FLOOD CONTROL
ON THE
RIO SASKA

A Thesis
Presented to
The Faculty of Engineering
The University of Manitoba

In Partial Fulfillment
of the Requirements for the Degree
Master of Science in Civil Engineering

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by
Dieter Hubert Lindner
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ABSTRACT

This thesis represents the flood control component of a multi-purpose water resource development of a region.

The purpose of this thesis is to develop a useful teaching tool that in future years may be used as an assignment and a reference for graduate students and junior engineers.

A hypothetical situation has been created, such that the four major flood control measures that may ordinarily be employed, must be analysed before the most economic project is determined.

In order to retain realism, the situation somewhat resembles that of Winnipeg, Manitoba. The actual flow record of the South Saskatchewan River at Saskatoon, Saskatchewan was used.

Part I of this thesis, consisting of Chapter I outlines the problems and provides the background data necessary to analyze the possible flood control measures. As such it comprises the assignment.

Part II, consisting of Chapters II to VI inclusive, represents the Engineering Design. Chapter II deals with basic considerations such as hydrologic investigations which are necessary or common to the remainder of the engineering analysis. Subsequent chapters deal with diking systems, diversions, storage reservoirs and channel improvements. Simplifying assumptions are made to reduce the time required for analysis. The design of the components of projects are preliminary in nature, however, sufficiently accurate to obtain realistic cost estimates. Sample calculations are presented and results were tabulated and plotted to facilitate understanding and following the design procedures.

Part III consisting of Chapters VII and VIII represents the Economic Analysis of the projects considered. Initially the proposed individual projects were analysed. Subsequently, these individual projects were combined in varying proportions to determine which combination of projects would represent the optimum development. In Chapter VIII some of the limitations of this study were outlined, conclusions were drawn and recommendations made.

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PART I

ASSIGNMENT

CHAPTER I

LETTER OF INTENT

Saska Consult,
University of Manitoba,
Winnipeg, Manitoba.

Dear Sir:

In accordance with powers vested in me as the Governor of the Province of Assiniboia, I herewith award the contract to investigate comprehensive flood protection measures for the City of Portage to your firm.

This investigation shall include a review of existing reports, and present studies in diking, upstream storage reservoirs, diversions, and channel improvements on the Rio Saska, and combinations thereof. These investigations shall include appropriate economic analyses and recommendations as to which measures or combinations thereof are to be implemented.

The above mentioned aspects shall be dealt with; however, the study shall not be limited to only these aspects, if further study is warranted.

The results of this investigation, complete with recommendations for implementation and further investigations if required shall be completed and presented on or before October, 1970.

Yours truly,

Governor of Assiniboia.

1-2 Preamble

The City of Portage in the Province of Assiniboia has been exposed to floods over the years. A particularly severe flood was experienced in 1969 causing an estimated $50 million damage in the City of Portage. This flood caused a major disruption of the social and economic life of the city.

As a result, immediate remedial measures were initiated. A low diking system was introduced in 1969 along the Rio Saska to give protection up to elevation 703.50.

However, it was realized that stop gap measures of this nature were inadequate to protect the city from future flooding. Consequently, the City of Portage Engineering Department reviewed potential flood protection measures and gathered background information for the present comprehensive flood control investigation. The results of this reconnaissance are as follows:

1-3 Topographic Information

Maps of the Rio Saska drainage basin were obtained to define topographic features in order to assess possible protective measures. See Figure 1. The reference system used to define location along the Rio Saska was adopted as presented in the preliminary engineering report.

1-4 Protective Measures

a) Dikes: It is felt that the dikes introduced immediately following the 1969 flood, even though beneficial, do not represent the optimum development and thus should receive further study in order to upgrade protection. See Figures 2 and 3.

