Technical Memorandum

DAMBREAK STUDY FOR GUADALUPE FLOOD RETARDING STRUCTURE

Submitted to

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FCD 88-65

E124034

APRIL 1990

Greiner

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SCALE: 1"= 600'

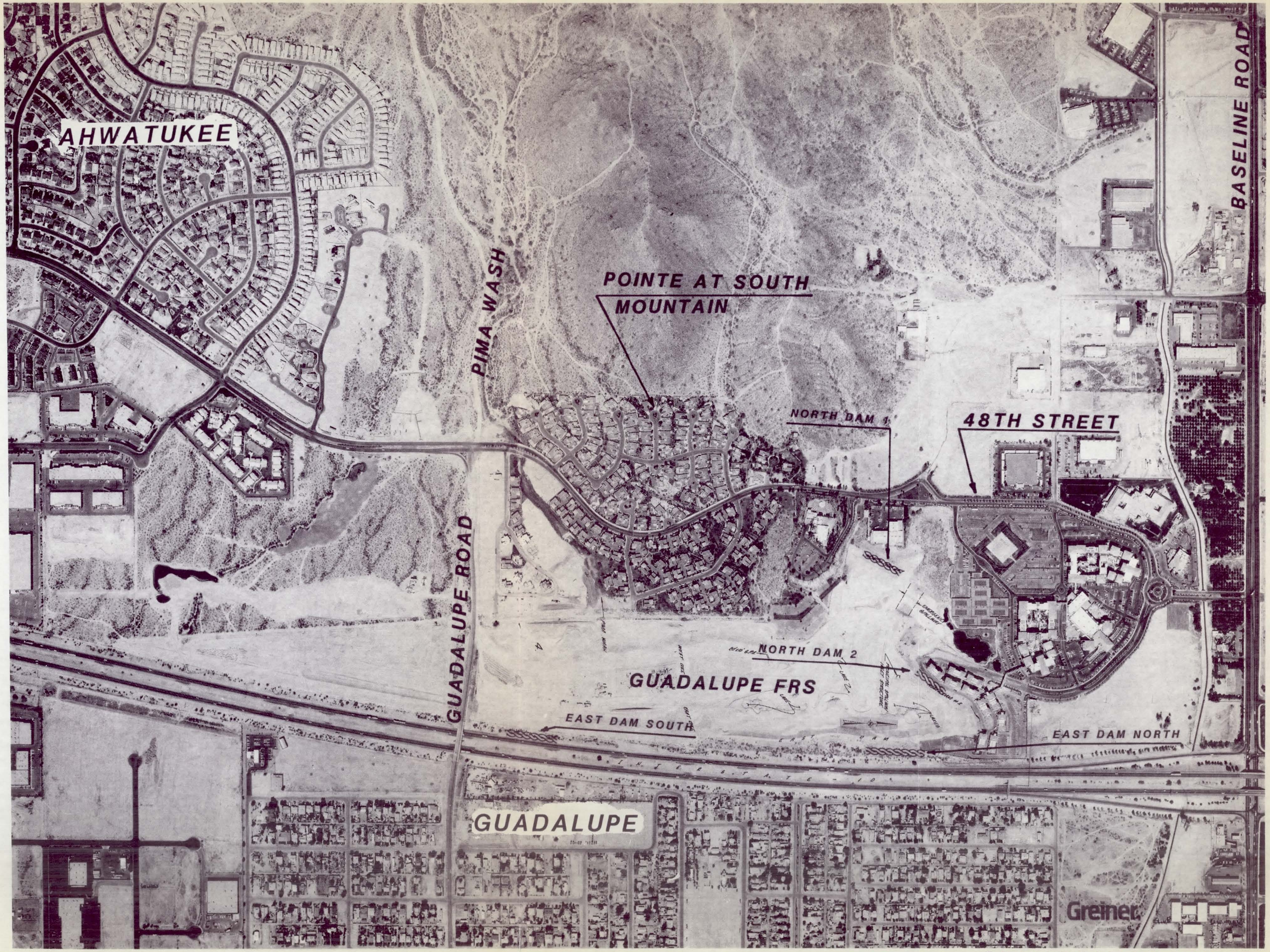


FIGURE 2 SITE LOCATION MAP

1. INTRODUCTION

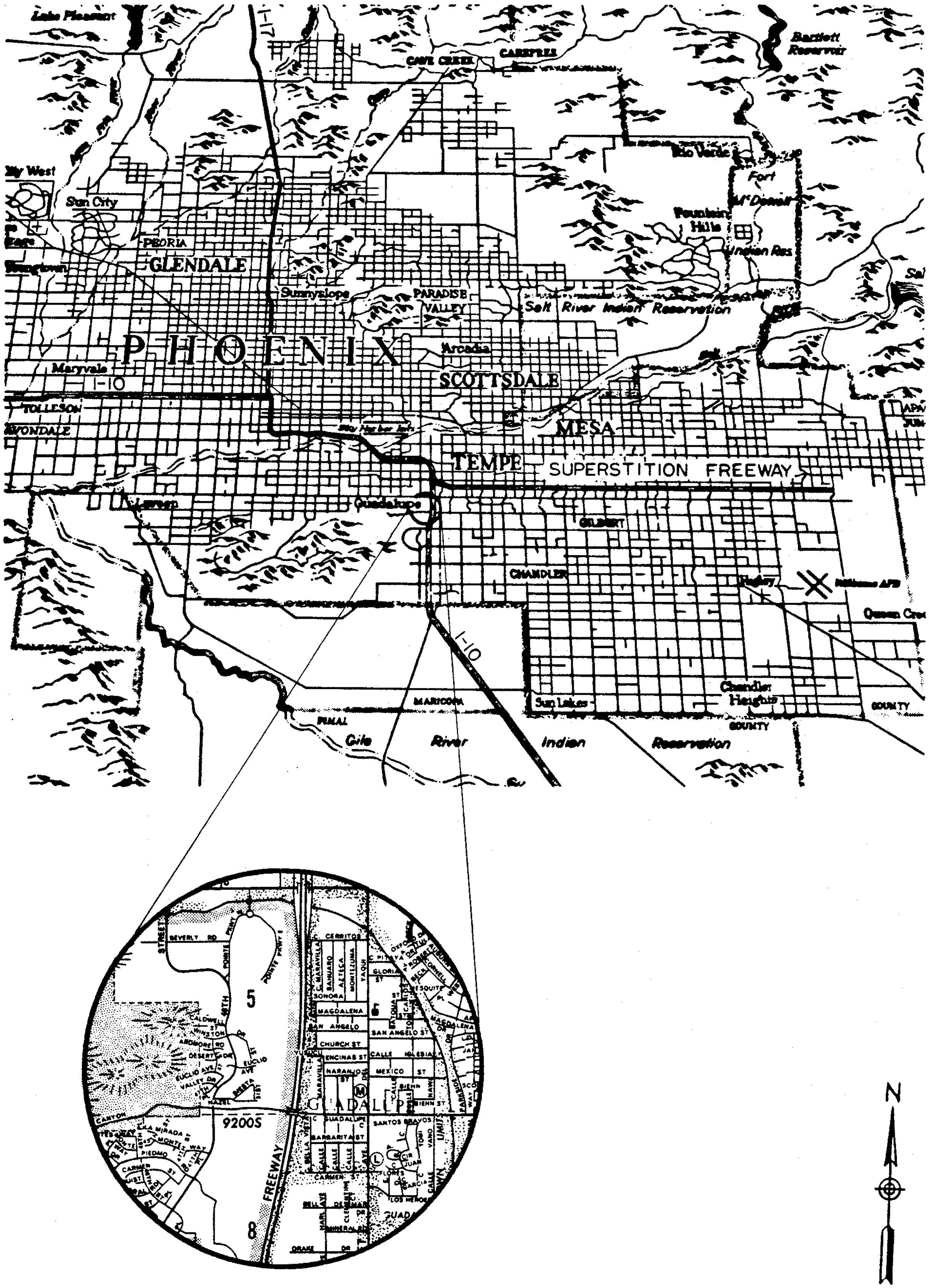
This Technical Memorandum is prepared to document all the methodology and procedures used by Greiner, Inc. for the dambreak study of the Guadalupe Flood Retarding Structure. Information concerning inundation area, structural inventory and damage cost due to dam failure is provided in the main study report, "Dambreak Study Report for Guadalupe Flood Retarding Structure," dated March 1990.

The Guadalupe Flood Retarding Structure (GFRS) is a small flood control structure located midway between Baseline Road and Guadalupe Road just west of Interstate Highway 10 (I-10), Phoenix, Arizona (Figure 1). The GFRS consists of three dams, North Dam No. 1 (Saddledike section), North Dam No. 2 (center section) and East Dam (I-10 section). An emergency overflow spillway, approximately 200 feet wide and with crest at elevation 1,274 feet, is located between North Dam No. 1 and the Pointe Parkway. The principal spillway is a 30-inch controlled low level outlet pipe structure extended through North Dam No. 2. The pipe conveys floodwater released from GFRS to the Western Canal, which is located approximately 5,470 feet to the north. The upstream inlet consists of an ungated standpipe with a gated inlet at the bottom. The site location map is shown in Figure 2.

The GFRS was constructed in 1975 by the Soil Conservation Service (SCS) to reduce the potential for floodwater and sediment damage from the 100-year flood to the community of Guadalupe and the surrounding portions of the Cities of Phoenix and Tempe. Significant flood damages had been incurred as recently as 1969 (Reference 1).

The GFRS was originally constructed on state-owned land. As the local sponsor, the Flood Control District of Maricopa County (FCD) obtained a perpetual easement from the state for the purpose of maintaining the structure. In 1981, Gosnell Development Corporation, developer of the Pointe at South Mountain, obtained ownership of the site through land trades with the state. Subsequently, the structure was incorporated into the resort's golf course. The FCD still maintains operational control of the GFRS and is responsible for the structural and functional integrity of the facility. The developer is responsible for maintaining the emergency spillway, erosion control of the embankments and landscaping. As part of its Safety of Dams Program, the Arizona Department of Water Resources (ADWR) requested the FCD to prepare Emergency Action Plans for all dams within its jurisdiction. The plans would include maps, delineating the area that will be inundated should the dams fail, and evacuation plans. The FCD subsequently determined that the dambreak analysis will be performed using the National Weather Services (NWS) DAMBRK computer program. The DAMBRK model was first presented by Daniel Fread, Senior Research Hydrologist of NWS, in 1977 and was updated several times since then. The latest version of the model, DAMBRK-88, was used for this project. This computer model is used to develop an outflow hydrograph from a dam and hydraulically route the flood through the downstream valley.

Greiner, Inc. was contracted by the FCD in August, 1989 to perform this analysis and to prepare the inundation area maps for evacuation use in the event of dam failure.



PROJECT VICINITY MAP

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FIGURE I

2. PURPOSE AND OBJECTIVES

PURPOSE

This study was conducted for the purpose of determining the flooded areas wherein loss of life and property damage could be substantial in the event of dam failure at the GFRS. Inundation maps are developed for the area downstream of the GFRS based on the best available information, hydrologic and hydraulic methodologies, and current ADWR dam safety criteria.

OBJECTIVES

The technical memorandum is prepared to document the technical study procedures in the development of the breach hydrograph and downstream routing utilizing computer models.

The computer simulation models include: 1) the HEC-1 Flood Hydrograph Package computer program developed by the U.S. Army Corps of Engineers for calculating the inflow hydrographs to the GFRS, 2) DAMBRK and, 3) BREACH. Both DAMBRK and BREACH programs were developed by the Hydrologic Research Laboratory, National Weather Service, NOAA, Silver Spring, Maryland.

The project consists of hydrology/hydraulics, dambreak analyses and development of inundation maps. The objectives are:

- Collect and review all available published and unpublished hydrological, geotechnical and as-built information.
- Conduct site investigations to verify existing hydrological and hydraulic data, and research for unfound information.
- Perform hydrological model simulations for the inflow hydrograph.
- Perform sensitivity analysis on various models to obtain representative breach width and time to failure for each dam.
- Complete dambreak analysis by defining breach parameters, selecting breach locations, developing failure outflow hydrographs, and performing downstream routing.
- Prepare inundation maps based on the interpretation of DAMBRK results.
- Estimate the structural inventory within the inundation areas.
- Analyze the economic and social impacts to the downstream areas when any of the dams fail.
- Prepare technical memorandums, hydrological report and dambreak report for this study with supporting tables, graphs and drawings.

3. INFLOW HYDROGRAPH

The GFRS is located at the mouth of Pima Wash between Guadalupe and Baseline Roads and west of the Maricopa Freeway (I-10). The GFRS is grass lined and in use as a golf course.

The tributary watershed is 1.81 square miles. The lower 0.3 square mile (including the GFRS itself) is developed with recreational, resort, commercial and multi-residential development. The remainder of the watershed is undeveloped desert. Drainage subareas are delineated within the watershed and are shown in Figure 3.

The inflow hydrograph for the watershed to the GFRS was calculated by Greiner utilizing the HEC-1 Flood Hydrograph Package computer program, which was developed by the U.S. Army Corps of Engineers.

Six-hour Probable Maximum Precipitation (PMP) was developed per Hydrometeorological Report No. 49. The PMP was calculated using the general method, the average depth method, and areal distribution method. The method that yielded the greatest total storm depth (areal distribution method) was then used. After total PMP inflow into the GFRS was calculated, each hydrograph ordinate was reduced by the specified ratio of 0.5. This yielded the one-half PMP hydrograph that was used as the inflow hydrograph to the reservoir in the dambreak analysis (see Table 1).

The detailed documentation of PMP calculations, its procedures and methodology are included in the "Hydrology Report - Dambreak Study for the Guadalupe Flood Retarding Structure," dated September 1989 (Reference 2).

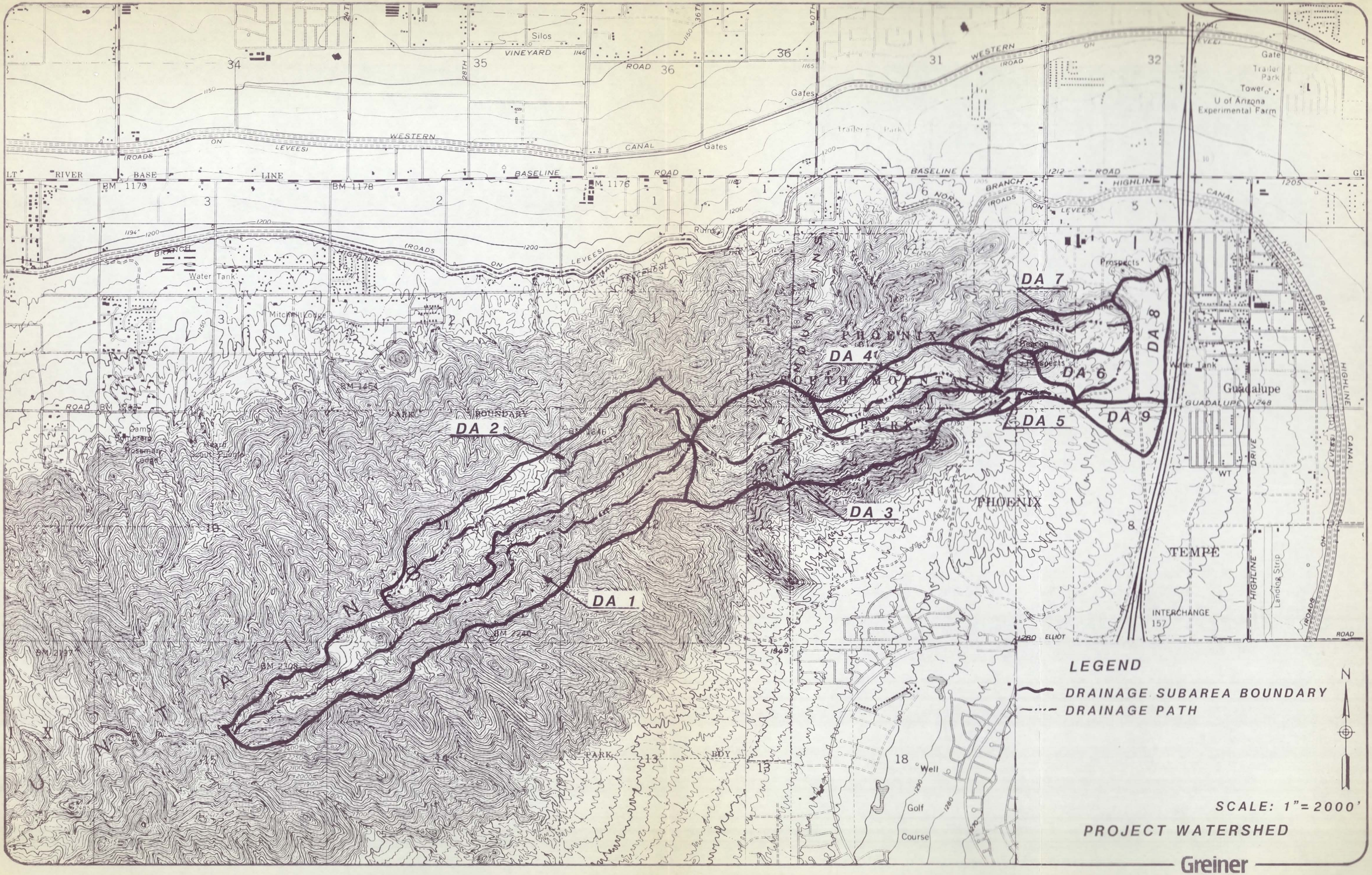

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FIGURE 3

3. INFLOW HYDROGRAPH

Table 1

One-Half PMP Hydrograph

Time (Hour)	Inflow (cfs)	Time (Hour)	Inflow (cfs)	Time (Hour)	Inflow (cfs)
0.00*	0.0	2.53*	4,856.0	4.60*	731.0
0.53	1.0	2.60	5,306.0	4.67	681.0
0.60	1.0	2.67	5,683.0	4.73	633.0
0.67	2.0	2.73	5,939.0	4.80	589.0
0.73	3.0	2.80*	6,039.0	4.87	549.0
0.80	5.0	2.87	5,971.0	4.93	515.0
0.87	8.0	2.93	5,742.0	5.00	486.0
0.93	11.0	3.00	5,390.0	5.07	458.0
1.00	15.0	3.07*	4,956.0	5.13	425.0
1.07	20.0	3.13	4,477.0	5.20	394.0
1.13	27.0	3.20	4,003.0	5.27	368.0
1.20	37.0	3.27	3,562.0	5.33	345.0
1.27	48.0	3.33	3,167.0	5.40	325.0
1.33	60.0	3.40	2,823.0	5.47	304.0
1.40	74.0	3.47	2,529.0	5.53	284.0
1.47	89.0	3.53	2,277.0	5.60	264.0
1.53	106.0	3.60	2,061.0	5.67	245.0
1.60	124.0	3.67	1,877.0	5.73	226.0
1.67	142.0	3.73	1,720.0	5.80	206.0
1.73	161.0	3.80	1,587.0	5.87	194.0
1.80	179.0	3.87*	1,475.0	5.93	181.0
1.87	196.0	3.93	1,382.0	6.00	170.0
1.93	213.0	4.00	1,304.0	6.07	158.0
2.00	228.0	4.07	1,230.0	6.13	144.0
2.07	507.0	4.13	1,146.0	6.20	131.0
2.13*	1,425.0	4.20	1,068.0	6.27	120.0
2.20	2,562.0	4.27	1,001.0	6.33	110.0
2.27	3,600.0	4.33	944.0	6.40	101.0
2.33	4,164.0	4.40	890.0	6.47	91.0
2.40	4,265.0	4.47	837.0	6.53	82.0
2.47	4,456.0	4.53	784.0	6.60*	72.0

*Data used in BREACH/DAMBRK model

4. DAM BREACH ANALYSIS

Selection of Breach Location at the Dam

There are two potential breach locations at the East Dam that will result in different downstream flooding. One breach location was selected near the south portion of the dam so that larger flooded areas will be covered in the event of dam failure. Another breach location is near the north end of the dam where the largest breach flow will occur. The selection of breach locations for the other dams was based on the potential of producing the largest flow. The dams and sections map is depicted in Figure 4. Figures 5 through 8 show the cross sections of the dams at the potential breach locations.

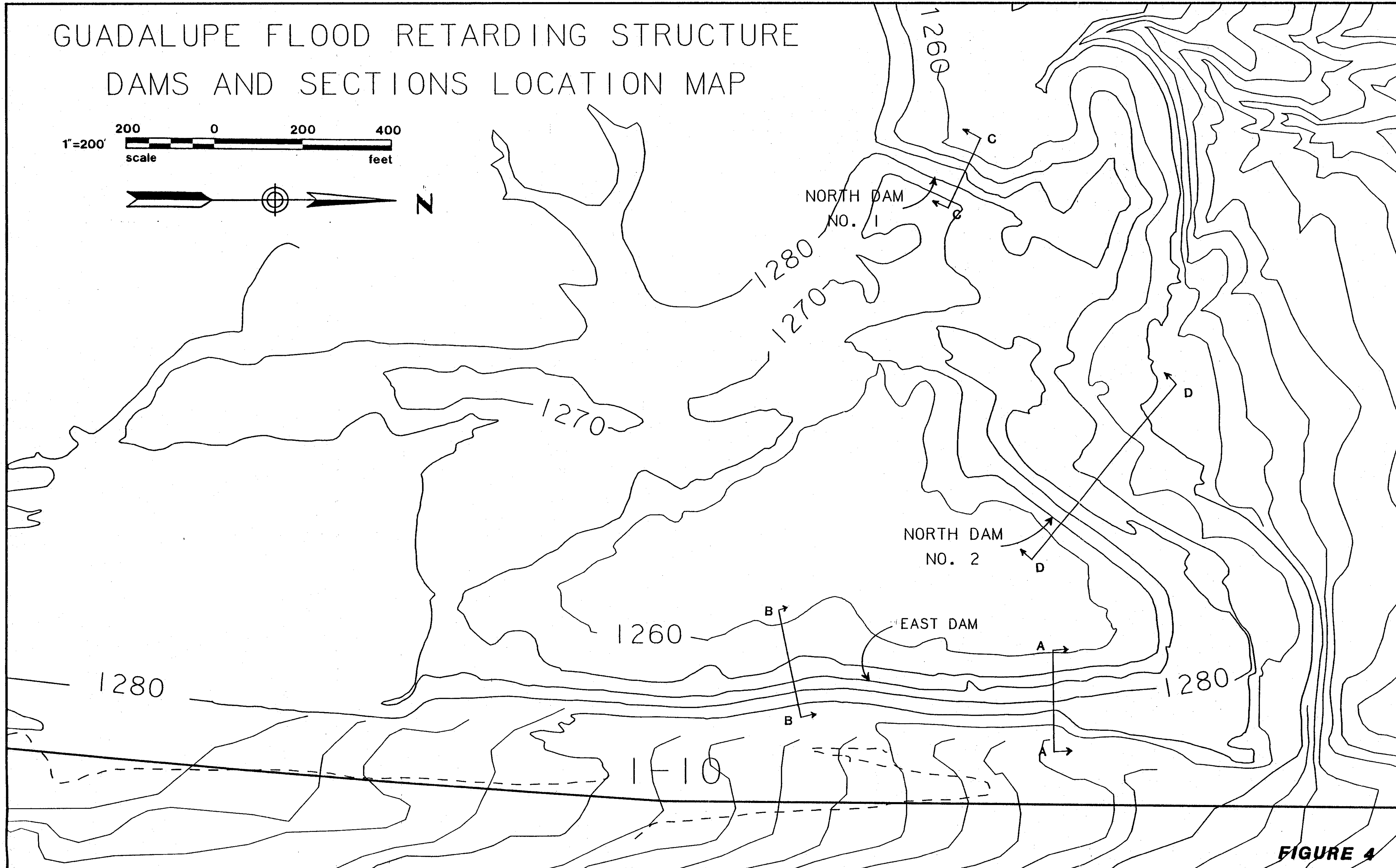
GUADALUPE FLOOD RETARDING STRUCTURE
DAMS AND SECTIONS LOCATION MAP
200 0 200 400
1"=200'
scale
feet
N
NORTH DAM NO. 1
NORTH DAM NO. 2
EAST DAM
A
B
C
D
1260
1270
1280
I-10

FIGURE 4

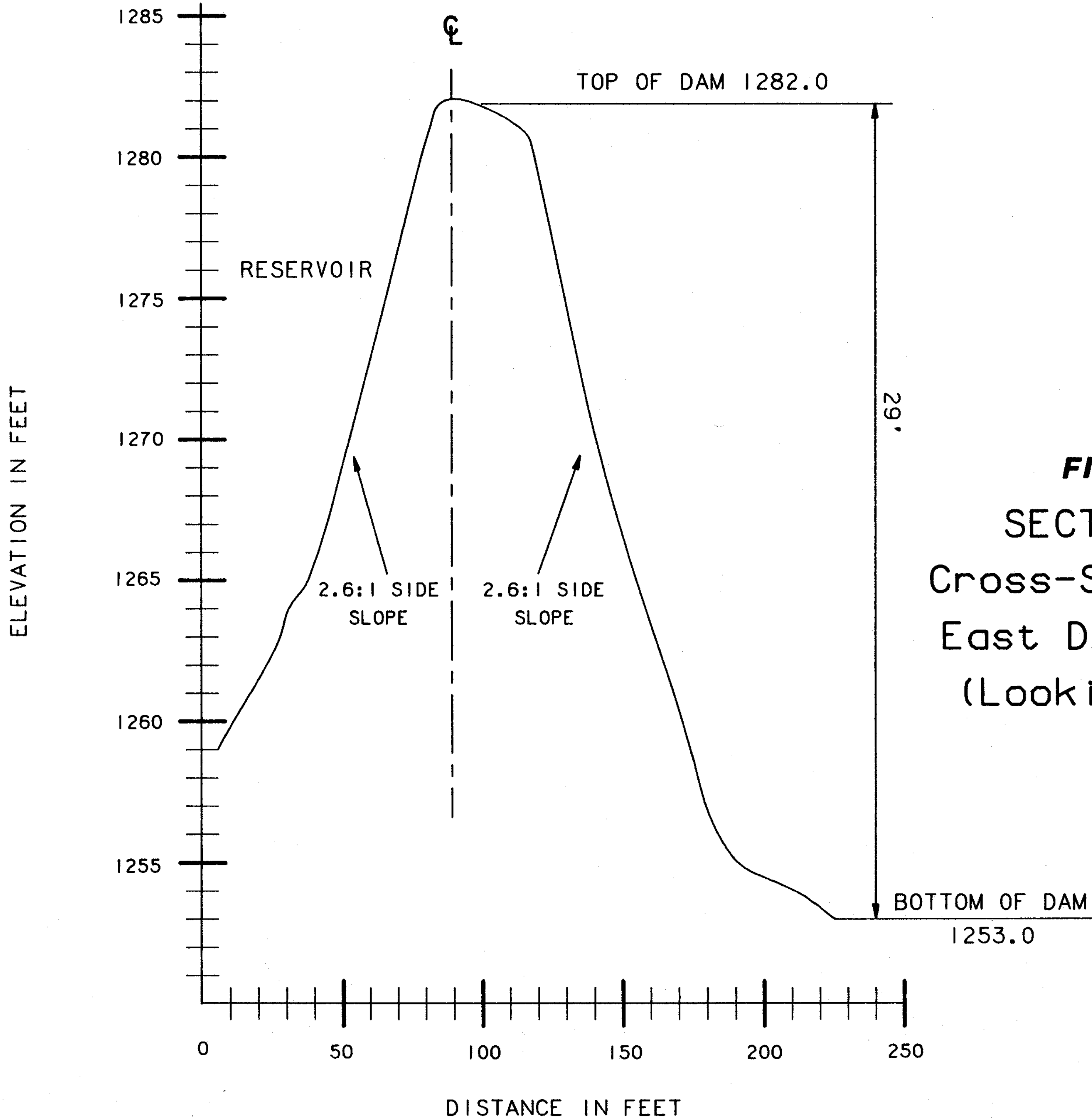

FIGURE 5

SECTION A-A

Cross-Section for East Dam (North) (Looking North)

H: 1":50'
V: 1":5'

