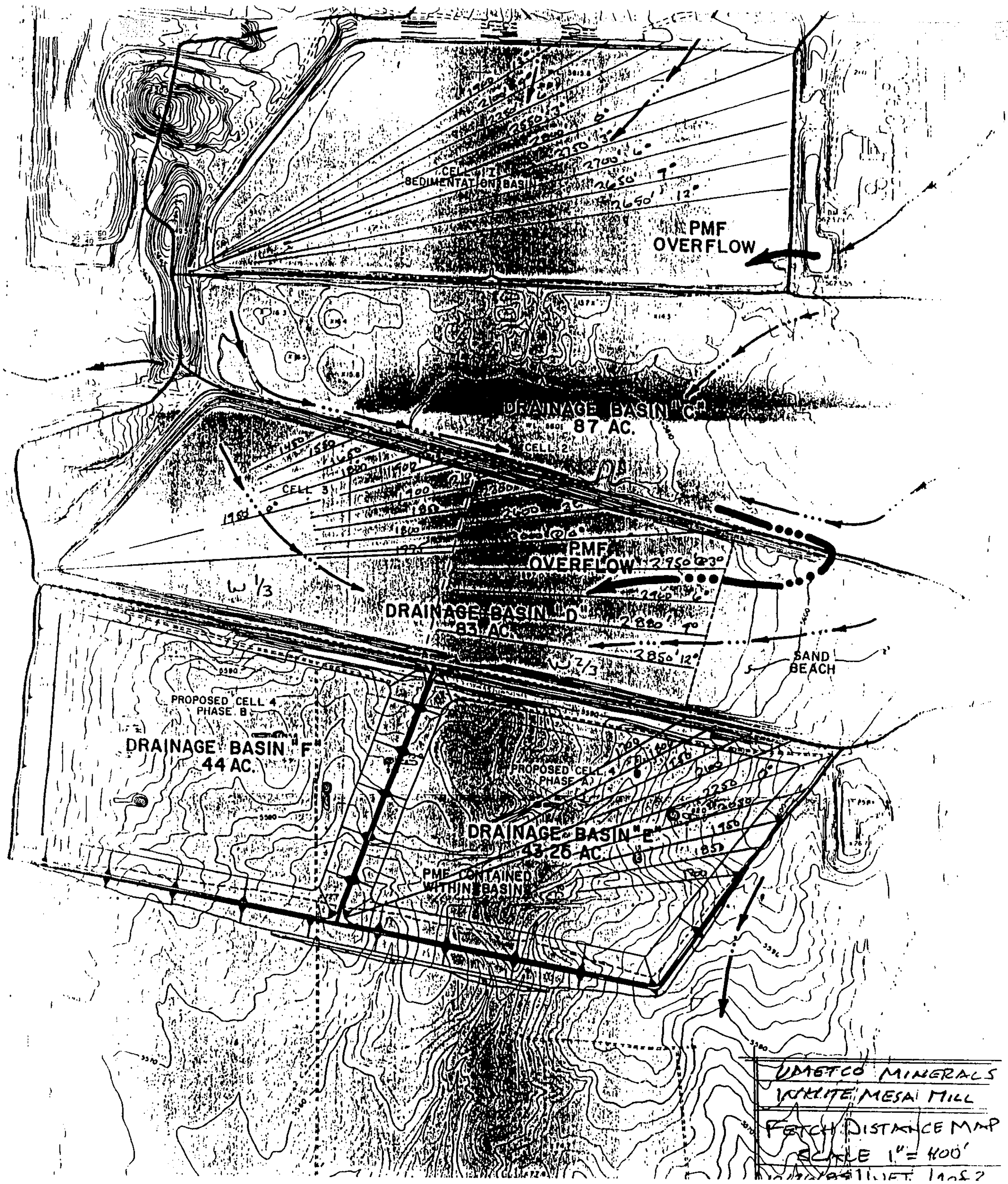
$R = 1.225(.45) = 0.551$

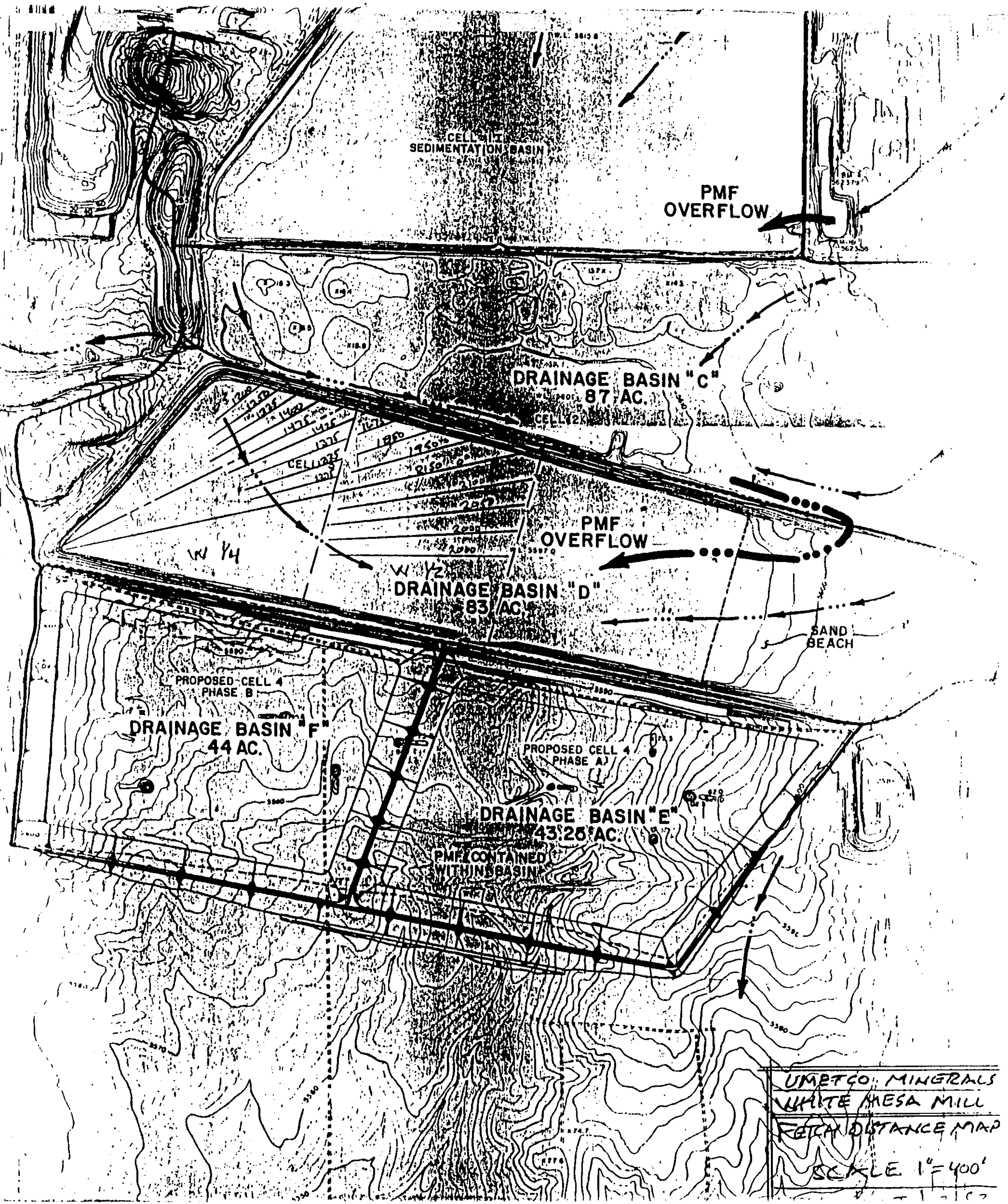
Fig 7.13 CORRECTION FACTOR

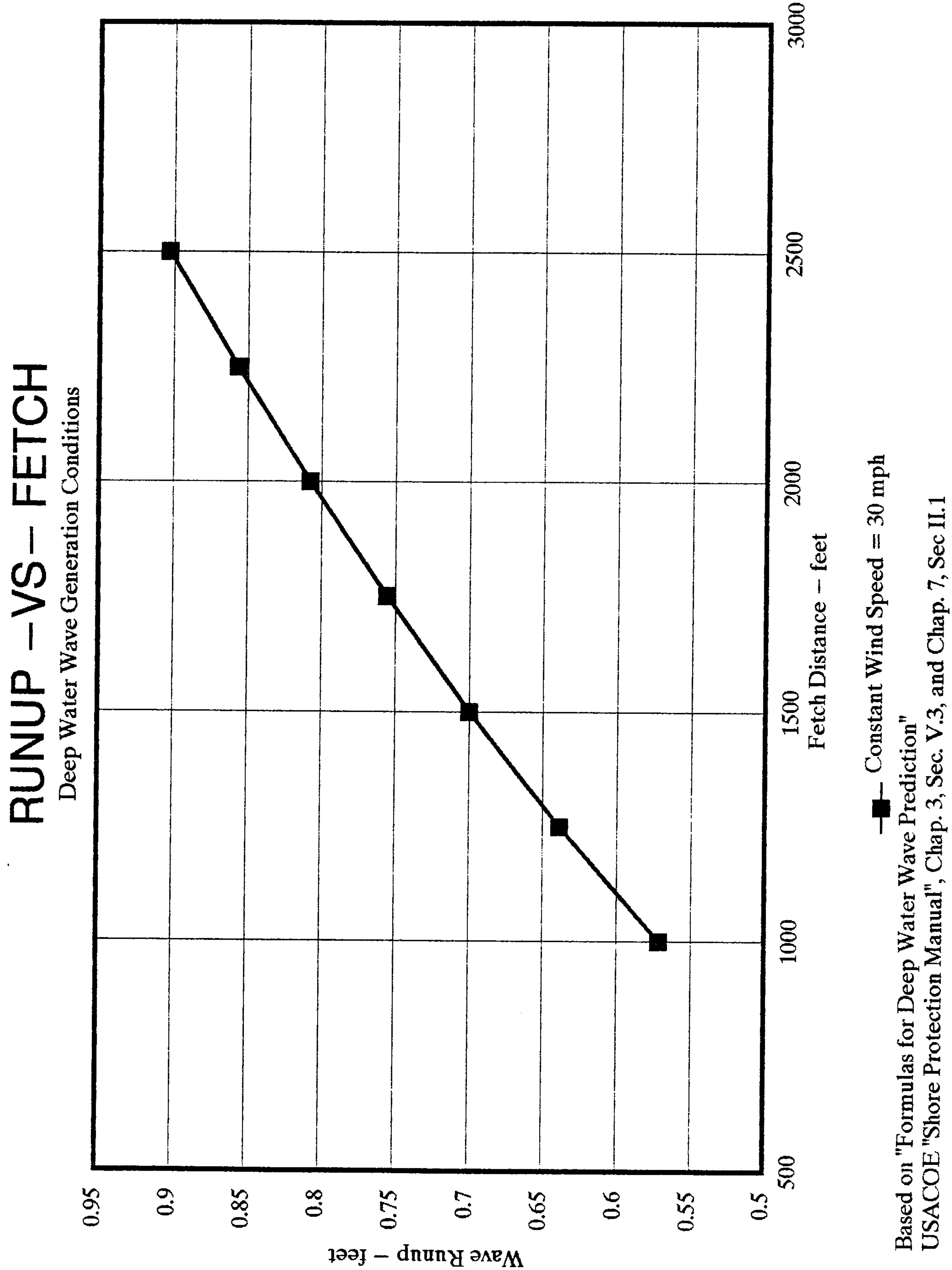
$K = 1.12$

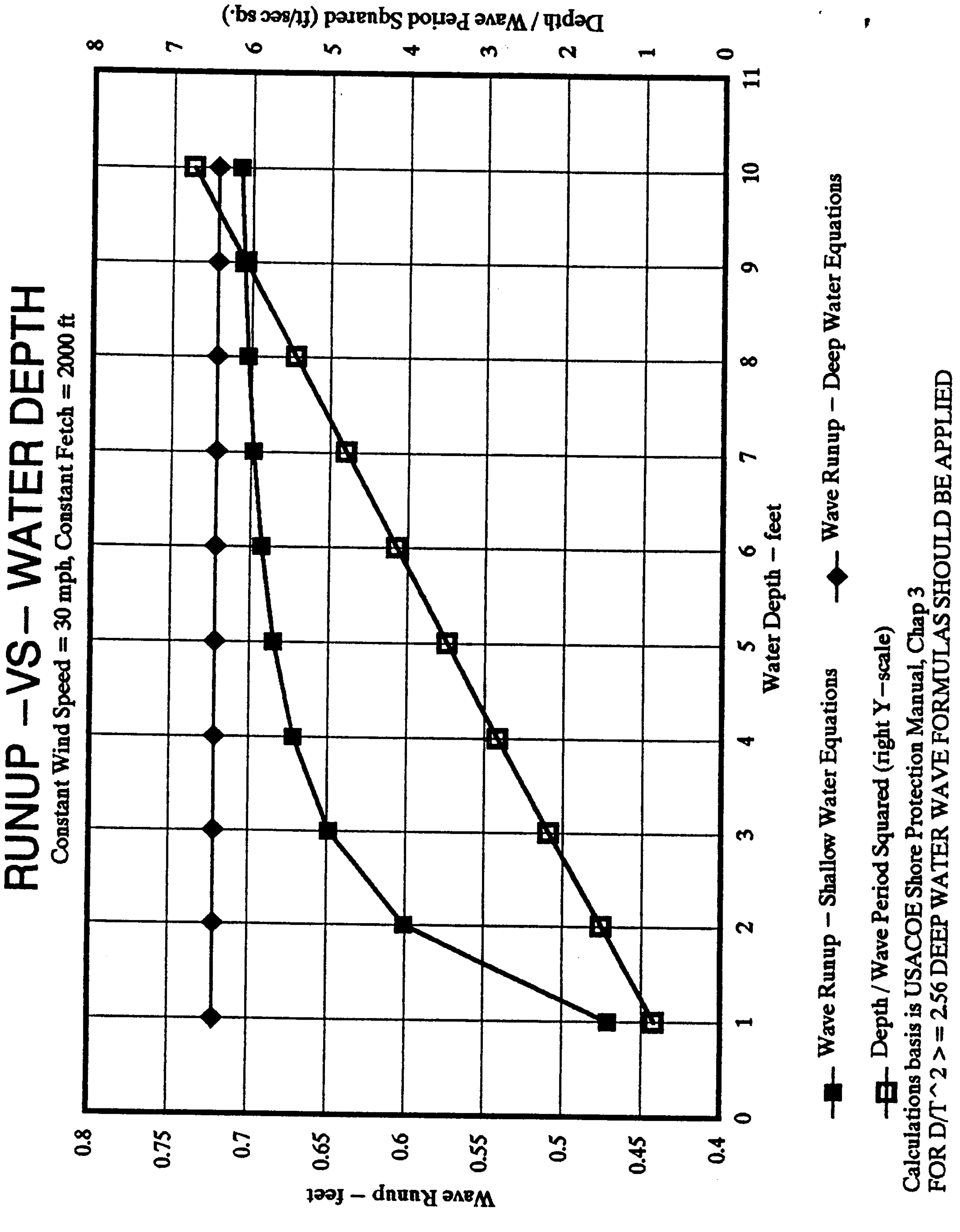
RUNUP = 1.12(.551) = 0.617

SAY 0.62'


PMF
OVERFLOW
CELL 1
SEDIMENTATION BASIN
DRAINAGE BASIN "C"
87 AC.
CELL 2
CELL 3
PMF
OVERFLOW
DRAINAGE BASIN "D"
83 AC.
SAND
BEACH
PROPOSED CELL 4
PHASE B
DRAINAGE BASIN "F"
44 AC.
PROPOSED CELL 4
PHASE A
DRAINAGE BASIN "E"
43.26 AC.
PMF CONTAINED
WITHIN BASIN
UMETCO MINERALS
WHITE MESA MILL
FETCH DISTANCE MAP
SCALE 1" = 400'


CELL 1-I
SEDIMENTATION BASIN
PMF
OVERFLOW
DRAINAGE BASIN "C"
87 AC.
CELL 2
CELL 3
PMF
OVERFLOW
W 1/4
W 1/2
DRAINAGE BASIN "D"
83 AC.
SAND
BEACH
PROPOSED CELL 4
PHASE B
DRAINAGE BASIN "F"
44 AC.
PROPOSED CELL 4
PHASE A
DRAINAGE BASIN "E"
43.26 AC.
PMF CONTAINED
WITHIN BASIN
UMETCO MINERALS
WHITE MESA MILL
FETCH DISTANCE MAP
SCALE 1"=400'


RUNUP –VS– FETCH
Deep Water Wave Generation Conditions
Wave Runup – feet
0.95
0.9
0.85
0.8
0.75
0.7
0.65
0.6
0.55
0.5
500
1000
1500
2000
2500
3000
Fetch Distance – feet
Constant Wind Speed = 30 mph
Based on "Formulas for Deep Water Wave Prediction"
USACOE "Shore Protection Manual", Chap. 3, Sec. V.3, and Chap. 7, Sec II.1


RUNUP -VS- WATER DEPTH
Constant Wind Speed = 30 mph, Constant Fetch = 2000 ft
Wave Runup – feet
0.4
0.45
0.5
0.55
0.6
0.65
0.7
0.75
0.8
Depth / Wave Period Squared (ft/sec sq.)
0
1
2
3
4
5
6
7
8
Water Depth – feet
0
1
2
3
4
5
6
7
8
9
10
11
Wave Runup – Shallow Water Equations
Wave Runup – Deep Water Equations
Depth / Wave Period Squared (right Y-scale)
Calculations basis is USACOE Shore Protection Manual, Chap 3
FOR D/T^2 >= 2.56 DEEP WATER WAVE FORMULAS SHOULD BE APPLIED

V. SIMPLIFIED METHODS FOR ESTIMATING WAVE CONDITIONS

When estimates of wave heights, periods, and directions are needed, the most accurate procedures are the numerical methods discussed in Chapter 3, Section III. However, there are often cases where neither the time available nor the cost justifies using complex numerical methods. In these cases, a simplified method may be justified. Chapter 3, Section V,3 presents a series of equations and nomograms that give significant wave height by H_{m_o} and period of the spectral peak, T_m, for a given windspeed and fetch or duration. Estimating surface winds is treated in Chapter 3, Section IV. Estimating fetch length is treated in Chapter 3, Section V,1.

The spectrally based significant wave height H_{m_o} is four times the square root of the variance of the sea surface elevation. In deep water H_{m_o} is approximately equal to the significant wave height H_s, which is based on counting and measuring individual waves (see Chapter 3, Section II,5). In shallow water, H_{m_o} becomes less than H_s. In both deep and shallow water, H_{m_o} is based on the wave energy; this is not true for H_s.

The following assumptions pertain to these methods. The methods will be used for cases where fetches are short (80 to 120 kilometers (50 to 75 miles) or less) and the wind can be assumed uniform and constant over the fetch. Cases where the wind field varies rapidly in time or with distance over the fetch or where swell from distant sources propagates into the area are best treated numerically. Since these conditions are rarely met and wind fields are not usually estimated accurately, do not assume the results are more accurate than warranted by the accuracy of the input or the simplicity of the method. Good, unbiased estimates of all parameters for input to the wave equations should be sought and the results interpreted conservatively. Individual input parameters should not each be estimated conservatively, since to do so may bias the results.