Two distinct diking possibilities present themselves at Portage. It is possible to improve the existing dikes adjacent to the Rio Saska somewhat at indicated in Figure 4. Even though minor difficulties have been encountered with bank stability, as indicated in the report by The City of Portage Engineering Department, and confirmed by an inspection trip to the failure sites, it was observed that they were only local failures which it was felt could be corrected with proper design and careful construction procedures. However, it was decided that an upper limit be placed on the permissible dike height adjacent to the river. This elevation was set equal to 706.50 allowing for a safe water level of 705.50 and one foot of freeboard. + ½ 662

The freeboard was arbitrarily set equal to one foot, to provide for settlement of the crest.

The upper limit was set primarily because of two considerations. Firstly, any construction in a residential area is necessarily associated with impeding normal activities of adjacent home owners. In any event land would have to be expropriated and trees along the bank removed to permit dike construction. If dikes were built too high, major infringements on homes would occur which then necessitates expropriation of the homes.

Secondly, it was felt that the previously mentioned stability problems should not be entirely ignored.

For a higher dike it becomes essential that the row of homes adjacent to the river be expropriated and moved in order to permit dyke construction adjacent to the existing road which would be retained in its present condition. See Figure 5 for a typical section of proposed high diking scheme.

It is estimated in the low dike proposal that the property owners should receive compensation of $2,500 each. Land would also have to be acquired immediately upstream and downstream of the City in order to permit construction of the dike. The estimated cost is $1.00 per square foot, based on a 50 ft. by 100 ft. lot selling for approximately $5,000. Diking costs were estimated to be $0.50 per cubic yard.

In the event that expropriation of the river-side homes becomes necessary it is felt that the cost would be $30,000 per home owner. Since this likely is associated with a high dike adjacent to the road, compensation must also be paid to the owners on the opposite side of the street who would be subjected to a lowering of property values. This compensation would amount to $2000 per owner.

b) Diversion: Several diversion sites, immediately upstream of the City of Portage were found. A diversion to Lake Manitou located approximately 18 miles north of the City appears promising. This area has been accurately mapped, and a soils survey was also undertaken. See Figures 6 and 7.

Diversions to Lake Manitou will encounter both major highways and railways. Estimates obtained from the Provincial Highways Department suggest that a simple crossing would cost $500 per lineal foot. A similar estimate for the railway crossing was $300 per lineal foot.

Agricultural land in the vicinity of Portage is valued at $100 per acre.

c) Storage Reservoir: A potential dam site exists upstream of the City at Tobin Rapids. The vicinity of Tobin Rapids was mapped and an exploratory drilling program was undertaken to assess foundation conditions. The results are presented in Figures 8 and 9.

The site at Tobin Rapids was initially investigated to assess its suitability for power development. During that study, various heights of dams complete with spillways were investigated. A Stage-Storage Curve as well as a Capital Cost Curve for this site were developed and are present in Figures 10 and 11 respectively.

d) Channel Improvements: It is felt that several measures are possible to improve the discharge characteristics of the Rio Saska downstream of Portage. An accurate survey of approximately 20 miles of the river from upstream of Portage extending downstream beyond Lister Rapids was made and is presented in Figures 12, 13 and 14.

From the topographic maps and the profile and cross-sections of the Rio Saska and Lister Rapids it was apparent that three distinct reaches have to be considered. Reach A extending from Lister Rapids to about Mile 650 has a relatively steep gradient of about 0.00038, while Reach B and the City Reach, being about 8.5 and 3.5 miles long respectively, have a bottom slope of about 0.00017. The

Rio Saska generally has a level flood plain of about 2,000 feet on either side, with the banks gradually rising at a slope of about 0.01. In Reach A a typical width is approximately 500 feet which then in Reach B gradually widens to 600 feet. The bank-full depth in the channel generally is about twenty feet. At about Mile 645 the river drops abruptly to elevation 637.0 at a point known as Lister Rapids. The Rio Saska has scoured its way through bedrock and the channel is only 350 feet wide. See Figures 13 and 14. The banks are generally wooded with both scrub brush and trees present.

1-5 Hydrologic Information

A gauging station has been in operation on the Rio Saska for the past 42 years. Thus a reliable record of the stages and discharges is available. The discharge record is presented in Table I. The Engineering Department also engaged the services of a hydrologist who developed Typical Flood Hydrograph presented in Figure 15.