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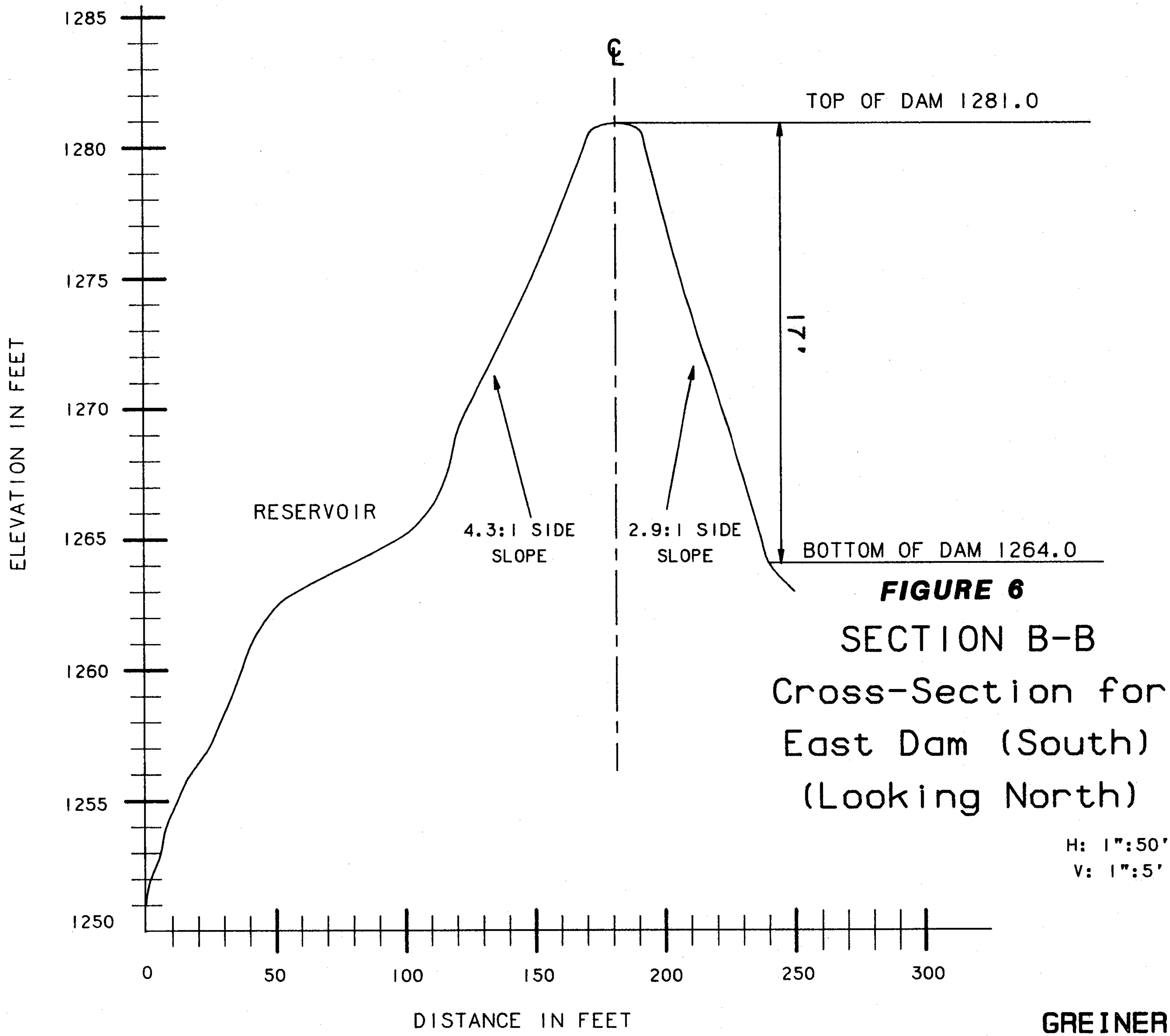

FIGURE 6

SECTION B-B
Cross-Section for
East Dam (South)
(Looking North)

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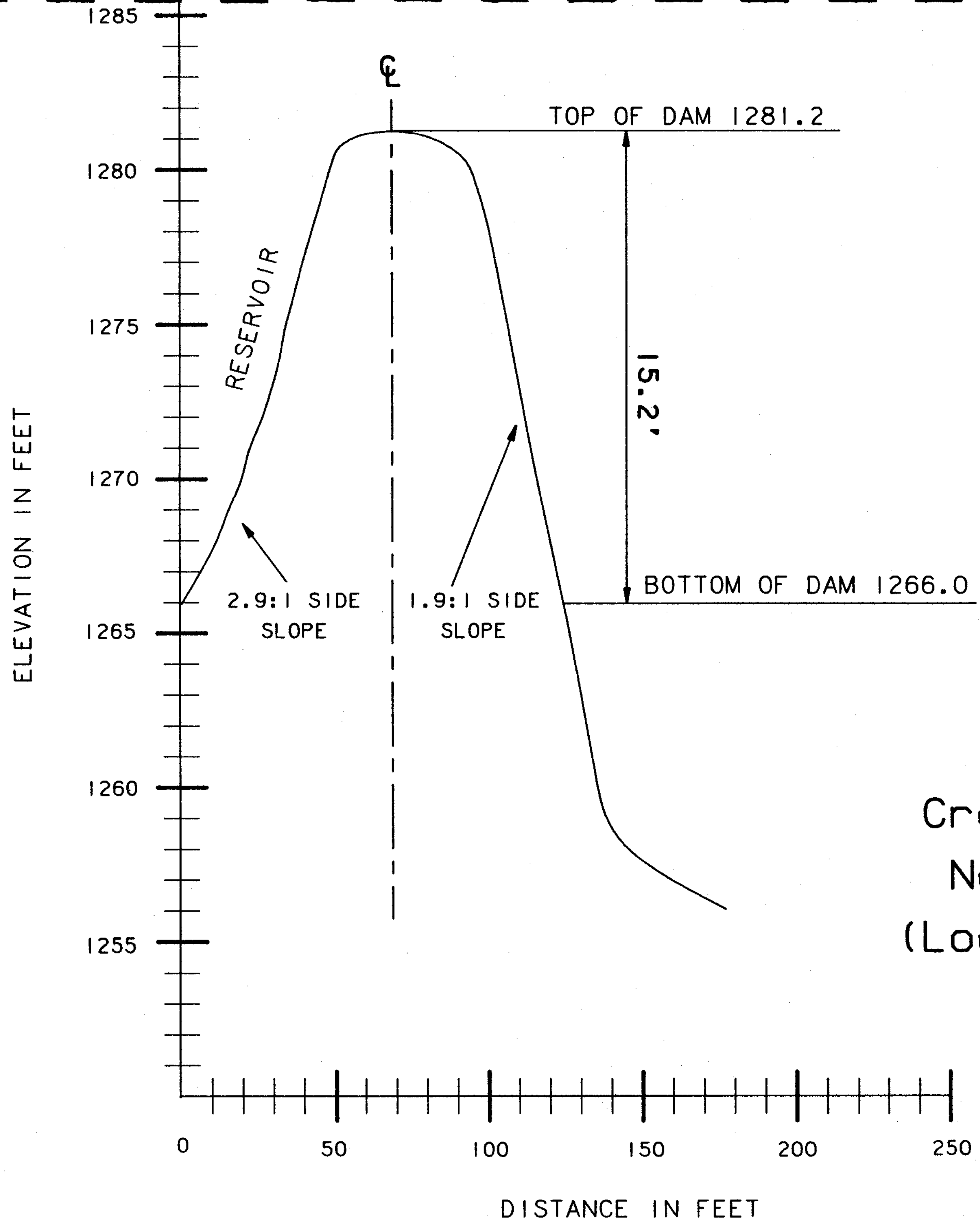

FIGURE 7

SECTION C-C

Cross-Section for North Dam No. 1 (Looking Southwest)

H: 1":50'
V: 1":5'

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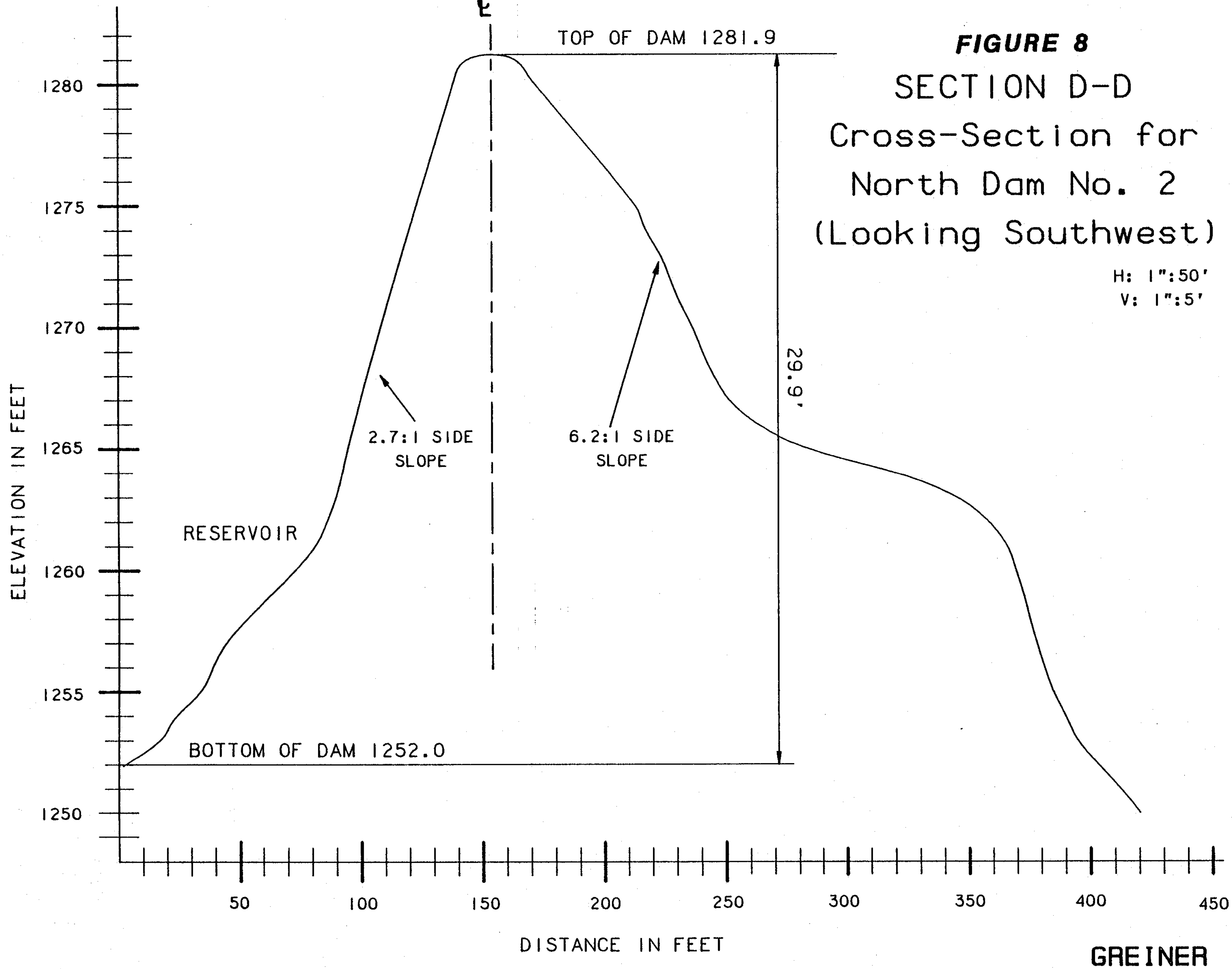
FIGURE 8

SECTION D-D
Cross-Section for
North Dam No. 2
(Looking Southwest)

H: 1":50'
V: 1":5'



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4. DAM BREACH ANALYSIS

Type of Dam Failure

To define the type of dam failure (overtopping or piping) in the simulation model, the maximum water level in the reservoir during storm with one-half PMP intensity of peak flow of 6,039 cfs was checked. The top of the dam is at elevation 1,281.0 and the crest of the emergency spillway is at elevation 1,274.0. The maximum outflow discharge from the emergency spillway at elevation 1,281.0 (top of East Dam) is 10,400 cubic feet per second. With computer routing justification, it is evident that dam failure shall occur as a result of piping condition, but not overtopping. See Reservoir Storage Routing and Dynamic Routing in Section 6.

Selection of Breach Parameters

MacDonald and Langrige-Monopolis (1984) and Froelich (1987) statistically derived dam breach predictors for b and T.

$$\bar{b} = 9.5\ k_o\ (V_r\ h_d)^{0.25}$$

$$T = 0.8\ (V_r/h_d^2)^{0.50}$$

In which b is average breach width (feet), T is the time of failure (hours), k_o = 1.0 for piping and 1.4 for overtopping, V_r is volume (acre-foot) and h_d is the height (feet) of water over the breach bottom which is usually about the height of the dam. Standard error of estimate for b was ±94 feet which is an average error of ±54 percent of b, and the standard error of estimate for T was ±0.9 hours, which is an average error of ±70 percent of T. The statistically derived dam breach parameters are included in Table 2.

Table 2

Summary of Statistically Derived Dam Breach Parameters

Parameter	East Dam North	East Dam South	North Dam No. 1	North Dam No. 2
Height (ft)*	22	11	9	23
Volume (Ac-ft)	339	256	227	358
b	88	69	64	90
b maximum	136	106	99	139
b minimum	40	32	29	41
T	0.67	1.16	1.34	0.66
T maximum	1.14	1.97	2.28	1.12
T minimum	0.20	0.35	0.40	0.20

*Dam height is based on breach formation at full lake water surface elevation of 1,275 feet.

4. DAM BREACH ANALYSIS

Parametric Approach

The parametric model uses empirical observations of previous dam failures such as the breach width-depth relation, time of breach formation, and depth of breach to develop the outflow hydrograph. The equations for this model are:

$$H_{k1} = \frac{0.04\ C \cos \phi}{1 - \cos (90 - \phi)}$$

$$H_{k2} = \frac{0.04\ C \cos \phi \sin (45 + \phi/2)}{1 - \cos (45 - \phi/2)}$$

$$H_{k3} = \frac{0.04\ C \cos \phi \sin (22.5 + 3\phi/4)}{1 - \cos (22.5 - \phi/4)}$$

$BB = 5/3\ H_d$ if $0.8\ H_d \leq H_{k1}$
$BB = 5/3\ H_{k1}$ if $0.8\ H_d > H_{k1}$
$0 = 90$ if $0.8\ H_d \leq H_{k1}$
$0 = 45 + \phi/2$ if $0.8\ H_d > H_{k1}$
$0 = 22.5 + 3\phi/4$ if $0.8\ H_d > H_{k2}$
$0 = 11.3 + 7\phi/8$ if $0.8\ H_d > H_{k3}$

Where ϕ = angle of friction of dam material, degree
C = cohesion strength, psf
BB = breach width, feet
H_d = difference in elevation between water surface and breach bottom, feet
0 = side slope angle of breach

For earthen dams:

$0.1 \leq TFH \leq 0.5$ (where TFH = Time of Failure in Hour)

$H_d \leq W \leq 5\ H_d$

A summary of the dam breach parameters derived from this method is included in Table 3. Detailed calculations are also provided in the Appendix.

Table 3

Summary of Parametrically Derived Dam Breach Parameters

Parameter	East Dam North	East Dam South	North Dam No. 1	North Dam No. 2
Height (ft)*	22	11	9	23
Breach width, BB (ft)	36.7	18.3	15	38.3
0 (degrees)	90	90	90	90

*Dam height is based on breach formation at full lake water surface elevation of 1,275 feet.

4. DAM BREACH ANALYSIS

Physical Approach

Another means of determining the breach properties is the use of physically-based breach erosion models. The BREACH model for earthen dams, developed by the National Weather Service, is a physically-based mathematical model which predicts the breach characteristics (size, shape, time of formation) and the discharge hydrograph emanating from a breach earthen dam.

The BREACH erosion model is based on principles of hydraulics, sediment transport, and soil mechanics. It uses equations of weir or orifice flow to simulate the outflow entering a channel that is gradually eroded through an earthen man-made dam. Conservation of reservoir inflow, storage volume, and outflow (crest overflow, spillway flow and breach flow) determines the time-dependent reservoir water elevation which along with the predicted breach bottom elevation determines the head controlling the reservoir outflow. A sediment transport relation, the Meyer-Peter and Muller equation modified for steep channels is used to predict the transport capacity of the breach flow whose depth is determined by a quasi-steady uniform flow relation (the Manning equation applied at each time step during the breach simulation). Breach enlargement is governed by the rate of erosion which is a function of the breach bottom slope and depth of flow and by the extent of collapse that occurs to the sides of the breach due to one or more sequential slope failures. The breach material properties (internal friction angle (ϕ) and cohesive strength (C)) are critical in determining the extent of enlargement of the trapezoidal-shaped breach. The Manning "n" used to compute the flow depth in the breach channel may be predicted on the basis of the grain size of the breach material by the Strickler equation or via the Moody relations. The dam may consist of two different materials: an outer portion of the dam, and a thin layer along the downstream face of the dam. The latter is grass covered or of a grain size substantially larger than that of the outer portion of the dam. The sequence of computations in the model are iterative since the flow into the breach is dependent on the bottom elevation of the breach and its width, while the breach properties are depended on the sediment transport capacity of the breach flow, and the transport capacity is dependent on the breach size and flow. A simple iterative algorithm is used to account for the mutual dependence of the flow, erosion and breach properties. An estimated incremental erosion depth is used at each time step to start the iterative computation. This estimated value can be extrapolated from previously computed, incremental erosion depths after the first few time steps.

4. DAM BREACH ANALYSIS

Parameters Used in BREACH Model Physical Approach Analysis

Table 4 lists the values and sources for all the parameters used in the analysis:

Table 4

Summary of Parameters Used in the BREACH Analysis

Description	Parameter	East Dam North	East Dam South	North Dam No. 1	North Dam No. 2	Source of Information
Elevation on top of dam	HU	1,282.0	1,281.0	1,281.2	1,281.9	Reference 13
Elevation of bottom of dam	HL	1,253.0	1,264.0	1,266.0	1,252.0	Reference 13
Plasticity index	PI	5	5	5	5	For little to non-plasticity - Reference 9
C_A	CA	0.01	0.01	0.01	0.01	Middle of range used - Reference 3
C_B	CB	0.7	0.7	0.7	0.7	Middle of range used - Reference 3
Slope of upstream face	Zu	2.6:1	4.3:1	2.9:1	2.7:1	Reference 13
Slope of downstream face	ZD	2.6:1	2.9:1	1.9:1	6.2:1	Reference 13
D_{50} of outer material (mm)	D_{50S}	0.76	0.76	0.76	0.76	Reference 15
Porosity Ratio	PORS	0.26 Void Ratio 0.347	0.26 Void Ratio 0.347	0.25 Void Ratio 0.334	0.28 Void Ratio 0.386	Reference 9
Unit weight pcf (dry)	UWS	123	123	124	119	Reference 9
Manning n	CNS	0.035	0.035	0.035	0.035	Reference 9
Friction angle	AFRS	37.5	37.5	38	37	Reference 9

Table 4 (Continued)

Summary of Parameters Used in the BREACH Analysis

Description	Parameter	East Dam North	East Dam South	North Dam No. 1	North Dam No. 2	Source of Information
Cohesive strength psf	COHS	280	280	280	280	Reference 15
D_{90} (mm) of outer material	UNFCS	10	10	10	10	(1)
D_{30} (mm) of outer material	UNFCS	10	10	10	10	(1)
Width of crest of dam (ft)	WC	14	14	16	24	Reference 13
Length of dam (ft)	CRL	1,500	1,500	400	400	Measured on map - Reference 13
Bottom slope of downstream (ft/mi)	SM	65	65	78	78	Reference 13
D_{50} (mm) of downstream face of dam	D_{50DF}	0.76	0.76	0.76	0.76	(2)
D_{90} (mm) of outer material	UNFCDF	10	10	10	10	(1)
D_{30} (mm) of outer material	UNFCDF	10	10	10	10	(1)

(1) No available data for D_{90} and D_{30}. The values for UNFCS and UNFCDF were assumed.

(2) Assume $D_{50DF} = D_{50S} = 0.76$ mm

4. DAM BREACH ANALYSIS

Reservoir Stage Versus Storage

The data for the reservoir stage versus storage were obtained in the final design report prepared by Gosnell Development Corporation in 1986. See Table 5 and curve in Figure 9.

Emergency Spillway

The crest elevation of the emergency spillway is at 1,274.0. Figure 10 depicts the stage versus discharge for the emergency spillway.

Manning "n"

The n value used in the breach analysis is 0.035.

Table 5

Stage Versus Storage Data for the GFRS Reservoir

Elevation (Feet)	End Area (Ac)	Average End Area (Ac)	Incremental Storage (Ac - Ft.)	Accumulated Storage (Ac - Ft.)
1,250*	1.995			0
		2.160	2.160	
1,251	2.326			2.160
		2.491	2.491	
1,252	2.656			4.651
		3.368	3.368	
1,253	4.079			8.019
		4.790	4.790	
1,254	5.501			12.809
		6.022	6.022	
1,255*	6.542			18.831
		7.062	7.062	
1,256	7.582			25.893
		8.069	8.069	
1,257	8.556			33.962
		9.043	9.043	
1,258	9.530			43.005
		9.913	9.913	
1,259	10.295			52.918
		10.677	10.677	
1,260*	11.059			63.595
		11.595	11.595	
1,261	12.130			75.190
		12.575	12.575	
1,262	13.201			87.765
		13.524	13.524	
1,263	13.846			101.289
		14.169	14.169	
1,264	14.491			115.458
		15.234	15.234	
1,265*	15.976			130.692
		16.718	16.718	
1,266	17.460			147.410
		17.983	17.983	
1,267	18.506			165.393
		19.029	19.029	
1,268	19.552			184.422
		20.315	20.315	
1,269	21.078			204.737
		21.841	21.841	
1,270*	22.603			226.578
		23.419	23.419	

Table 5 (Continued)

Stage Versus Storage Data for the GFRS Reservoir

Elevation (Feet)	End Area (Ac)	Average End Area (Ac)	Incremental Storage (Ac - Ft.)	Accumulated Storage (Ac - Ft.)
1,271	24.234			249.997
		25.049	25.049	
1,272	25.865			275.046
		26.388	26.388	
1,273	26.910			301.434
		27.432	27.432	
1,274	27.954			328.866
		28.750	28.750	
1,275*	29.545			357.616
		30.341	30.341	
1,276	31.136			387.957
		32.110	32.110	
1,277*	33.084			420.067
		34.058	34.058	
1,278	35.032			454.125
		35.544	35.544	
1,279	36.056			489.669
		36.568	36.568	
1,280	37.080			526.237
		37.497	37.497	
1,281*	37.913			563.734

*Data used for computer input

Source: Data from the final design report prepared by Gosnell Development Corporation, dated June 25, 1986.

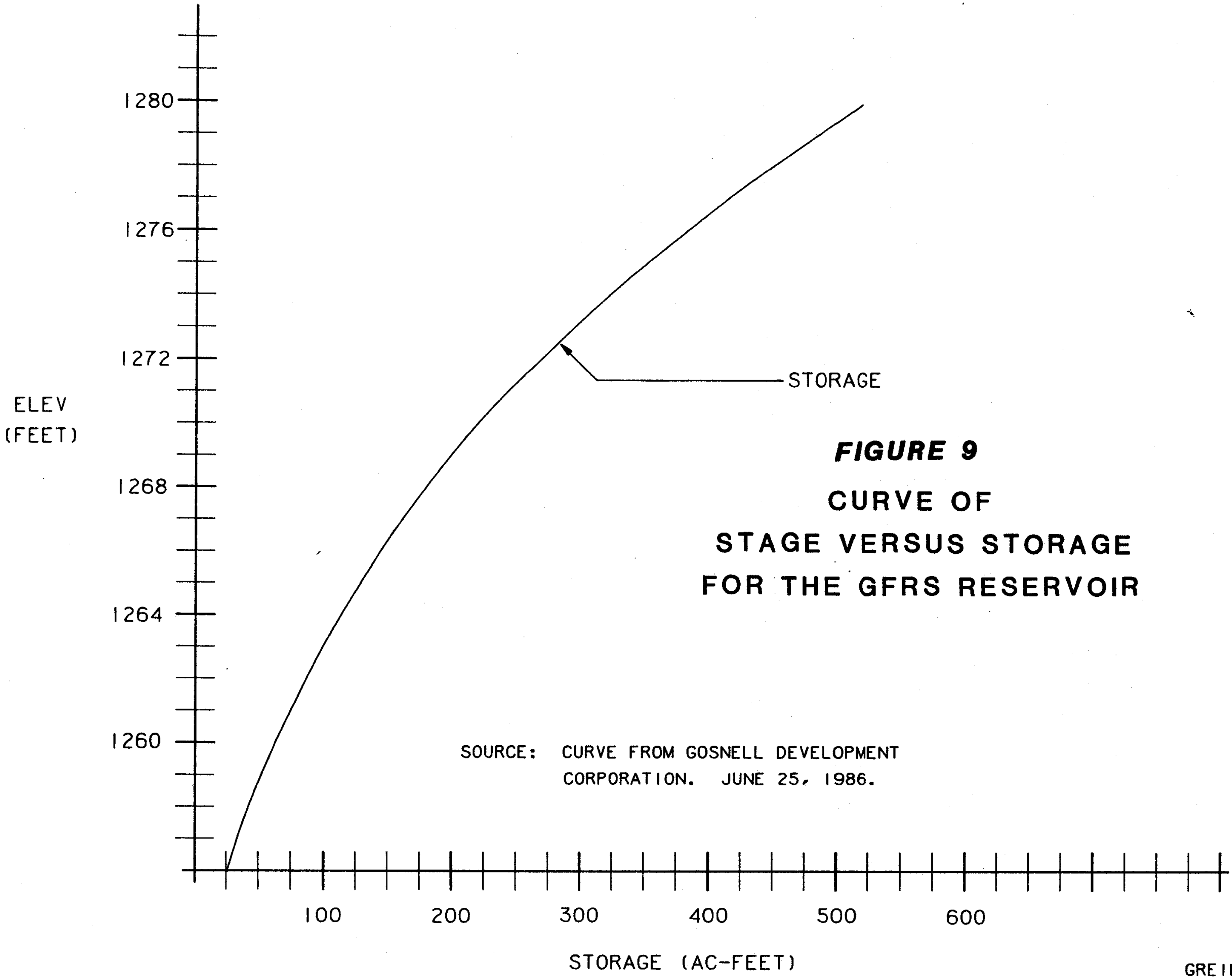

FIGURE 9

CURVE OF STAGE VERSUS STORAGE FOR THE GFRS RESERVOIR

SOURCE: CURVE FROM GOSNELL DEVELOPMENT CORPORATION. JUNE 25, 1986.