1. Delineating a Fetch.

A fetch has been defined subjectively as a region in which the windspeed and direction are reasonably constant. Confidence in the computed results begins to deteriorate when wind direction variations exceed 15°; confidence deteriorates significantly when direction deviations exceed 45°. The computed results are sensitive to changes in windspeed as small as 1 knot (0.5 meter per second), but it is not possible to estimate the windspeed over any sizable region with this precision. For practical wave predictions it is usually satisfactory to regard the windspeed as reasonably constant if variations do not exceed 5 knots (2.5 meters per second) from the mean. A coastline upwind from the point of interest always limits a fetch. An upwind limit to the fetch may also be provided by curvature or spreading of the isobars as indicated in Figure 3-20 (Shields and Burdwell, 1970) or by a definite shift in wind direction. Frequently the discontinuity at a weather front will limit a fetch, although this is not always so.

Estimates of the duration of the wind are also needed for wave prediction. Computer results, especially for short durations and high windspeeds may be sensitive to differences of only a few minutes in the duration. Complete synoptic weather charts are prepared only at 6-hour intervals. Thus interpolation to determine the duration may be necessary. Linear interpolation is adequate for most uses, and, when not obviously incorrect, is usually the best procedure. Care should be taken not to interpolate if short-duration phenomena, such as frontal passage or thunderstorms, are present.

The effect of fetch width on limiting ocean wave growth in a generating area may usually be neglected since nearly all ocean fetches have widths about as large as their lengths. In inland waters (bays, rivers, lakes, and reservoirs), fetches are limited by landforms surrounding the body of water. Fetches that are long in comparison to width are frequently found. It is not clear what measure of width is important in limiting the growth of waves.

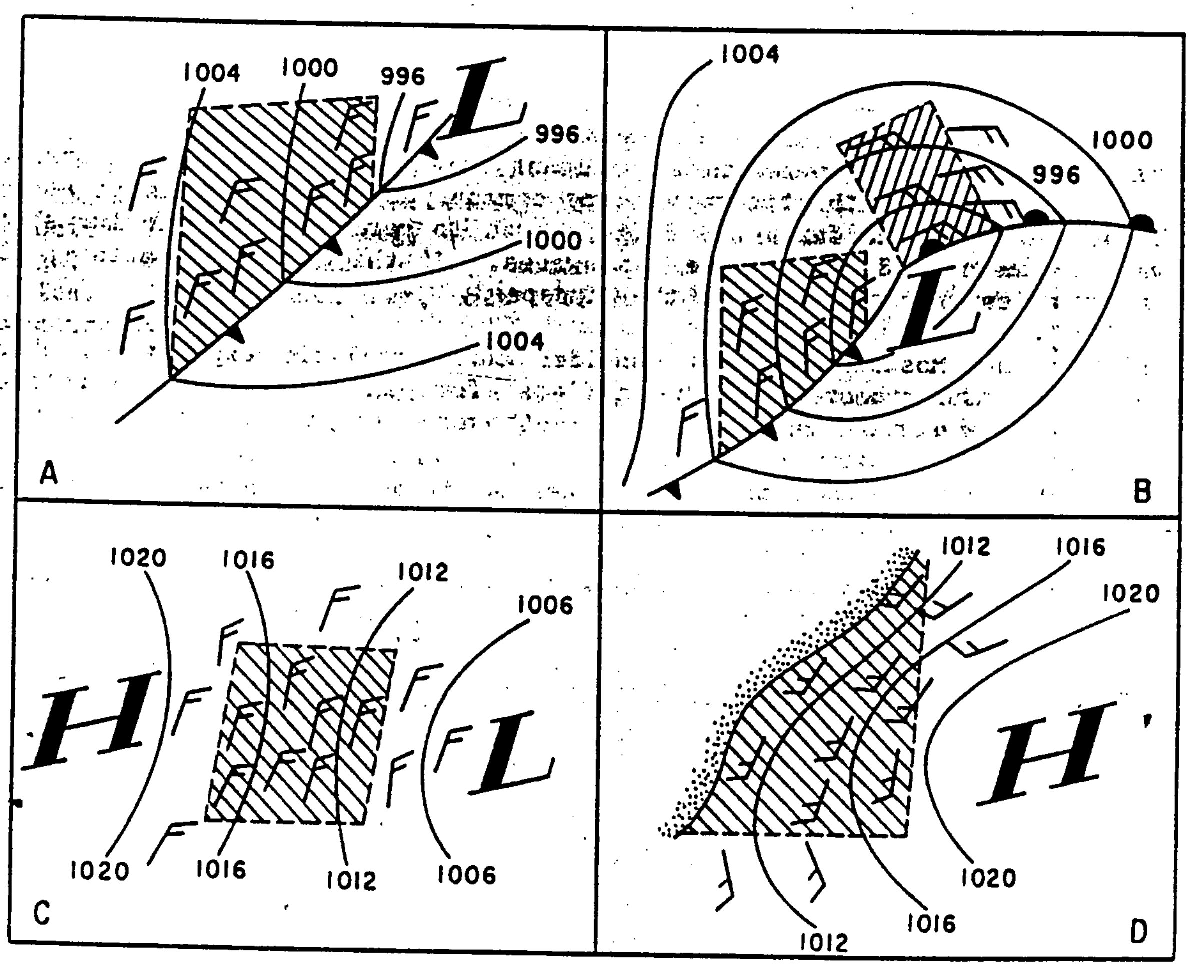


Figure 3-20. Possible fetch limitations.

Shorelines are usually irregular, and a more general method for estimating fetch must be applied. A recommended procedure for determining the fetch length consists of constructing nine radials from the point of interest at 3-degree intervals and extending these radials until they first intersect the shoreline. The length of each radial is measured and arithmetically averaged. While 3-degree spacing of the radials is used in this example, any other small angular spacing could be used.

2. Simplified Wave-Prediction Models.

Use of the wave prediction models discussed in Chapter 3, Section III (Wave Field) requires an enormous computational effort and more meteorological data than is likely to be found outside of a major forecasting center or laboratory.

The U.S. Navy operates an oceanic forecast facility at Monterey, California, and the Corps of Engineers is developing a wave climate for U.S. coastal areas using a sophisticated numerical model. The results of the latter study are being published as a series of climatological reports by the U.S. Army Engineer Waterways Experiment Station.

Computational effort required for the model discussed in Chapter 3, Section III,1 (Development of a Wave Field) can be greatly reduced by the use of simplified assumptions, with only a slight loss in accuracy for wave height calculations, but sometimes with significant loss of detail on the distribution of wave energy with frequency. One commonly used approach is to assume that both duration and fetch are large enough to permit an equilibrium state between the mean wind, turbulence, and waves. If this condition exists, all other variables are determined by the windspeed.

Pierson and Moskowitz (1964) consider three analytic expressions which satisfy all the theoretical constraints for an equilibrium spectrum. Empirical data described by Moskowitz (1964) were used to show that the most satisfactory of these is

$$E(\omega)\ d\omega = (\alpha g^2/\omega^5) e^{-\beta(\omega_0^4/\omega^4)}\ d\omega \qquad (3\text{-}31)$$

where

$\alpha = 8.1 \times 10^{-3}$ (dimensionless constant)

$\beta = 0.74$ (dimensionless constant)

$\omega_0 = g/U$

g = acceleration of gravity

U = windspeed reported by weather ships

ω = wave frequency considered

Equation (3-31) may be expressed in many other forms. Bretschneider (1959, 1963) gave an equivalent form, but with different values for α and β A similar expression was also given by Roll and Fischer (1956). The condition in which waves are in equilibrium with the wind is called a *fully arisen sea*. The assumption of a universal form for the fully arisen sea permits the

$$b = \exp - \left[\frac{(f - f_m)^2}{2\ \sigma^2\ f_m^{\ 2}} \right]$$

f_m is the frequency of the spectral peak, and α, σ, and γ are coefficients either fit to an observed spectrum or calculated as functions of dimensionless fetch (Hasselmann et al., (1973, 1976). This formula is called the Joint North Sea Wave Project (JONSWAP) spectral shape after the field experiment on which it is based. Frequently, a single peaked spectrum is fitted to this form if parametric analytic spectra are required for mathematical analysis.

Similar formulas can also be developed empirically from wind and wave observations. A combined empirical-analytical procedure was used by Sverdrup and Munk (1947) in the first widely used wave prediction system. The Sverdrup-Munk prediction curves were revised by Bretschneider (1952, 1958) using empirical data. This prediction system is therefore often called the Sverdrup-Munk-Bretschneider (SMB) method.

More recent field data (Mitsuyasu, 1968; Hasselman et al., 1973) have resulted in some revisions to this method. The resulting curves are given in the next section. This wave prediction system is convenient when limited data and time are available.

3. Formulas for Predicting Waves in Deep Water.

It is desirable to have a simple method for making wave estimates. This is possible only if the geometry of the waterbody is relatively simple and if the wave conditions are either fetch-limited or duration-limited. Under fetch-limited conditions, winds have blown constantly long enough for wave heights at the end of the fetch to reach equilibrium. Under duration-limited conditions, the wave heights are limited by the length of time the wind has blown. These two conditions represent asymptotic approximations to the general problem of wave growth. In most cases the wave growth pattern at a site is a combination of the two cases. Equations (3-33) to (3-38) (Table 3-2) were obtained by simplifying the equation used to develop the parametric model (Hasselmann et al., 1976). Two dimensionless plots for wave growth are given in Figures 3-21 and 3-22, which also include adjustments for shallow water discussed in Chapter 3, Section IV.

In the fetch-limited case, the parameters required are the fetch, F and the wind-stress factor U_A (adjusted windspeed), where U_A has been adjusted as described in Chapter 3, Section IV, and represents a relatively constant average value over the fetch. The spectral wave height H_{m_o} and peak spectral period T_m are the parameters predicted.

$$\frac{gH_{m_o}}{U_A^2} = 1.6x10^{-3} \left(\frac{gF}{U_A^2} \right)^{1/2} \qquad (3-33)$$

$$\frac{gT_m}{U_A} = 2.857x10^{-1} \left(\frac{gF}{U_A^2} \right)^{1/3} \qquad (3-34)$$

and

$$\frac{gt}{U_A} = 6.88\times10^1 \left(\frac{gF}{U_A^2}\right)^{2/3} \quad (3\text{-}35)$$

Note that $T_{1/3}$ is given as 0.95 T_m. The preceding equations are valid up to the fully developed wave conditions given by

$$\frac{gH_{m_o}}{U_A^2} = 2.433\times10^{-1} \quad (3\text{-}36)$$

$$\frac{gT_m}{U_A} = 8.134 \quad (3\text{-}37)$$

$$\frac{gt}{U_A} = 7.15\times10^4 \quad (3\text{-}38)$$

where

H_{m_o} = the spectrally based significant wave height

T_m = the period of the peak of the wave spectrum

F = the fetch

t = the duration

U_A = the wind-stress factor

Often in applying the wave growth formulas, the engineer must determine if the design situation is fetch limited or duration limited. In these cases estimates of a one half- to 5-, etc. hour windspeeds with some return period (often 25 or 50 years) may be available. The objective is to find the largest wave height that occurs under these conditions. For example, a given return period, the 30-minute windspeed, will be higher than the 1- to 3-, etc. hour windspeeds, but because of its short duration it may produce a smaller wave height than the 1-hour windspeed.

A given calculation for a duration should be checked to ensure that it has not exceeded the maximum wave height or period possible for the given wind-stress factor and fetch. The nomograms in Figures 3-23 and 3-24 show wave prediction curves of empirical values which can be used to check the reasonableness of the mathematical solutions. For example, for U_A = 20 meters per second a duration of 5 hours yields a height of 2.5 meters. However, if the fetch were only 30 kilometers long, the maximum wave height can only be 1.75 meters for a wind-stress factor of 20 meters per second. If the wind-stress factor is 20 meters per second and its duration is only 3 hours, the fetch-limited wave height of 2.5 meters for a fetch of 30 kilometers would not be reached; therefore, the wave height is duration limited. It is essential that fetch-limited wave calculations be checked to see if they are duration limited; likewise, duration-limited cases should be

Table 3-2. Deepwater wave forecasting equation.

Dimensionless	Metric Units	
	$H(m)$, $T(s)$, $U_A(m/s)$, $F(m)$, $t(s)$	$H(m)$, $T(s)$, $U_A(m/s)$, $F(km)$, $t(hr)$
FETCH LIMITED, (F, U)		
$\frac{gH_{m_o}}{U_A^2} = 1.6\times10^{-3}\left(\frac{gF}{U_A^2}\right)^{1/2}$ (3-33)	$H_{m_o} = 5.112\times10^{-4}\, U_A F^{1/2}$ (3-33a)	$H_{m_o} = 1.616\times10^{-2}\, U_A F^{1/2}$ (3-33b)
$\frac{gT_m}{U_A} = 2.857\times10^{-1}\left(\frac{gF}{U_A^2}\right)^{1/3}$ (3-34)	$T_m = 6.238\times10^{-2}\left(U_A F\right)^{1/3}$ (3-34a)	$T_m = 6.238\times10^{-1}\left(U_A F\right)^{1/3}$ (3-34b)
$\frac{gt}{U_A} = 6.88\times10^{1}\left(\frac{gF}{U_A^2}\right)^{2/3}$ (3-35)	$t = 3.215\times10^{1}\left(\frac{F^2}{U_A}\right)^{1/3}$ (3-35a)	$t = 8.93\times10^{-1}\left(\frac{F^2}{U_A}\right)^{1/3}$ (3-35b)
FULLY DEVELOPED		
$\frac{gH_{m_o}}{U_A^2} = 2.433\times10^{-1}$ (3-36)	$H_{m_o} = 2.482\times20^{-2}\, U_A^2$ (3-36a)	$H_{m_o} = 2.4821\times10^{-2}\, U_A^2$ (3-36b)
$\frac{gT_m}{U_A} = 8.134$ (3-37)	$T_m = 8.30\times10^{-1}\, U_A$ (3-37a)	$T_m = 8.30\times10^{-1}\, U_A$ (3-37b)
$\frac{gt}{U_A} = 7.15\times10^{4}$ (3-38)	$t = 7.296\times10^{3}\, U_A$ (3-38a)	$t = 2.027\, U_A$ (3-38b)
NOTATIONS	$g = 9.8\ m/s^2$	$g = 9.8\ m/s^2$ 1 kilometer = 1000 m 1 hour = 3600 s

	English Units	
$H(ft)$, $T(s)$, $U_A(ft/s)$, $F(ft)$, $t(s)$	$H(ft)$, $T(s)$, $U_A(mi/hr)$ $F(m)$, $t(hr)$	$H(ft)$, $T(s)$, U_A (kn), $F(nmi)$, $t(hr)$
FETCH LIMITED (F, U)		
$H_{m_o} = 2.82\times10^{-4}\, U_A F^{1/2}$ (3-33c)	$H_{m_o} = 3.01\times10^{-2}\, U_A F^{1/2}$ (3-33d)	$H_{m_o} = 3.714\times10^{-2}\, U_A F^{1/2}$ (3-33e)
$T_m = 2.825\times10^{-2}\left(U_A F\right)^{1/3}$ (3-34c)	$T_m = 5.59\times10^{-1}\left(U_A F\right)^{1/3}$ (3-34d)	$T_m = 6.14\times10^{-1}\left(U_A F\right)^{1/3}$ (3-34e)
$t = 2.16\times10^{1}\left(\frac{F^2}{U_A}\right)^{1/3}$ (3-35c)	$t = 1.603\left(\frac{F^2}{U_A}\right)^{1/3}$ (3-35d)	$t = 1.680\left(\frac{F^2}{U_A}\right)^{1/3}$ (3-35e)
FULLY DEVELOPED		
$H_{m_o} = 7.553\times10^{-3}\, U_A^2$ (3-36c)	$H_{m_o} = 1.625\times10^{-2}\, U_A^2$ (3-36d)	$H_{m_o} = 2.154\times10^{-2}\, U_A^2$ (3-36e)
$T_m = 2.53\times10^{-1}\, U_A$ (3-37c)	$T_m = 3.706\times10^{-1}\, U_A$ (3-37d)	$T_m = 4.244\times10^{-1}\, U_A$ (3-37e)
$t = 2.220\times10^{3}\, U_A$ (3-38c)	$t = 9.045\times10^{-1}\, U_A$ (3-38d)	$t = 1.04\, U_A$ (3-38e)
NOTATIONS $g = 32.2\ ft/s^2$	$g = 32.2\ ft/s^2$ 1 mile = 5280 ft miles per hour = 1.467 ft/s 1 hour = 3600 s	$g = 32.2\ ft/s^2$ 1 nautical mile = 6080 ft 1 knot = 1.689 ft/s 1 hour = 3600 s

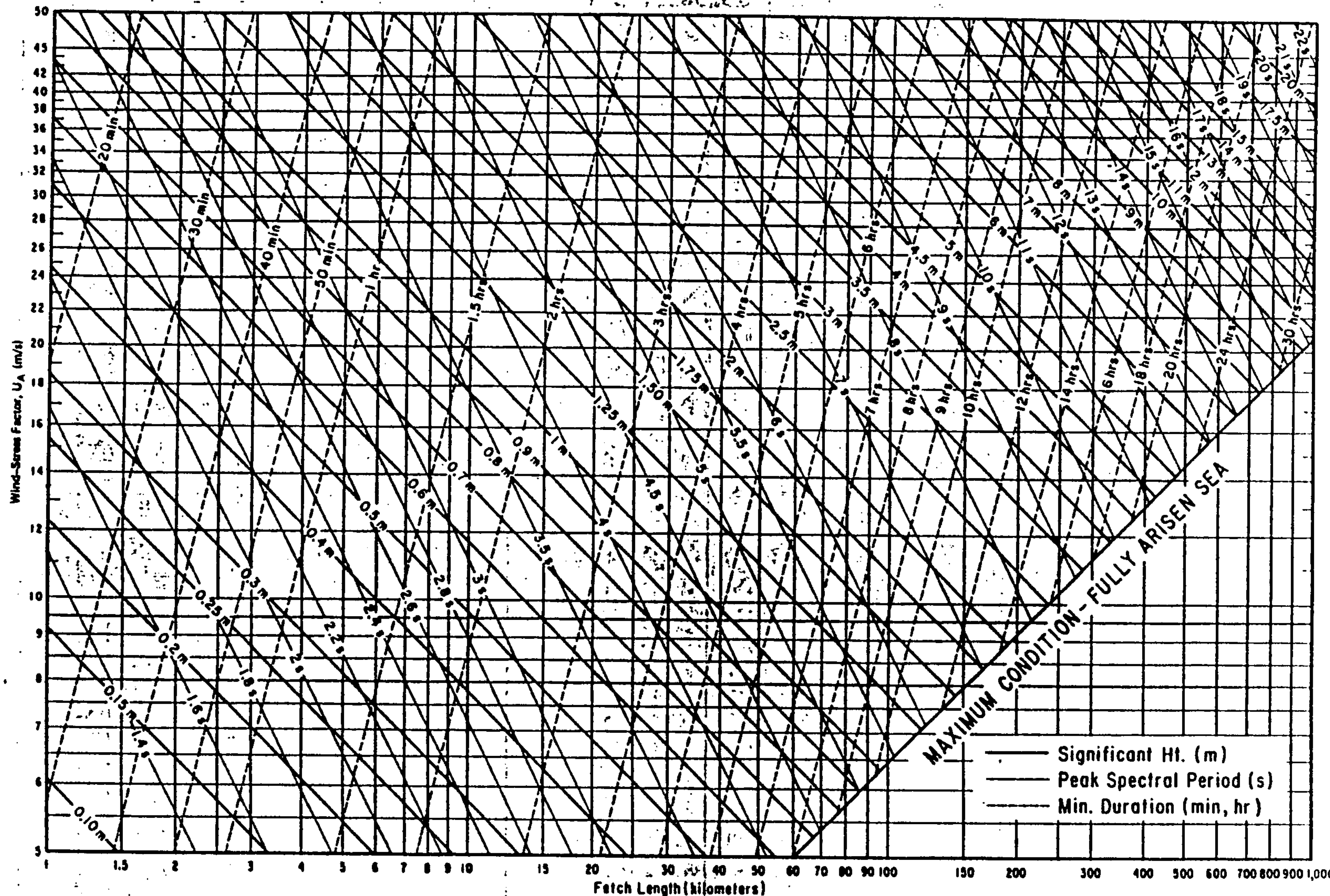


Figure 3-23. Nomograms of deepwater significant wave prediction curves as functions of windspeed, fetch length, and wind duration (metric units).