1-6 Discharge Characteristics of Rio Saska at Portage

Observations at the bridge crossing the Rio Saska in Portage indicate that when the water level is at elevation 703.50, the effective top of the existing dikes, the discharge is approximately 80,000 cfs. During the 1969 flood of 138,000

TABLE I

RIO SAȘKA - MAXIMUM ANNUAL FLOWS

Year	Flow, cfs	Year	Flow, cfs
1929	42,700	1950	40,200
1930	35,100	1951	28,000
1931	111,000	1952	29,340
1932	93,000	1953	39,050
1933	69,700	1954	48,200
1934	41,760	1955	50,200
1935	35,560	1956	55,500
1936	62,180	1957	32,700
1937	50,000	1958	64,160
1938	37,800	1959	44,510
1939	119,800	1960	22,340
1940	31,800	1961	41,300
1941	37,000	1962	33,690
1942	26,000	1963	65,370
1943	82,300	1964	85,450
1944	87,800	1965	19,050
1945	108,000	1966	44,990
1946	30,200	1967	54,370
1947	20,600	1968	84,270
1948	111,000	1969	138,900
1949	38,200	1970	49,860

cfs the water level was accurately measured to be 707.50. The Rating Curve of the Rio Saska at Portage is shown in Figure 16.

1-7 Damage Survey

Subsequent to the 1969 flood the City undertook a major program to assess the destruction caused by the flood water and the associated economic losses, and also estimated the potential losses for higher stages. From this study a Stage-Demage Curve was developed for the City of Portage and is presented in Figure 17. The elevation was referenced to Mile 662.

1-8 Cost Data

In the preliminary study conducted by the Engineering Department of the City of Portage, an effort was made to gather information on construction cost in the Province of Assiniboia. Such data was collected and is presented in Appendix D-1.

PART II

ENGINEERING DESIGN

CHAPTER II

BASIC CONSIDERATIONS

2-1 Introduction

In accordance with the terms of the contract for the Engineering studies for flood control for the City of Portage the following investigations were conducted.

2-2 Hydrologic Investigations

The preliminary report prepared by the City of Portage Engineering Department was reviewed. The entire investigation, leading to the development of the Basic Flood Hydrograph as shown in Figure 15 was thoroughly reviewed and found to be satisfactory for this type of study, and hence accepted as presented.

The data obtained from the gauging station on the Rio Saska as presented in Table I, was used to develop the Flood Frequency Curves as shown in Figures 18 and 19. Initially, the Hazen Plotting Method was used to calculate flood frequencies.

$$\text{Frequency} = \frac{2M - 1}{2N}$$

where: M = Order Number

N = Length of Record

Sample Calculation:

Largest Mean Daily Discharge on Record = 138,000 cfs

The Order Number M = 1

Length of Record N = 42 Years

$$\text{Frequency} = \frac{2(1) - 1}{2(42)} = 0.0119$$

In a similar manner the other frequencies were calculated, tabulated and plotted, as shown in Table II and Figure 18, respectively. The straight line through the points was drawn by visual inspection.

In order to check the accuracy of this frequency curve the Weibull Plotting Method was employed through an existing computer program. The frequencies were calculated and plotted. A straight line was drawn through the points, representing the best fit line using the Theory of Least Squares. The results are presented in Figure 19. By inspection of Figures 18 and 19 it is evident that the curves obtained are in close agreement. It was decided to use the Frequency Curve obtained through the Weibull Method, Figure 19, in all subsequent calculations.

This selection was largely arbitrary since either method is commonly used. It was felt that a Least Square Fit perhaps was a more scientific approach than visual inspection, even though the results are not necessarily better.