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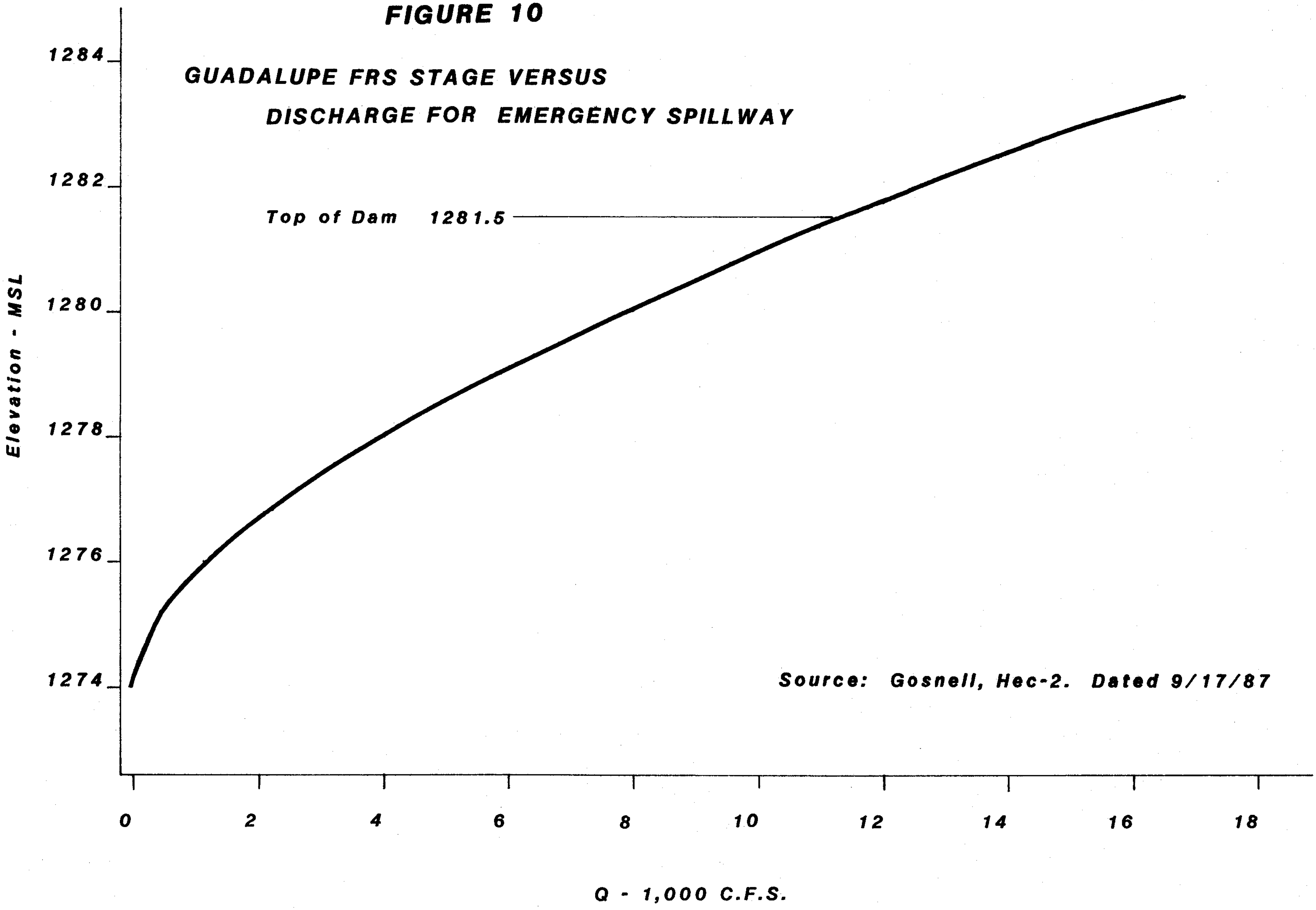
FIGURE 10



GUADALUPE FRS STAGE VERSUS
DISCHARGE FOR EMERGENCY SPILLWAY
Top of Dam 1281.5
Source: Gosnell, Hec-2. Dated 9/17/87
1284
1282
1280
1278
1276
1274
Elevation - MSL
0
2
4
6
8
10
12
14
16
18
Q - 1,000 C.F.S.

5. DAM BREACH PARAMETER SENSITIVITY, VERIFICATION AND HYDROGRAPH GENERATION

Selection of breach parameters, either before a breach forms or in the absence of observation, introduces a varying degree of uncertainty in the downstream flooding results of a dambreak analysis. For conservative forecasts which err on the side of larger flood waves, values for breach bottom and side slope should produce an average breach width in the upper most range for a certain type of dam. Failure time should be selected in the lower range to produce a maximum outflow. However, the reality of the dam breach formation and its parameters are a function of dam type, size and material, and also depend on initial lake water surface elevation, inflow hydrograph, spillway elevation and capacity, and maintenance of the dam.

The following procedures were employed to derive the dam breach hydrograph for the typical dam shown in Figure 11:

1. Dam breach parameters estimation: By utilizing the statistical indicator method, it was found that the breach width varies from 20 feet to 140 feet, and breach time is in the range of 0.2 hour to 2.2 hours. The parameter estimation indicates that breach width is limited to not wider than 42 feet.

2. Initial piping elevation selection: The breach model was used for physical erosion test runs in the combination of:

   - initial breach elevation 1,250 feet, 1,252 feet, 1,254 feet, 1,256 feet and 1,260 feet which are 0 feet, 2 feet, 4 feet, 6 feet and 10 feet above the dam bottom respectively

   - downstream channel in widths of 10 feet, 20 feet, 40 feet, 60 feet and 100 feet

   - channel shape of trapezoidal and rectangular

   - initial water surface elevation is 1,275

   The variation of the downstream channel is to minimize the submergence correction for the breach formation to induce the maximum breach condition. The results of these test runs are included in Table 6. The results of maximum breach width of 25 feet for a typical dam section indicate that in order to maximize the breach width, as derived by Step 1, the initial breach elevation shall be set close to the bottom of the dam. The breach time is between 0.1 hour to 0.16 hour.

3. Maximum breach width determination: In order to define the maximum breach that can possibly occur, a prolonged high inflow hydrograph was incorporated in the breach model. The results indicate that the maximum breach width is 28.4 feet for this particular material and the configuration of the dam.

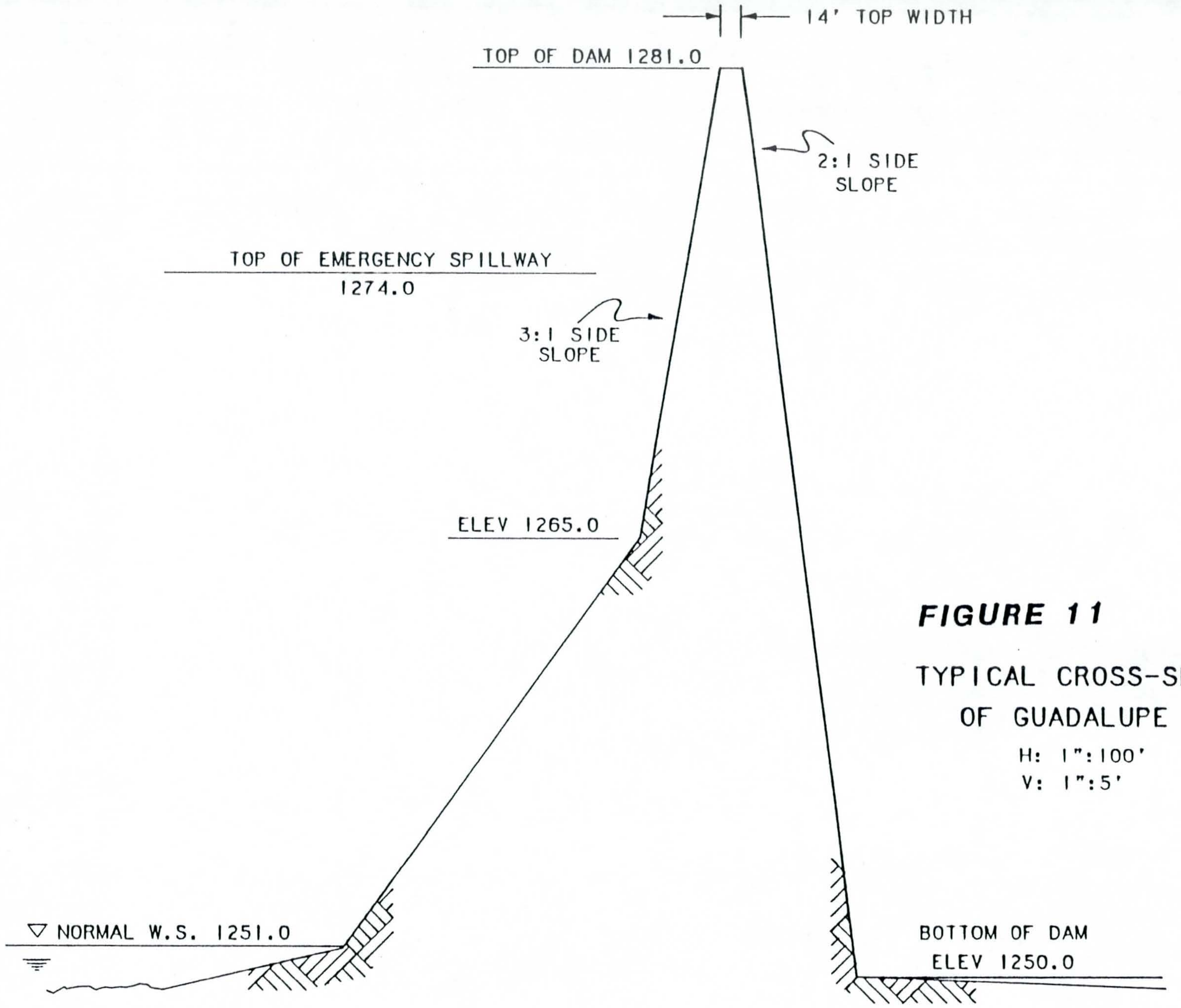


FIGURE 11

TYPICAL CROSS-SECTION
OF GUADALUPE FRS

H: 1":100'
V: 1":5'

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Table 6

Summary of Parameters Obtained by Breach Erosion Method

Shape of Channel Immediately Downstream From Breach	Bottom of Breach Elevation	Bottom Width of Downstream Channel (ft)	Time to Peak TP (hr)	Peak Discharge QBP (cfs)	Final Top Width of Breach BRW (ft)	Final Elevation of Bottom of Breach (ft)
Rectangular		20	0.16	10,950	23.67	1,250.00
WS 1275	1,250	40	0.10	15,881	25.57	1,250.00
		60	0.10	19,043	27.38	1,250.00
		10	--	--	--	--
		20	0.13	11,004	23.52	1,250.00
		40	0.09	15,649	25.01	1,250.00
	1,252	60	0.07	17,451	25.24	1,250.00
		100	0.06	20,261	26.05	1,250.00
		10	--	--	--	--
		20	0.09	10,708	22.16	1,250.00
	1,254	40	0.07	14,133	22.24	1,250.00
		60	0.04	14,683	22.25	1,250.00
		100	0.04	16,753	22.96	1,250.00
		10	0.10	6,870	18.66	1,250.00
		20	0.05	9,305	19.40	1,250.00
	1,256	40	0.03	10,395	18.58	1,250.00
		60	0.03	10,718	18.24	1,250.00
		100	0.03	10,935	18.38	1,250.00
		10	--	--	--	--
		20	0.04	7,641	17.32	1,250.00
	1,258	40	0.03	8,731	17.23	1,250.00
		60	0.03	8,718	17.23	1,250.00
		100	0.03	8,718	17.23	1,250.00

Table 6 (Continued)

Summary of Parameters Obtained by Breach Erosion Method

Shape of Channel Immediately Downstream From Breach	Bottom of Breach Elevation	Bottom Width of Downstream Channel (ft)	Time to Peak TP (hr)	Peak Discharge QBP (cfs)	Final Top Width of Breach BRW (ft)	Final Elevation of Bottom of Breach (ft)
		10	0.04	4,732	13.73	1,252.57
		20	0.02	4,909	15.39	1,252.31
	1,260	40	0.02	4,910	15.39	1,252.31
		60	0.02	4,910	15.39	1,252.31
		100	0.02	4,190	15.39	1,252.31
Trapezoidal		10	0.15	10,473	23.17	1,250.00
WS 1275		20	0.10	12,970	23.68	1,250.00
	1,252	40	0.08	16,310	25.11	1,250.00
		60	0.07	18,338	25.62	1,250.00
		100	0.06	20,802	26.17	1,250.00
		10	0.09	10,387	21.75	1,250.00
		20	0.08	12,271	22.57	1,250.00
	1,254	40	0.06	14,606	22.98	1,250.00
		60	0.04	15,647	22.77	1,250.00
		100	0.04	16,903	23.07	1,250.00
		10	0.06	8,860	19.27	1,250.00
		20	0.04	9,746	18.92	1,250.00
	1,256	40	0.03	10,640	18.79	1,250.00
		60	0.03	10,762	18.28	1,250.00
		100	0.03	10,935	18.38	1,250.00
		10	0.04	6,875	16.16	1,250.00
		20	0.04	8,230	17.66	1,250.00
	1,258	40	0.03	8,724	17.24	1,250.00
		60	0.03	8,718	17.23	1,250.00
		100	0.03	8,718	17.23	1,250.00
		10	0.03	5,221	13.63	1,252.21
		20	0.02	4,910	15.39	1,252.31
	1,260	40	0.02	4,910	15.39	1,252.31
		60	0.02	4,910	15.39	1,252.31
		100	0.02	4,910	15.39	1,252.31

4. Breach outflow hydrograph generation: Two extreme conditions were possible when the dam breached: 1) the lake could have been full when PMP started, and 2) the lake could have been empty when PMP started. The test runs of DAMBRK models indicate that the empty lake will result in a higher breach flow of 11,504 cfs as the breach occurs during the rising limb of the hydrograph. The lake full condition of peak flow of 9,673 cfs indicates that damage shall arise if the dam is breached when it is full during a less intensified flood.

5. Downstream routing damage test: The outflow hydrographs generated from Step 4 are routed downstream by the dambreak model. The variation of simulated flood peaks at critical downstream locations were compared (see Table 7). This represents that risk of damage may occur as a result of a lesser flood event.

6. Downstream routing sensitivity tests: These tests were conducted by running different models with various breach widths of 20 feet, 30 feet and 40 feet, and time of breach at 0.1 hour, 0.3 hour and 0.5 hour. Two water surface elevations were used in these tests at 1,258 (lake empty) and 1,275 (lake full). The results are provided in Tables 8 through 11.

7. Finalize breach parameter to be used in the dambreak model: Final breach parameters were selected based on the comparison of maximum breach width, derived from breach models, to the statistically and parametrically derived parameters. The configuration of Guadalupe dams and its reservoirs were also considered. The discharge generated from the DAMBRK model was also compared.

By following the same procedure, the breach parameters for each of the potential dam failure location were obtained for the final dambreak modeling (Table 12).

Table 7

Downstream Routing Damage Test for Typical Dam

Lake Empty			Lake Full		
Distance From Dam Mile	Maximum Water Elevation Feet	Maximum Flow cfs	Distance From Dam Mile	Maximum Water Elevation Feet	Maximum Flow cfs
0.023	1,256.02	11,727	0.023	1,255.65	9,514
0.108	1,250.33	11,658	0.108	1,250.20	9,554
0.203	1,247.17	11,576	0.203	1,246.84	9,490
0.305	1,240.62	11,488	0.305	1,240.49	9,395
0.443	1,230.53	11,408	0.443	1,230.30	9,285
0.532	1,222.47	11,368	0.532	1,222.33	9,233
0.629	1,219.37	11,276	0.629	1,219.18	9,133
0.773	1,209.80	11,191	0.773	1,209.63	9,027
0.905	1,204.60	11,129	0.905	1,204.42	8,934
1.030	1,195.19	11,082	1.030	1,195.05	8,875
1.138	1,191.34	11,030	1.138	1,191.02	8,805
1.269	1,188.49	10,802	1.269	1,187.97	8,620
1.421	1,181.09	10,782	1.421	1,181.03	8,556
1.574	1,176.14	10,639	1.574	1,175.82	8,379
1.790	1,174.49	9,613	1.790	1,174.36	7,475

Table 8

Summary of Downstream Routing Sensitivity Tests for Typical Dam

Full Lake TFH = 0.1 Hour

	BB = 20 Feet		BB = 30 Feet		BB = 40 Feet	
Distance From Dam Mile	Maximum Elevation Feet	Maximum Flow cfs	Maximum Elevation Feet	Maximum Flow cfs	Maximum Elevation Feet	Maximum Flow cfs
0.023	1,255.25	7,321	1,255.91	10,976	1,256.46	14,404
0.108	1,250.03	7,284	1,250.28	10,850	1,250.49	14,340
0.203	1,246.44	7,185	1,247.01	10,389	1,247.47	13,903
0.305	1,240.31	6,975	1,240.55	10,356	1,240.75	13,558
0.443	1,230.00	6,894	1,230.40	10,176	1,230.69	13,269
0.532	1,222.14	6,866	1,222.39	10,107	1,222.59	13,152
0.629	1,218.96	6,812	1,219.26	9,957	1,219.49	12,911
0.773	1,209.40	6,764	1,209.70	9,823	1,209.94	12,695
0.905	1,204.22	6,711	1,204.48	9,726	1,204.70	12,541
1.030	1,194.85	6,672	1,195.13	9,654	1,195.30	12,431
1.138	1,190.68	6,638	1,191.13	9,576	1,191.50	12,304
1.269	1,187.51	6,551	1,188.14	9,330	1,188.80	11,834
1.421	1,180.95	6,192	1,181.05	9,271	1,181.10	11,798
1.574	1,175.51	6,407	1,175.92	9,063	1,176.30	11,520
1.790	1,174.24	5,813	1,174.41	8,138	1,174.50	10,129

*Sensitivity tests for empty lake and TFH = 0. hour are not feasible since the lake is never filled up when the breach time comes.

BB = Breach Width

Table 9

Summary of Downstream Routing Sensitivity Tests for Typical Dam

Empty Lake TFH = 0.3 Hour

	BB = 20 Feet		BB = 30 Feet		BB = 40 Feet	
Distance From Dam Mile	Maximum Elevation Feet	Maximum Flow cfs	Maximum Elevation Feet	Maximum Flow cfs	Maximum Elevation Feet	Maximum Flow cfs
0.023	1,255.51	8,775	1,256.02	11,727	1,256.49	1,4627
0.108	1,250.14	8,715	1,250.33	11,658	1,250.50	14,566
0.203	1,246.71	8,652	1,247.17	11,576	1,247.55	14,462
0.305	1,240.43	8,552	1,240.62	11,488	1,240.79	14,360
0.443	1,230.21	8,476	1,230.53	11,408	1,230.78	14,251
0.532	1,222.27	8,440	1,222.47	11,368	1,222.64	14,197
0.629	1,219.12	8,357	1,219.37	11,276	1,219.57	14,086
0.773	1,209.54	8,281	1,209.80	11,191	1,210.01	13,981
0.905	1,204.36	8,224	1,204.60	11,129	1,204.80	13,894
1.030	1,194.98	8,181	1,195.19	11,082	1,195.37	13,827
1.138	1,190.93	8,142	1,191.34	11,030	1,191.68	13,749
1.269	1,187.83	8,026	1,188.49	10,802	1,189.23	13,292
1.421	1,181.02	7,984	1,181.09	10,782	1,181.14	13,339
1.574	1,175.73	7,873	1,176.14	10,639	1,176.55	13,138
1.790	1,174.36	7,173	1,174.49	9,613	1,174.54	11,864

BB = Breach Width

Table 10

Summary of Downstream Routing Sensitivity Tests for Typical Dam

Full Lake TFH = 0.3 Hour

Distance From Dam Mile	BB = 20 Feet Maximum Elevation Feet	BB = 20 Feet Maximum Flow cfs	BB = 30 Feet Maximum Elevation Feet	BB = 30 Feet Maximum Flow cfs	BB = 40 Feet Maximum Elevation Feet	BB = 40 Feet Maximum Flow cfs
0.023	1,255.12	6,833	1,255.65	9,514	1,256.12	12,136
0.108	1,249.98	6,779	1,250.20	9,554	1,250.35	11,932
0.203	1,246.34	6,691	1,246.84	9,490	1,247.22	11,955
0.305	1,240.28	6,534	1,240.49	9,395	1,240.65	11,837
0.443	1,229.92	6,408	1,230.30	9,285	1,230.55	11,727
0.532	1,222.10	6,382	1,222.33	9,233	1,222.50	22,649
0.629	1,218.91	6,329	1,219.18	9,133	1,219.38	11,519
0.773	1,209.35	6,285	1,209.63	9,027	1,209.84	11,360
0.905	1,204.17	6,241	1,204.42	8,934	1,204.60	11,243
1.030	1,194.80	6,209	1,195.05	8,875	1,195.22	11,144
1.138	1,190.60	6,174	1,191.02	8,805	1,191.33	11,018
1.269	1,187.41	6,116	1,187.97	8,620	1,188.51	10,717
1.421	1,180.93	6,073	1,181.03	8.556	1,181.08	10,669
1.574	1,175.44	6,009	1,175.82	8,379	1,176.14	10,418
1.790	1,174.21	5,509	1,174.36	7,475	1,174.45	9,091

BB = Breach Width

Table 11

Summary of Downstream Routing Sensitivity Tests for Typical Dam

Full Lake TFH = 0.5 Hour

	BB = 30 Feet	
Distance From Dam Mile	Maximum Elevation Feet	Maximum Flow cfs
0.023	1,255.45	8,406
0.108	1,250.11	8,334
0.203	1,246.65	8,325
0.305	1,240.41	8,281
0.443	1,230.18	8,237
0.532	1,222.25	8,202
0.629	1,219.09	8,148
0.773	1,209.54	8,073
0.905	1,204.33	8,006
1.030	1,194.97	7,959
1.138	1,190.88	7,892
1.269	1,187.79	7,790
1.421	1,181.00	7,742
1.574	1,175.71	7,657
1.790	1,174.32	6,997

Table 12

Summary of Breach Parameters

Dam	Breach Width (Ft)	Breach Time Empty Lake (Hr)	Breach Time Full Lake (Hr)	Peak Discharge (cfs)
East-North	27	0.28	0.14	8,903
East-South	17	0.24	0.12	5,205
North Dam No. 1	15	0.26	0.13	5,095
North Dam No. 2	30	0.32	0.15	9,981

6. RESERVOIR STORAGE ROUTING AND DYNAMIC ROUTING

A series of tests were performed to compare the Reservoir Storage Routing (RSR) to the Reservoir Dynamic Routing (RDR). This required four separate runs using three program options. To simplify the analysis procedures, the typical cross section for the East Dam and its downstream area was considered for routing.

Routing Options

The RSR model used Options 2 and 8, while the RDR model used Options 5 and 8. Each of these options is described in detail in Appendix A of the NWS DAMBRK User's Manual (Reference 3).

Option 2 was used to perform a RSR and compute the outflow hydrograph from the breached dam and route the hydrograph through the downstream channel as supercritical flow.

Option 5 does the same as Option 2, except it was used to perform a RDR. Since this option creates non-conveyance problems through the downstream routing, Option 8 was used to route the hydrograph through the downstream channel supercritical flow. The non-conveyance errors may be due to some internal program problems as acknowledged by the NWS staff.

In order to compare the results of the RSR and RDR models, Option 8 was also applied to the RSR for downstream routing.

Breach Conditions

The following is the input for RSR and RDR model runs based on the results of breach analysis:

Parameter	RSR (Option 2)	RDR (Option 5)
Initial Water Surface Elevation, Feet (Empty Lake)	1,258	1,258
Breach Elevation, Feet	1,250	1,250
Maximum Breach Width, Feet	20	20
Time to Maximum Failure Width, Hour	0.3	0.3
Top of Dam Elevation, Feet	1,281	1,281
Water Surface Elevation at Time of Breach, Feet	1,274	1,274
Breach Side Slope	Vertical	Vertical
Elevation of Spillway Crest, Feet	1,274	1,274
Discharge Coefficient of Weir Flow	-1,254*	3 x Length of Top of Dam

*Beginning elevation in feet for piping

Dynamic Routing

The reservoir is a golf course with a non-uniform shape. It consists of a main channel section, where the routing begins, and a large storage section adjacent to the main channel. These two sections are connected by culverts. The RDR will route the one-half PMP inflow hydrograph through the reservoir as a wave.

6. RESERVOIR STORAGE ROUTING AND DYNAMIC ROUTING

Reservoir and Downstream Cross Sections

All reservoir and downstream cross sections for routing were based on the latest USGS Topographic Maps, Topographic Maps of the Cities of Phoenix and Tempe, and the Post-Development Topographic Maps of the golf course area at South Mountain provided by Gosnell Development Corporation (References 10 through 13). The information was input to the CADD system in order to generate a base map of desirable scale. The area covered in the base map extends from I-10 and the Superstition Freeway in the north, Guadalupe Road in the south, McClintock Drive in the east, and 24th Street in the west. The contour intervals for the base map are five feet for steep slopes and two feet for mild slopes. Mile streets, railroad, highways, canals and river were also shown on the map.

Each of the reservoir cross sections consists of a main channel and a storage portion. The main channel has a substantially smaller volume. The main channel had a uniform slope, but its downstream elevation did not equal the breach elevation. Therefore, it was necessary to lower the minimum elevation of some of the cross sections and smooth them for the model to route the wave downstream to the dam. The flow in the reservoir was considered subcritical.

The preliminary flood boundaries for the East Dam are delineated on a topographic work map of 200 scale which is overlayed on an aerial map (Reference 14). The station centerline for the flooded area is located at or close to the lowest elevation of the channel. In order to simulate channel flow, each cross section layout was in curve with convex shape. Cross sections were located at intervals along the channel where changes occur in slope, land-use or roughness. They extend across the entire floodplain and are perpendicular to the anticipated flow lines (approximately parallel to contour lines).

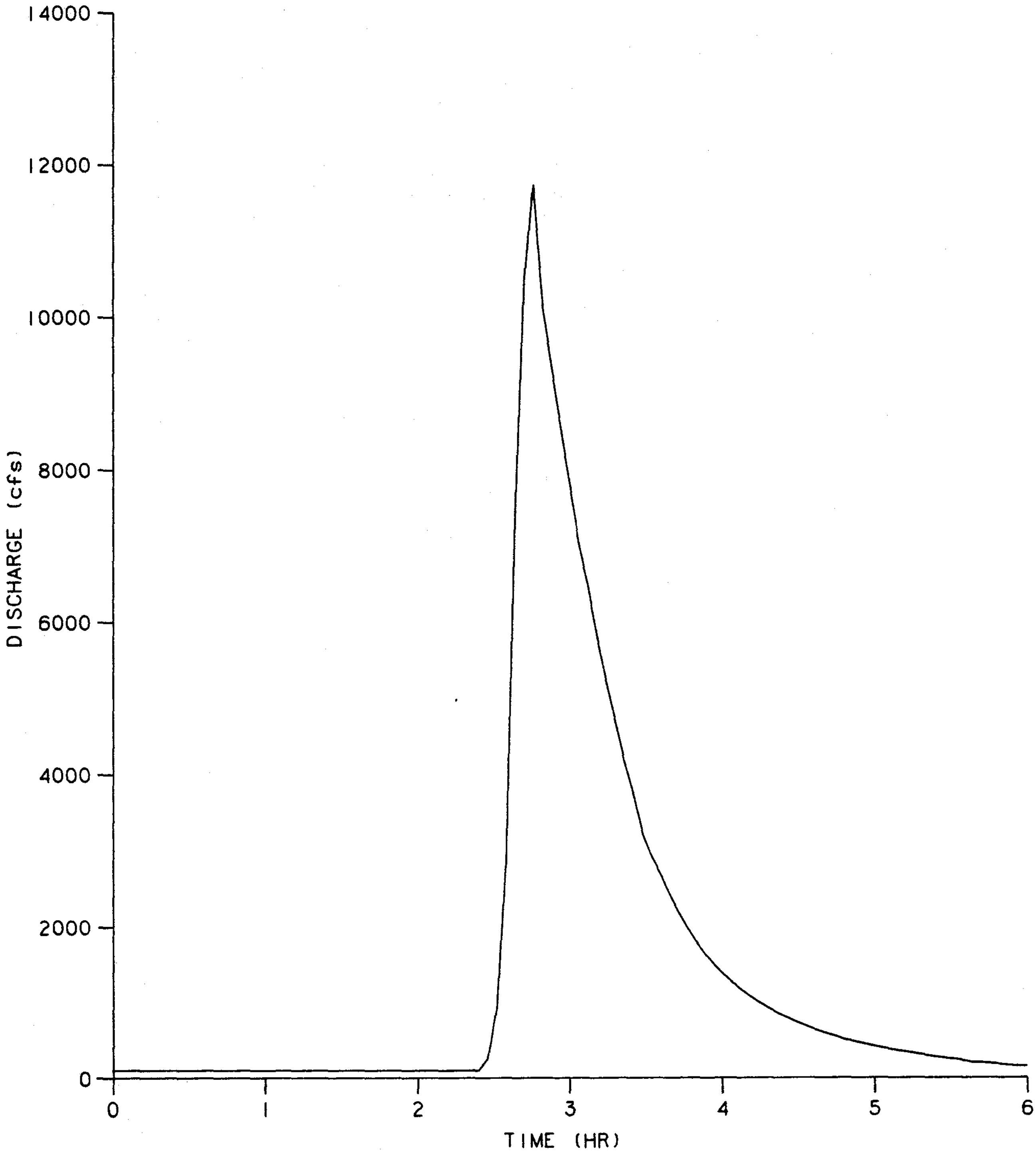
Since the input elevation versus top width data must provide a smooth transition, the cross section locations must be selected carefully to accurately model the downstream channel. Top width versus water depth for each cross section was plotted and compared to the upstream and downstream sections. New cross sections with a similar shape of the original cross section were drawn that more closely matched the adjacent sections. As a result, a set of new cross sections was produced that transitioned smoothly from upstream to downstream for the RDR and RSR runs.