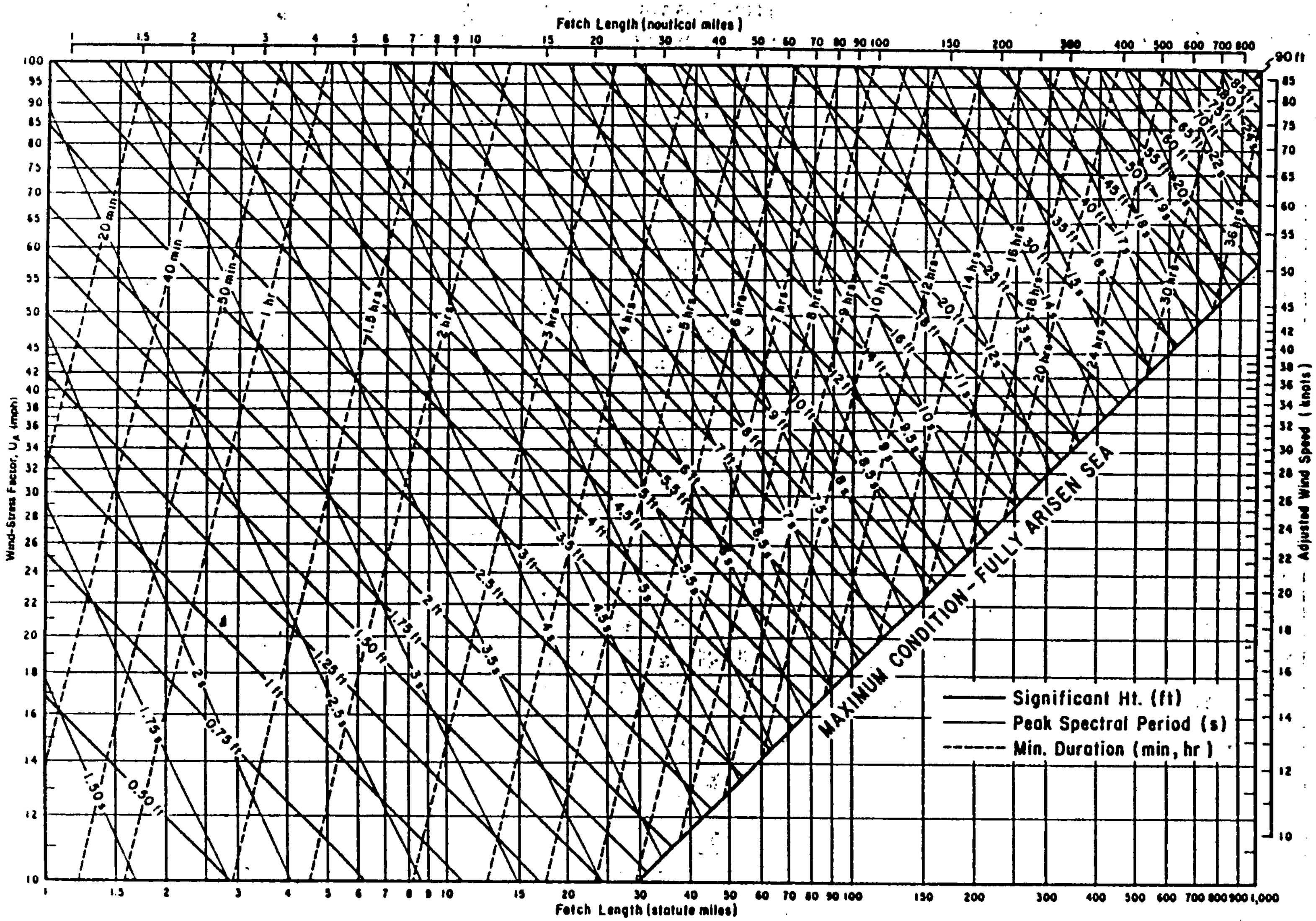


Figure 3-24. Nomograms of deepwater significant wave prediction curves as functions of windspeed, fetch length, and wind duration (English units).

checked to see if they are really fetch limited. If the formulas are used rather than the nomograms, wave conditions should also be checked to see if they exceed the fully developed condition.

Wave growth with duration is not as well understood as wave growth with fetch length. Equation (3-36) ensures that the growth of H_{m_0} and T_m with time reaches the fetch-limited value at about the same duration specified by equation (3-39). The approximation works well except for long dimensionless fetches (relatively long-fetch; low-windspeed cases).

Inevitably, estimating wave height and period requires that checks be made between fetch, duration, and fully developed limitations. Many design situations require iteration between these approaches and the appropriate averaged durations. The wave growth formulas must use the wind-stress factor and not windspeed. The proper averaging times for the winds (as related to the duration and fetch) must be used. This approach is approximate, and the number of iterations and adjustments used should reflect this limited accuracy.

4. Narrow Fetch Conditions.

When early users of the SMB curves applied them to reservoirs and small lakes, calculated wave heights were much larger than observed wave heights. It was thus assumed that the narrowness of the fetch was affecting wave growth. The concept of an effective fetch was introduced which reduced fetch length to account for the narrowness of the fetch. The adjustment provided improved wave estimates. When the growth curves presented here were applied to similar situations (Resio and Vincent, 1979) the effective fetch calculation resulted in wave heights that were too low, while a straight-line fetch provided wave heights closer to observed values (Fig. 3-25). Data from inland reservoirs were checked by computing H_s based on an effective fetch and on straight-line fetch (Fig. 3-26). The straight-line fetch shows reasonable agreement with the growth curves.

The reason an effective fetch adjustment is required for the SMB curves is that these curves overpredict wave heights for small values of F more than do recent data. The effective fetch method implicitly assumes a cosine directional spread for wind input to the sea. More recent data suggest that a cosine to the 10th power describes the directional distribution near the peak frequency of the spectrum. This is a much narrower spread. *Effective fetch should not be used with the growth curves presented herein.* There may be a critical fetch width where width becomes important, but this is not known at this time.

* * * * * * * * * * * * * * * * EXAMPLE PROBLEM 4 * * * * * * * * * * * * * * * *

GIVEN: Eight consecutive hourly observations of fastest mile windspeed U_O = 20 meters per second are observed at an elevation of Z_L = 6 meters, approximately 5 kilometers inland from shore. The observation site is at an airport weather station. The air-sea temperature difference was estimated to be −6° C.

VI. WAVE FORECASTING FOR SHALLOW WATER

1. Forecasting Curves.

Water depth affects wave generation. For a given set of wind and fetch conditions, wave heights will be smaller and wave periods shorter if generation takes place in transitional or shallow water rather than in deep water. Several forecasting approaches have been made, including the method given by Bretschneider as modified using the results of Ijima and Tang (1966). Bretschneider and Reid (1953) consider bottom friction and percolation in the permeable sea bottom.

There is no single theoretical development for determining the actual growth of waves generated by winds blowing over relatively shallow water. The method presented here is based on successive approximations in which wave energy is added due to wind stress and subtracted due to bottom friction and percolation. This method uses deepwater forecasting relationships (Chapter 3, Section V) to determine the energy added due to wind stress. Wave energy lost due to bottom friction and percolation is determined from the relationships developed by Bretschneider and Reid (1953). Resultant wave heights and periods are obtained by combining the above relationships by numerical methods. The basic assumptions applicable to development of deepwater wave generation relationships as well as development of relationships for bottom friction loss (Putnam and Johnson, 1949) and percolation loss (Putnam, 1949) apply. The duration should be considered approximate.

These shallow-water forecasting curves (Fig. 3-27 through 3-36) represent an interim method for wave forecasting in shallow water. Modifications to the shallow-water forecasting equations were made to provide a transition between the revised deepwater forecasting equations and the shallow-water forecasting model. Research is underway that may revise the shallow-water forecasting model. Until the results of this new research are available, the curves should be used. The curves are plotted from the following equations:

$$\frac{gH}{U_A^2} = 0.283 \tanh\left[0.530\left(\frac{gd}{U_A^2}\right)^{3/4}\right] \tanh\left\{\frac{0.00565\left(\frac{gF}{U_A^2}\right)^{1/2}}{\tanh\left[0.530\left(\frac{gd}{U_A^2}\right)^{3/4}\right]}\right\} \qquad (3\text{-}39)$$

$$\frac{gT}{U_A} = 7.54 \tanh\left[0.833\left(\frac{gd}{U_A^2}\right)^{3/8}\right] \tanh\left\{\frac{0.0379\left(\frac{gF}{U_A^2}\right)^{1/3}}{\tanh\left[0.833\left(\frac{gd}{U_A^2}\right)^{3/8}\right]}\right\} \qquad (3\text{-}40)$$

and

$$\frac{gt}{U_A} = 5.37 \times 10^2 \left(\frac{gT}{U_A}\right)^{7/3} \qquad (3\text{-}41)$$

The wind-stress factor U_A (adjusted windspeed) is obtained by estimating the surface wind U_s in meters per second via Chapter 3, Section IV and then setting $U_A = 0.71\ U_s^{1.23}$. Each figure is plotted for a constant water depth d. Linear interpolation between figures is sufficiently accurate for determining intermediate wave heights and periods. For water depths greater than 15 meters (50 feet) and less than 90 meters (300 feet), use equations (3-39) to (3-41). For depths greater than 90 meters (300 feet), the revised deepwater forecasting equations should be used.

The minimum duration t has been added to the shallow-water forecasting curves to simplify determining the wind-stress factor U_A. Waves with periods less than a specified value are noted as deepwater waves on each figure. The duration equation used, therefore, is a transposed, simplified approximation of the deepwater duration equation.

* * * * * * * * * * * * * * * * EXAMPLE PROBLEM 5 * * * * * * * * * * * * * * * *

GIVEN: Fetch length $F = 24.4$ km (80,000 ft)

Wind-stress factor $U_A = 22$ m/s (50 mi/hr)

Constant depth $d = 11$ m (35 ft)

FIND: Wave height H_s

Wave period T

SOLUTION:

From Figure 3-33a or equation (3-39) and (3-40)

$H_s = 1.5$ m (4.9 ft)

and

$T = 4.4$ s

* *

2. Propagation Over Flooded, Vegetated Land.

When waves travel across a shallow flooded area, the initial heights and periods of the waves may increase; i.e., when the wind stress exceeds the frictional stress of the ground and vegetation underlying the shallow water. The initial wave heights may decay at other times when the frictional stress exceeds the wind stress.

Camfield (1977) presents an *approximate method* for estimating the growth or decay of wind waves passing over areas with high values of bottom friction. It is assumed that the high friction values can be accounted for by can reasonably be assumed to occur simultaneously at the site. Where hurricanes cross the coast, high water levels resulting from storm surge and extreme wave action generated by the storm occur together and usually provide critical design conditions. Design water levels and wave conditions are needed for refraction and diffraction analyses, and these analyses must follow establishment of design water levels and design wave conditions.

The frequency of occurrence of adopted design conditions and the frequency of occurrence and duration of a range of reasonable combinations of water level and wave action are required for an adequate economic evaluation any proposed shore protection scheme.

II. WAVE RUNUP, OVERTOPPING, AND TRANSMISSION

1. Wave Runup

a. Regular (Monochromatic) Waves. The vertical height above the stillwater level to which water from an incident wave will run up the face of a structure determines the required structure height if wave overtopping cannot be permitted (see Fig. 7-7 for definitions). Runup depends on structure shape and roughness, water depth at structure toe, bottom slope in front of a structure, and incident wave characteristics. Because of the large number of variables involved, a complete description is not available of the runup phenomenon in terms of all possible ranges of the geometric variables and wave conditions. Numerous laboratory investigations have been conducted, but mostly for runup on smooth, impermeable slopes. Hall and Watts (1953) investigated runup of solitary waves on impermeable slopes; Saville (1956) investigated runup by periodic waves. Dai and Kamel (1969) investigated the runup and rundown of waves on rubble breakwaters. Savage (1958) studied effects of structure roughness and slope permeability. Miller (1968) investigated runup of undular and fully broken waves on three beaches of different roughnesses. LeMehaute (1963) and Freeman and LeMehaute (1964) studied long-period wave runup analytically. Keller et al. (1960), Ho and Meyer (1962), and Shen and Meyer (1963) studied the motion of a fully broken wave and its runup on a sloping beach.

Figures 7-8 through 7-13 summarize results for small-scale laboratory tests of runup of regular (monochromatic) waves on smooth impermeable slopes (Saville, 1958a). The curves are in dimensionless form for the relative runup R/H_o' as a function of deepwater wave steepness and structure slope, where R is the runup height measured (vertically) from the SWL and H_o' is the *unrefracted deepwater wave height* (see Figure 7-7 for definitions). Results predicted by Figures 7-8 through 7-12 are probably smaller than the runup on prototype structures because of the inability to scale roughness effects in small-scale laboratory tests. *Runup values from Figures 7-8 through 7-12 can be adjusted for scale effects by using Figure 7-13.*

Runup on impermeable structures having quarrystone slopes and runup on vertical, stepped, curved and Galveston-type recurved seawalls have been studied on laboratory-scale models by Saville (1955, 1956). The results are

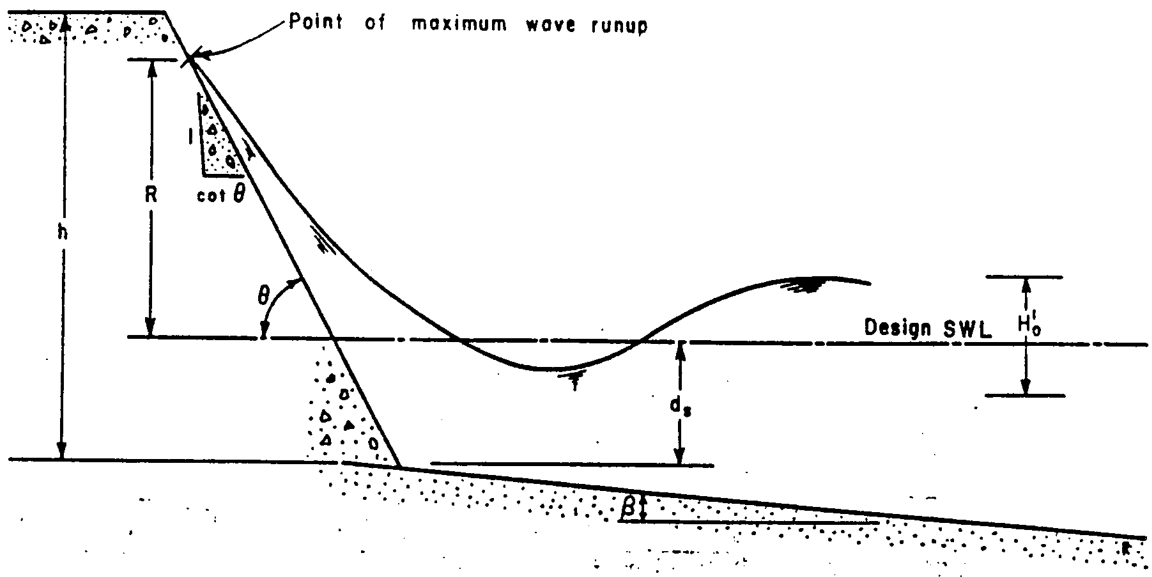


Figure 7-7. Definition sketch: wave runup and overtopping.

shown in Figures 7-14 through 7-18. Effects of using graded riprap on the face of an impermeable structure (as opposed to quarrystone of uniform site for which Figure 7-15 was obtained) are presented in Figure 7-19 for a 1 on 2 graded riprap slope. Wave rundown for the same slope is also presented in Figure 7-19. Runup on *permeable rubble slopes* as a function of structure slope and H'_o/gT^2 is compared with runup on smooth slopes in Figure 7-20. Corrections for scale effects, using the curves in Figure 7-13, should be applied to runup values obtained from Figures 7-8 through 7-12 and 7-14 through 7-18. The values of runup obtained from Figure 7-19 and 7-20 are assumed directly applicable to prototype structures without correction for scale effects.