TABLE II

HAZEN PLOTTING METHOD

Order No. M	Flow cfs	Frequency $= \frac{2M-1}{2N}$	Order No. M	Flow cfs	Frequency $= \frac{2M-1}{2N}$
1	138,900	0.0119	22	44,510	0.512
2	119,800	0.0357	23	42,700	0.535
3	111,000	0.0595	24	41,760	0.560
4	111,000	0.0834	25	41,300	0.583
5	108,000	0.107	26	40,200	0.607
6	93,000	0.131	27	39,050	0.631
7	87,800	0.154	28	38,200	0.655
8	85,400	0.178	29	37,800	0.679
9	84,270	0.202	30	37,000	0.701
10	82,300	0.226	31	35,560	0.726
11	69,700	0.250	32	35,100	0.750
12	65,370	0.273	33	33,690	0.774
13	64,160	0.298	34	32,700	0.797
14	62,180	0.321	35	31,800	0.821
15	55,500	0.345	36	30,200	0.845
16	54,370	0.369	37	29,340	0.869
17	50,200	0.392	38	28,000	0.893
18	50,000	0.416	39	26,000	0.916
19	49,860	0.440	40	22,340	0.840
20	48,200	0.464	41	20,600	0.965
21	44,990	0.488	42	19,050	0.989

2-3 Simulation of Rio Saska Rating Curve

At this point an investigation of the discharge characteristics of the Rio Saska was made for a location just upstream of the City of Portage at Mile 662 to simulate the Rating Curve given in Figure 16. It is necessary to develop this model in order to be able to determine what the stages at Portage will be if such modifications as a diking system or channel improvements are introduced.

To determine the discharge curve it was necessary to investigate the reach of the river from Lister Rapids, located approximately 13.5 miles downstream of the City, to a point upstream of Portage.

The Rio Saska channel can be broken into distinct reaches as shown in Figure 20. To facilitate computations, the cross-sections have also been idealized as shown in Figure 21.

In order to obtain an estimate of discharge at Portage, backwater calculations are necessary to assess the effect of Lister Rapids.

While there are several ways to estimate the water surface profile, a simplification of the Bresse Equation as suggested by Bossen was used primarily because of its simplicity and application. This method approximates the distance required to halve the difference between the water level and the normal depth of the channel. Manning's

Equation was used to obtain the normal depth of both the channel and the bank section.

This requires an estimate of Manning's roughness "n" in both the channel and overbank section. It was felt that n = 0030 to 0040 for the channel and n ranging from 0060 to 0080 for the overbank represent a realistic estimate. It was decided to use channel values of 0.030, 0.035, and 0.040 in conjunction with an overbank value of 0.070 in order to make the necessary calculations. The normal depths were established for the Rio Saska and suitable curves were plotted. See Figures 22 and 23.

Sample calculation:

To find Normal Rating Curve:

Manning's Equation $v = \frac{1.49\ S^{1/2}\ R^{2/3}}{n}$

where: v = average velocity

n = Manning's roughness

S = channel slope

R = hydraulic radius

also, Q = Av where: Q = total discharge

A = cross-sectional area

v = average velocity

also, q = vd where: q = discharge per unit width

d = area per unit width

Combining these expressions yields: $q = \frac{1.49\ S^{1/2}\ d^{2/3}\ d}{n}$

where: R = d, which is a reasonable approximation in wide channels.

For channel:

$$n = 0.035,\ \text{slope} = 0.00038,\ \text{width } W = 500 \text{ feet}$$

$$q = \frac{1.49\ (.00038)^{1/2} d^{\ 5/3}}{0.035}$$

$$Q = 0.830\ d^{5/3}\ W = 0.830\ (30)^{5/3}\ 500 = 120{,}500 \text{ cfs}$$

For overbank:

$$n = 0.070,\ \text{slope} = 0.00038,\ \text{average width} = 4{,}000+10(100)$$

$$Q = 0.415\ d^{5/3}\ W \text{ where depth on banks} = 30-20 = 10 \text{ feet}$$

$$Q = 0.415\ (10)^{5/3}\ 5{,}000 = 96{,}800 \text{ cfs}$$

Total discharge = 120,500 + 96,800 = 217,300 cfs.