Outflow Hydrographs

Figures 12 and 13 are computer plots of the outflow hydrographs for storage routing and dynamic routing respectively. The RDR outflow hydrograph begins later than the RSR and consequently has a higher peak and greater volume. This will have an effect on both the downstream maximum flow versus distance curve, and time of maximum water surface elevation versus distance curves.

The outflow hydrographs were used as the input hydrographs in Option 8 for the downstream dynamic routings.

FIGURE 12



OUTFLOW HYD. (STORAGE ROUTING)

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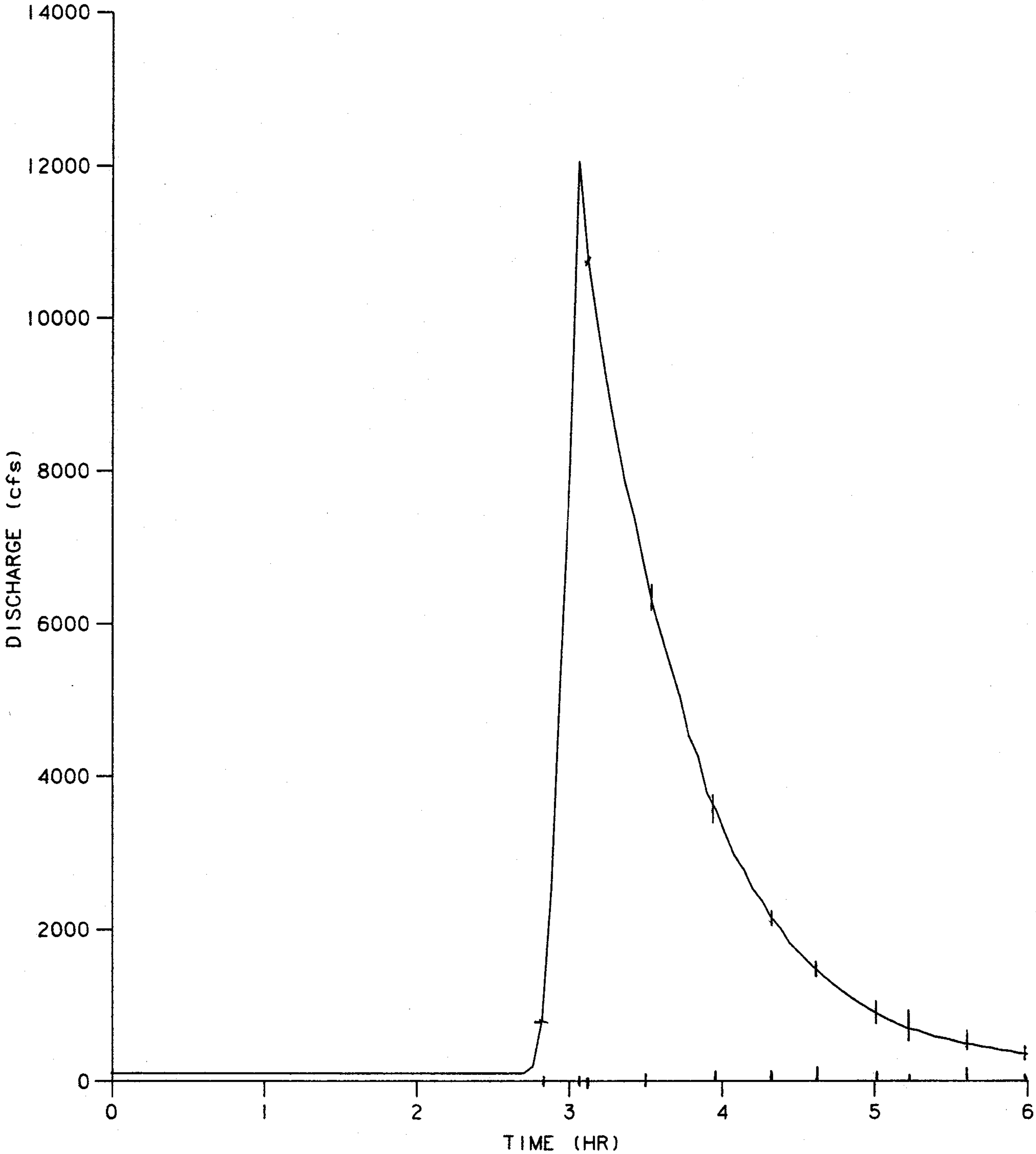
FIGURE 13



OUTFLOW HYD. (DYNAMIC ROUTING)

GREINER

6. RESERVOIR STORAGE ROUTING AND DYNAMIC ROUTING

To better analyze the results, six graphs were produced for each type of routings (see Graphs A through L in Appendix B):

1. Maximum water elevation versus distance from dam
2. Water depth versus distance from dam
3. Top width of maximum water elevation versus distance from dam
4. Maximum flow velocity versus distance from dam
5. Time of maximum water elevation versus distance from dam
6. Maximum discharge versus distance from dam

All of the graphs have similar shape and slope; however, two of the graphs, time of maximum water elevation versus distance downstream of dam and maximum discharge versus distance downstream of dam, have different magnitudes. This can be related to the dam outflow hydrographs, where the RDR outflow hydrograph peaks approximately 0.25 hour later than the RSR hydrograph. The discharge is 5.6 percent greater for the RDR than the RSR. This is an insignificant difference and both runs can be considered equal. The magnitudes for the remaining graphs were equal for both types of routings. Table 13 summarizes the maximum water elevation in feet and maximum flow in cubic feet per second versus distance from dam in miles for both RSR and RDR models.

These tests indicate that, regardless of which routing method was used, similar results would be obtained.

Table 13

Summary of Reservoir Storage Routing and Reservoir Dynamic Routing for Typical Dam

Reservoir Storage Routing			Reservoir Dynamic Routing		
Distance From Dam Mile	Maximum Elevation Feet	Maximum Flow cfs	Distance From Dam Mile	Maximum Elevation Feet	Maximum Flow cfs
0.792	1,201.53	8,511	0.792	1,201.55	8,902
0.806	1,200.66	8,504	0.806	1,200.68	8,894
0.820	1,199.79	8,495	0.820	1,199.82	8,882
0.834	1,198.93	8,494	0.834	1,198.96	8,868
0.847	1,198.08	8,497	0.847	1,198.10	8,850
0.861	1,197.23	8,498	0.861	1,197.25	8,828
0.875	1,196.38	8,496	0.875	1,196.41	8,803
0.889	1,195.55	8,493	0.889	1,195.57	8,803
0.903	1,194.72	8,487	0.903	1,194.75	8,817
0.916	1,193.91	8,478	0.916	1,193.94	8,827
0.930	1,193.11	8,475	0.930	1,193.14	8,833
0.944	1,192.33	8,479	0.944	1,192.36	8,836
0.958	1,191.56	8,479	0.958	1,191.59	8,836
0.972	1,190.81	8,477	0.972	1,190.84	8,831
0.992	1,190.09	8,467	0.992	1,190.12	8,814
1.013	1,189.18	8,451	1.013	1,189.21	8,792

Table 13 (Continued)

Summary of Reservoir Storage Routing and
Reservoir Dynamic Routing for Typical Dam

Reservoir Storage Routing			Reservoir Dynamic Routing		
Distance From Dam Mile	Maximum Elevation Feet	Maximum Flow cfs	Distance From Dam Mile	Maximum Elevation Feet	Maximum Flow cfs
1.034	1,188.35	8,455	1.034	1,188.37	8,769
1.055	1,187.48	8,448	1.055	1,187.51	8,739
1.076	1,186.63	8,434	1.076	1,186.67	8,756
1.097	1,185.78	8,424	1.097	1,185.82	8,778
1.117	1,184.93	8,425	1.117	1,184.97	8,788
1.138	1,184.08	8,416	1.138	1,184.11	8,769
1.160	1,183.74	8,385	1.160	1,183.77	8,711
1.182	1,182.88	8,387	1.182	1,182.91	8,674
1.204	1,182.50	8,370	1.204	1,182.53	8,688
1.225	1,181.66	8,350	1.225	1,181.69	8,691
1.247	1,181.28	8,348	1.247	1,181.30	8,677
1.269	1,180.43	8,338	1.269	1,180.47	8,651
1.291	1,180.06	8,314	1.291	1,180.09	8,612

7. INUNDATION AREA ROUTING

By utilizing the DAMBRK model, the breach outflow hydrograph was routed downstream from the breach location of each dam. However, the results of the computer runs show that the inundation areas for the East and North Dam's two breach locations are very close. Therefore, the inundation area maps were prepared for the extreme cases in which the largest impact to the downstream areas will occur from a single breach location.

The DAMBRK model utilized a dynamic routing technique by solving the <u>one-dimensional equations of unsteady flow</u> which allows variable time and accounts for the acceleration effects associated with the dambreak wave.

For a well defined channel with overbank reaches in a rural area, the geometrical descriptions for the downstream routing would be the same as those used in the floodplain delineation programs such as the HEC-2 water surface profiles. The HEC-2 program is intended for calculating water surface profiles for <u>steady gradually varied flow</u> which is instantaneous in time and based on the one-dimensional energy equation.

For a well urbanized area with no defined channels, such as the downstream of Guadalupe Dam, special considerations should be made in defining the downstream geometric descriptions. Under the steady flow conditions, such downstream cross sections can be coded from the topographic contours and neglecting the buildings and other structures by adjusting the natural "n" value to an artificial number of 0.18 or 0.20. This approach is commonly accepted in the floodplain delineation practice as the purpose is to define a conservative flood stage only. Also, this approach is justified because the flood wave can be assumed to be very similar in shape and magnitude. However, utilizing the geometrical description of this approach in an unsteady flow model will generate an unreasonable simulation with potential deviations in results of lower velocity, higher flood stage at upstream end and lower flood stage at the downstream end with a major discrepancy in conservation of mass.

A modified geometric description was used for the unsteady flow modeling approach.

The cross section data for downstream routing was input into the DAMBRK model according to the net top widths versus stage. The net top width for water to pass through is equal to the total top width at this elevation minus the widths occupied by buildings and obstructions. The width between the block walls where water cannot pass through was considered non-effective and assumed zero. The curves for the net top width versus water depth were adjusted to obtain smooth flow transition in the downstream sections and eliminate non-conveyance problems (see graphs provided in Figures 14 through 17).

Two conditions were considered when the routing models were developed: with- and without-block walls. The block walls constructed at different downstream locations of the dams would greatly affect the flow path. For the without-block-wall conditions, the walls were assumed to collapse during flooding.

Once the geometric description was completed, the program was tested in subcritical, supercritical and mixed flow DAMBRK routines. The subcritical flow models were initiated by utilizing the "n" value of 0.18 and gradually reducing its value in order to eliminate the deficiencies in conservation of mass. It has been determined that problems exist under an "n" value of 0.025. The supercritical flow was modeled reversely, until a maximum "n" value could be used without an error message. Under such an approach, the contraction and expansion coefficient was not used as it is compensated for in the Saint-Venant equation. It was found that the program output is supercritical flow from the dam site, gradually reducing its energy to critical and then transitioning to subcritical. However, the mixing flow option is not running at this time (verified by NWS), so the supercritical flow option was used in this study. This is to be considered the most appropriate simulation.

As the result will provide the most accurate prediction of flow travel time as well as instantaneous discharge, total volume and velocity along the main effective flow area.

The "n" value used for the downstream routing is summarized in Table 14.

7. INUNDATION AREA ROUTING

Table 14

"n" Value used for the Downstream Routing

Location	Between Station Nos.	"n" Value	Comment
East Dam North (w/o block walls)	All Downstream Stations	0.025	Because the area of inundation is considered 100 percent developed and the walls adjacent to the main flow path (Baseline Road and Kyrene Road) are considered flattened, the flow path will be overvegetated cover around trees and cars. .025 is a reasonable median "n" value for these conditions.
North Dam 2 (w/o block walls)	All Downstream Stations	0.018	The North Dam flow path has large areas of undeveloped land. This was considered when choosing an appropriate "n" value.
East Dam North (w/block walls)	Station 0.017 to Station 0.1174	0.024	From the dam to Station 0.1174, the flow path is within the City of Guadalupe which is 100 percent developed, but has no major flow path to contain the flow. It will be allowed to spread out and, consequently, have shrub trees and broken walls in its flow path. An "n" value of .024 was used for this area.
	Station 0.1174 to Station 0.7509	0.022	From Station 0.1174 to Station 0.7509, the flow is partially restricted to Priest Road and Baseline Road. Roughness is decreased due to increased percentage of roadway area. An "n" value of .022 was used to simulate these conditions.
	Station 0.7509 to Station 1.3229	0.020	From Station 0.7509 to the end, the flow will be contained in Baseline Road and Kyrene Road with their right-of-ways. An "n" value of 0.020 was used for this area.
North Dam 2 (w/block walls)	Station 0.0379 to Station 2.9242	0.020	From Station 0.00 to Station 2.9242, the flow will follow 48th Street with some of the flow conveyed in the landscaped right-of-way area. An "n" value of 0.020 is appropriate for these conditions.

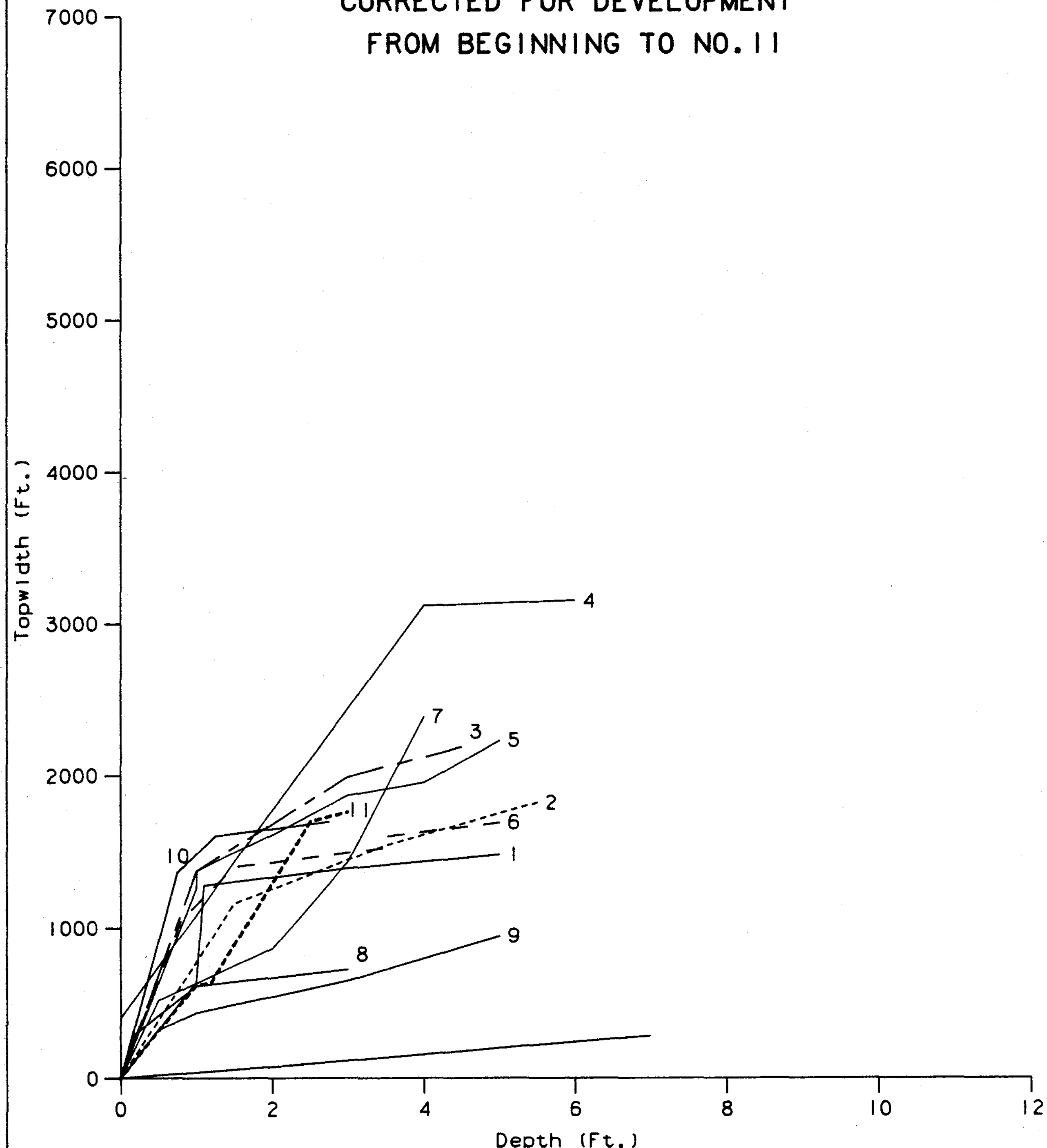
Table 14 (Continued)

"n" Value used for the Downstream Routing

Location	Between Station Nos.	"n" Value	Comment
North Dam 2 (w/block walls) (continued)	Station 2.9242 to Station 3.4943	0.022	From Station 2.9242 to Station 3.4943, the flow is not completely contained in 48th Street and can break out into areas of development. The increase in "n" value is appropriate to simulate these flow conditions.
	Station 3.4943 to Station 4.1411	0.024	From Station 3.4943 to Station 4.1411, the flow splits and runs in Broadway Road, south of Broadway Road in the industrial area, north of Broadway Road in the industrial area, and in a collector channel adjacent to I-10. To model these conditions, a composite "n" value of .024 was used.
	Station 4.1411 to Station 4.8987	0.035	For the remainder of the flow reach to the Salt River, the flow has mostly left Broadway Road and is following a very rough non-defined channel area. An "n" value of 0.035 was used for this area.
Typical Dam (East Dam)	All Downstream Stations	0.020	This is considered a good median "n" value to be used to simulate the average roughness characteristics between north and east downstream valleys.