As previously discussed, Figures 7-8 through 7-20 provide design curves for smooth and rough slopes, as well as various wall configurations. As noted, there are considerable data on smooth slopes for a wide range of d_s/H'_o values, whereas the rough-slope data are limited to values of $d_s/H'_o > 3$. It is frequently necessary to determine the wave runup on permeable rubble structures for specific conditions for which model tests have not been conducted, such as breaking waves for $d_s/H'_o < 3$. To provide the necessary design guidance, Battjes (1974), Ahrens (1977a), and Stoa (1978) have suggested the use of a roughness and porosity correction factor that allows the use of various smooth-slope design curves for application to other structure slope characteristics. This roughness and porosity correction factor, r , is the ratio of runup or relative runup on rough permeable or other nonsmooth slope to the runup or relative runup on a smooth impermeable slope. This is expressed by the following equation:

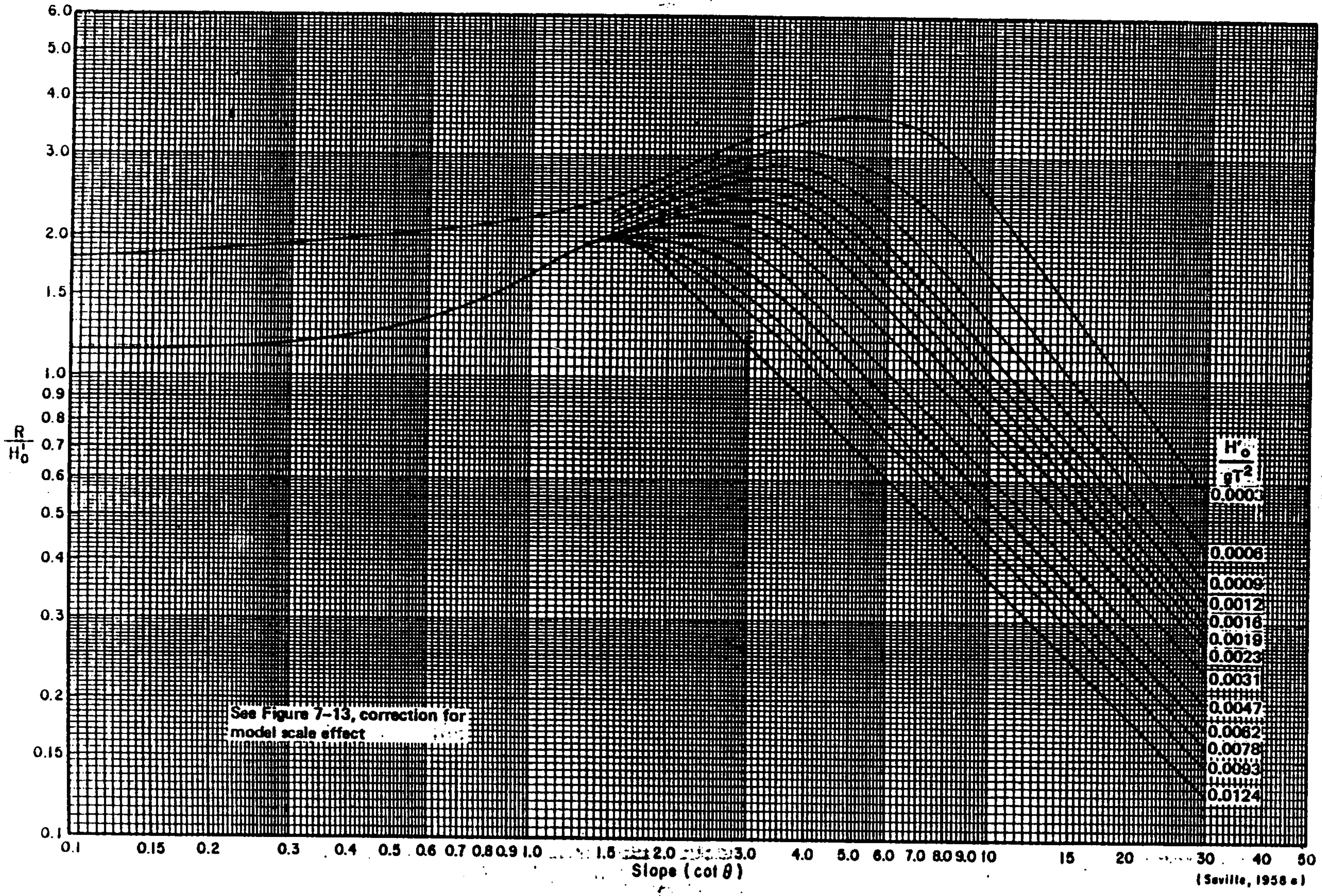


Figure 7-12. Wave runup on smooth, impermeable slopes when $d_s/H'_o \geq 3.0$.

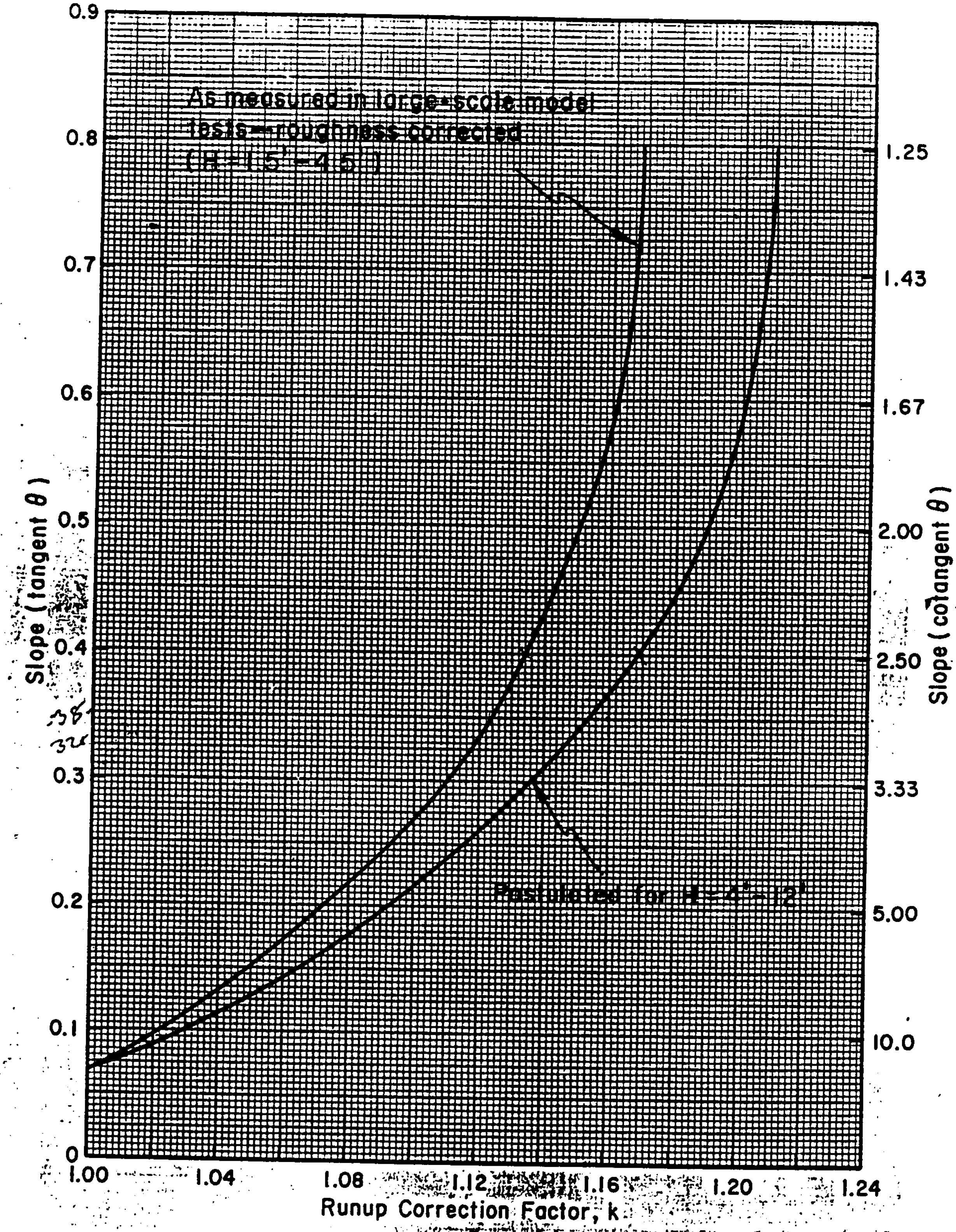


Figure 7-13. Runup correction for scale effects.

FLOOD DETENTION POND

FLOOD DETENTION POND

A flood detention pond was constructed in Drainage Basin B1 to attenuate the PMF flood allowing a smaller capacity for Ditch 2. The dike material is classifed as an ML-CL soil. The compaction tests average 90.3% of the maximum dry density determined by ASTM D698. The dike is approximately 325 feet long with a minimum crest elevation of 5648.5. At the maximum section the crest width is 11.8 feet, 4.97 feet high with 2 horz:1 vert downstream slope and a 1.8 horz:1 vert upstream slope. The pond will retain water only during a storm event. The maximum water surface in Ditch 2 during a PMF is elevation 5647.99 leaving 0.51 feet freeboard to the top of detention pond dike. Ditch 2 drains the detention pond when not in use. Construction of the ditch will be completed by March 31, 1990.

The stability analysis for the dike was prepared by using a typical shear strength value (ϕ) of 31.8 degrees for the sandy silty clay soil. Using stability charts from Bureau of Mines manual RI8564, a factor of safety equal to 1.65 was determined. The required safety factor is 1.5 so the dike is stable.

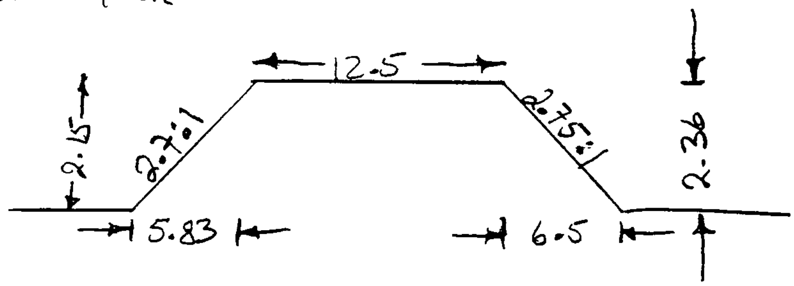
In the event of a dike failure, the water stored in the pond would flow downstream and overtop a mill road approximately 50 feet downstream. Roughly 4 acre feet would be stored behind the mill road and the remainder would flow into Drainage Basin B3 eventually reaching Cell 1I. Some of the water would be retained in the soil between the cell and the pond. Assuming the entire 20 acre foot volume from the pond reached Cell 1I the water surface in Cell 1I would increase by .38 feet.

White Mesa Mill
Flood Retention Dike

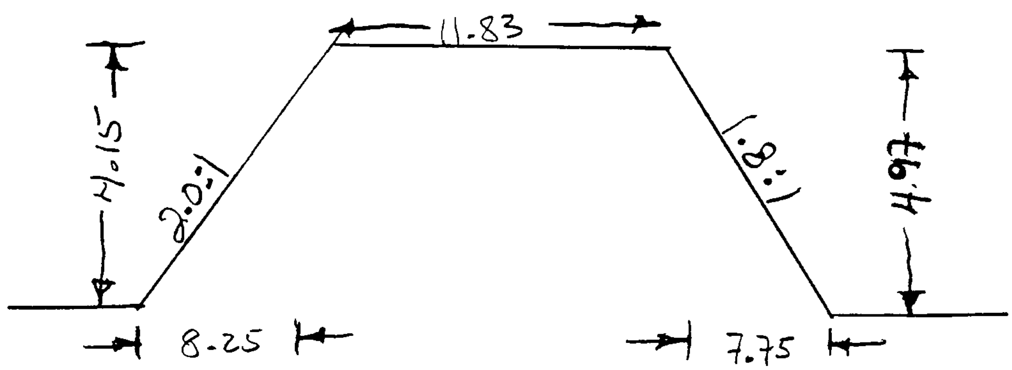
11-6-89
Not to Scale

North →
(Looking West)

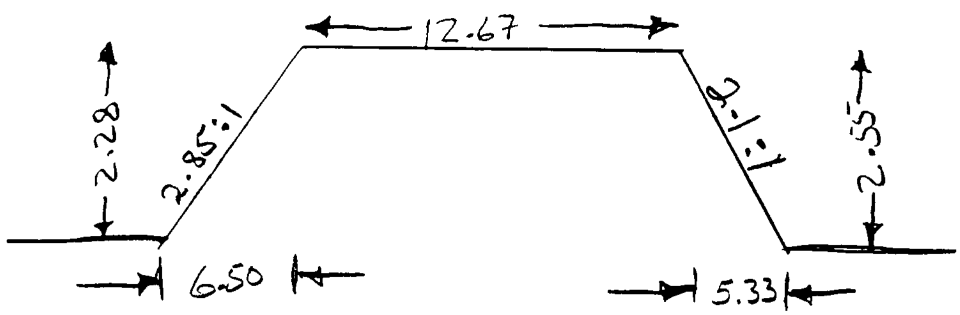
East Section



Middle Section



West Section



WHITE MESA MILL
FLOOD RETENTION DIKE
STABILITY ANALYSIS

Material Properties:

ASTM D698 Proctor Curve	
Max. Dry Density	110.2 PCF
Optimum Moisture	15.6%
Visual Classification	Sandy Silty Clay
Atterburg Limits	
PI	7
LL	25
Unified Soil Classification	ML-CL
Avg. Compaction	90.3% of Max.
Avg. Moisture Content	7.2% of optimum
Avg. Compacted unit weight (γ)	99.8 pcf

Shear Strength Parameters:

Values taken from averages listed in USBR "Small Dams" page 137

Tanϕ = .62 ϕ_1 = 31.8

Cohesion - Co (psi) C^1 = Co = 9.7 psi

The factor of safety was determined using RI 8564 Bureau of Mines Report on Investigations/1981 "Factor of Safety Charts for Estimating the Stability of Saturated and Unsaturated Tailings Pond Embankments.

1. $C^1/\gamma H$ = 9.7 psi/99.8 pcf (4.3 ft) = .0226

2. Avg Slope = 2.0 Horz to 1 Vert.

3. Appendix A Figure A-4 F=1.65$>$1.5 so embankment is stable

Excerpts from the manual are attached along with the material properties.

F-4 SOIL SAMPLING LOG

SAMPLE NO. 111

DATE 11/6/89

SAMPLED BY H. Kuebler

PROJECT NO. Plant Drainage

DELIVERED TO LABORATORY

DATE 11/6/89

LOCATION Berm N of Plant

(EXAMPLE: STOCKPILE, BORROW AREA, TRUCK, FILL)

DEPTH 0-1'

SAMPLE TYPE Bulk

(EXAMPLE: LARGE BULK SAMPLE, DRIVE CYLINDER, ECT.)

VISUAL CLASSIFICATION Sandy Silty Clay

INTENDED USE Berm Construction

(EXAMPLE: CLAYEY BORROW, RANDOM FILL, ETC.)

TESTING PROGRAM Sieve, PI, Proctor

(EXAMPLE: STANDARD COMPACTION TEST, ATTERBERG LIMITS, ETC.)

SOIL/AGGREGATE - MOISTURE DENSITY RELATIONS

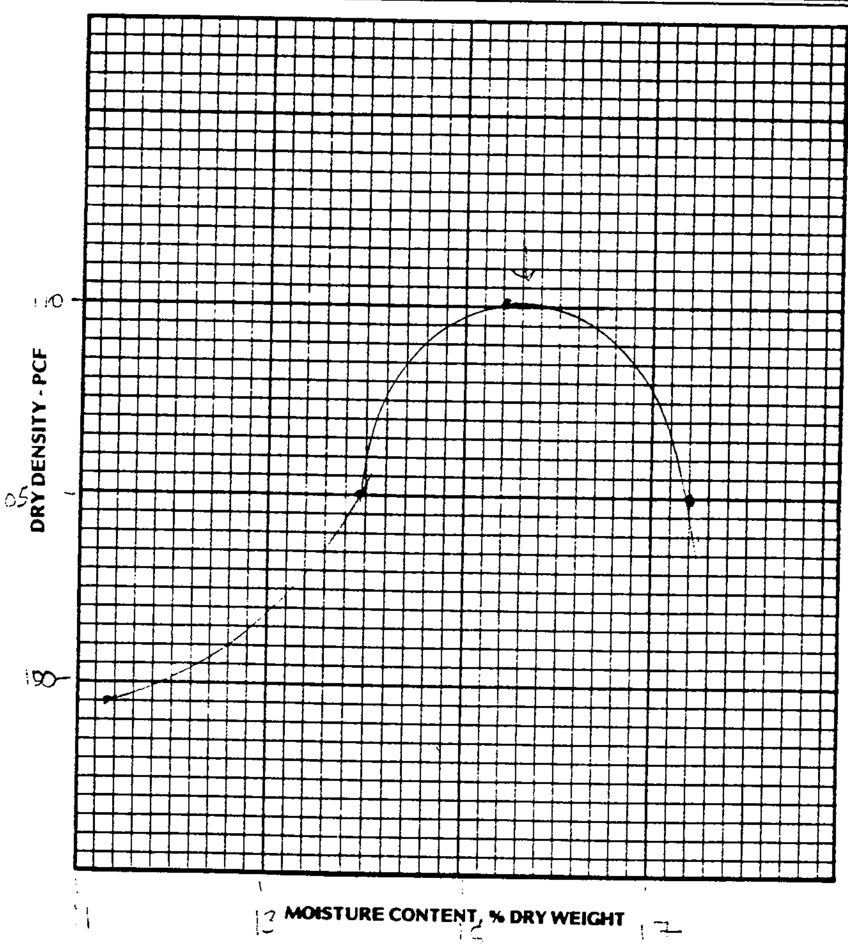
#111

Job No. C-4A-WM

Lab./Invoice No.

Type of Material	Sandy Silty Clay	Sampled By	H Kuecler	Date 11-7-89
Source of Material	Borrow - North [illegible]	Submitted By	↓	Date ↓
Test Procedure	ASTM 698	Tested/Calc. By	↓	Date ↓
		Reviewed By		Date

Trial No.	1	2	3	4	5	6	7
Water, estimated %							
Water, cc	300	250	[illegible]	200			
Wt. Sample + Mold	6133	6037	6072	5902.			
Wt. Mold	4222.	→	→	→			
Wt. Wet Sample, gm	1925	1809	[illegible]	[illegible]			
Wt. Wet Sample, lbs.	4.24	[illegible]	[illegible]	[illegible]			
Wet Density, pcf	[illegible]	[illegible]	[illegible]	[illegible]			
Moisture Sample, wet	395.0	[illegible]	281.9	329.6			
Moisture Sample, Dry	342.3	[illegible]	240.2	296.2			
Wt. Moisture	52.7	[illegible]	41.7	33.4			
Moisture, %	[illegible]	13.9	17.4	11.3			
Dry Density, pcf	110.2	105.1	105.0	99.5			



Max. Dry density, pcf 110.2

Optimum Moisture Content, % 15.6

Diameter of Mold, in. [illegible]

Height of Mold, in. [illegible]

No. of Layers

Blows per Layer

Wt. of Hammer, lbs. [illegible]

Height of Drop [illegible]

Material Used [illegible]

F-10 GRADATION ANALYSIS WITH HYDROMETER #[illegible]

WORKSHEET

TECHNICIAN: [illegible] PROJECT NO: [illegible]
APPROVED BY: ______ DATE: [illegible]-20

SAMPLE NO. #111
VISUAL DESCRIPTION: Sandy Silty Clay

RUN BY ______

SAMPLE PREPERATION

SIEVING TIME ______

SIEVE SIZE		3"	1 1/2"	3/4"	3/8"	NO.4
OF PAN AND SAMPLE						
WT. OF PAN						
DRY WT. RETAINED						0
DRY WT. PASSING						830.4
% OF TOTAL PASSING						100
					W% =	

SAMPLE WEIGHTS	WET	DRY
TOTAL SAMPLE	861.1	830.4
RETAINED ON NO. 4		
PASSING NO. 4		

RUN BY ______

SIEVE AND HYDROMETER ANALYSIS

SIEVING TIME ______

FACTOR = W% / W = ______ = ______

SIEVE NO.	WEIGHT RETAINED	WEIGHT PASSING	% OF TOTAL PASSING
8 (10)	3.0 / [illegible].3	8[illegible] / [illegible]261	100 / 100
16	3.0	32[illegible].8	99.0
30 (40)	[illegible].3 / [illegible]	[illegible] / [illegible]	96.0 / 98
50	[illegible]	[illegible]	97.
100	[illegible]4.0	[illegible]	94.
200	323.3	[illegible]	60.0
PAN			—
TOTAL			—

MOISTURE DETERMINATION

	+4 MATERIAL	-4 MATERIAL	HYGRO. MOISTURE	HYDRO. SAMPLE
DISH NO.				
WT. WET SOIL AND DISH				
WT. DRY SOIL AND DISH				
WT. DISH				
WT. OF DRY SOIL		= W		
% MOISTURE				

RUN BY ______

HYDROMETER ANALYSIS

CYLINDER NO. ______ SPECIFIC GRAVITY ______ DISPERSING AGENT ______
DISH NO. ______ DATE ______ AMOUNT ______ ml DATE CALIB. ______