Similar calculations were performed for the other depths using a simple computer program, the results of which are in Appendix A-1. These results are also graphically illustrated in Figure 22. Similarly, a Normal Depth Curve was developed for Reach B and the City Reach. See Figure 23.

2-4 The Bossen Formula

A computer program for the Bossen water surface calculations was then written to use the Normal Curve data as input to calculate the water level at Portage at Mile 662.

Using Manning's n = 0.035 for the channel in conjunction with an overbank n value of 0.070 yielded a Rating Curve almost perfectly duplicating the observed water levels at Portage. See Figure 24. At this point it was felt that a suitable model had been established which could now be used confidently to estimate the water levels at Portage. For the computer program see Appendix B-1.

Sample calculation:

To illustrate the Bossen Formula:

$$\Delta x = \frac{0.25\ DN}{SB}$$

where Δx = distance required to halve the difference between water depth and normal depth

DN = normal depth

SB = bottom slope

For discharge: $Q = 140{,}000$ cfs

At Lister Rapids:

$$\text{slope} = \frac{765 - 737}{4{,}200} = 0.0088$$

$$q = \frac{Q}{W} = \frac{140{,}000}{350} = 400 \text{ cfs/ft.}$$

$$\text{But } dcr = \sqrt[3]{\frac{(400)^2}{32.2}} = 17.10 \text{ feet}$$

Flow area = 350 (17.10) = 5,980 sq. ft.

Using Manning's equation

$$v = 1.49\ S^{1/2}\ R^{2/3} \qquad \text{using: } n = 0.040$$

$$R = d \text{ (approximately)}$$

$$= 1.49\ (0.0088)^{1/2}\ (17.10)^{2/3} = 23.40 \text{ fps.}$$

Thus discharge Q would be:

$$Q = Av$$

$$= 5{,}980\ (23.40) = 140{,}000 \text{ cfs.}$$

It is thus established that critical depth is reached.

If a lower roughness coefficient had been assumed, such as n = 0.035, the velocity would have exceeded 23.40 fps, and the subsequent discharge would have exceeded 140,000 cfs, indicating that supercritical flow exists in Lister Rapids at this discharge. With the foregoing estimate it is reasonable to accept that critical depth is established at the control point at Mile 645.

Reach A:

The normal depth in Reach A from Figure 22 is DN = 26.30 feet.

when: $n = 0.035$, slope A = 0.00038

$$\Delta x = \frac{0.25\ (26.30)}{0.00038} = 17{,}300 \text{ feet}$$

Depth upstream of Lister Rapids is 17.10 + 5.00 = 22.10 feet.

Difference between DN and water depth = 26.30 - 22.10 = 4.20 ft.

Depth at 17,300 feet upstream of Lister Rapids is

22.10 + 0.50 (4.20) = 24.20 feet.

Length of Reach B = 5.0 (5,280) = 26,400 feet

Again, Δx = 17,300 feet.

Difference in depth = 2.10 feet.

Increase in depth over next 17,300 ft. is 1.05 ft. by linear interpolation estimate depth at end of Reach A.

$$\text{Increase in depth} = 1.05\ \frac{(26{,}400 - 17{,}300)}{17{,}300} = 0.55 \text{ feet.}$$

Depth at end of Reach A is 24.20 + 0.55 = 24.75 feet.

Reach B:

The normal depth in Reach B from Figure 23 is

DN = 28.70 feet

when: $n = 0.035$, slope B = 0.00017

$$\Delta x = \frac{0.25\ (28.7)}{.00017} = 42,200 \text{ feet}$$

Difference in depth = 28.70 - 24.75 = 3.95 feet.

Depth at 42,200 feet upstream of Mile 650 = 24.75 + 0.50 (3.95)

= 26.72 feet.

Length of Reach B and City Reach = 12 miles = 63,380 feet.

Again, Δx = 42,200 feet.

By linear interpolation increase in depth is

$$\frac{0.99\ (63,380 - 42,000)}{42,200} = 0.47 \text{ feet.}$$

Depth at Mile 662 = 26.72 + 0.47 = 27.19 feet.