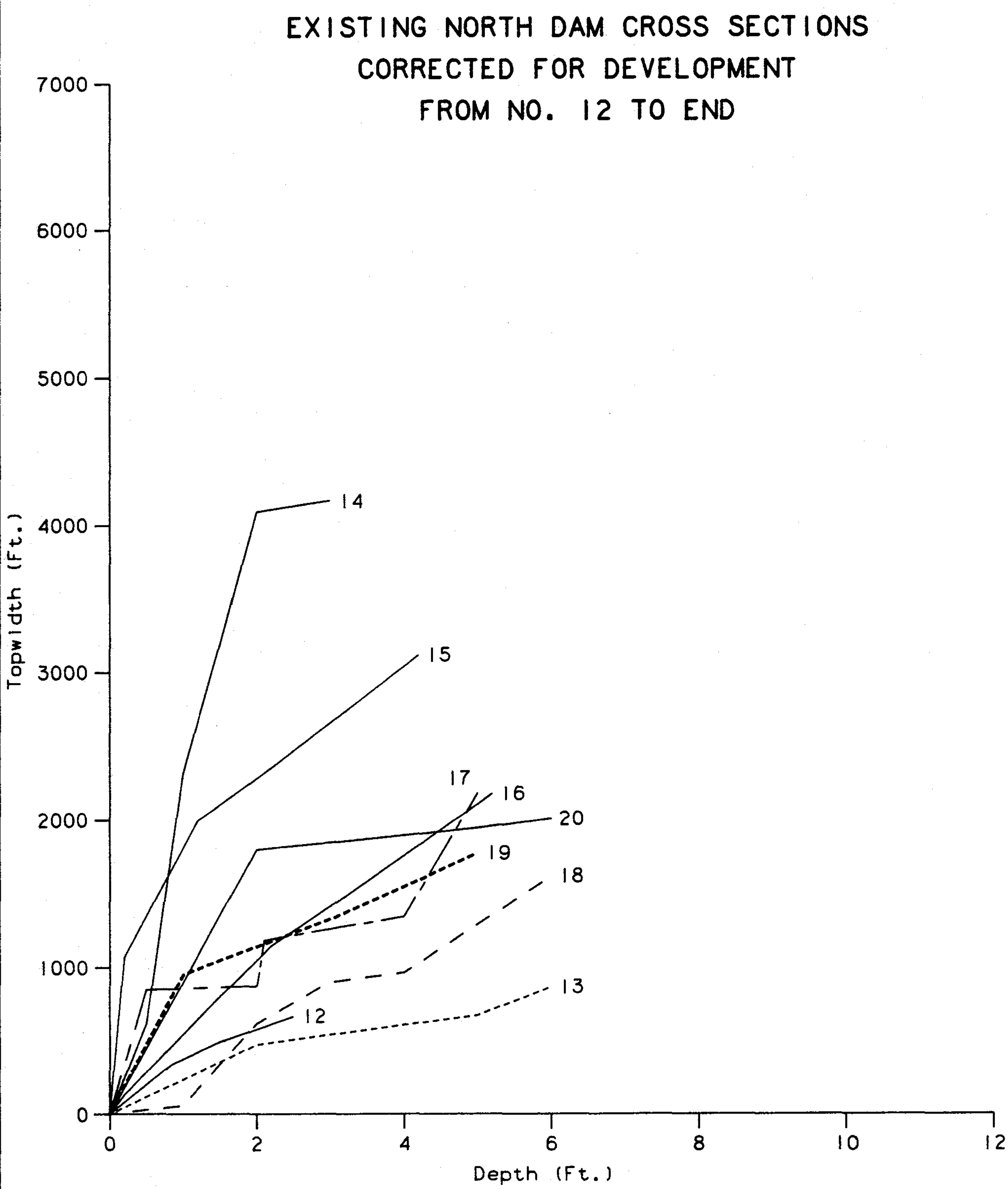
FIGURE 14A

EXISTING NORTH DAM CROSS SECTIONS
CORRECTED FOR DEVELOPMENT
FROM BEGINNING TO NO.11



GREINER

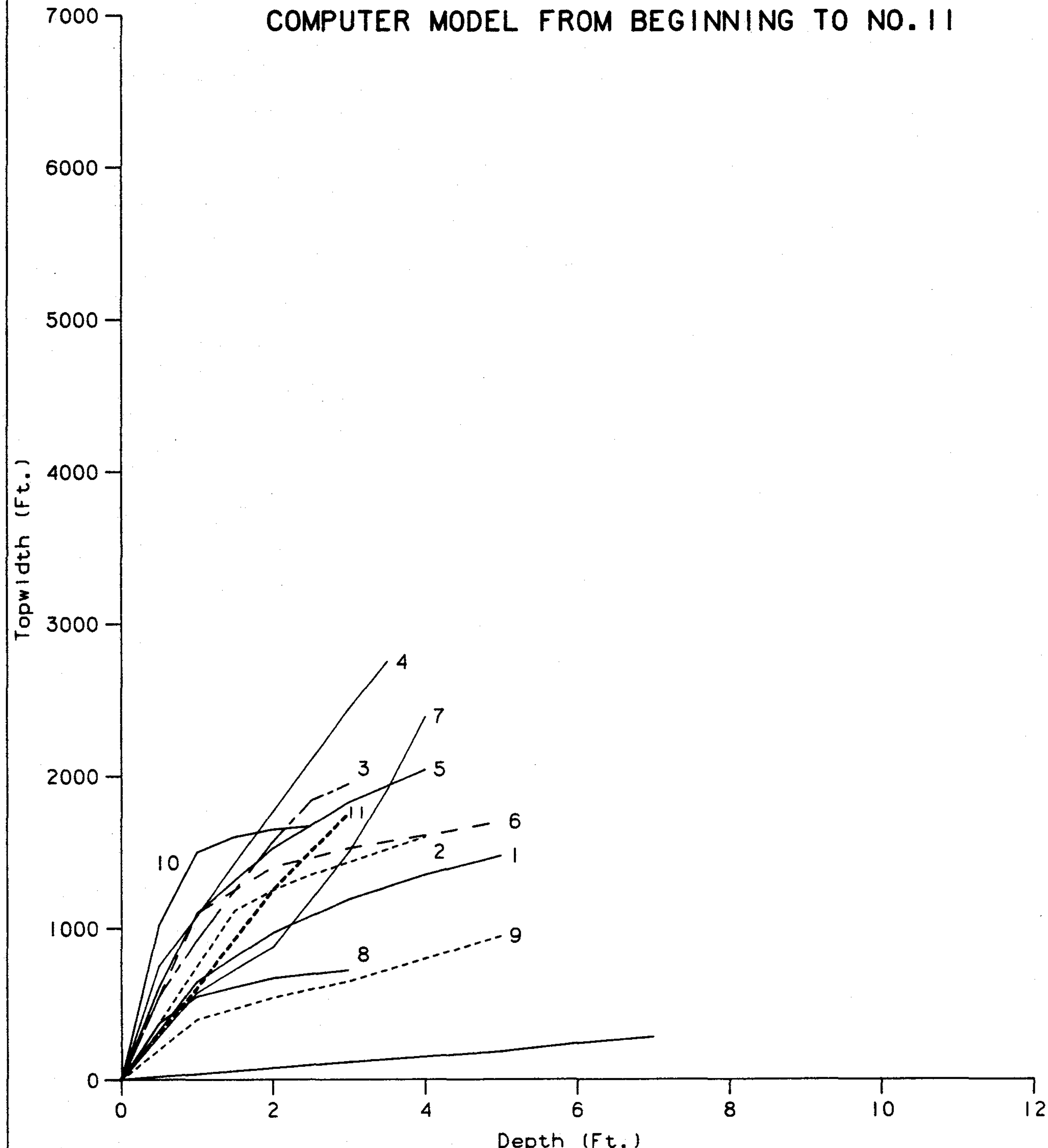
FIGURE 14B



GREINER

FIGURE 15A

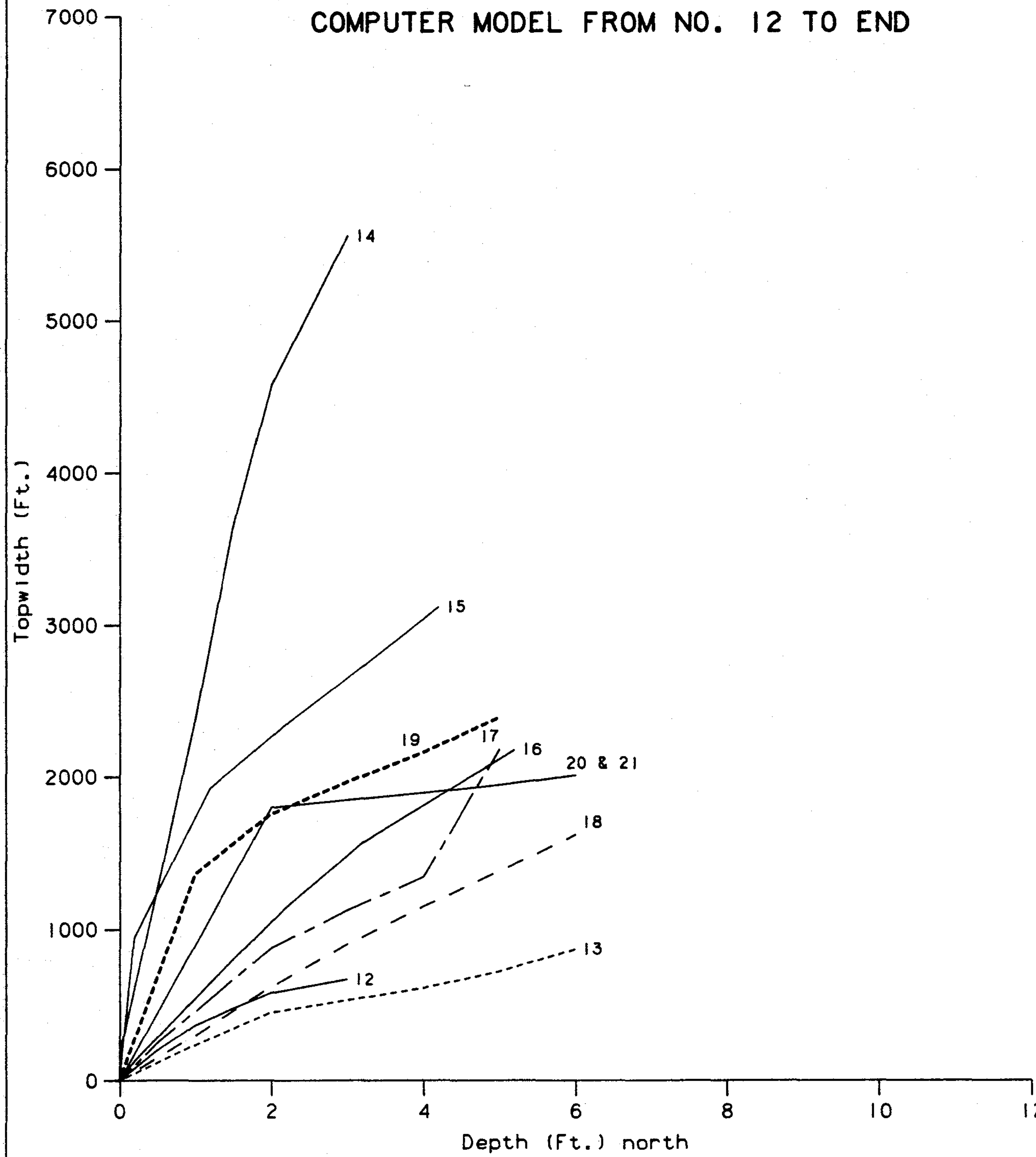
NORTH DAM CROSS SECTIONS USED IN
COMPUTER MODEL FROM BEGINNING TO NO.11



GREINER

FIGURE 15B

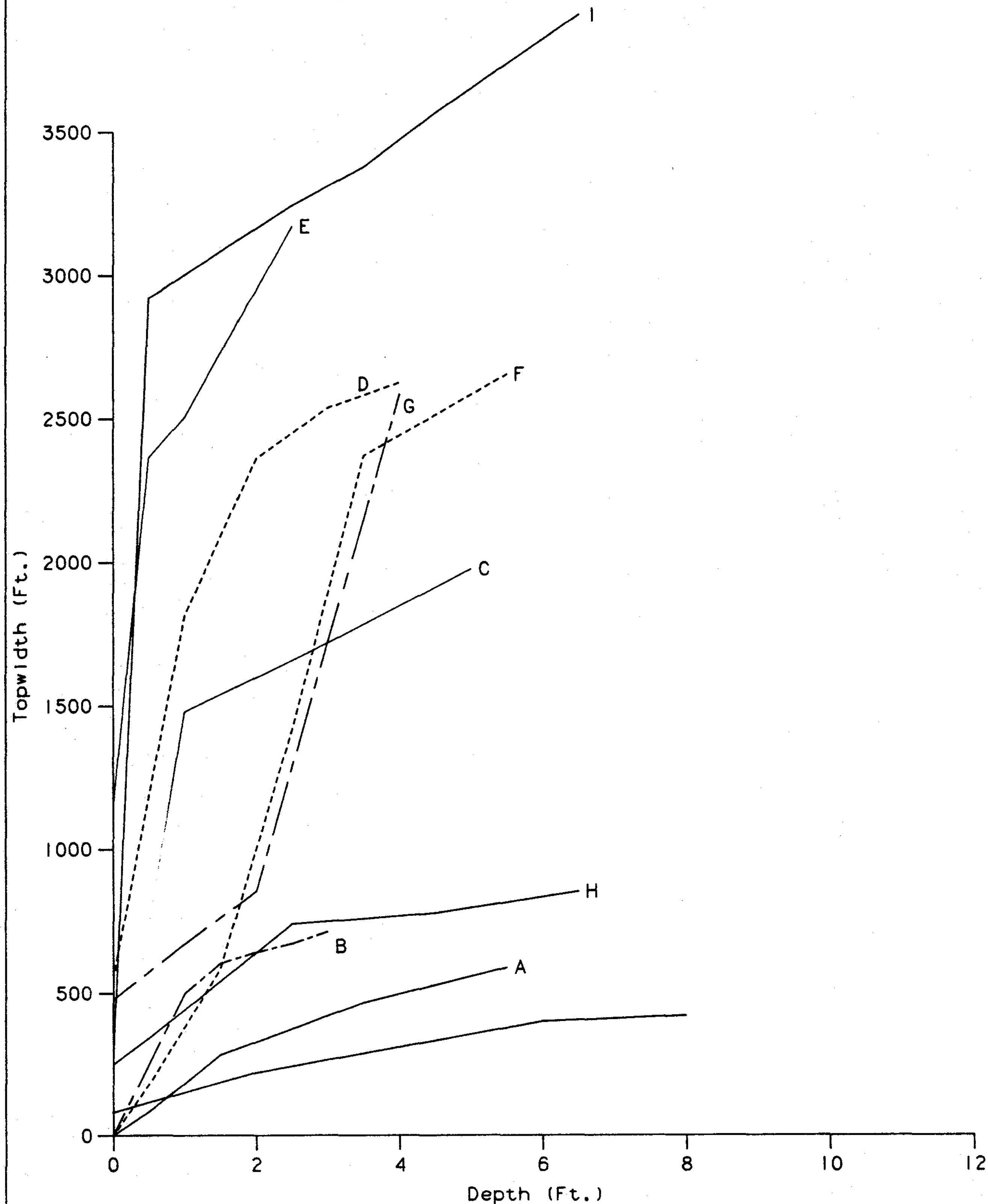
NORTH DAM CROSS SECTIONS USED IN COMPUTER MODEL FROM NO. 12 TO END



GREINER

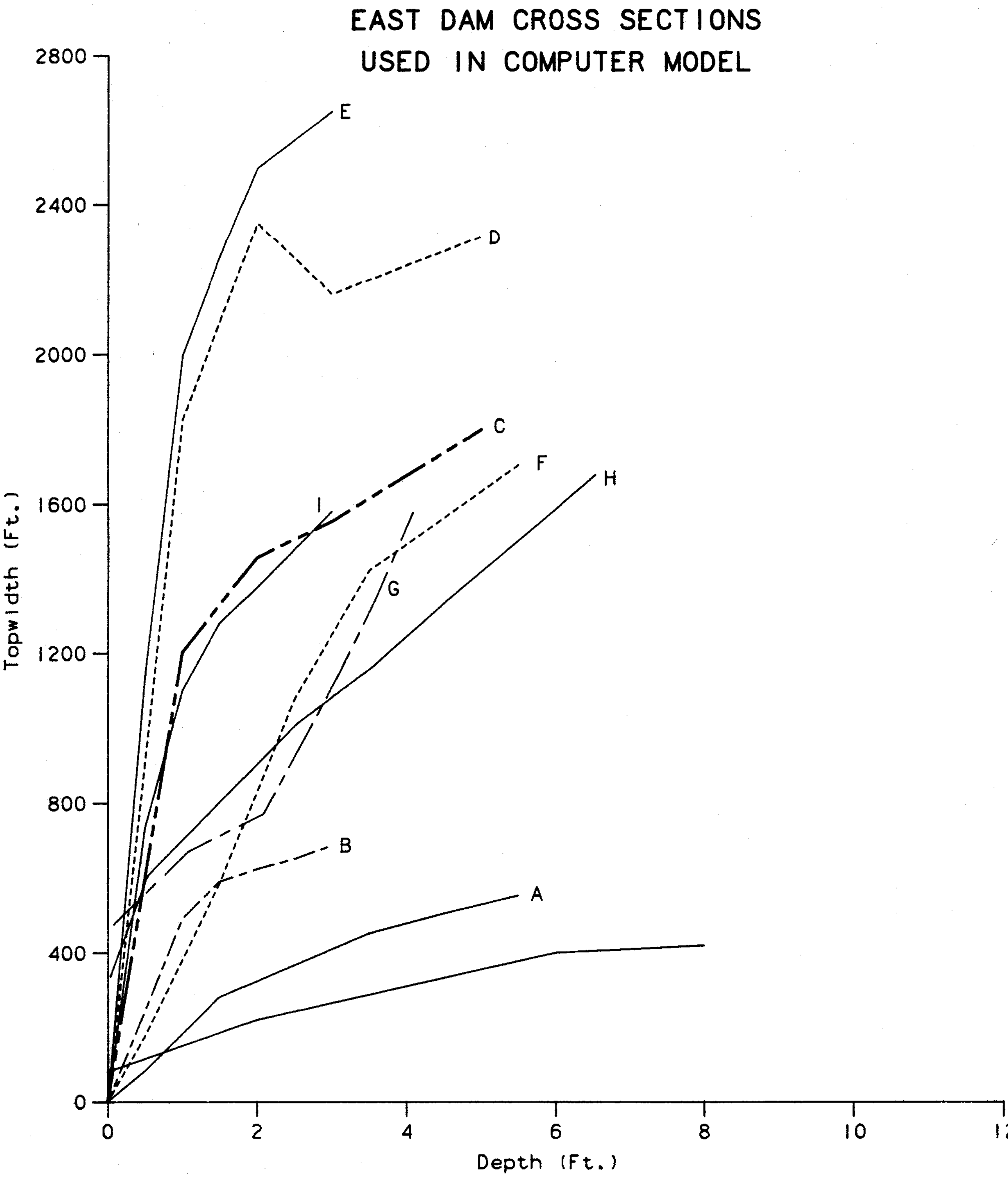
FIGURE 16

EXISTING EAST DAM CROSS SECTIONS
CORRECTED FOR DEVELOPMENT



GREINER

FIGURE 17



GREINER

On the east side of the Interstate Highway 10 (I-10), existing block walls could be seen within an area enclosed by 56th Street, Kyrene Road, Superstition Freeway and Baseline Road. Some block walls also exist along the Highline Canal and its neighborhood. On the west side of I-10, block walls were constructed along the Highline Canal and Western Canal between I-10 and 48th Street, along 48th Street between Southern Avenue and Baseline Road, within the neighboring subdivisions, and between Maricopa Freeway and Broadway Road within the inundation areas.

In the event of East Dam failure, water will flow to the area between 56th Street and I-10 due to the block wall located along the Highline Canal on the east of 56th Street. The main exits for the flow could be 56th Street, Baseline Road, Hardy Road and Kyrene Road with water ponding between 56th Street and I-10 and south of the Superstition Freeway.

In the event of either North Dam failure, water will flow mainly along 48th Street and some small streets in the neighboring subdivisions. When the water gets to Broadway Road, it flows mainly along this road with water ponding in areas between Broadway Road and the Maricopa Freeway due to the existing block walls located in streets such as 40th Street and other quarter-mile streets.

For the downstream routing of North Dams, the results for both with- and without-block-wall conditions are shown in Tables 15 and 16 respectively. The graphs in Appendix D show the velocity, maximum depth and discharge versus the distance along the centerline from the dam. The water profile along the routing channel is also provided in the Appendix.

Table 15

Summary of Dambreak Results for North Dam - Without-Block-Wall Condition

Cross Section Number	Distance From Dam Mile	Maximum Elevation Feet	Maximum Flow cfs	Time to Maximum Elevation Hours	Time to Flood Elevation* Hours
1	0.133	1,242.43	9,912	0.32	**
2	0.322	1,226.32	9,859	0.33	**
3	0.417	1,216.28	9,783	0.37	**
4	0.549	1,207.59	9,799	0.37	**
5	0.682	1,198.34	9,788	0.39	**
6	0.824	1,190.16	9,777	0.40	**
7	1.051	1,180.13	9,703	0.43	0.22
8	1.178	1,172.96	9,694	0.45	**
9	1.269	1,169.69	9,686	0.46	0.26
10	1.557	1,158.58	9,644	0.48	**
11	1.600	1,157.23	9,641	0.50	0.31
12	1.701	1,154.44	9,611	0.52	0.32
13	1.867	1,148.78	8,523	0.70	0.35
14	2.296	1,138.33	7,782	0.90	**

Table 15 (Continued)

Summary of Dambreak Results for North Dam - Without-Block-Wall Condition

Cross Section Number	Distance From Dam Mile	Maximum Elevation Feet	Maximum Flow cfs	Time to Maximum Elevation Hours	Time to Flood Elevation* Hours
15	2.609	1,133.18	7,829	1.05	**
16	2.924	1,130.80	8,577	1.18	0.50
17	3.206	1,126.38	7,587	1.43	0.54
18	3.494	1,123.89	8,446	1.68	0.78
19	3.824	1,118.71	7,295	1.98	**
20	4.141	1,114.19	7,142	2.26	1.31
21	4.899	1,100.79	7,160	2.93	**

*Flood elevation was set at two feet above channel bottom.
**Maximum water elevation is less than two feet deep.

Table 16

Summary of Dambreak Results for North Dam - With-Block-Wall Condition

Cross Section Number	Distance From Dam Mile	Maximum Elevation Feet	Maximum Flow cfs	Time to Maximum Elevation Hours	Time to Flood Elevation* Hours
1	0.133	1,244.50	9,613	0.36	0.1
2	0.322	1,228.58	8,641	0.54	0.2
3	0.417	1,218.28	8,226	0.63	0.3
4	0.549	1,209.69	7,493	0.88	0.4
5	0.682	1,200.35	6,960	1.00	0.5
6	0.824	1,192.31	6,653	1.15	0.5
7	1.051	1,181.85	6,140	1.38	0.7
8	1.178	1,174.21	6,013	1.38	0.8
9	1.269	1,170.93	5,980	1.45	0.9
10	1.557	1,160.42	5,672	1.68	1.1
11	1.600	1,157.41	5,646	1.68	1.1
12	1.701	1,154.25	5,624	1.74	1.1
13	1.867	1,148.75	5,608	1.74	1.2
14	2.296	1,139.72	4,124	2.52	1.6
15	2.609	1,134.24	3,755	3.27	2.5
16	2.924	1,131.19	3,486	3.72	2.8
17	3.206	1,126.26	3,430	4.09	3.0
18	3.494	1,124.30	3,274	4.46	3.3
19	3.824	1,119.47	3,228	4.88	3.6
20	4.141	1,116.56	3,030	5.28	4.1
21	4.899	1,103.84	2,668	6.24	5.1

*Flood elevation was set at two feet above channel bottom.

Comparatively, the water depth along the downstream routing under with-block-wall condition is higher. The increase in water depth at the same location between these two conditions varies from zero to three feet. The maximum flow is highly decreased with with-block-wall condition. The discharge to the Salt River is 2,668 cfs instead of 7,160 cfs for without-block-wall conditions. The model with with-block-wall condition is considered as the most appropriate. The attenuation of the peak dam breach flow from 10,003 cfs to 2,668 cfs is a result of the vast floodplain storage capacity.

For the East Dam, the results for both with- and without-block-wall conditions are shown in Tables 17 and 18 respectively. The graphs in Appendix C show the velocity, maximum depth and discharge versus the distance along the centerline from the dam. The water profile along the routing channel is also provided in the Appendix.

Table 17

Summary of Dambreak Results for East Dam - Without-Block Wall Condition

Cross Section Letter	Distance*** From Dam Mile	Maximum Elevation Feet	Maximum Flow cfs	Time to Maximum Elevation Hours	Time to Flood Elevation* Hours
A	0.1975	1,247.49	8,861	0.28	0.11
B	0.3220	1,237.16	8,837	0.29	0.17
C	0.4782	1,224.51	8,782	0.31	**
D	0.6487	1,213.41	8,722	0.34	**
E	0.8258	1,204.32	8,647	0.39	**
F	1.0653	1,190.94	8,496	0.44	0.32
G	1.2311	1,184.10	8,424	0.48	0.41
H	1.3182	1,181.51	8,109	0.57	0.54
I	1.4157	1,176.44	8,045	0.67	**

*Flood elevation was set at two feet above channel bottom.
**Maximum water elevation is less than two feet deep.
***For station conversions and computer outputs, see Table 19.

Table 18

Summary of Dambreak Results for East Dam - With-Block-Wall Condition

Cross Section Letter	Distance** From Dam Mile	Maximum Elevation Feet	Maximum Flow cfs	Time to Maximum Elevation Hours	Time to Flood Elevation* Hours
A	0.1975	1,247.43	8,827	0.29	0.1
B	0.3220	1,237.70	8,687	0.32	0.2
C	0.4782	1,225.62	8,176	0.40	0.2
D	0.6487	1,215.07	7,306	0.65	0.3
E	0.8258	1,205.56	6,719	0.90	0.6
F	1.0653	1,190.61	6,580	1.02	0.8
G	1.2311	1,185.24	6,425	1.14	0.9
H	1.3182	1,182.05	6,223	1.31	0.9
I	1.4157	1,177.36	6,098	1.39	1.0

*Flood elevation was set at two feet above channel bottom.
**For station conversions and computer outputs, see Table 19.

Comparatively, the water depth along the downstream routing under with-block-wall condition is higher and varies in the range of 0 to 1.5 feet. The maximum flow for with-block-wall conditions is approximately 2,000 cfs lower.

The inundation maps were prepared based on the results of with-block-wall conditions because they simulate more closely the actual conditions of dam failure.

The inundation area maps were developed based on the results of the dambreak study for the downstream routings of the dams with a scale of 1 inch = 1,000 feet. All cross-section lines used for modeling were shown on the maps with the floodplain boundaries delineated. Since approximately 90 percent of the flooding area was common for either breaching location at the East Dam, and 80 percent common for either breaching at North Dam No. 1 or North Dam No. 2 location, the inundation area map was prepared for the breach location covering the maximum impact areas.

From the results of inundation area routing, the areas with different water depths were identified on the inundation area map with 1-foot increments. The cross section number and its distance along the channel, maximum stage, time to maximum stage, estimated flood elevation and time to flood elevation were tabulized in Figure 18. The peak time to a flood depth of two feet for water traveling downstream of the dam is shown with 0.25 hour intervals for the East Dam and 0.5-hour intervals for the North Dams (see Figure 19).

The maximum inundation areas during the breach of either North Dam No. 1 or North Dam No. 2 was identified on the map (see Figure 18). The east and north boundaries of the inundation area are close to the west and south edges of I-10 excluding the high elevation location near the Westcourt in the Butte. The west and south boundaries are at approximately one-quarter mile west of 48th Street and one-quarter mile south of Broadway Road to the Salt River. The total inundation area is approximately 2.8 square miles and is broken down into smaller areas for different water depths.

The study results indicated that when either of the North Dams breaches, half of the areas between 48th Street and I-10, and north to Southern Avenue will be inundated with a water depth of more than three feet. This area is approximately 0.7 square mile. It covers almost 95 percent of the Gosnell's development located north of the North Dams, including the warehouses. Approximately 65 percent of this area is residential subdivisions located between Baseline Road and Southern Avenue. Some commercial properties at the intersection of Southern Avenue and 48th Street will be subject to three feet or more inundation. Another area which will suffer a water depth of more than three feet is located north of Broadway Road between 32nd Street and 42nd Street. Most of this area is used for light industry and multi-family residences.

7. INUNDATION AREA ROUTING

Table 19

Station Table for East Dam Inundation Map

Inundation Map and Report Stationing	East Dam Computer Stationing
*	0.0170
0.1975	0.1174
0.3220	0.2348
0.4782	0.3883
0.6487	0.5701
0.8258	0.7509
1.0653	0.9716
1.2311	1.1382
1.3182	1.2254
1.4157	1.3229

*Not shown on map.

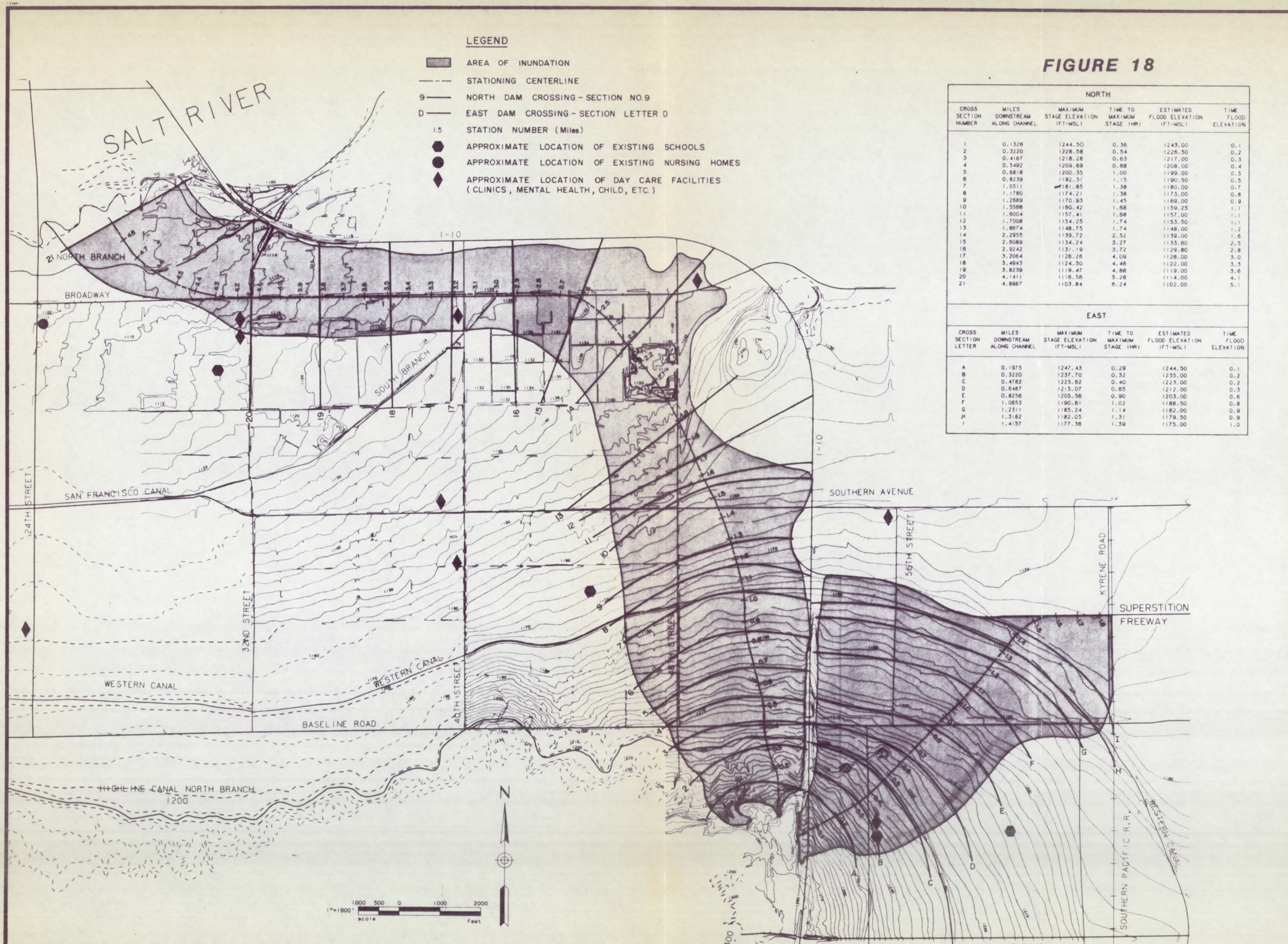


FIGURE 18

NORTH					
CROSS SECTION NUMBER	MILES DOWNSTREAM ALONG CHANNEL	MAXIMUM STAGE ELEVATION (FT-MSL)	TIME TO MAXIMUM STAGE (HR)	ESTIMATED FLOOD ELEVATION (FT-MSL)	TIME FLOOD ELEVATION
1	0.1326	1244.50	0.36	1243.00	0.1
2	0.3220	1228.58	0.54	1226.50	0.2
3	0.4187	1218.28	0.63	1217.00	0.3
4	0.5492	1209.69	0.88	1208.00	0.4
5	0.6818	1200.35	1.00	1199.00	0.5
6	0.8239	1192.31	1.15	1190.50	0.5
7	1.0511	1181.85	1.38	1180.00	0.7
8	1.1780	1174.21	1.38	1173.00	0.8
9	1.2689	1170.93	1.45	1169.00	0.9
10	1.5568	1160.42	1.68	1159.25	1.1
11	1.6004	1157.41	1.68	1157.00	1.1
12	1.7008	1154.25	1.74	1153.50	1.1
13	1.8674	1148.75	1.74	1148.00	1.2
14	2.2955	1139.72	2.52	1139.00	1.6
15	2.6089	1134.24	3.27	1133.80	2.5
16	2.9242	1131.19	3.72	1129.80	2.8
17	3.2064	1126.26	4.09	1126.00	3.0
18	3.4943	1124.30	4.46	1122.00	3.3
19	3.8239	1119.47	4.88	1119.00	3.6
20	4.1411	1116.56	5.28	1114.00	4.1
21	4.8987	1103.84	6.24	1102.00	5.1

EAST					
CROSS SECTION LETTER	MILES DOWNSTREAM ALONG CHANNEL	MAXIMUM STAGE ELEVATION (FT-MSL)	TIME TO MAXIMUM STAGE (HR)	ESTIMATED FLOOD ELEVATION (FT-MSL)	TIME FLOOD ELEVATION
A	0.1975	1247.43	0.29	1244.50	0.1
B	0.3220	1237.70	0.32	1235.00	0.2
C	0.4782	1225.62	0.40	1223.00	0.2
D	0.6487	1215.07	0.65	1212.00	0.3
E	0.8258	1205.56	0.90	1203.00	0.6
F	1.0653	1190.61	1.02	1188.50	0.8
G	1.2311	1185.24	1.14	1182.00	0.9
H	1.3182	1182.05	1.31	1179.50	0.9
I	1.4157	1177.36	1.39	1175.00	1.0

GUADALUPE FLOOD RETARDING STRUCTURE
DAM FAILURE INUNDATION MAP - 1 of 2

Greiner
7310 N. 16th Street Suite 160 Phoenix Arizona 85020 602 275 3400
555 East River Road Suite 100 Tucson Arizona 85704 602 887 1800

FLOOD CONTROL DISTRICT OF
MARICOPA COUNTY
3335 W. Durango Street Phoenix, Arizona 85009 / 602 262-1501

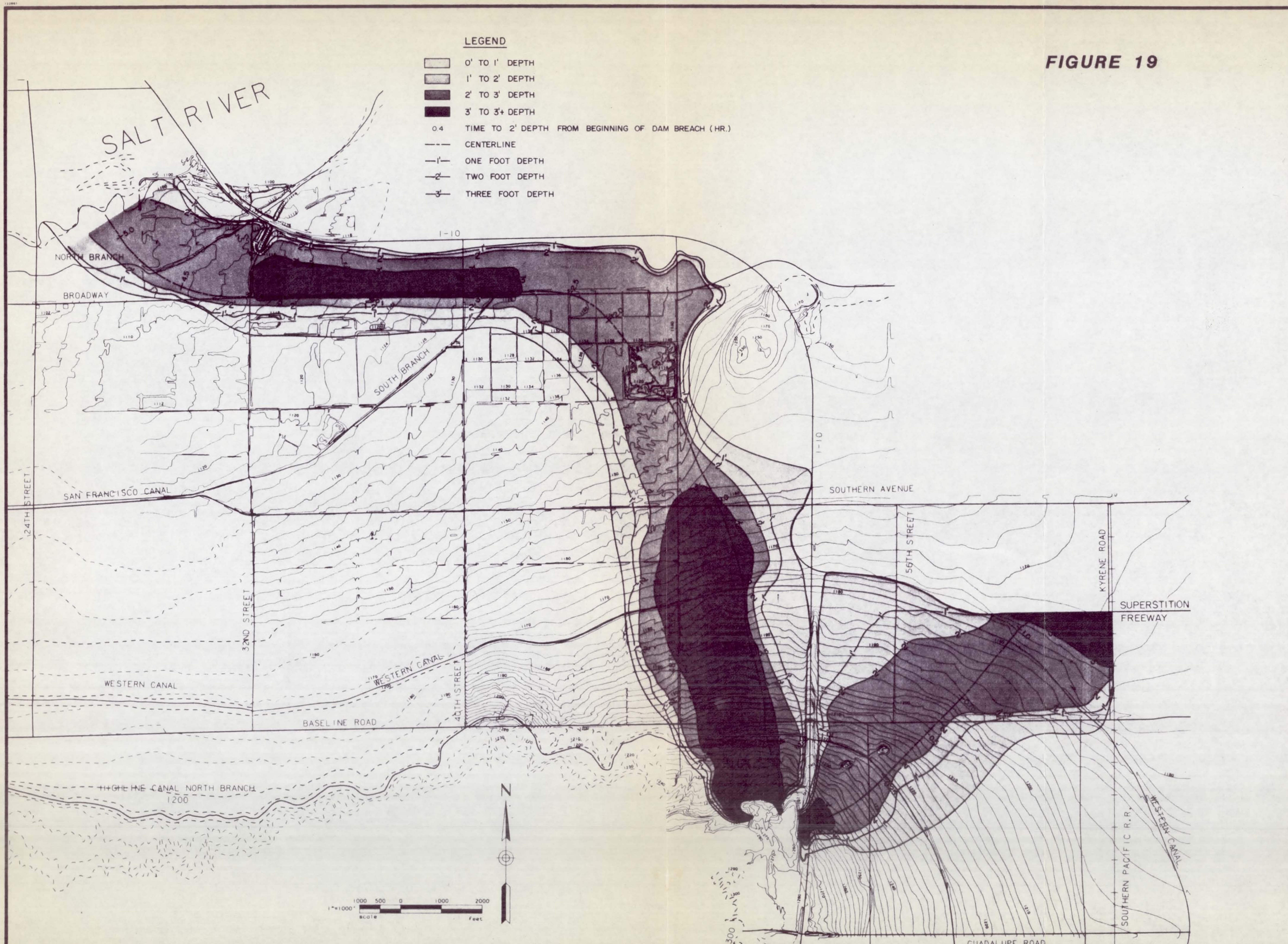


FIGURE 19

GUADALUPE FLOOD RETARDING STRUCTURE
DAM FAILURE INUNDATION MAP – 2 of 2

Greiner
7310 N 16th Street Suite 160 Phoenix Arizona 85020 602 275-5400
555 East River Road Suite 100 Tucson Arizona 85704 602 887-1800

FLOOD CONTROL DISTRICT OF
MARICOPA COUNTY
3335 W. Durango Street Phoenix, Arizona 85009 / 602 262-1501

7. INUNDATION AREA ROUTING

There is a 1/16-square-mile pit located west of 48th Street and 1/4-mile south of Broadway Road. This pit area was treated as off-site storage in the dambreak modeling and will be filled up with two to three feet of water above the level ground.