CLOCK TIME	TEST TIME	TEMP. C°	HYD. READ	HYD.* CORR	CORR READ	FACTOR X CORRECTED READING = % OF TOTAL PASSING	% OF TOTAL PASSING	PARTICLE DIAMETER	
	START MIX	—	—	—	—		—	—	
	STOP MIX	—	—	—	—		—	—	
	0.5 min							0.050 mm	
	1.0 min							0.037 mm	
	4.0 min							0.019 mm	
	19 min							0.009 mm	
	60 min							0.005 mm	
	7h 15 min							0.002 mm	
	25h 45 min							0.001 mm	

GRAVEL ______ % SAND ______ % CLAY-SILT ______ % STORAGE LOCATION ______

* CORRECTION INCLUDES TEMP., MENISCUS, AND DEFLOCULENT

#111

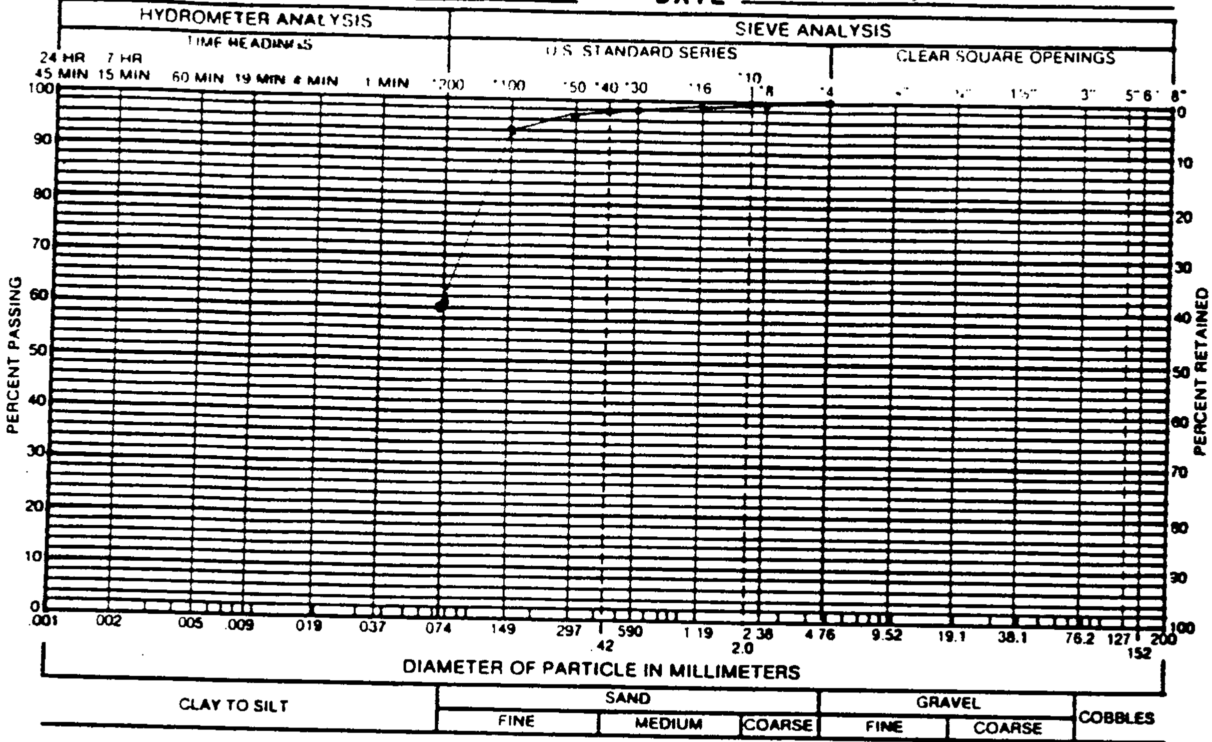
F-12 GRADATION TEST RESULTS

TECHNICIAN H K[illegible]

PROJECT NO. C-4-A-[illegible]

APPROVED BY

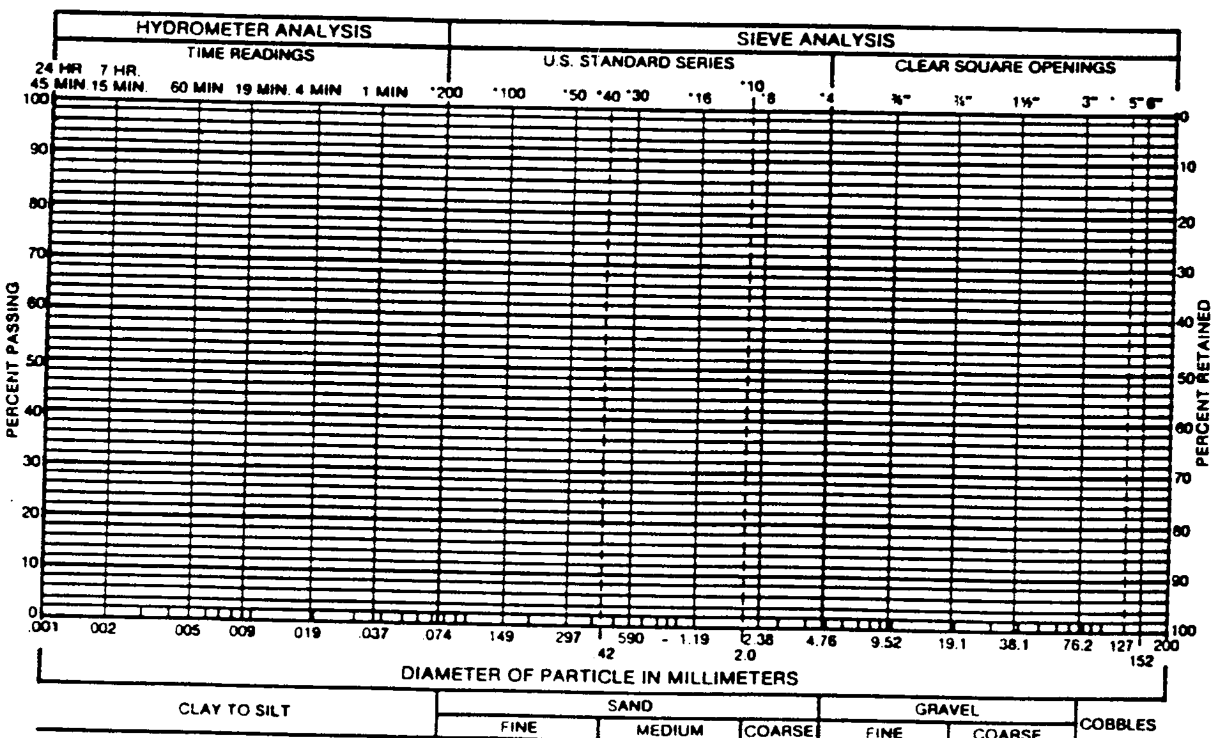
DATE 11-7-89



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT 25 % PLASTICITY INDEX 7 %

SAMPLE OF FROM



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF FROM

F-14 ATTERBERG, -200, MOISTURE & DENSITY
WORKSHEET

TECHNICIAN: [illegible]	PROJECT NO. [illegible]
APPROVED BY	DATE 11-7-89

SAMPLE NO. # [illegible]

SAMPLE DESCRIPTION Sandy silty Clay COLOR Red

ATTERBERG LIMITS

PREP. DISH ______ RUN BY ______

	PL	LL
NO. OF BLOWS	—	30
DISH NO.	17	[illegible]
WT. OF WET SOIL & DISH	10.23	15.56
WT. OF DRY SOIL & DISH	[illegible]	13.[illegible]
WT. OF DISH	7.66	11.[illegible]
WT. OF WATER	.39	[illegible]
WT. OF DRY SOIL	2.22	[illegible]
WATER CONTENT	13	26.

LIQUID LIMIT, LL 25

PLASTIC INDEX, PI 7

-200

RUN BY ______

DISH NO.	
WT. OF DISH & DRY SOIL	
WT. OF DISH & WASHED SOIL	
WT. OF DISH	
WT. OF -200	
WT. OF TOTAL SOIL, DRY	

PERCENT -200 ______ %

MOISTURE CONTENT

RUN BY ______

DISH NO.	
WT. OF DISH & WET SOIL	566.[illegible]
WT. OF DISH & DRY SOIL	[illegible]
WT. OF DISH	—
WT. OF WATER	[illegible]
WT. OF DRY SOIL	[illegible]

MOISTURE CONTENT 3.7 %

DENSITY

RUN BY ______

LENGTH	
DIAMETER	
VOLUME	
WT. OF WET SOIL	
WT. OF DRY SOIL	

DRY DENSITY ______ PCF

REMARKS: ______

11/7/89

Berm N. of Plant Proctor - 110.2 at 15.6%

	Density	% Moist	% Compaction
W. End	99.6	7.6	90%
	99.7	7.1	90%
	100.2	6.9	91%

distribution. The procedure for determining which of many submitted samples should be tested is in itself conducive to obtaining a representative range of values, since samples were selected from the coarsest, finest, and average soil within a potential source.

For each soil property listed, the average and its 90 percent confidence limits are given where sufficient data were available to determine them. Since all laboratory tests, except large-sized permeability tests, were made on the minus No. 4 fraction of the soil, data on average values for the gravels are not available for most properties. However, an indication as to whether these average values will be greater than or less than the average values for the corresponding sand group is given in the table. The averages shown are subject to uncertainties that arise from sampling fluctuations, and they tend to vary from the true averages more widely if the number of observations is small. The plus or minus limits given are determined mathematically from the number of observations and from the standard deviation of the data used to determine the average. These limits imply that the true average, obtained by securing and testing more and more samples under the same essential conditions, lies within the plus or minus values 9 chances out of 10 [4].

The values for Proctor maximum dry density and optimum water content were obtained by tests described in section 120. The other properties are based on tests made on samples compacted to Proctor maximum dry density at optimum water content. The value of void ratio, e_o, is the ratio of the portion of the volume of the soil mass occupied by water and air to the volume of the soil grains. It is derived from the Proctor maximum dry density and the specific gravity of the grains. The MH and CH soil groups have no upper boundary of liquid limits in the classification; hence, it is necessary to give the range of those soils included in the table. The maximum liquid limits for the MH and the CH soils tested were 81 and 88 percent, respectively. Soils with higher liquid limits than these will have inferior engineering properties.

(b) *Permeability.*—The voids in the soil mass provide passages through which water may move. Such passages are variable in size and the paths of flow are tortuous and interconnected. If, however, a sufficiently large number of paths of flow are considered as acting together, an average rate of flow for the soil mass can be determined under controlled conditions that will represent a property of the

TABLE 8.—*Average properties of soils*

Soil classification group	Proctor compaction		Void ratio, e_o	Permeability, k, feet per year	Compressibility		Shearing strength		
	Maximum dry density in pounds per cubic foot	Optimum water content, percent			@ 20 p.s.i., percent	@ 50 p.s.i., percent	C_o p.s.i.	C_{sat} p.s.i.	tan ϕ
GW	>119	<13.3	(*)	27,000±13,000	<1.4	(*)	(*)	(*)	>0.79
GP	>110	<12.4	(*)	64,000±34,000	<0.8	(*)	(*)	(*)	>0.74
GM	>114	<14.5	(*)	>0.3	<1.2	<3.0	(*)	(*)	>0.67
GC	>115	<14.7	(*)	>0.3	<1.2	<2.4	(*)	(*)	>0.60
SW	119±5	13.3±2.5	0.37±*	(*)	1.4±*	(*)	5.7±0.6	(*)	0.79±0.02
SP	110±2	12.4±1.0	0.50±0.03	>15.0	0.8±0.3	(*)	3.3±0.9	(*)	0.74±0.02
SM	114±1	14.5±0.4	0.48±0.02	7.5±4.8	1.2±0.1	3.0±0.4	7.4±0.9	2.9±1.0	0.67±0.02
SM-SC	119±1	12.8±0.5	0.41±0.02	0.8±0.6	1.4±0.3	2.9±1.0	7.3±3.1	2.1±0.8	0.66±0.07
SC	115±1	14.7±0.4	0.48±0.01	0.3±0.2	1.2±0.2	2.4±0.5	10.9±2.2	1.6±0.9	0.60±0.07
ML	103±1	19.2±0.7	0.63±0.02	0.59±0.23	1.5±0.2	2.6±0.3	9.7±1.5	1.3±*	0.62±0.04
ML-CL	109±2	16.8±0.7	0.54±0.03	0.13±0.07	1.0±0.2	2.2±0.0	9.2±2.4	3.2±*	0.62±0.06
CL	108±1	17.3±0.3	0.56±0.01	0.08±0.03	1.4±0.2	2.6±0.4	12.6±1.5	1.9±0.3	0.54±0.04
OL	(*)	(*)	(*)	(*)	(*)	(*)	(*)	(*)	(*)
MH	82±4	36.3±3.2	1.15±0.12	0.16±0.10	2.0±1.2	3.8±0.8	10.5±4.3	2.9±1.3	0.47±0.05
CH	94±2	25.5±1.2	0.80±0.04	0.05±0.05	2.6±1.3	3.9±1.5	14.9±4.9	1.6±0.86	0.35±0.09
OH	(*)	(*)	(*)	(*)	(*)	(*)	(*)	(*)	(*)

The ± entry indicates 90 percent confidence limits of the average value. * Denotes insufficient data, > is greater than, < is less than.

C. SEALY

RI 8564

Bureau of Mines Report of Investigations/1981

Factor of Safety Charts for Estimating the Stability of Saturated and Unsaturated Tailings Pond Embankments

By D. R. Tesarik and P. C. McWilliams

UNITED STATES DEPARTMENT OF THE INTERIOR

the SRC charts and the assumptions accompanying their use. The SRC charts (which appear in the appendixes) provide estimates of stability for technical personnel responsible for judging the stability of soil and tailings embankments in the minerals industries.

SLOPE STABILITY CHARTS

Slope stability charts reduce a multidimensional problem that includes the following listed parameters into a two-dimensional graphic display for quick and easy reference:

F = factor of safety.

γ = unit weight of soil, pounds per cubic foot.

H = height of the embankment, feet.

ϕ' or ϕ = internal friction angle, degrees.[4]

c' or c = cohesion, pounds per square inch (square foot).

$r_u = \frac{u}{\gamma h}$ = pore pressure ratio at a point, where u is the pore pressure at that point and h is the depth of the point below the soil surface.

β = slope of the embankment in degrees, sometimes expressed as increments in the x and y directions respectively; for example, 2:1 = 26.57°.

$D = \frac{L}{H}$ = depth factor where L is the distance from the top of the embankment to the stiff base (fig. 1).[5]

Previous studies have combined several parameters into different forms such as $c'/F\gamma H$ (<u>11</u>), $c/\gamma H\tan\phi$ (2), or $c/\gamma H$ (<u>10</u>). From the available literature reviewed, the chart format presented by Singh (<u>10</u>) for dry embankments is probably the easiest format to use if the factor of safety is desired, given that all other parameters are known (fig 2). The charts in this publication employ a "Singh-like" format for both dry and saturated embankments.

---

[4]The prime (') symbol indicates that the parameter is in terms of effective stress.

[5]Values of F for D = 1.00 and D = 1.50 are in the appendixes. Values of F for D = 1.25 were calculated but have been eliminated from the charts. Investigation of 2,409 values of F showed that values of F for D = 1.50 are more critical (than for D = 1.25) 81 pct of the time. When values of F for D = 1.25 are more critical, the difference is usually in the third decimal place.

for each slice. Further, the value of F was sometimes quite sensitive to small changes in r_u, necessitating the calculation of r_u to the nearest hundredth place for accuracy. This would require a large number of charts with r_u in increments of 0.01 or interpolation by the user.

Because of the above reasons, and in order to elimate the calculation of an average r_u, the authors chose the second alternative--to make γ a fixed parameter and limit the steady-state seepage charts to the condition of 10 percent freeboard.

The method used for generating the curves containing pore pressure is the same as described in the preceding section, except that c'/H is now plotted on the y axis instead of $c'/\gamma H$ since γ is now a fixed parameter. The location of the phreatic surface for input into the Bishop computer code was determined by running a finite-element program. Figure 7 illustrates the phreatic surface for an homogeneous slope of 2.5 to 1.

A natural question is whether linear interpolation is valid for the charts--for example, if $\gamma = 95$, what does one do? Because of the discreteness of the Bishop process, one does not find a smoothness between points. Thus, interpolation should be done with caution, particularlly if F varies significantly from chart to chart.

USING THE SLOPE STABILITY CHARTS

For Embankments With No Phreatic Surface

If the factor of safety is desired for an embankment with no phreatic surface, the following steps are necessary:

1. Calculate $c'/\gamma H$.

2. Determine the slope for the embankment and if the embankment is constructed on a stiff base. If the embankment is on a stiff base, use appendix A. If the stiff base is approximately H/2 feet below the embankment, use appendix B.

3. Locate the appropriate chart by using the information in the upper right-hand corner of the chart.

4. Find where the ordered pair (ϕ', $c'/\gamma H$) intersects the factor of safety contours. This is the critical factor of safety desired.

For Embankments With 10-Pct Freeboard

1. Calculate c'/H.

2. Determine the slope for the embankment, the density of the soil, and if the embankment is constructed on a stiff base. If the embankment is on a stiff base, use appendix C. If the stiff base is approximately H/2 feet below the embankment, use appendix D.

3. Locate the appropriate chart by using the information in the upper right-hand corner of the chart.

4. Find where the ordered pair (ϕ', $c'/\gamma H$) intersects the factor of safety contours. This is the critical factor of safety desired.

Example 1

The following physical parameters are part of the data collected by the Bureau of Mines from West Virginia coal refuse embankments (5).

$$\gamma_{natural} = 90.74 \text{ lb/ft}^3,$$

$$\gamma_{saturated} = 101.12 \text{ lb/ft}^3,$$

$$c' = 3.2 \text{ lb/in}^2,$$

$$\phi' = 33.48°.$$

If an embankment 100 feet high with slope 1.5 to 1 and no phreatic surface is to be constructed on a stiff base, the factor of safety is obtained as follows:

1. $$\frac{c'}{\gamma H} = \left(3.2 \frac{\text{lb}}{\text{in}^2}\right) \cdot \left(144 \frac{\text{in}^2}{\text{ft}^2}\right) \div \left\{\left(90.74 \frac{\text{lb}}{\text{ft}^3}\right) \cdot (100 \text{ ft})\right\} = 0.051.$$

2. The slope is 1.5 to 1, and the embankment is to be constructed on a stiff base.

3. The appropriate chart has 1.5 to 1 in the upper right-hand corner (fig. A-3). The ordered pair (ϕ', $c'/\gamma H$) = (33.48°, 0.051) lies close to the midpoint between the contour lines of F = 1.6 and F = 1.8, yielding a factor of safety of approximately 1.7.

To verify the result, the Bishop program was run for the above case. The result (F = 1.729) verifies the factor of safety obtained via the charts.

Example 2

Suppose an embankment with the same physical properties and of the same geometry as in example 1 is to be constructed; however, a phreatic surface at 10 percent freeboard is anticipated. The following steps are taken:

1. $$\frac{c'}{H} = \left(3.2 \frac{\text{lb}}{\text{in}^2}\right) \cdot \left(144 \frac{\text{in}^2}{\text{ft}^2}\right) \div 100 \text{ ft} = 4.61 \frac{\text{lb}}{\text{ft}^3}.$$

2. [6]The slope is 1.5 to 1, and the density is rounded to 100 lb/ft^3.