Bottom elevation of channel at Mile 666 = 680.50.

Water level elevation = 680.50 + 27.19 = 707.79,

which checks closely with computer calculated elevation of 707.75.

Similar calculations were performed such that Figure 24, Rating Curve at Portage, was produced.

2-5 Flood Forecasting

Through flood forecasting it is possible to reduce the damage that a flood will cause, and hence has to be considered a flood control measure. The effectiveness of this measure depends largely on accuracy in flood predictions and the reaction of the populace to warnings that are issued. While the damage can be reduced significantly it is difficult to determine benefits that accrue as a result of flood forecasting. Flood forecasting as a means of flood control will not be considered for the City of Portage.

2-6 Flood Zoning

Flood zoning is another administrative measure that can be employed to reduce flood damages. This measure can easily be introduced in Portage through zoning bylaws requiring expansion of the City to take place along its southern and northern limits instead of in the flood plain of the Rio Saska system and downstream of Portage.

2-7 Design Considerations

The design work of structures has been minimized since great detail and accuracy is not required in order to obtain a reasonable cost estimate, which can then be utilized in the economic analysis.

2-8 Cost Considerations

In estimating the capital costs of the various projects under consideration, the unit cost curves given in Appendix D-1 were used where applicable. Other cost data provided in the preliminary findings of the City of Portage, Engineering Department, upon review were accepted as presented. On occasion judgement had to be exercised in deciding as to which unit prices were to be used. Wherever it was found necessary to deviate substantially from the cost curves, an explanation is provided.

In order to account for uncertanties and allow for costs associated with the design, an item, Engineering and Contingencies, amounting to 30% has been allowed for. This particular item is frequently of this order of magnitude in studies of this nature.

CHAPTER III

DIKING

3-1 General Considerations

The findings of the Engineering Department were reviewed and found to be acceptable as a premises for design of a diking system. As such, it was necessary to determine the stages at Portage as a result of the introduction of higher dikes.

3-2 Determination of Rating Curve at Portage with Diking System

The previously developed computer program was employed to obtain Normal Rating Curves at Portage with dikes adjacent to the channel and 150 feet from the channel. A channel roughness of 0.035 and an overbank roughness of 0.070 were used. Results are graphically presented in Figure 25. From these curves the input for the Routing Program, Appendix B-1 was obtained. It should be noted that the input for Reach A and B was not altered. Only the data for the City Reach had to be changed. The results are presented in Figure 26

3-3 Cost Estimate - Low Diking Alternative

For the diking - Section shown in Figure

Diking Volume

Parallel to channel

$$= \frac{(8 + 18)\ (6)\ (3.5)\ (5280)}{27} \quad = \quad 107{,}000 \text{ cu. yds.}$$

Upstream and downstream of Portage

$$= \frac{26\ (6)\ (4{,}000)}{27} \quad = \quad \underline{23{,}000}$$

Total Dike Volume = $\underline{\underline{130{,}000}}$ cu. yds.

Number of Property owners affected

$$= \frac{3.5\ (5{,}280)}{50} \quad = \quad 370 \text{ owners}$$

In order to connect the dike adjacent to the river to the valley bank land has to be expropriated upstream and downstream of the City for dike construction:

Required Width = 2 (3) (6) + 8 = 44 ft., say 50 ft.

= 4,000 (50) = 200,000 sq. ft.

Costs

Diking Earthwork	= 130,000 (.50)	=	$ 65,000
Owner Compensation	= 370 (2,500)	=	925,000
Diking Land	= 200,000 (1.00)	=	200,000
Subtotal			$1,190,000
For both sides of Rio Saska			2,380,000
30% Engineering and Contingencies			715,000
Total Capital Cost			$3,093,000

It is felt that there is not much merit in costing lower diking proposals since the major cost component results from the compensation to be paid to the home owners along the Rio Saska. Even if the dikes were lowered, consequently being less beneficial, the cost components would not undergo a corresponding reduction. Hence, the height of dike will not be solely governed by economic aspects, but also by the previously mentioned engineering considerations which limit their height.