There are three daycare facilities, including clinics, mental health, child care, etc., which will be flooded with one to two feet of water. The approximate locations for these facilities are at Broadway Road near 32nd Street, 40th Street and 48th Street (see Figure 18).

Figure 20 also shows the time required to fill the inundation area with two feet of water after dam the is breached. It will take about five hours to reach the area near the Salt River which is located approximately five miles downstream from the North Dam.

The maximum inundation area during the breach of the East Dam was identified in Figure 18. The west and north boundaries of the inundation area are along the east edge of I-10 and south edge of the Superstition Freeway. The east and south boundaries are at approximately Kyrene Road and one-half mile south of Baseline Road running northeasterly to Kyrene Road and Baseline Road. The total inundation area is approximately 1.3 square miles. Figure 19 shows the inundated areas delineated with various water depths.

The study results indicate that during the breach of the East Dam, two areas will suffer serious flooding of more than three feet of water. The size of these areas is approximately 0.1 square mile. The first area is located right at the downstream of the dam. It covers an area of approximately 0.03 square mile within the City of Guadalupe. The second area is located near the intersections of the Superstition Freeway and Kyrene Road.

The size is approximately 0.07 square mile. Approximately half of this area covers a mobile home area. Water will pond at this low area to a depth at which water begins overflowing into the Superstition Freeway and exits to the north side of Kyrene Road. This remote area can reach a water depth of two feet in an approximate time of one hour. There are school and daycare facilities located near 56th Street and one-half mile south of Baseline Road which will be flooded with one to two feet of water during dam failure. Most of the inundated areas are residential with approximately 5 percent of commercial and industrial uses.

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Appendix A

9. APPENDICES

Appendix A
Calculations

Porosity

East Dam

Average void ratio = 0.347
Ve = 0.347 Vs

$$\text{Porosity} = \frac{Ve}{V} = \frac{Ve}{Ve + Vs} = \frac{0.347\ Vs}{0.347\ (Vs + Vs)} = \frac{0.347}{1.347} = 0.258$$

North Dam No. 1

Average void ratio = 0.334
Ve = 0.334 Vs

$$\text{Porosity} = \frac{0.334}{1.334} = 0.250$$

North Dam No. 2

Average void ratio = 0.386
Ve = 0.386 Vs

$$\text{Porosity} = \frac{0.386}{1.386} = 0.279$$

Breach Parametric Method

Breach in South Portion of East Dam

c = cohesion strength = 280 psf
ø = friction angle = 37.5°
Water surface elevation = 1,275.0
Elevation at base of dam (downstream side) = 1,263.0

$$H_{k1} = \frac{0.04\ (280)\ \text{Cos}\ (37.5)}{1 - \text{Cos}\ (90 - 37.5)} = 22.7$$

$$H_{k2} = \frac{= 0.04\ (280)\ \text{Cos}\ (37.5)\ \text{Sin}\ (45 - \frac{37.5}{2})}{1 - \text{Cos}\ (45 - \frac{37.5}{2})} = 77.3$$

$$H_{k3} = \frac{= 0.04\ (280)\ \text{Cos}\ (37.5)\ \text{Sin}\ (22.5 + \frac{3 \times 37.5}{4})}{1 - \text{Cos}\ (22.5 - \frac{37.5}{4})} = 262.9$$

H_d = 1,275 - 1,263 = 12 feet

0.8 H_d = 0.8 x 12 = 9.6 feet < H_{k1}

BB = breach width = $\frac{5}{3}$ (12) = 20 feet

0 = 90°

Breach in North Portion of East Dam

Water surface elevation = 1,275.0
Elevation at base of dam (downstream side) = 1,253.0

H_d = 1,275 - 1,253 = 22 feet

0.8 H_d = 0.8 x 22 = 17.6 feet < H_{k1}

BB = breach width = $\frac{5}{3}$ x 22 = 36.7 feet

0 = 90°

North Dam No. 1

Water surface elevation = 1,275.0
Elevation at base of dam (downstream side) = 1,256.0
ø = 38°
C = 280 psf

$$H_{k1} = \frac{0.04 \times (280)\ \text{Cos}\ (38)}{1 - \text{Cos}\ (90 - 38)} = 22.96$$

$$H_{k2} = \frac{= 0.04 \times (280)\ \text{Cos}\ (38)\ \text{Sin}\ (45 + \frac{38}{2})}{1 - \text{Cos}\ (45 - \frac{38}{2})} = 78.38$$

$$H_{k3} = \frac{0.04 \times (280)\ \text{Cos}\ (38)\ \text{Sin}\ (22.5 + \frac{3 \times 38}{4})}{1 - \text{Cos}\ (22.5 - \frac{38}{4})} = 267.61$$

H_d = 1,275 - 1,256 = 19.0 feet
0.8 H_d = 15.2 < H_{k1}

$$BB = \frac{5}{3} \times 19 = 31.7 \text{ feet}$$

$0 = 90°$

North Dam No. 2

Water surface elevation = 1,275.0
Elevation at base of dam (downstream side) = 1,245.0
$\phi = 37°$
$C = 280$ psf

$$H_{k1} = \frac{0.04 \times (280) \text{ Cos } (37)}{1 - \text{Cos } (90 - 37)} = 22.46$$

$$H_{k2} = \frac{0.04 \times (280) \text{ Cos } (37) \text{ Sin } (45 + \frac{37}{2})}{1 - \text{Cos } (45 - \frac{37}{2})} = 76.19$$

$$H_{k3} = \frac{0.04 \times (280) \text{ Cos } (37) \text{ Sin } (22.5 + \frac{3 \times 37}{4})}{1 - \text{Cos } (22.5 - \frac{37}{4})} = 258.34$$

$H_d = 1,275 - 1,245 = 30.0$ feet
$0.8\ H_d = 24 \text{ feet} > H_{k1}$

$$BB = \frac{5}{3}\ (22.46) = 37.43$$

$$0 = 45 + \frac{37}{2} = 63.5°$$

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Appendix B

9. APPENDICES

Appendix B

Graphs for Reservoir Storage Routing and
Reservoir Dynamic Routing for
Typical Dam
(Figures A Through L)

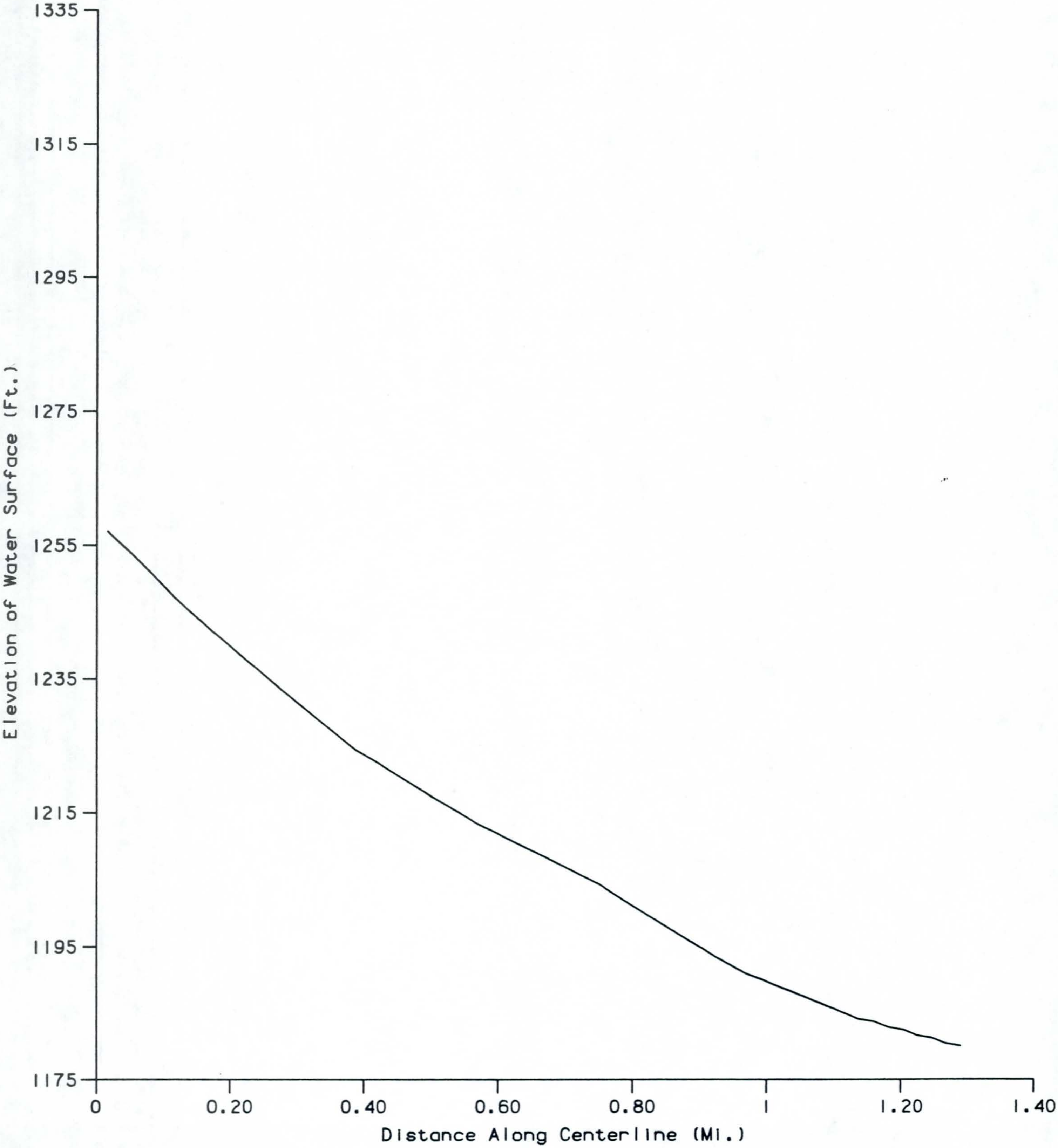

FIGURE A
ELEVATION (RESERVOIR ROUTING)

GREINER

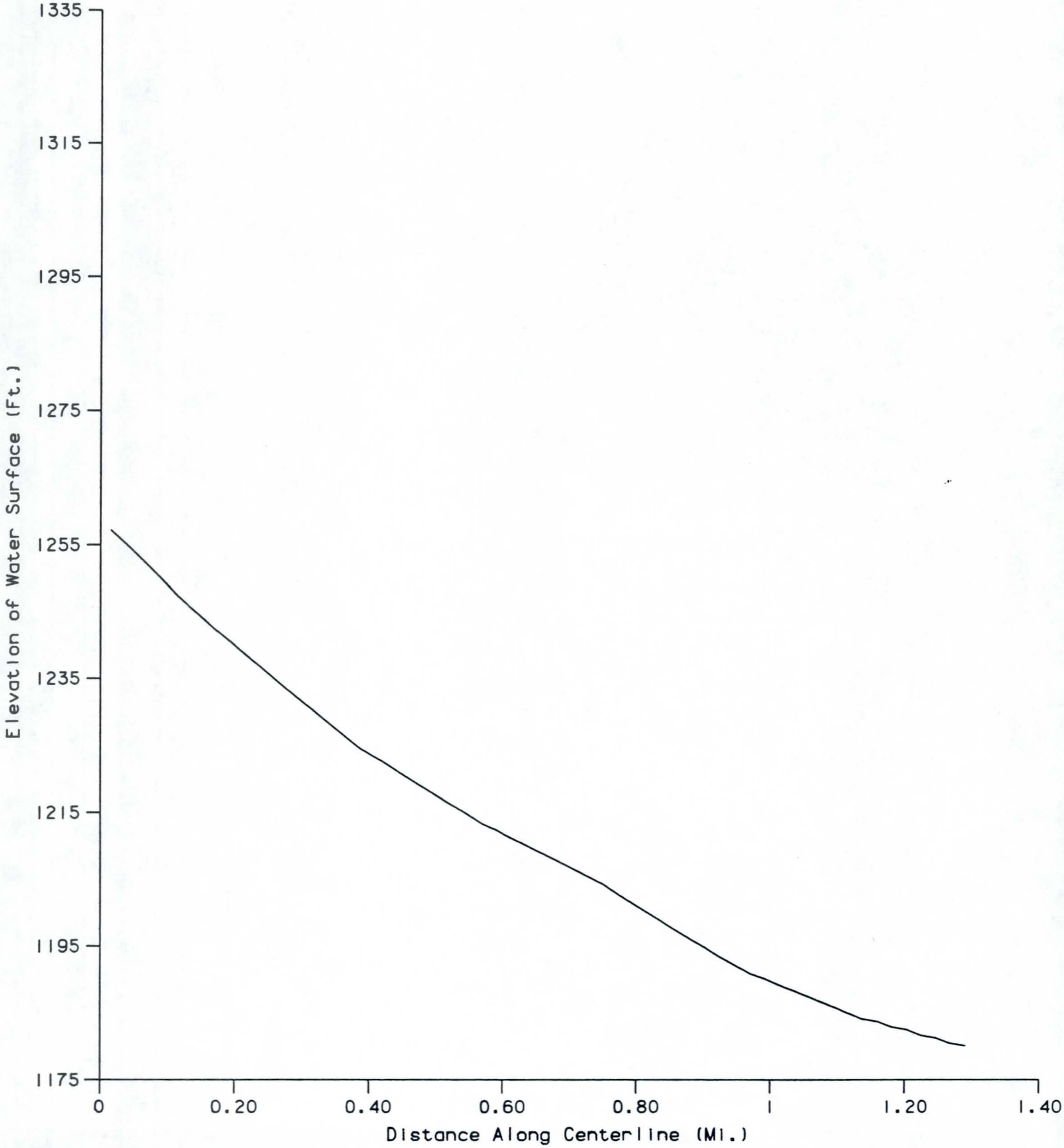

FIGURE B
ELEVATION (DYNAMIC ROUTING)

GREINER

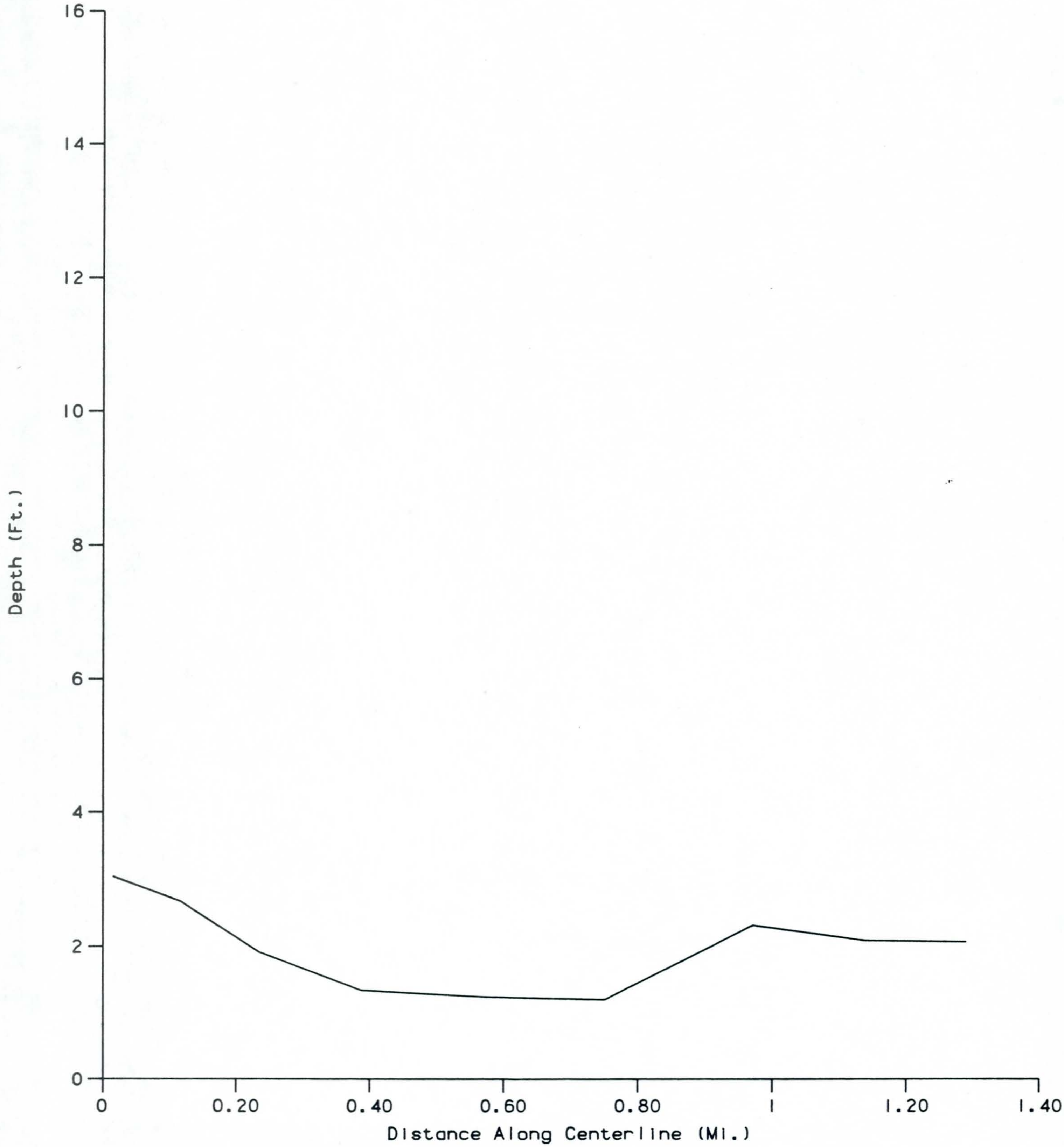

FIGURE C
DEPTH (RESERVOIR ROUTING)

GREINER

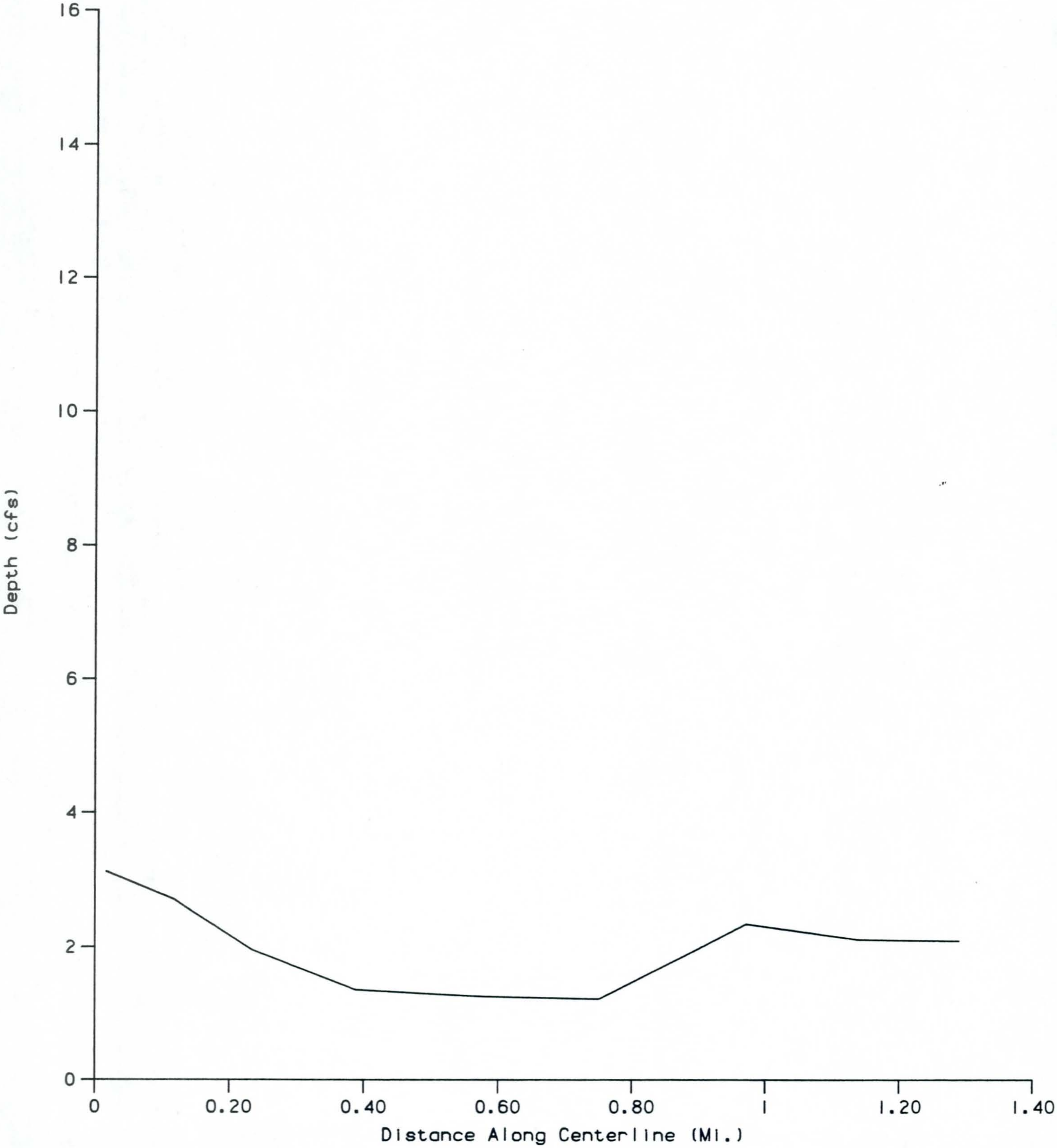

FIGURE D

DEPTH (DYNAMIC ROUTING)

GREINER

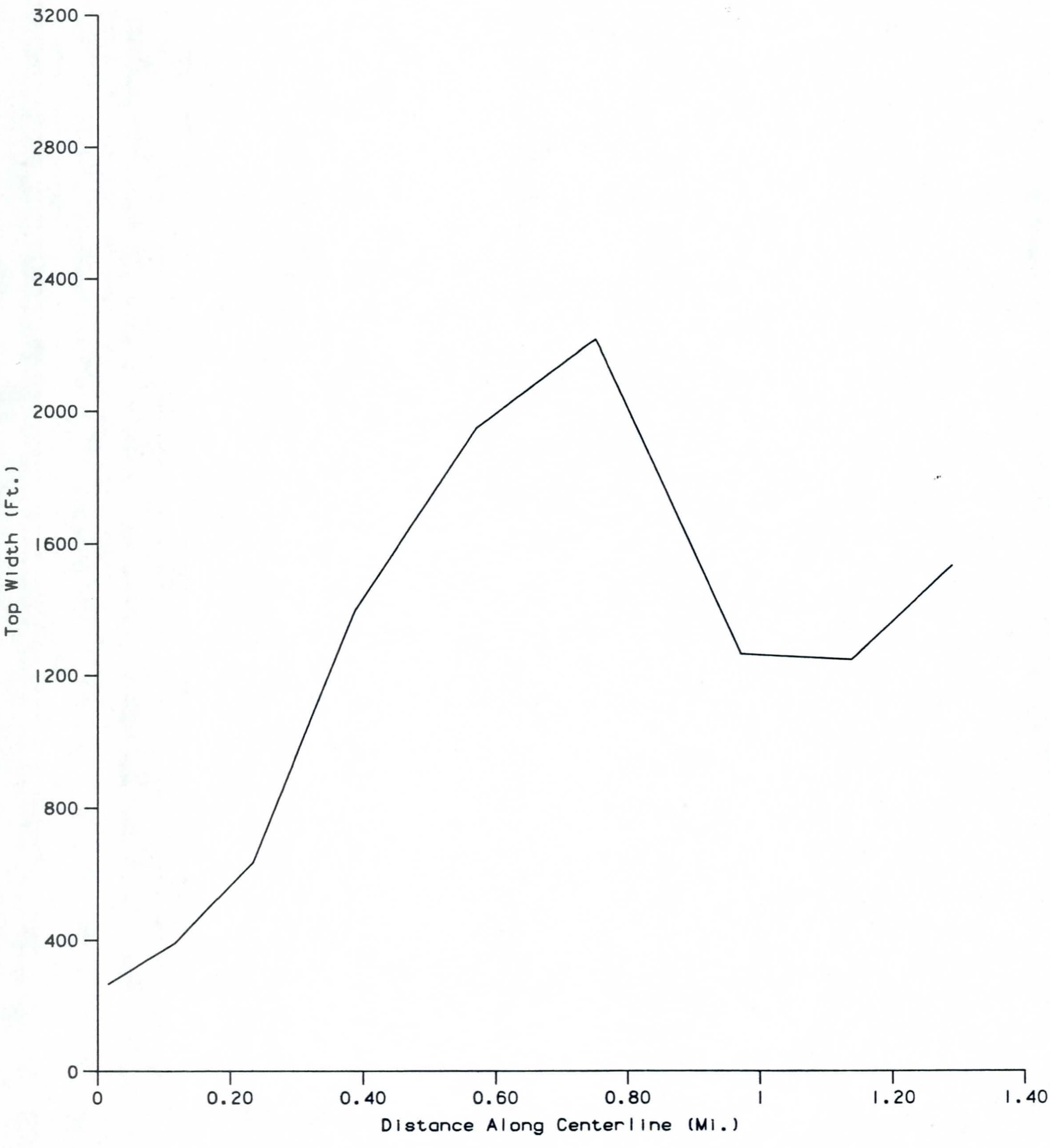

FIGURE E
TOP WIDTH (RESERVOIR ROUTING)