[6]A comparative computer run was made using 90.74 lb/ft^3 for density above the phreatic line and 101.12 lb/ft^3 for density below the phreatic line. The factor of safety was 1.205.

APPENDIX A.--STABILITY CHARTS FOR EMBANKMENTS WITH NO PHREATIC SURFACE AND DEPTH FACTOR = 1.00

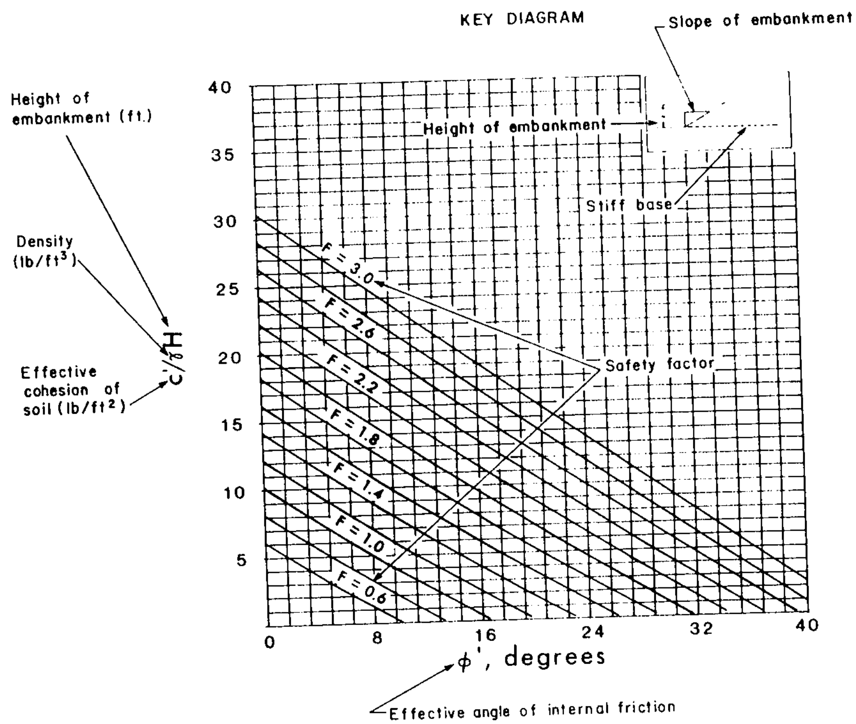


FIGURE A-1. - Stability charts for embankments with no phreatic surface and depth factor = 1.0.

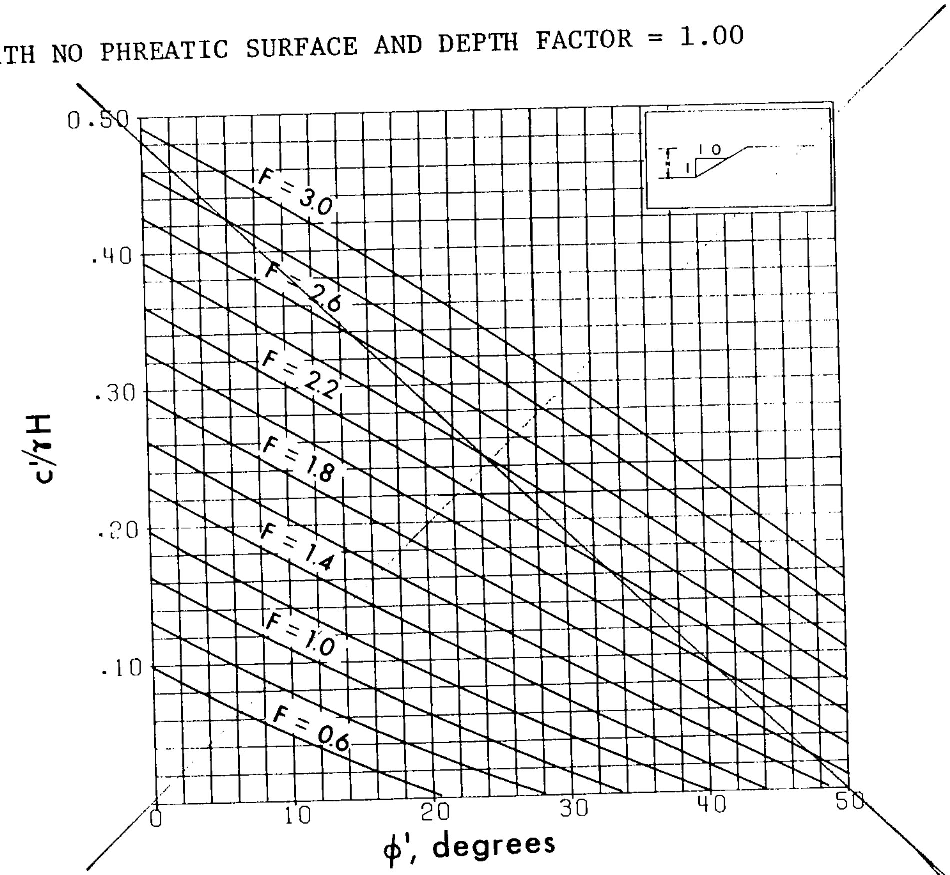


FIGURE A-2. - Factors of safety—1.0:1 slope, no phreatic surface, D = 1.00.

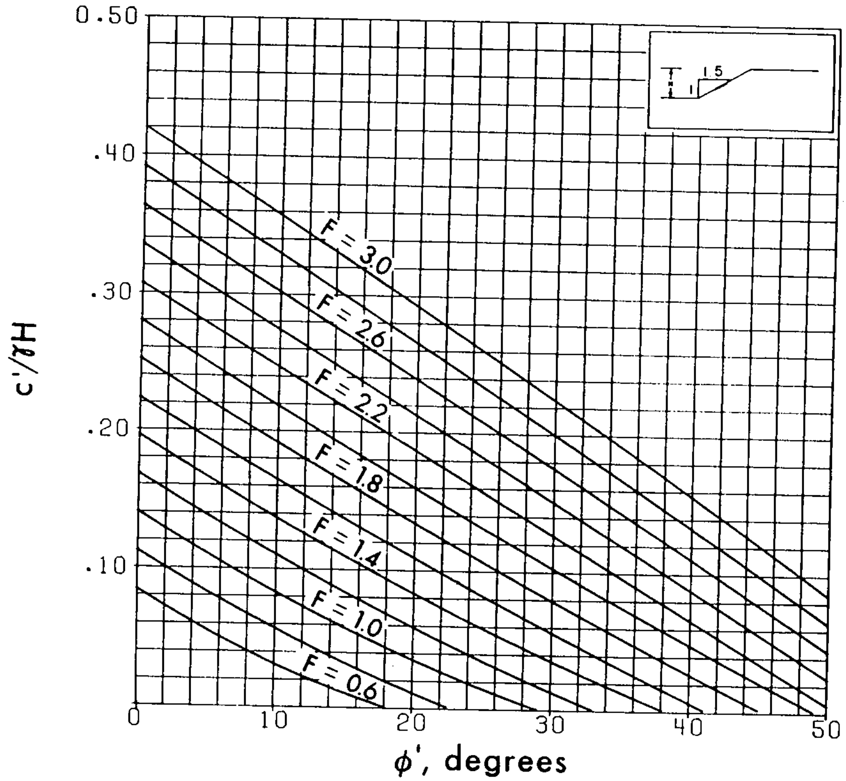


FIGURE A-3. - Factors of safety–1.5:1 slope, no phreatic surface, D = 1.00.

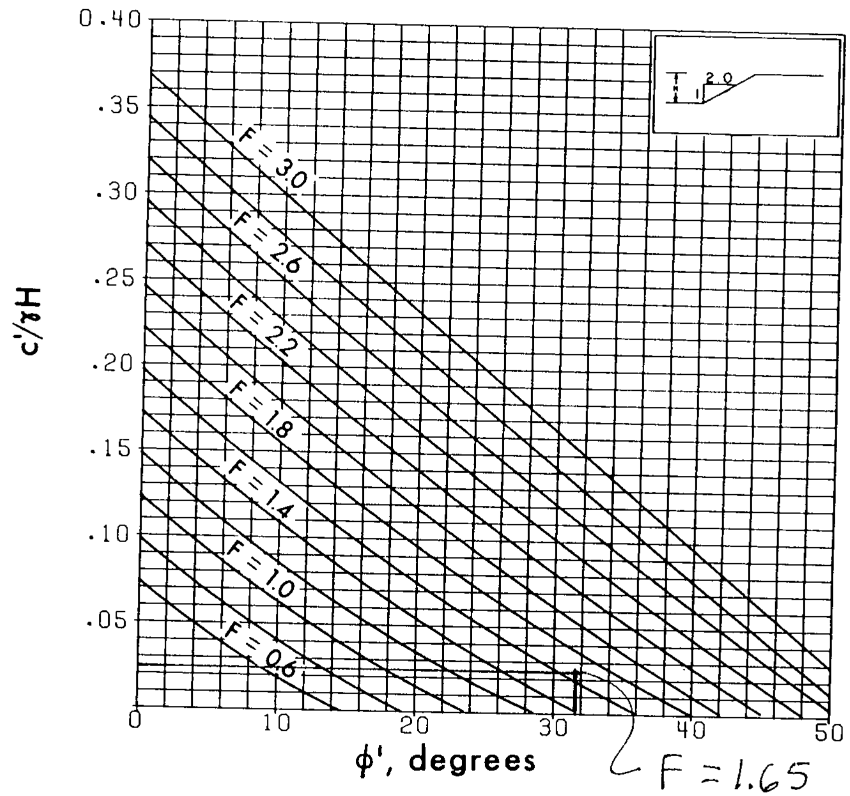


FIGURE A-4. - Factors of safety–2.0:1 slope, no phreatic surface, D = 1.00.

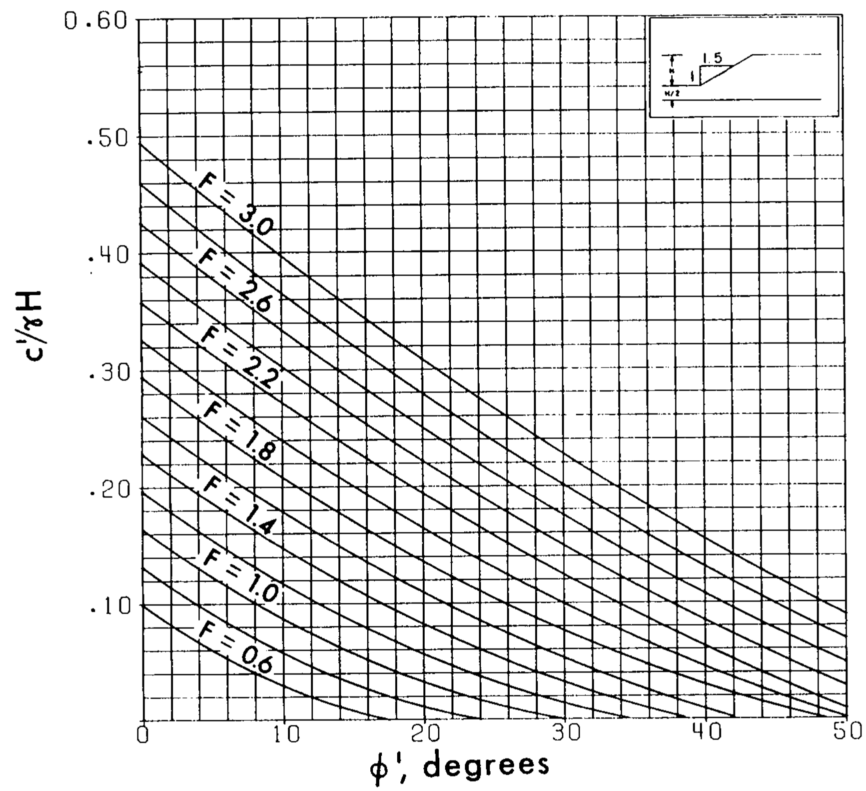


FIGURE B-3. - Factors of safety–1.5:1 slope, no phreatic surface, D = 1.50.

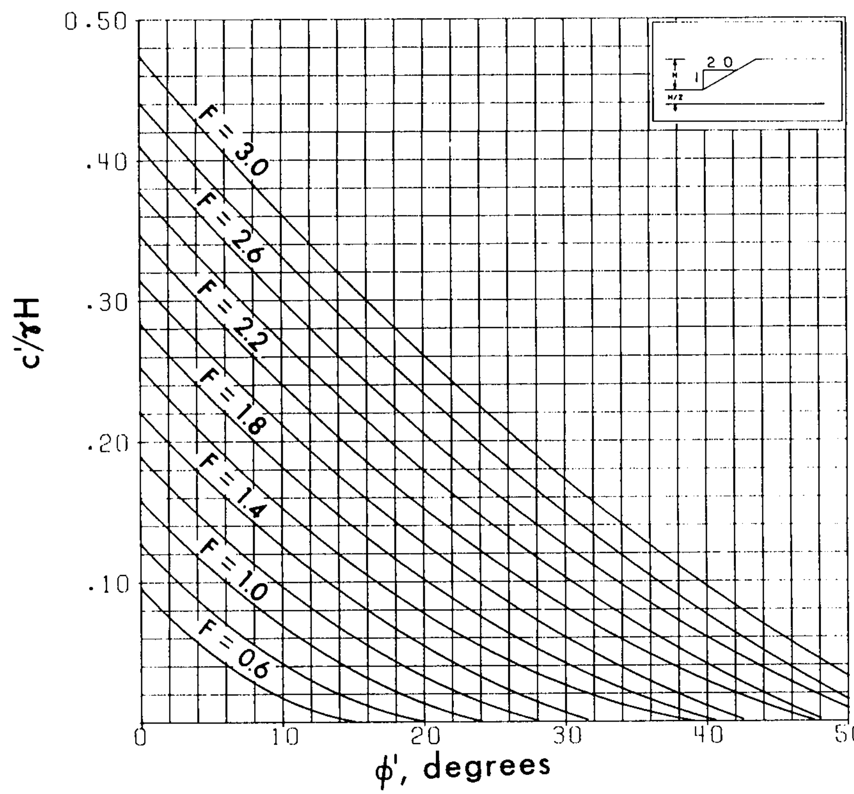


FIGURE B-4. - Factors of safety–2.0:1 slope, no phreatic surface, D = 1.50.

CELL 2 SPILLWAY

Cell 2 Spillway

Prior to completion of the interim cover on Cell 2, a spillway will be constructed from Cell 2 to Cell 3. The spillway will be lined with 40 mil PVC or HDPE liner having a low friction factor. The bottom width is 18 feet wide with 20 foot long transitions on the side slopes and a slope of 0.05 ft/ft. The depth required is 1.2 feet with a freeboard of 1 foot to accommodate a peak flow of 1283 cfs.

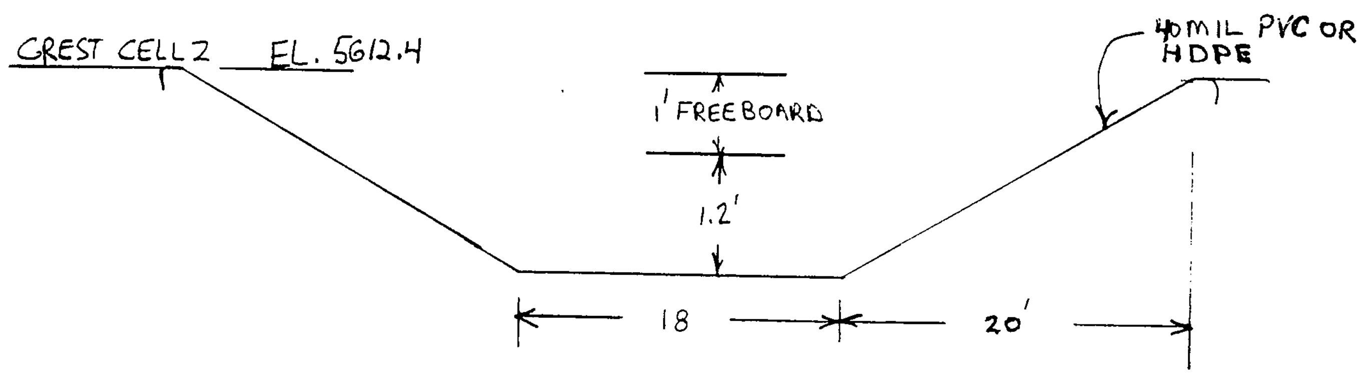
WHITE MESA MILL
SPILLWAY FROM CELL 2 TO CELL 3

PMP (6 HR. STORM)	10 INCHES
CELL 2 DRAINAGE	87 ACRES
CELL 3 DRAINAGE AREA	83 ACRES

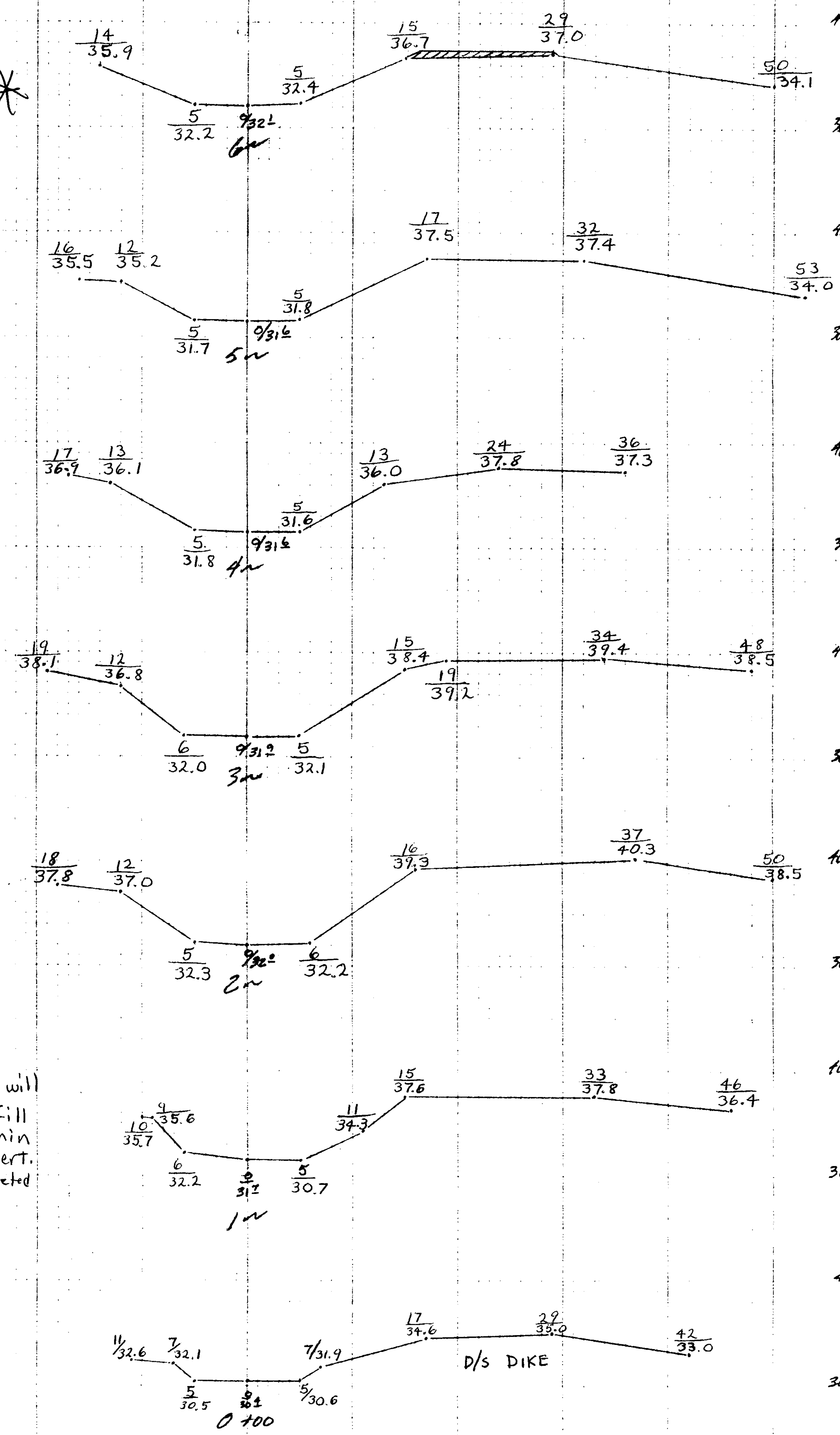
BASED ON TR 55 PROCEDURES THE PEAK RUNOFF FROM CELL 2 TO 3 WILL BE APPROXIMATELY 1283 CFS.

ASSUME THE SPILLWAY IS LINED WITH PVC AND 2 .2 FT BELOW CREST OF CELL 2 .

SLOPE =	.05 FT/FT	Q= 1283 CFS
MANNINGS N=	0.01	



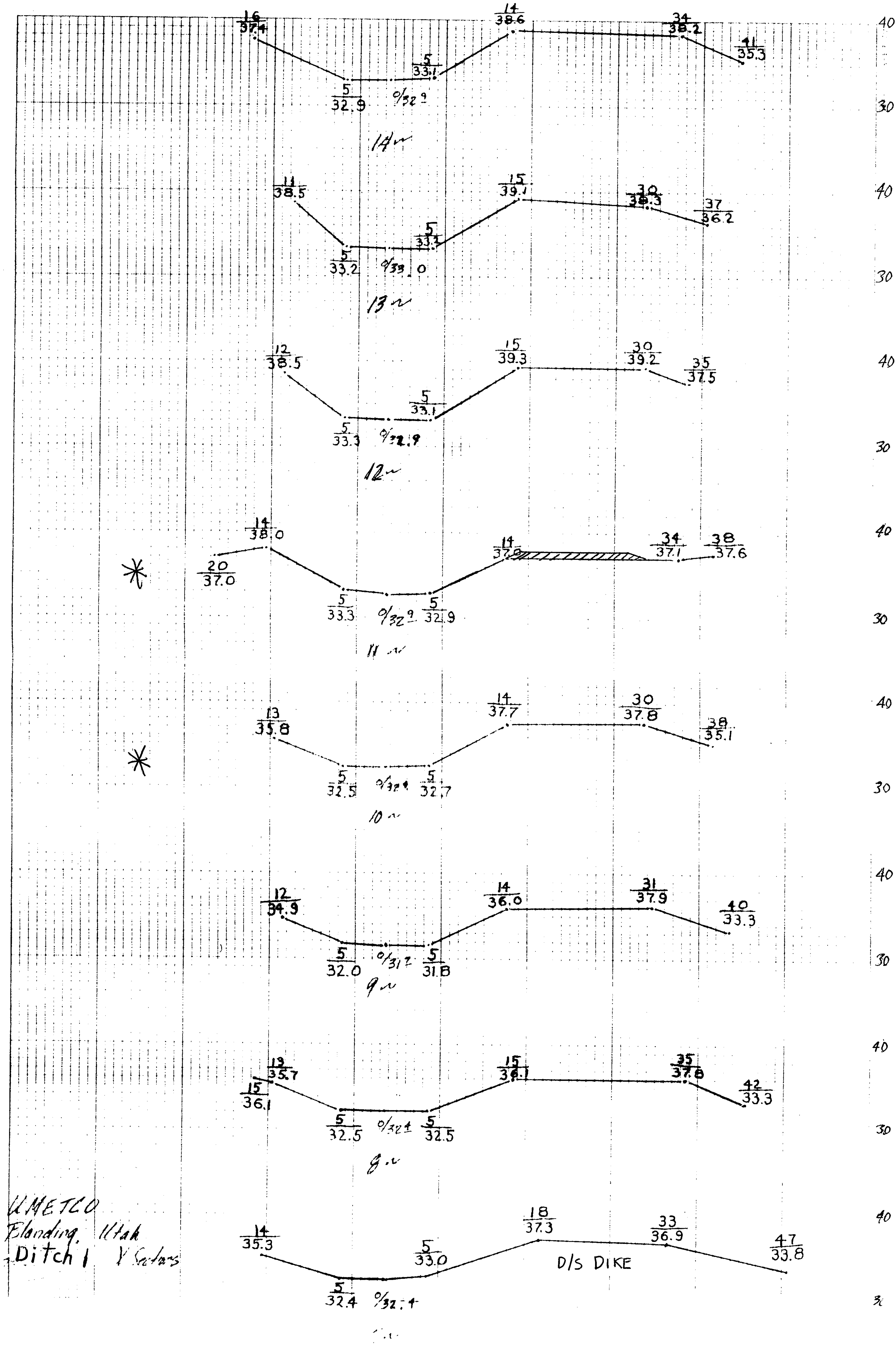
THE CHANNEL CAN BE 18 FT WIDE W/ 20 FT SLOPES AND A DEPTH OF 1.2 FT



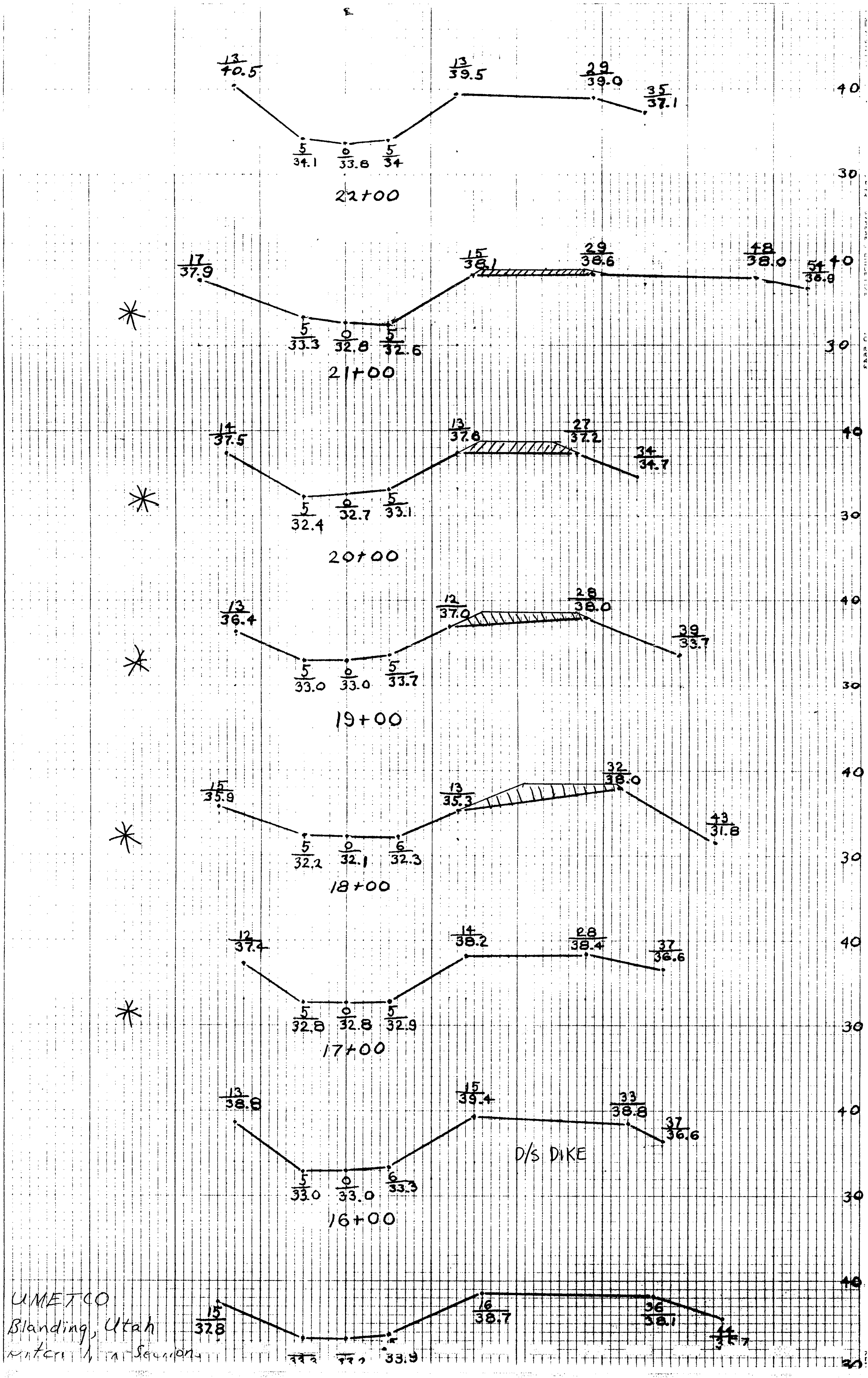
* Sections marked by * will require additional fill to raise the dikes a min 5.1' above design invert. This work will be completed by 3/31/90

UMETCO
Blanding, Utah
DITCH I X-Sections

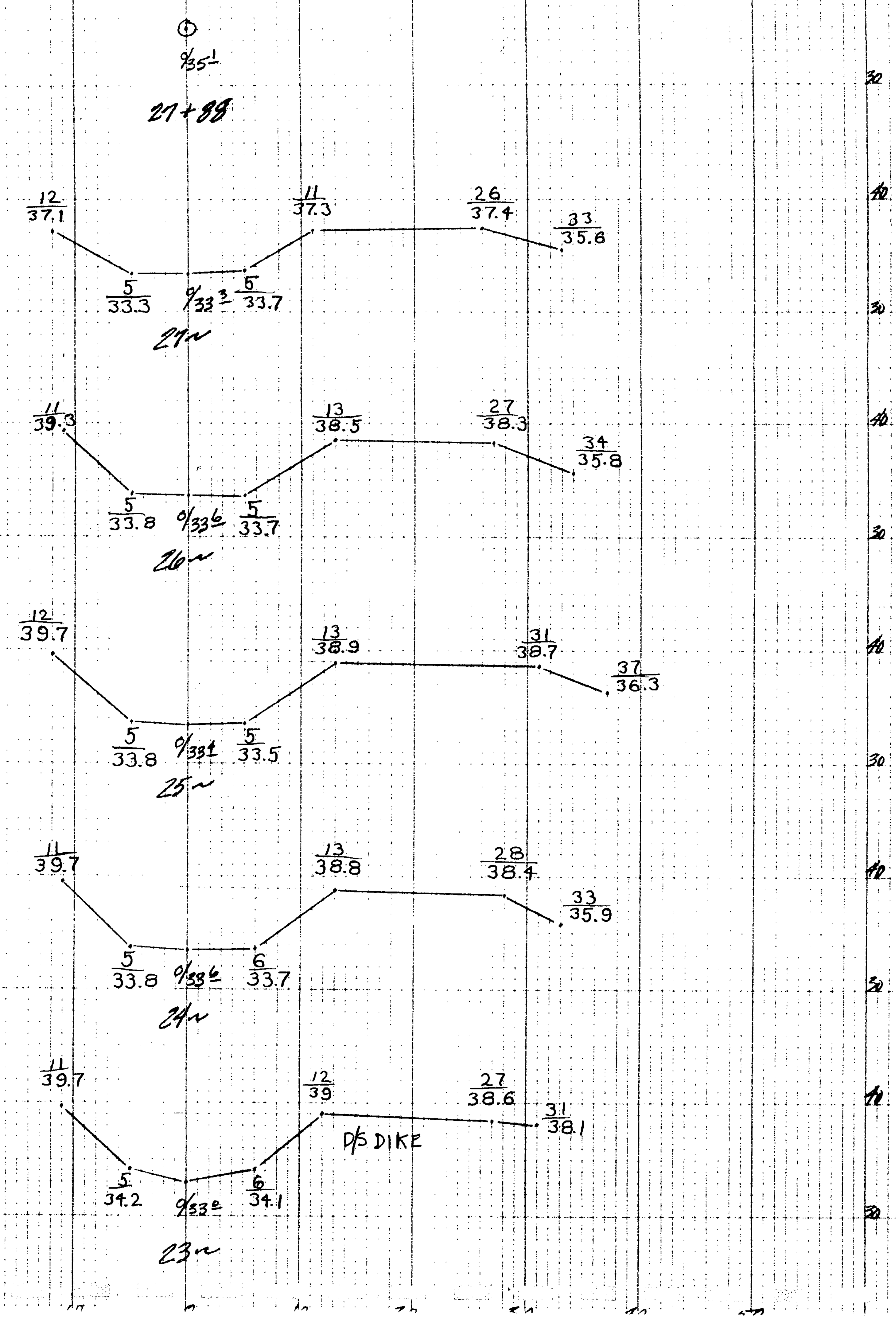
0 - 60 = 0⁰⁰ Section

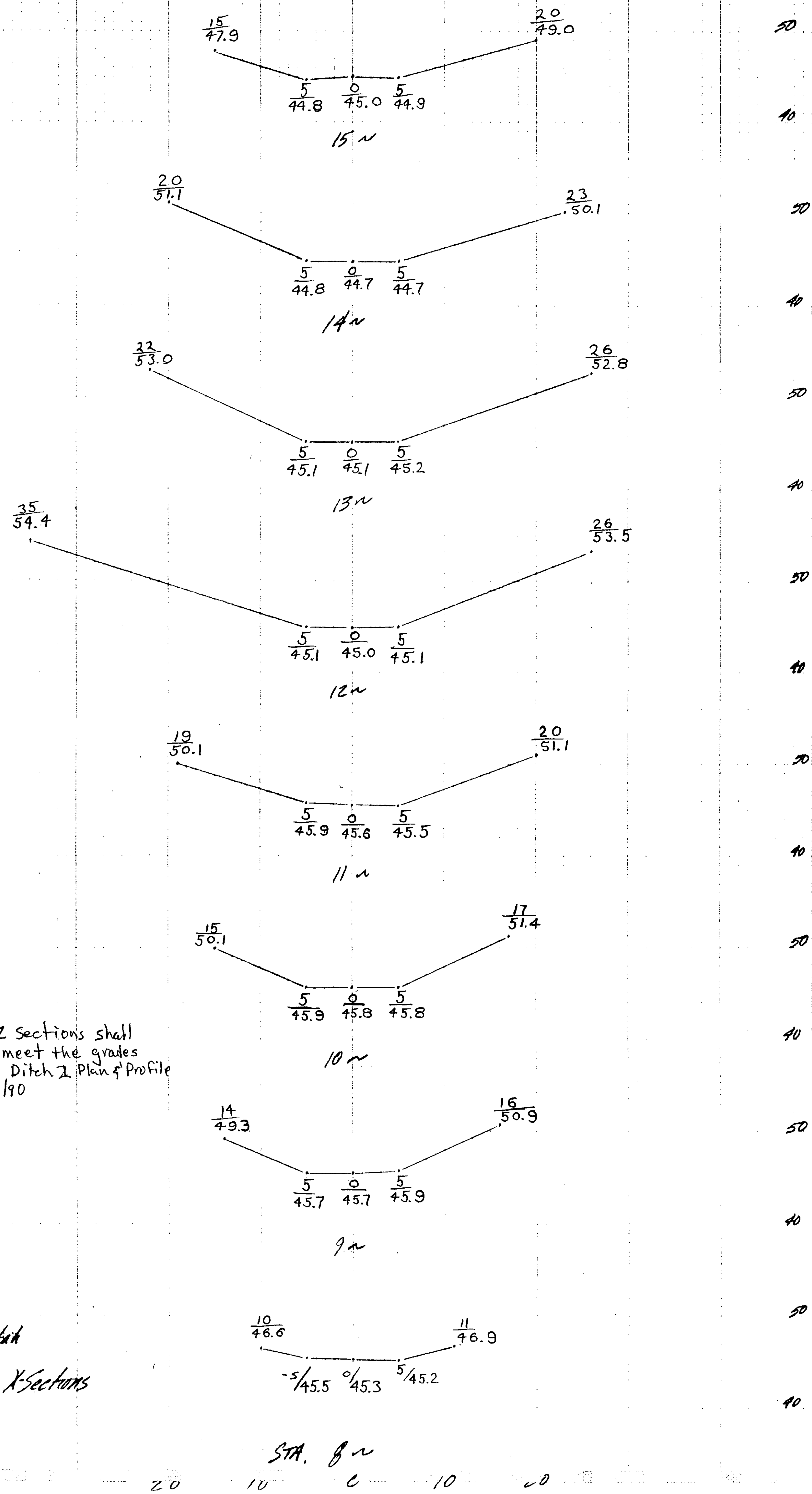


16/37.4
5/32.9
0/32.9
5/33.1
14/38.6
34/38.2
41/35.3
14 ~
11/38.5
5/33.2
0/33.0
5/33.1
15/39.1
30/38.3
37/36.2
13 ~
12/38.5
5/33.3
0/32.9
5/33.1
15/39.3
30/39.2
35/37.5
12 ~
20/37.0
14/38.0
5/33.3
0/32.9
5/32.9
14/37.8
34/37.1
38/37.6
11 ~
13/35.8
5/32.5
0/32.4
5/32.7
14/37.7
30/37.8
38/35.1
10 ~
12/34.9
5/32.0
0/31.7
5/31.8
14/36.0
31/37.9
40/33.3
9 ~
15/36.1
13/35.7
5/32.5
0/32.4
5/32.5
15/36.1
35/37.8
42/33.3
8 ~
UMETCO
Blanding, Utah
Ditch 1
14/35.3
5/32.4
0/32.4
5/33.0
18/37.3
33/36.9
47/33.8
D/S DIKE
40
30


22+00
21+00
20+00
19+00
18+00
17+00
16+00
D/S DIKE
UMETCO
Blanding, Utah

UMETCO
Blanding, Utah
Ditch 1 X-Sections

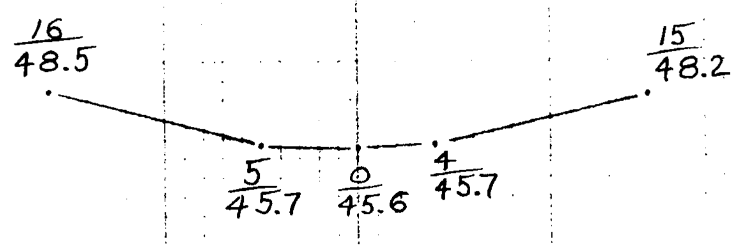




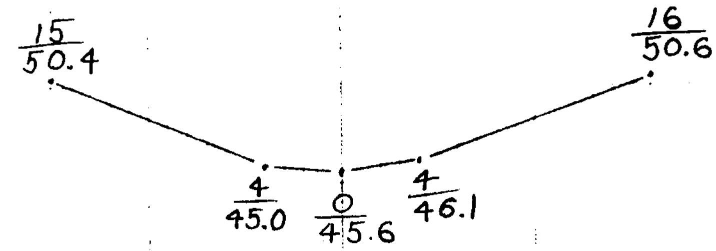
All the ditch 2 sections shall
be lowered to meet the grades
shown on the Ditch 2 Plan & Profile
sheet by 3/31/90

UMETCO
Blanding, Utah
Ditch 2 X-Sections

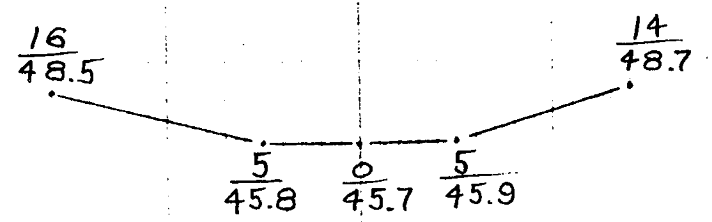
4+ = 0°° Section @ 44?