GREINER

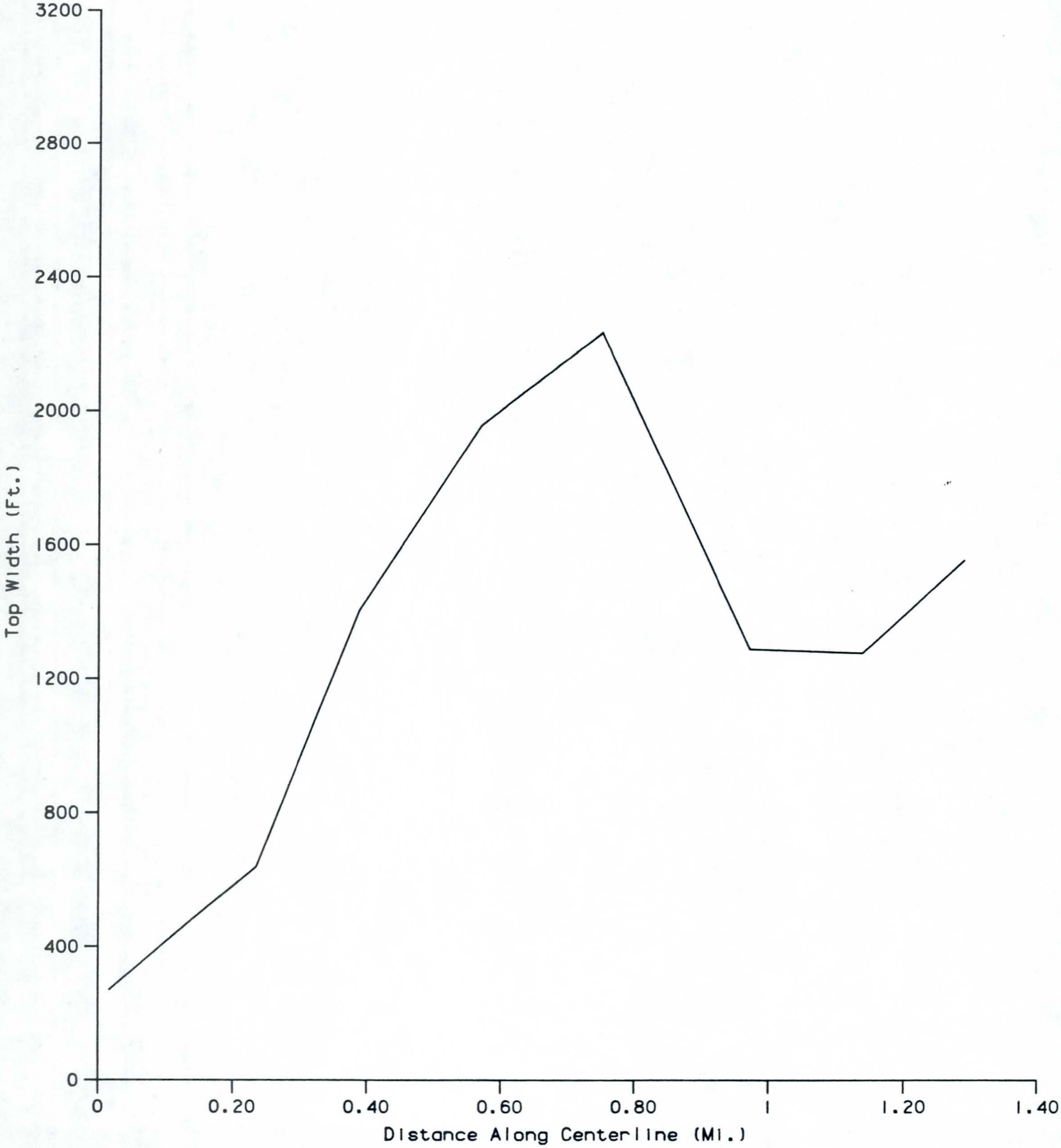

FIGURE F

TOP WIDTH (DYNAMIC ROUTING)

GREINER

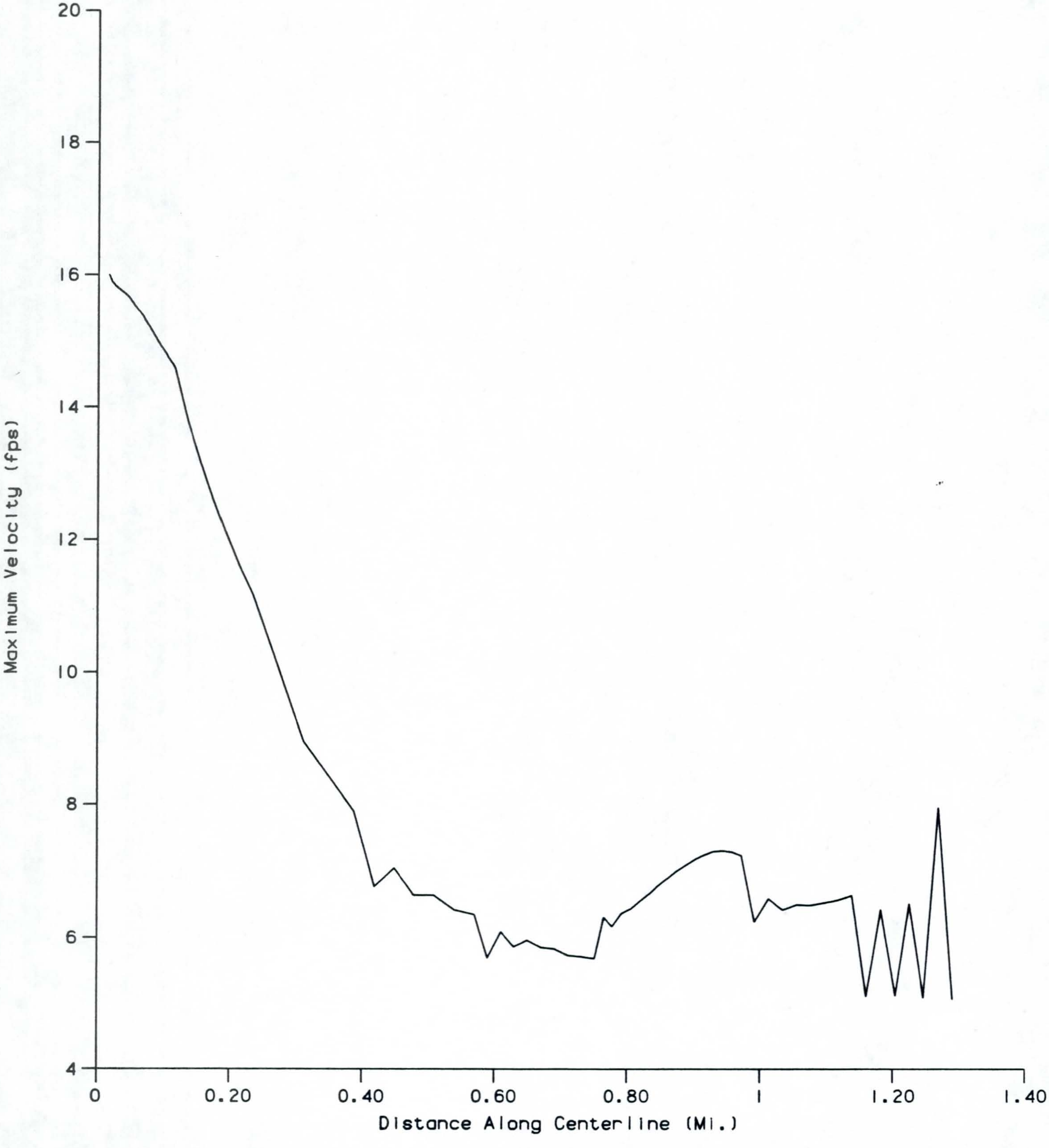

FIGURE G

VELOCITY (RESERVOIR ROUTING) GREINER

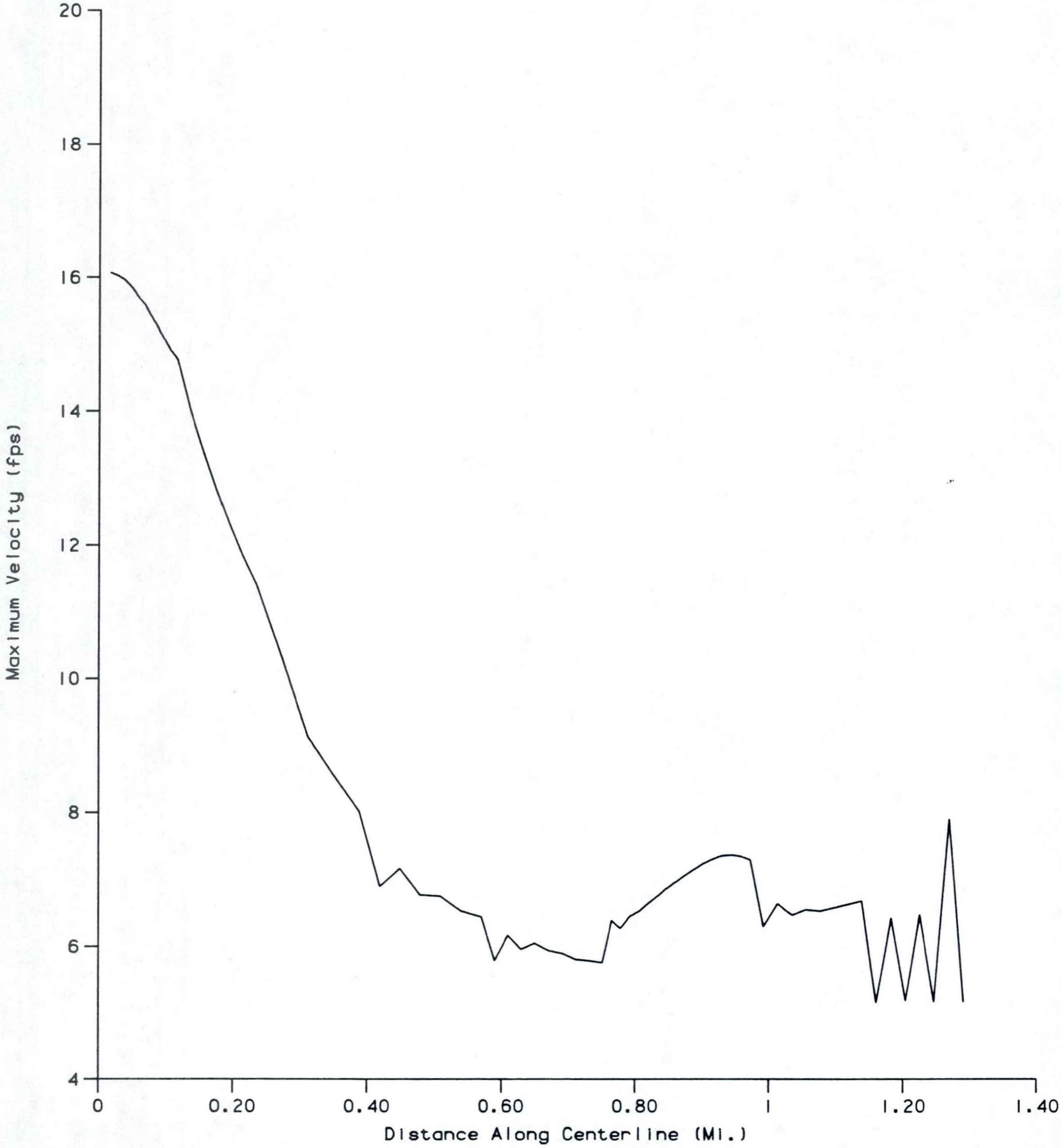

FIGURE H

VELOCITY (DYNAMIC ROUTING)

GREINER

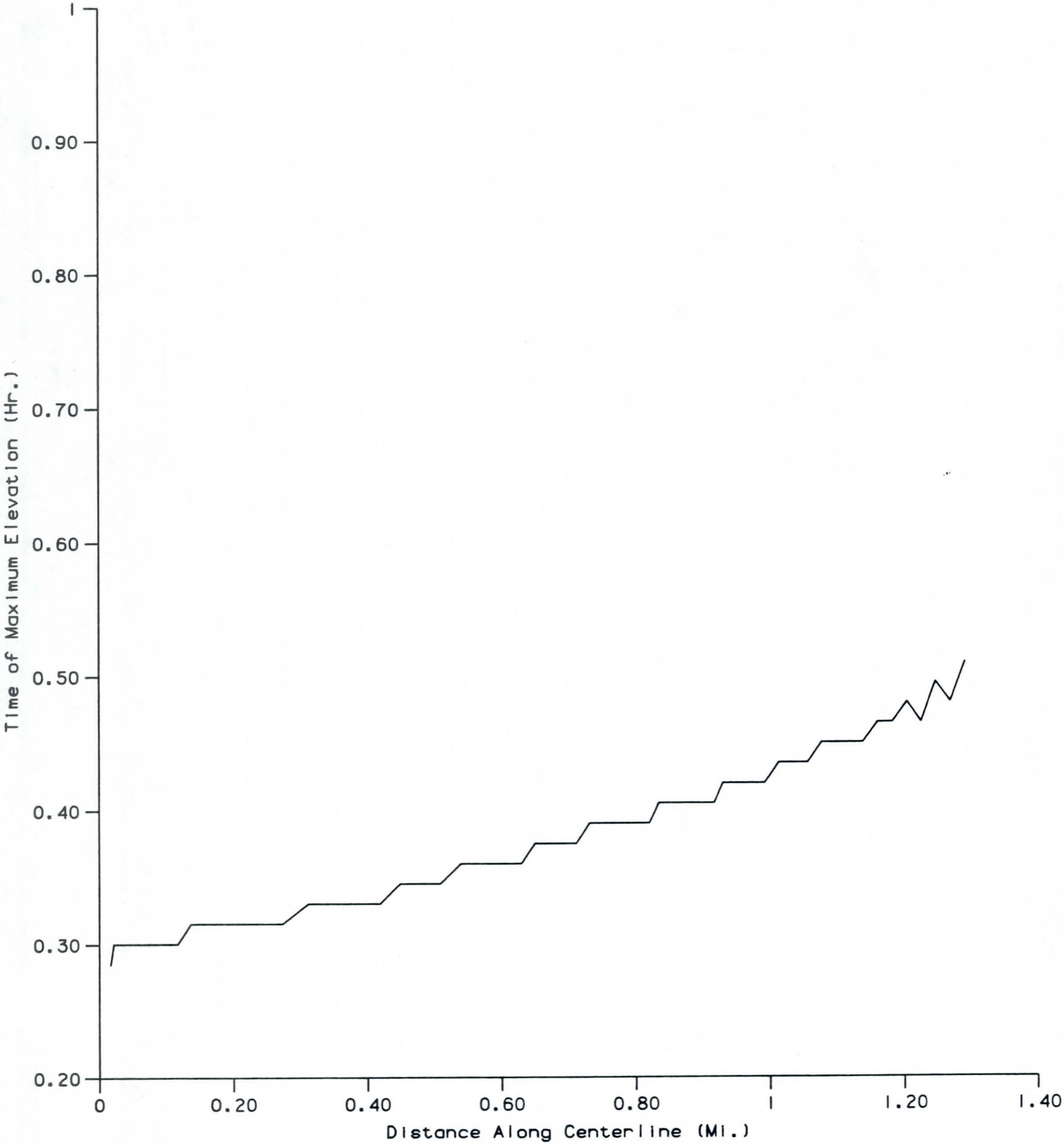

FIGURE I

TIME TO MAX. ELEVATION (RESERVOIR ROUTING) **GREINER**

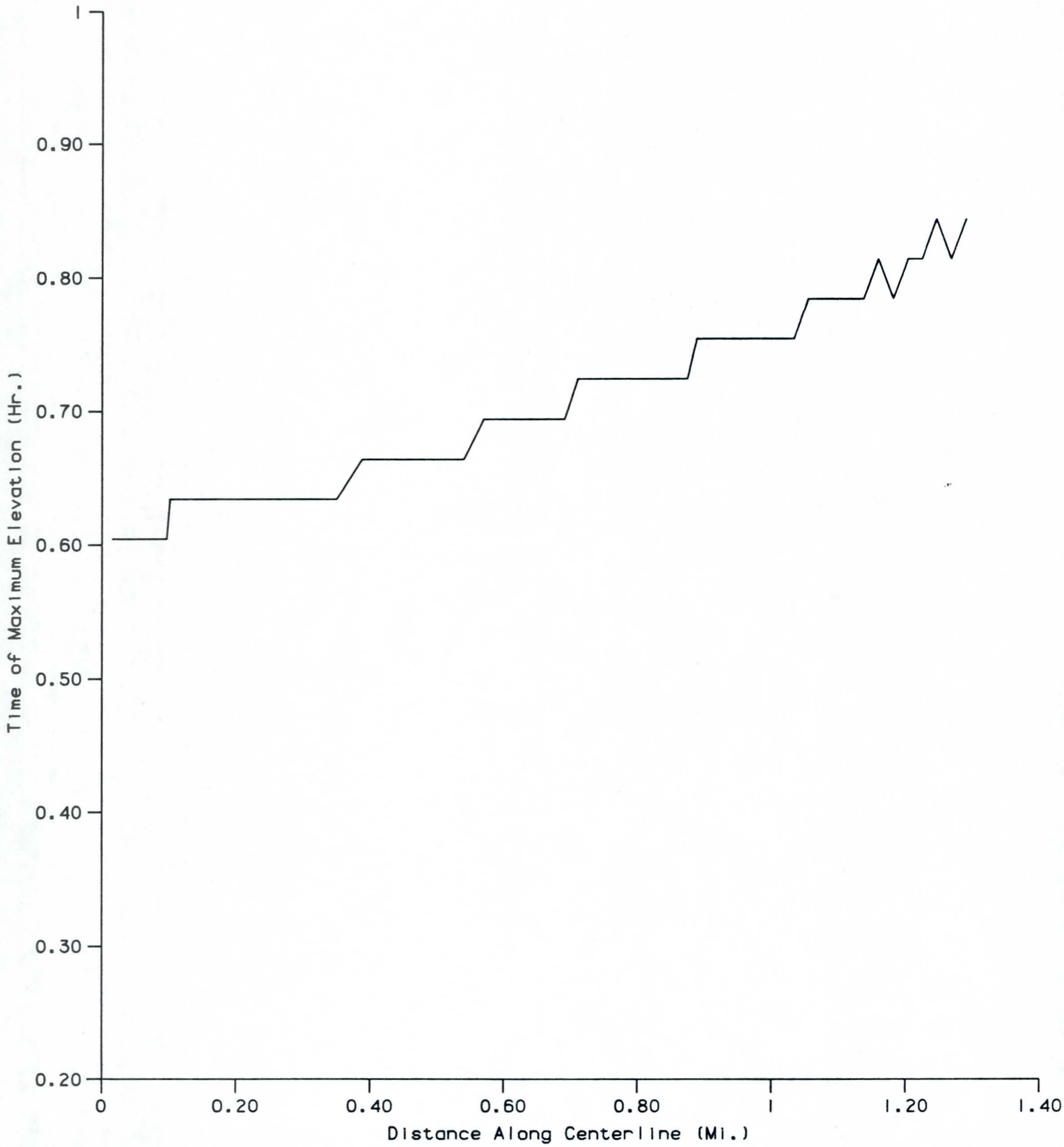

FIGURE J

TIME TO MAX. ELEVATION (DYNAMIC ROUTING) **GREINER**

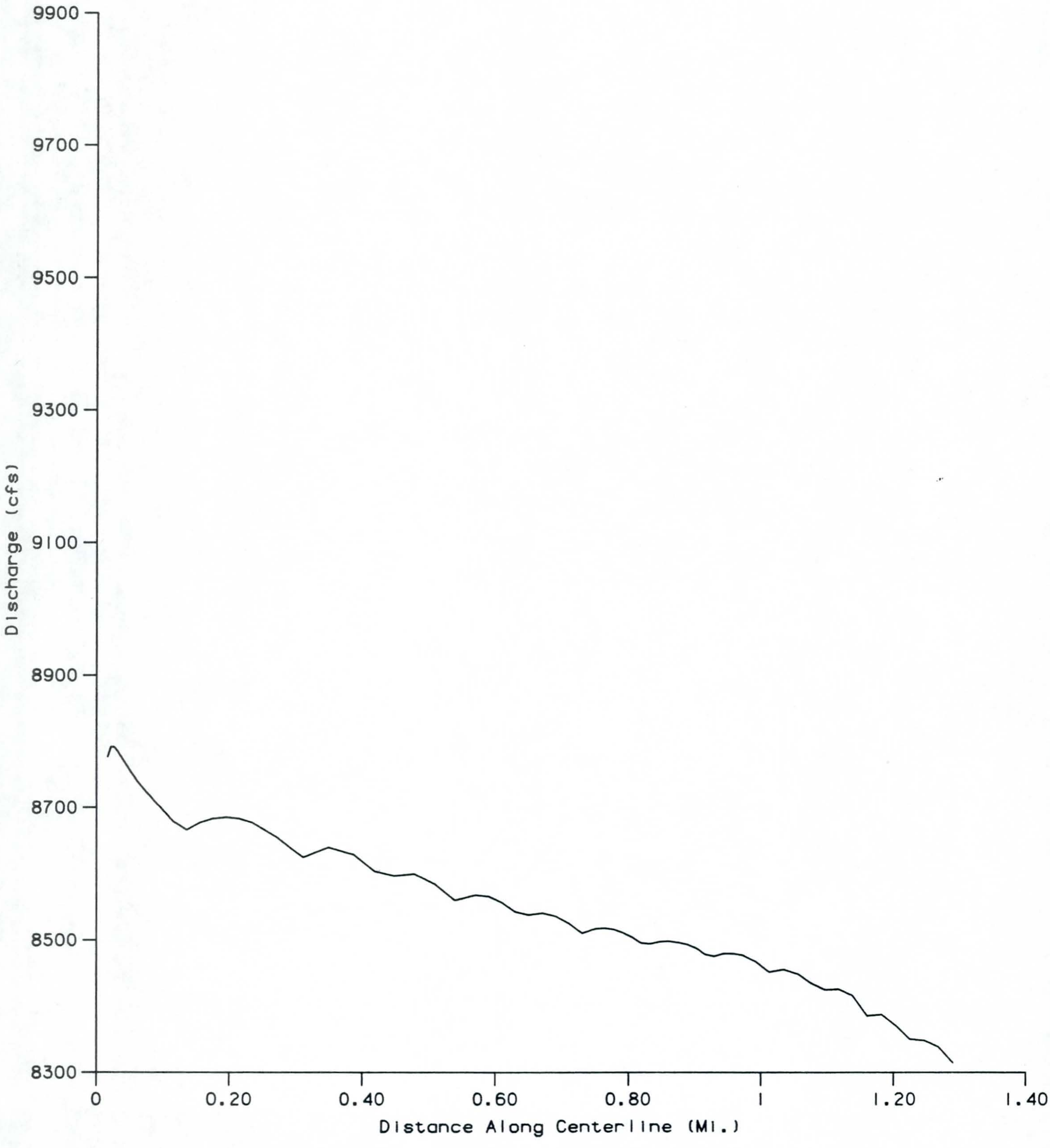

FIGURE K

DISCHARGE (RESERVOIR ROUTING)

GREINER

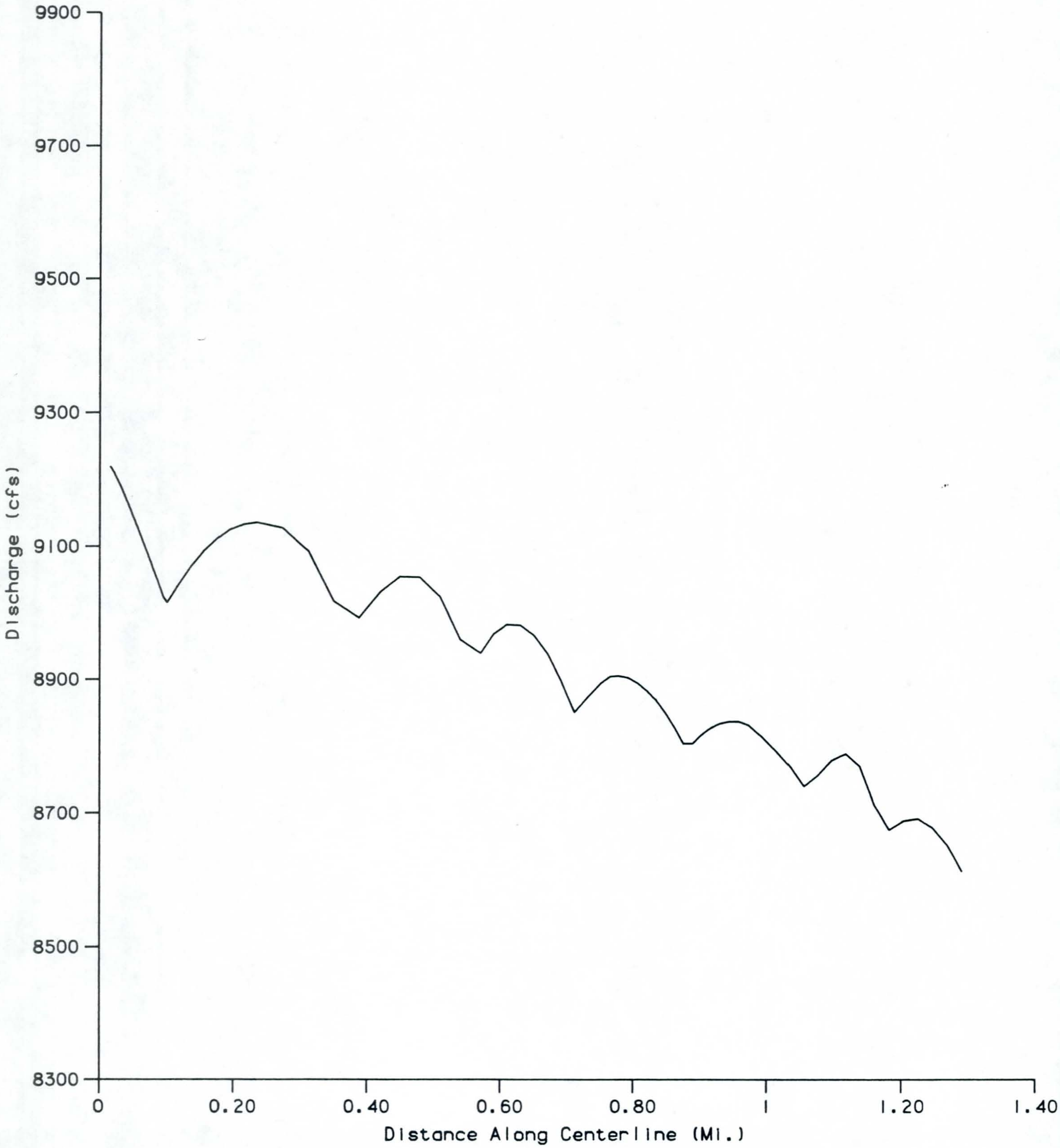

FIGURE L

DISCHARGE (DYNAMIC ROUTING)

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Appendix C

9. APPENDICES

Appendix C

Results of Downstream Routing for East Dam
(Figures A Through D)