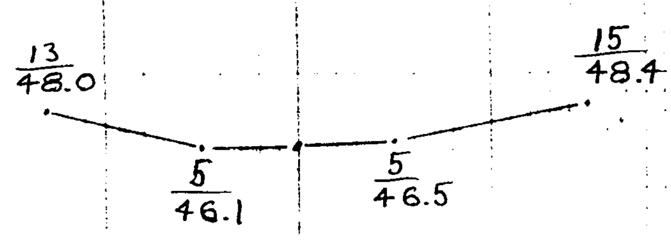
3+



2+



1+

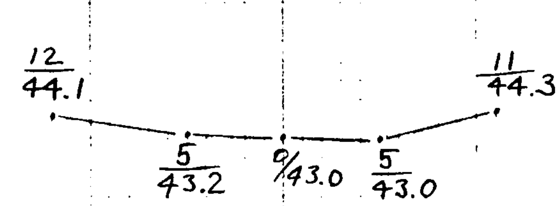


0+60

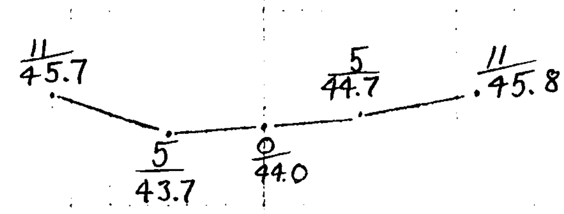
Ditch 3 X-Sections

0+ = 0°° Section @ 46?

19+

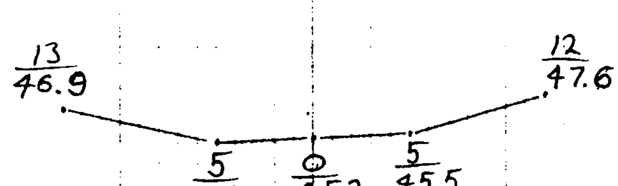


18+

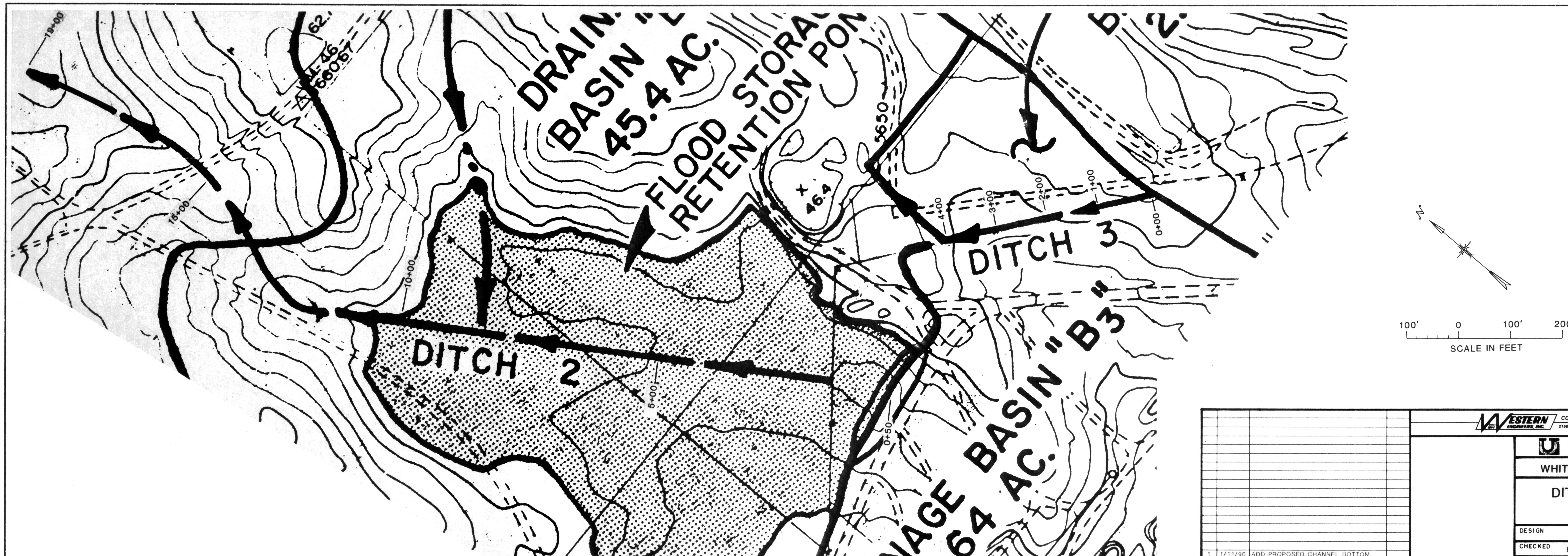


17+

UMETCO
Blending, Utah
Ditch 2 X-Sections



16+



WESTERN ENGINEERS, INC. CONSULTING ENGINEERS/LAND SURVEYORS

Umetco Minerals Corporation

WHITE MESA URANIUM MILL SITE

DITCH 2 & DITCH 3

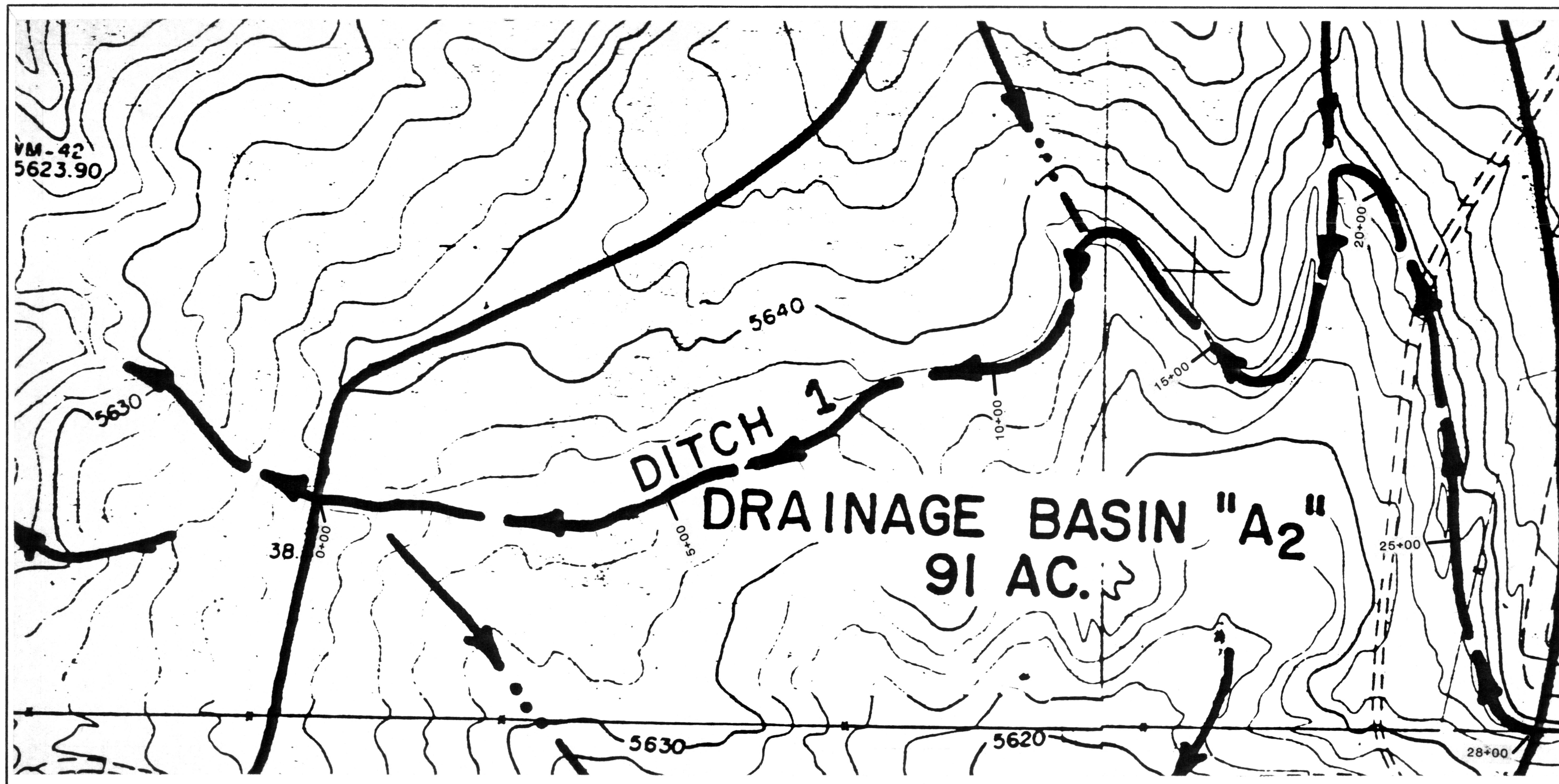
PLAN AND PROFILE

DESIGN	F.L.W.	DRAWN	R.W.Q.	SHEET 2 OF 2
CHECKED	F.L.W.	DATE	10/31/89	
UMETCO APPROVAL				

NO.	DATE	REVISION - DESCRIPTION	BY	W.E.I. DWG.
1	1/11/90	ADD PROPOSED CHANNEL BOTTOM		

PLAN: NOTE BOOK, No., SURVEYED, PLOTTED, ALIGNMENT CHECKED, RT. OF WAY CHECKED, BY, DATE

PROFILE: NOTE BOOK, No., SURVEYED, PLOTTED, GRADES CHECKED, B. M.'s NOTED, STRUCTURE NOTATIONS CHECKED, BY, DATE



1	1/11/90	ADD DIKE NOTES	
NO.	DATE	REVISION - DESCRIPTION	BY

WESTERN ENGINEERS, INC. CONSULTING ENGINEERS/LAND SURVEYORS

Umetco Minerals Corporation

WHITE MESA URANIUM MILL SITE

DITCH 1

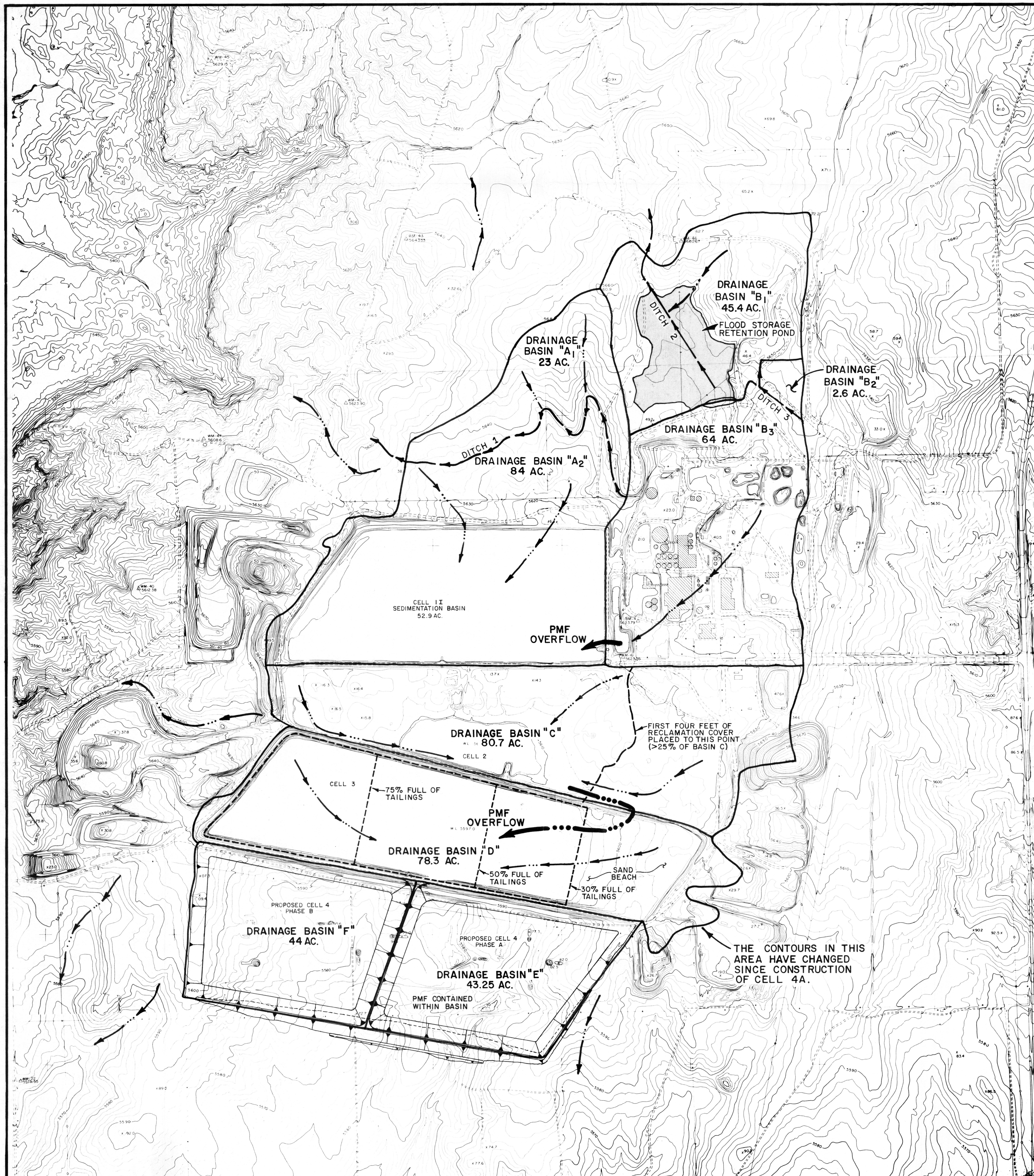
PLAN AND PROFILE

DESIGN	F.L.W.	DRAWN	R.W.Q.	SHEET 1 OF 2
CHECKED	F.L.W.	DATE	10/31/89	
UMETCO APPROVAL				

W.E.I. DWG.

PLAN — NOTE BOOK — No. — SURVEYED — PLOTTED — ALIGNMENT CHECKED — R/W CHECKED — BY — DATE

PROFILE — NOTE BOOK — No. — SURVEYED — PLOTTED — GRADES CHECKED — B. M.'s NOTED — STRUCTURE NOTATIONS CHECKED — BY — DATE



HYDROLOGY SUMAMRY

6-Hour Thunderstorm PMP [1] = 10.0 Inches
6-Hour General Storm PMP [1] = 5.63 Inches

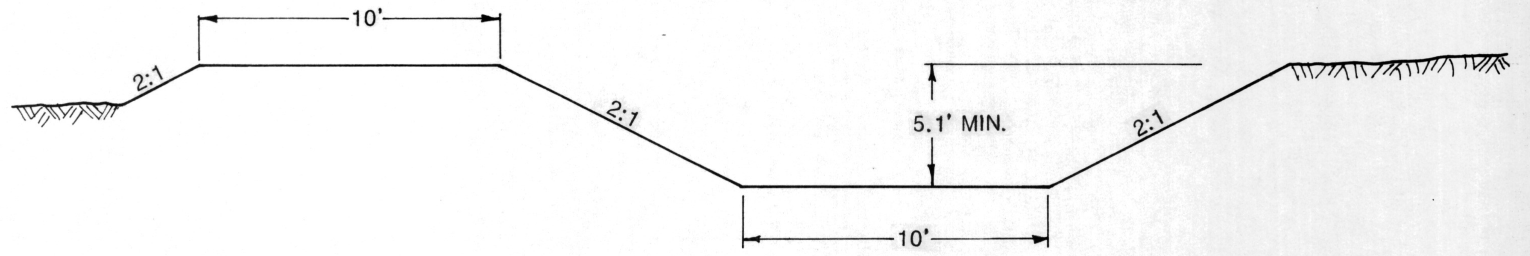
BASIN	TRIBUTARY AREA (Acres)
A_1	23
A_2	84
B_1	45.4
B_2	2.6
B_3	64
C	80.7
D	78.3
E	43.25
F [2]	44

1) PMP from NWS Hydrometeorological Report 49. 6-Hour thunderstorm used for design.

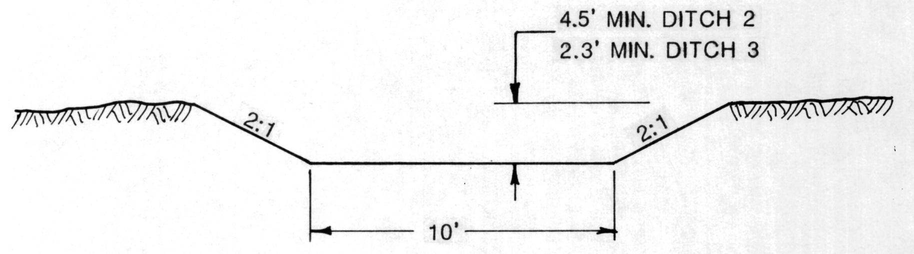
2) Cell 4 - Phase B to be constructed at future date.

CELL 3

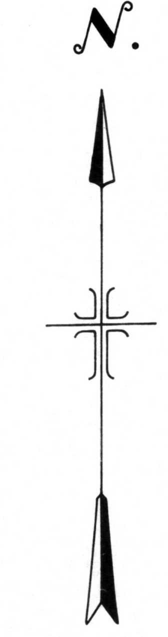
SURFACE AREA AVAILABLE FOR FLOOD STORAGE	PERCENTAGE OF CELL 3 FILLED WITH TAILINGS
51.2 Acres	35%
37.4 Acres	50%
18.1 Acres	75%



DITCH 1
TYPICAL SECTION
STA. 9+00



DITCH 2 & 3
TYPICAL SECTION



400' 200' 0 400' 800' 1200'
SCALE IN FEET
CONTOUR INTERVAL - 2 FEET

NO.	DATE	REVISION - DESCRIPTION	BY
4	1/8/90	Change Ditch 1, 2, & 3 Typ. Sections	
3	12/20/89	Add Notes Cell 2 & Cell 3	
2	11/10/89	Update Drainage Basins	
1	10/27/89	Addition of Ditches	

W.E.I. DWG.

WESTERN ENGINEERS, INC. CONSULTING ENGINEERS/LAND SURVEYORS
2150 Hwy. 6 & 50, Grand Junction, CO 81505 • 303/242-5202

Umetco Minerals Corporation

WHITE MESA URANIUM MILL SITE

CELL 4
REVISED DRAINAGE MAP

DESIGN	DRAWN D.R.H.	SHEET
CHECKED	DATE 3/23/89	C4-6
UMETCO APPROVAL		