FIGURE A

EAST DAM

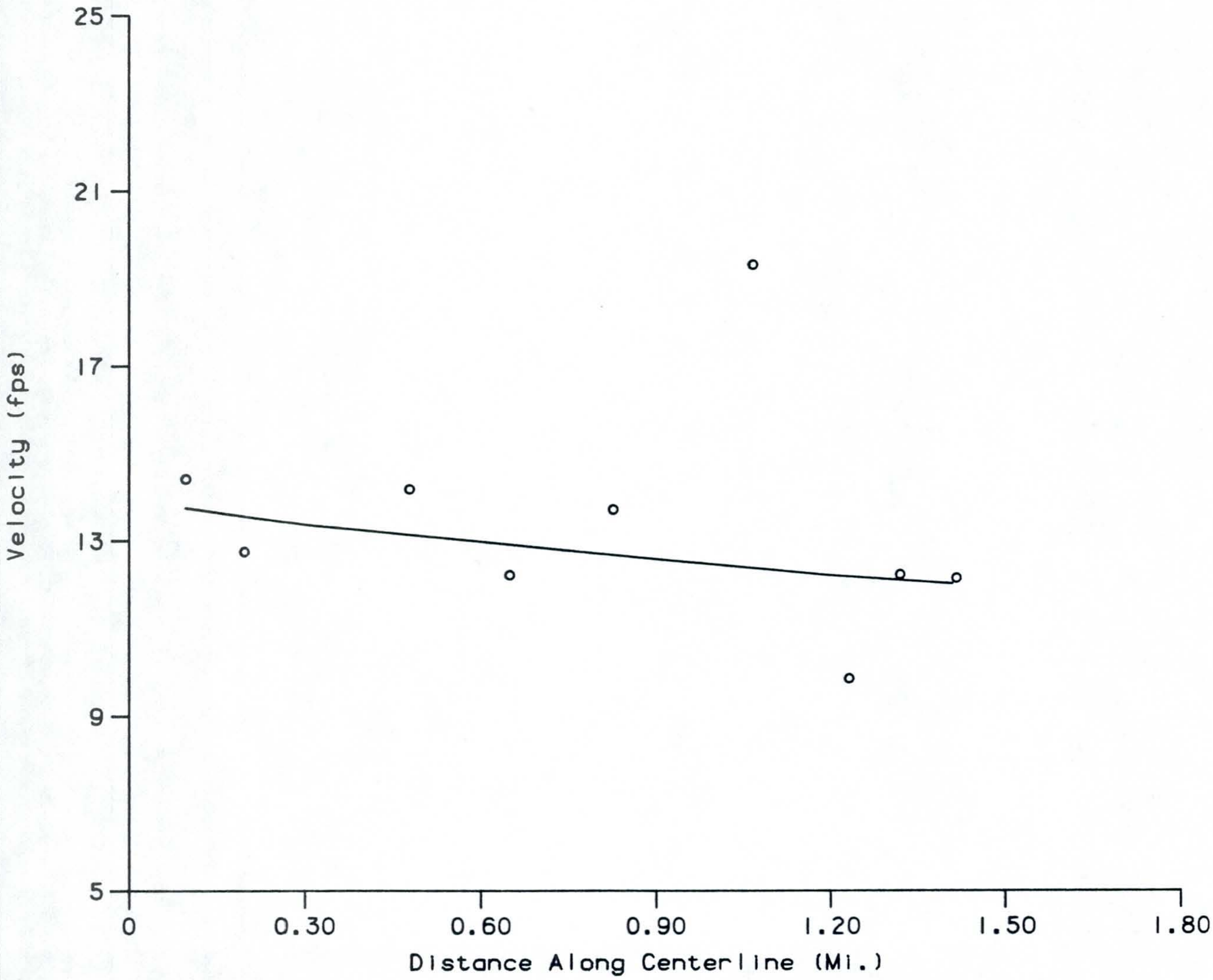

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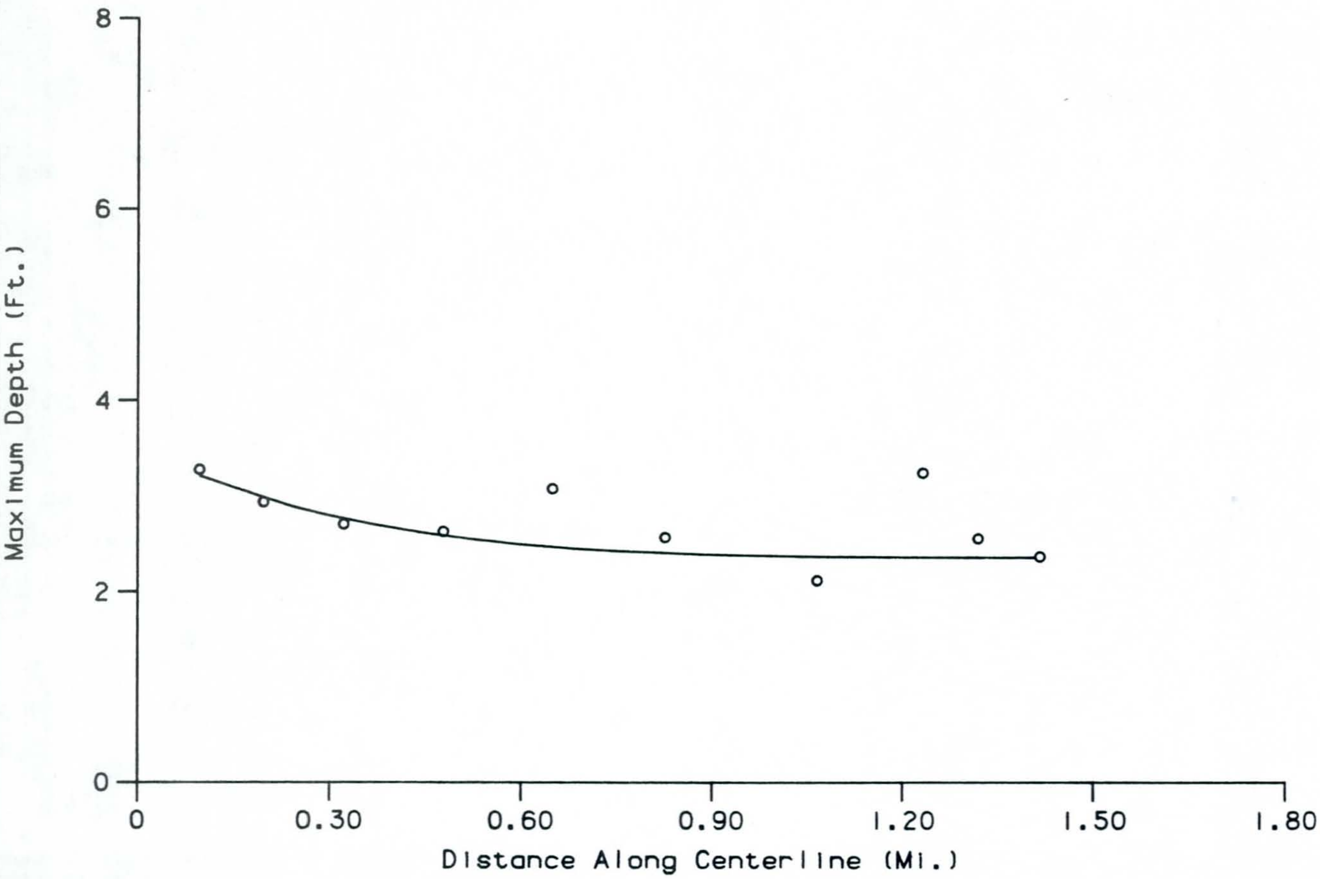
FIGURE B

EAST DAM



GREINER

FIGURE C

EAST DAM

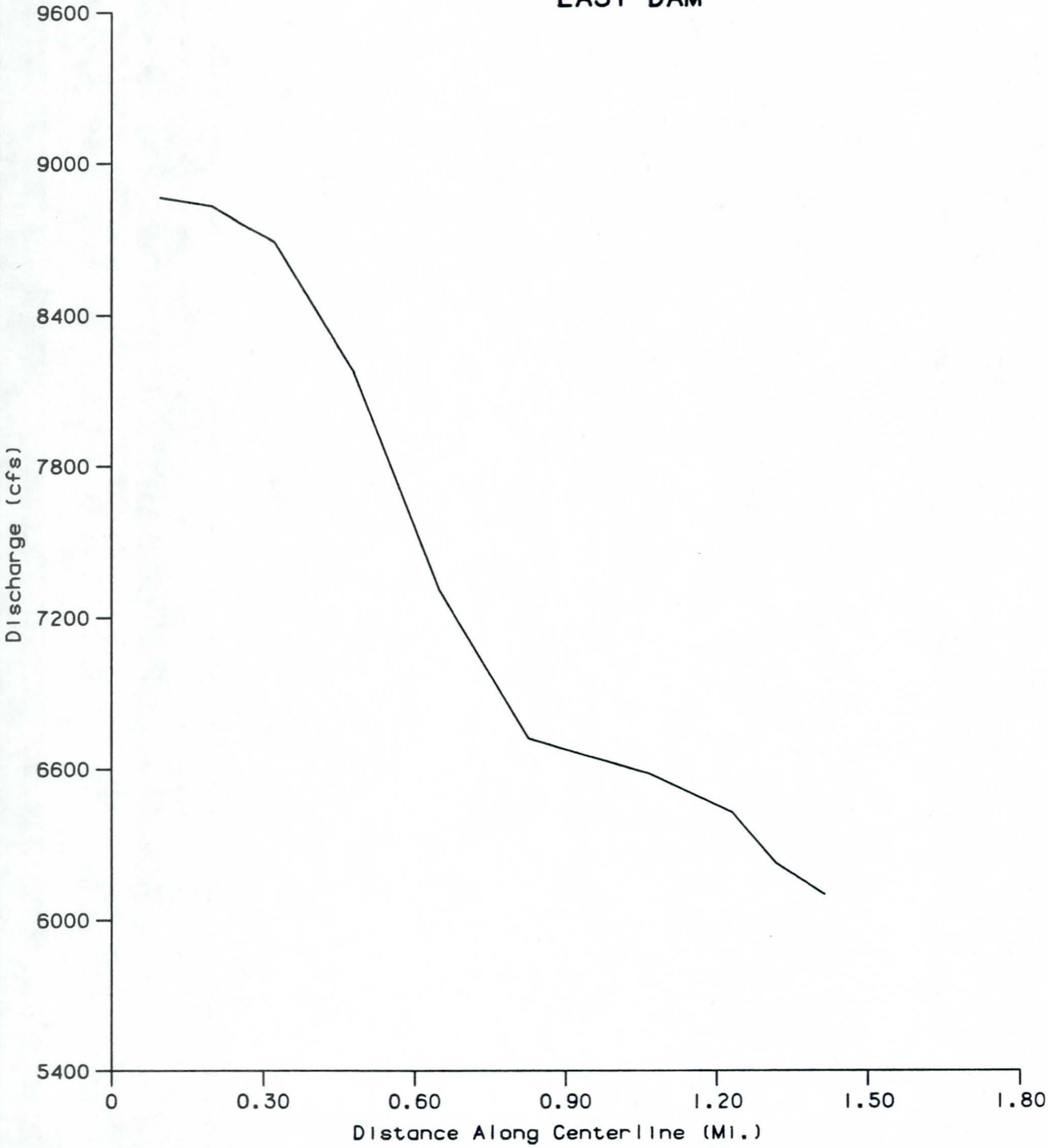


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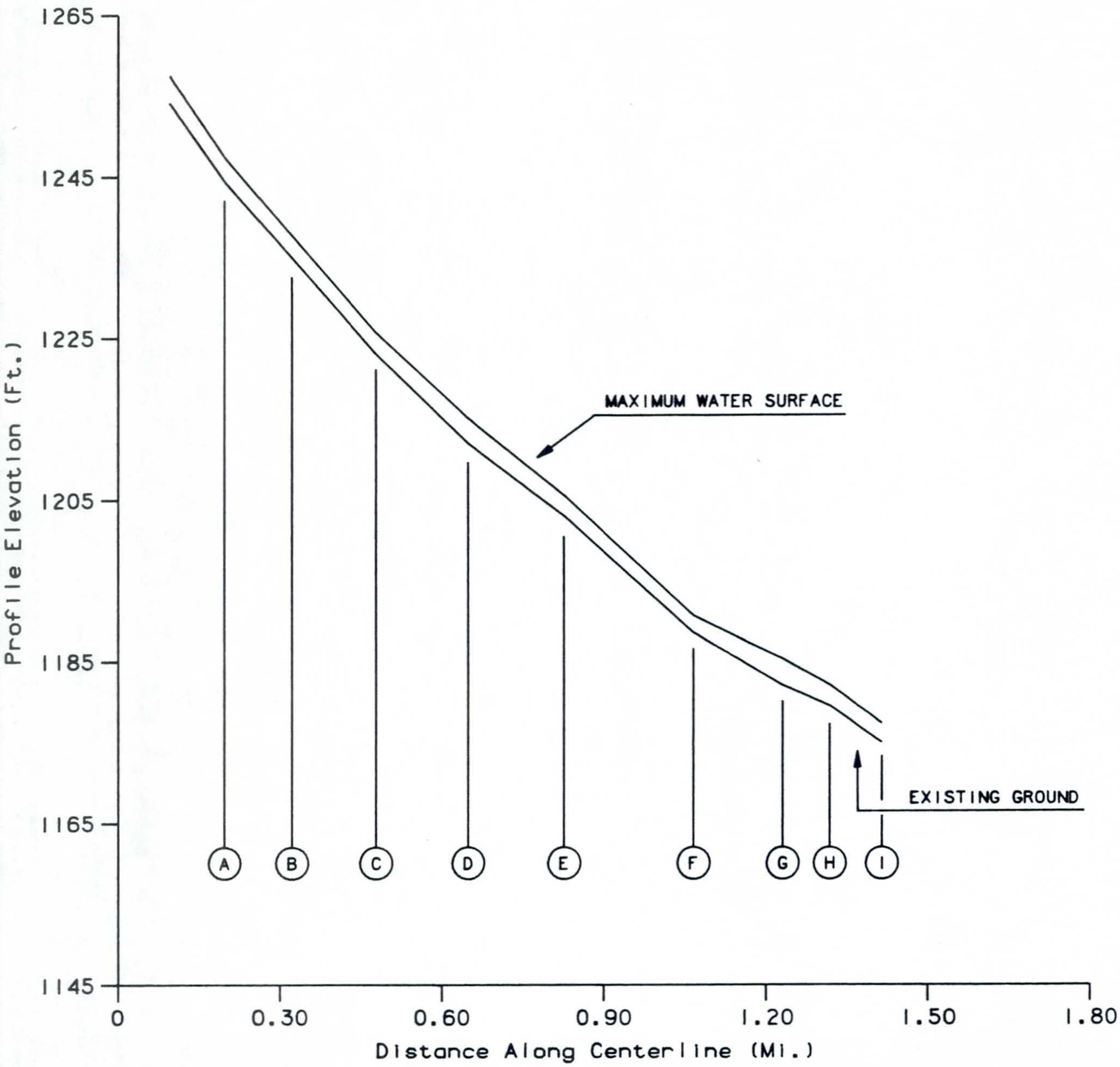
FIGURE D

EAST DAM



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Appendix D

9. APPENDICES

Appendix D

Results of Downstream Routing for North Dam
(Figures A Through D)

FIGURE A

NORTH DAM

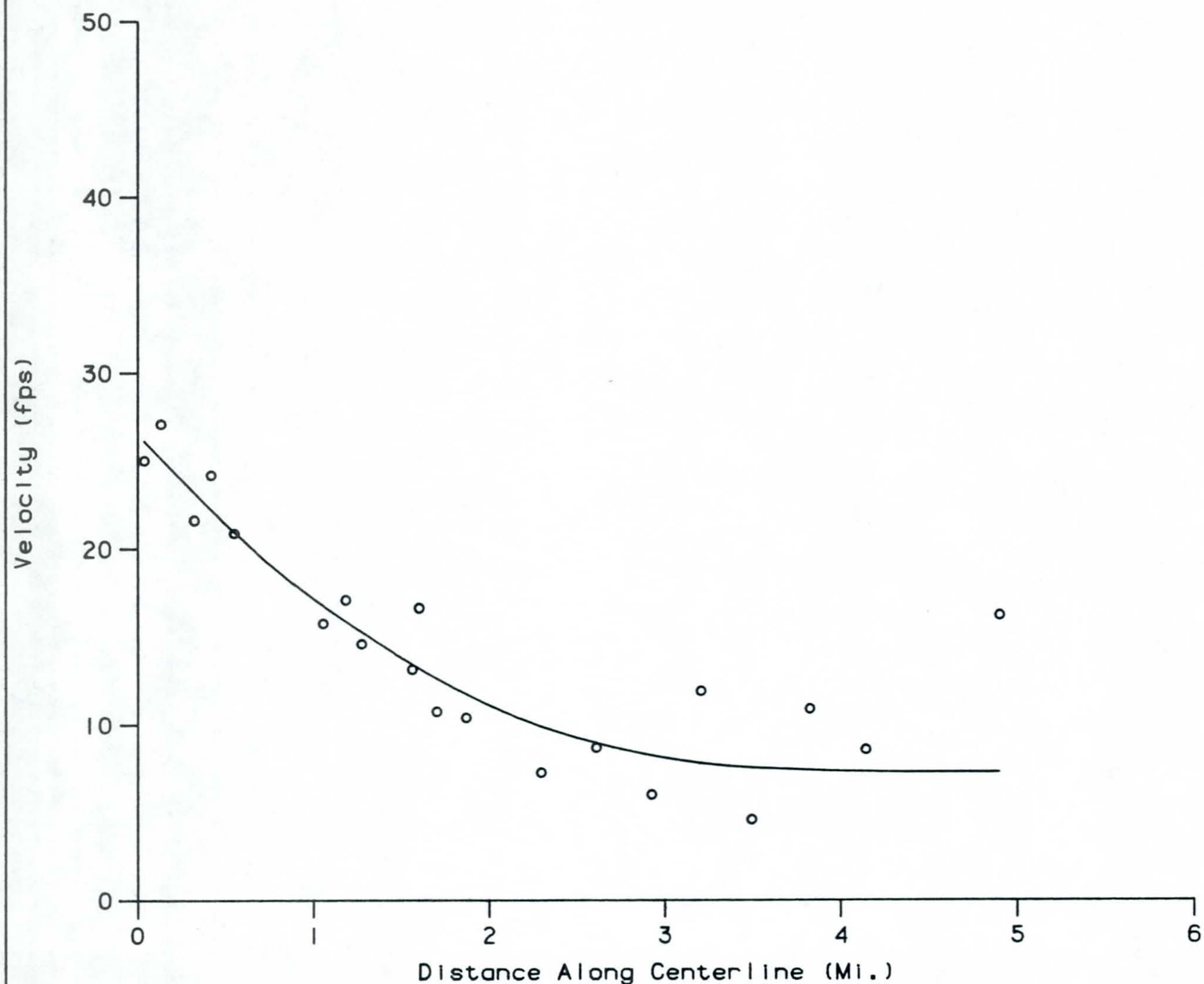

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FIGURE B

NORTH DAM

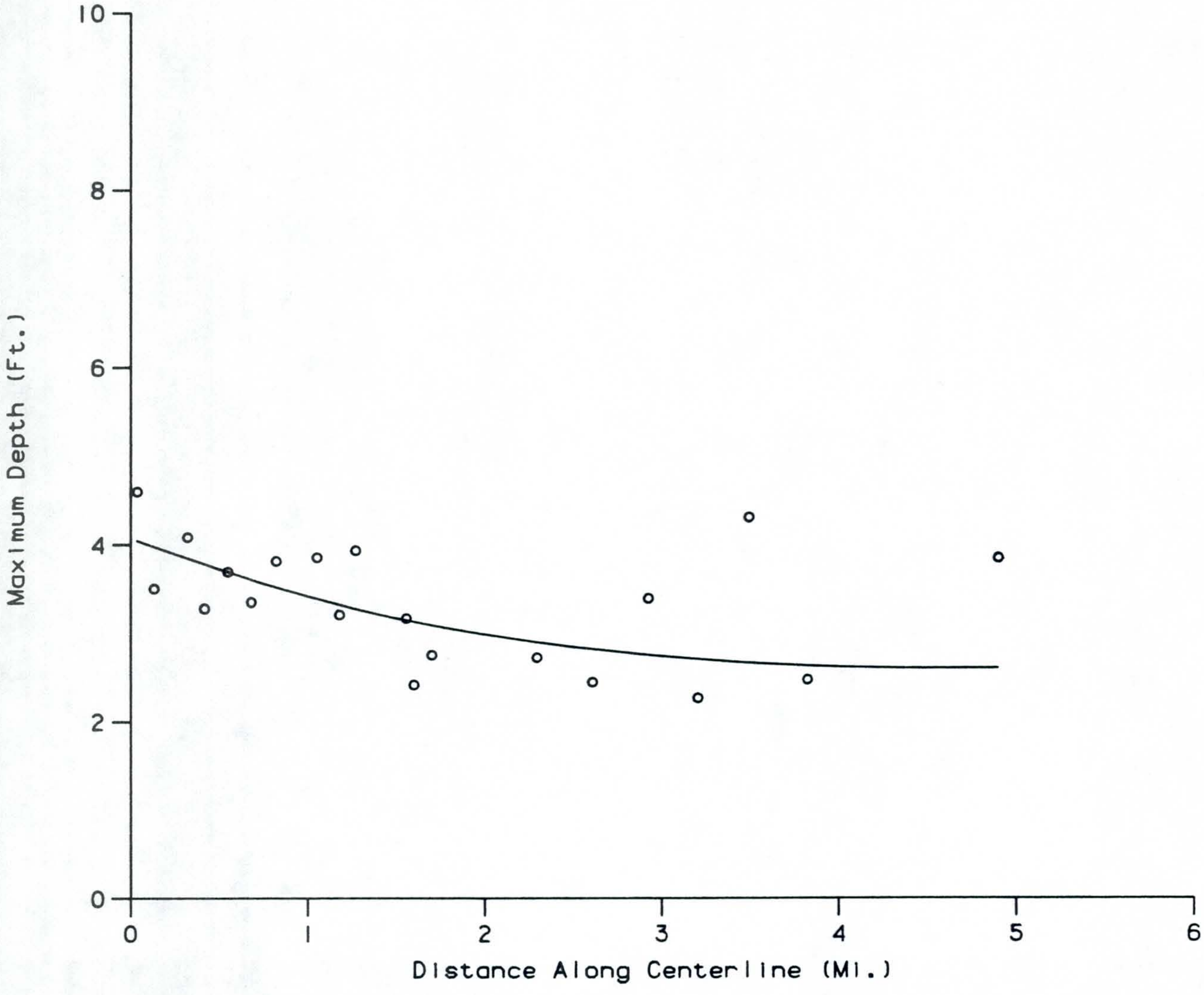

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FIGURE C

NORTH DAM

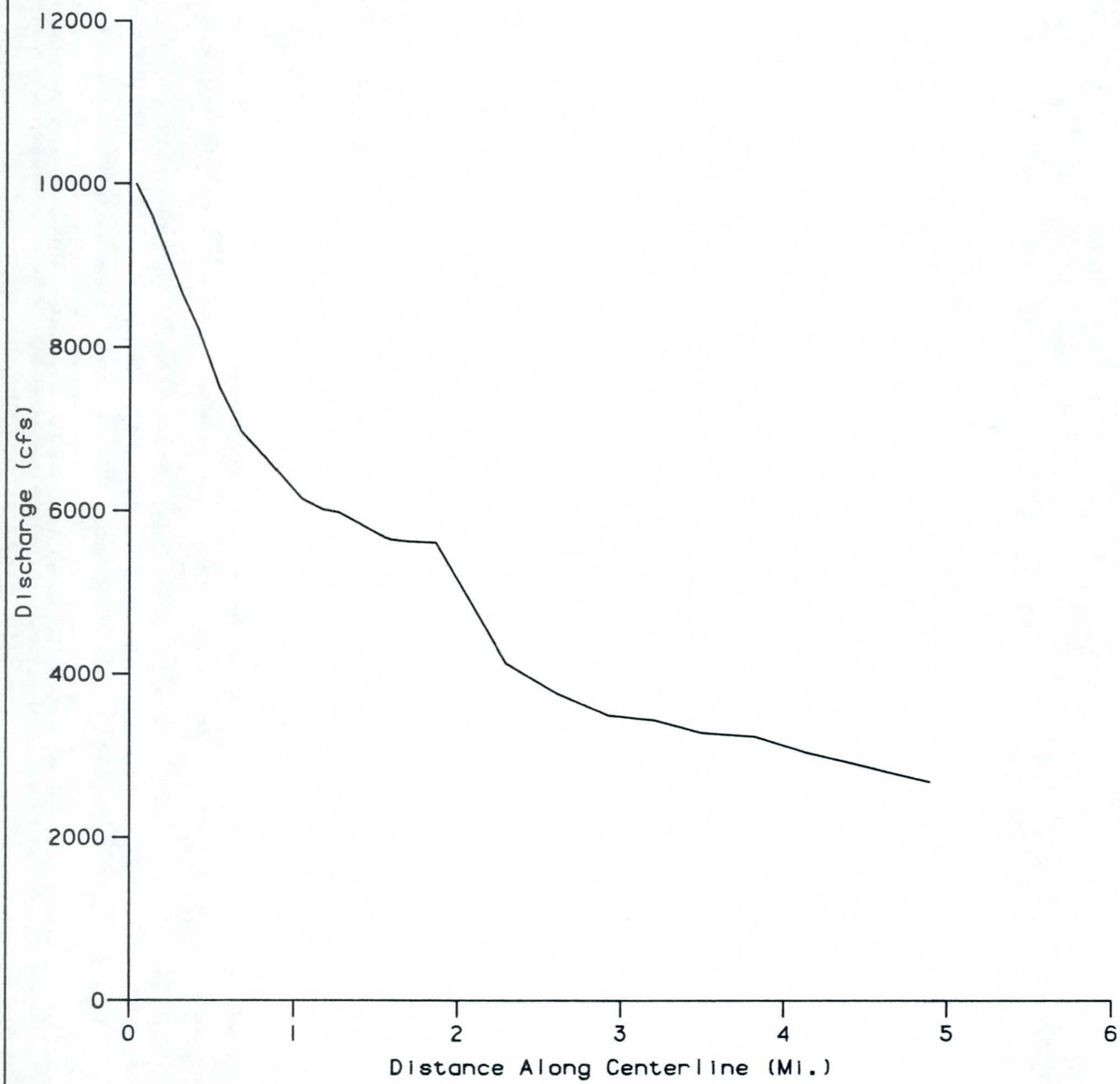

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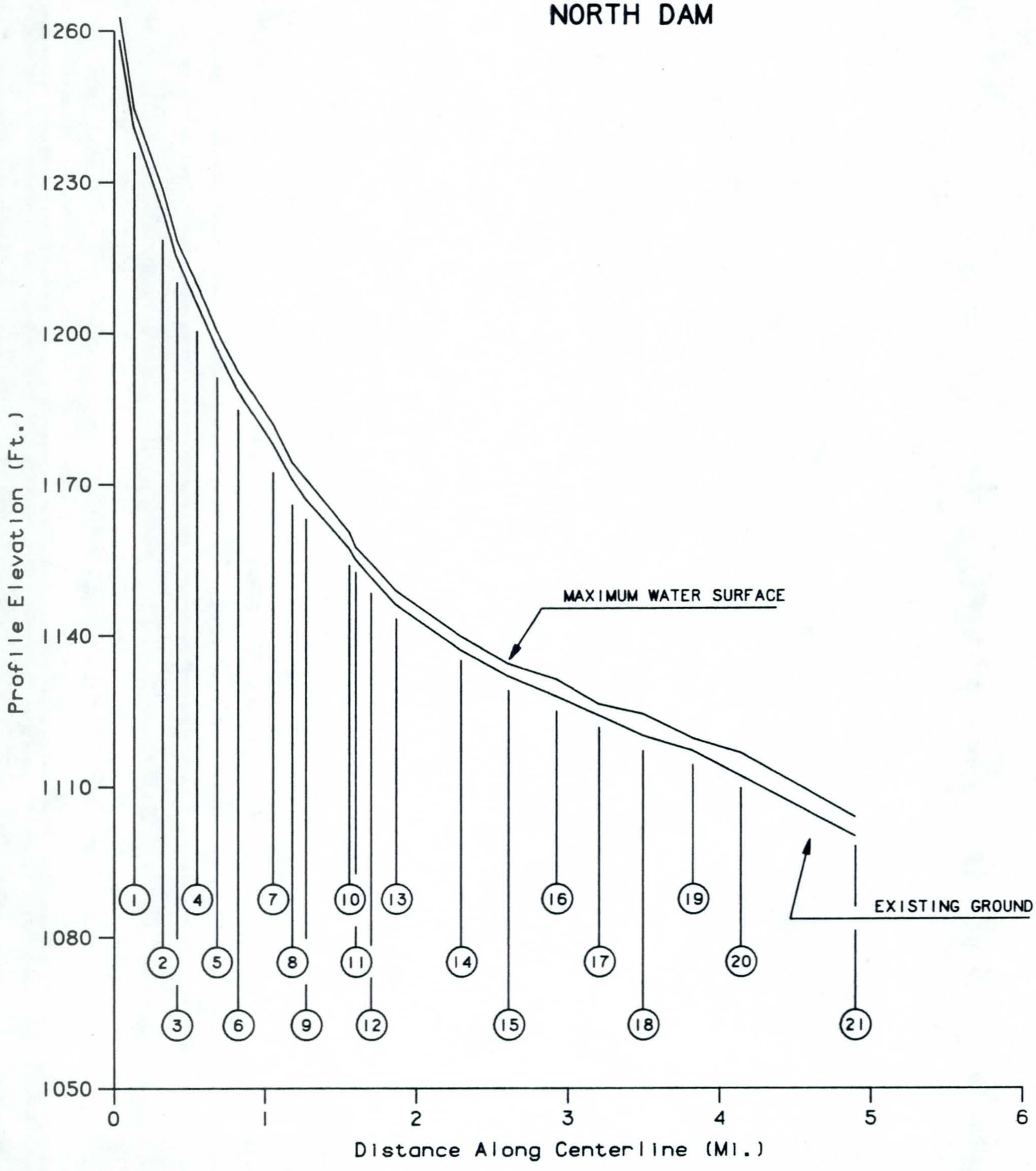
FIGURE D

NORTH DAM



GREINER