3-4 Cost Calculations - High Diking Alternative

Consider Dike to Elevation 708.00

Dike Volume

Parallel to channel

$$= \frac{(10 + 24)\ (8)\ (3.5)\ (5{,}280)}{27} = 186{,}000 \text{ cu. yds.}$$

Upstream and downstream of Portage

$$= \frac{(34)\ (8)\ (4{,}000)}{27} = \underline{40{,}000}$$

Total Diking Required $= \underline{\underline{226{,}000}}$ cu. yds.

Dike Land Required

$= 4{,}000$ ft. (75) $= 300{,}000$ sq. ft.

Expropriation of 370 properties complete with houses along either side of the river will be necessary.

Similarly, compensation to owners on opposite side of the street will be necessary since the construction of a dike approximately 10 feet high in front of their homes would adversely affect their property values.

Costs

Diking Earthwork	= (226,000) (.50)	=	$ 113,000
Home Expropriation	= 370 (30,000)	=	11,100,000
Diking Land	= 300,000 (1.00)	=	300,000
Owner Compensation	= 370 (2,000)	=	740,000
Subtotal			$12,253,000
For both sides of Rio Saska			$24,506,000
30% Engineering and Contingencies			$ 7,370,000
Total Capital Cost			$31,876,000

In a similar manner the costs of a diking system providing virtually complete protection were estimated. A dike extending to elevation 719.00 and containing a flow of about 350,000 cfs would meet this requirement.

Dike Volume

Parallel to channel

$$= \frac{(60 + 10)\ (20)\ (3.5)\ (5{,}280)}{27} = 959{,}000 \text{ cu. yds.}$$

Upstream and downstream of Portage

$$= \frac{70\ (20)\ 6{,}000}{27} = 311{,}000$$

Total Volume 1,270,000 cu. yds.

Dike Land Required

= 6,000 ft. (150) = 900,000 sq. ft.

Unit Cost = $ 1.00 per sq. ft.

Compensation to owners across the street would have to be considerably higher since a high dike is more unpleasant.

Unit Cost = $4,000 per owner.

Costs

Diking Earthwork	=	1,270,000 (.50) =	$ 635,000
Home Expropriation	=	370 (30,000) =	11,100,000
Diking Land	=	900,000 (1.00) =	900,000
Owner Compensation	=	370 (4,000) =	1,480,000
Subtotal			$14,135,000
For both sides of Rio Saska			$28,270,000
30% Engineering and Contingencies			$ 8,630,000
Total Capital Cost			$36,900,000

Further cost estimates could be made in order to develop a Capital Cost Curve for the High Dike Alternative. However, this has not been done since this alternative has quite a number of adverse features which will likely preclude its implementation. If the economic appraisal reveals that this alternative is highly desirable such a curve can still be developed, in that event, in order to determine optimum level of development.

CHAPTER IV

DIVERSION OF THE RIO SASKA

4-1 General Considerations

By inspection of Figure 6 it becomes apparent that two particularly suitable locations exist for a diversion: these being at Mile 662 immediately upstream of the City and at Mile 667 about five miles upstream of Portage.

Drill holes obtained by the City of Portage Engineering Department indicate that the soil profile is relatively uniform, hence not favouring either location to any extent.

Topographic features seem to favour a diversion at Mile 662 since less excavation is probably involved. The elevation of the land is somewhat lower at this point and the diversion route appears to be approximately a half mile shorter. Storage capacity in the valley is limited at either location, and hence routing of floods is not required.

As can be seen from the topographic maps, both diversions will require three major highway bridges as well as two railway crossings.

4-2 Main Structures Associated with a Diversion

Generally the diversion scheme will involve the construction of a relatively low control dam across the valley of the Rio Saska, as well as the excavation of a diversion channel as schematically illustrated in Figure 27

The control dam will have a gated structure capable of passing normal river flows with a limited head. This structure will be gated to permit regulation of the diversion of flow between the river and the diversion channel.

4-3 Mode of Operation

With a certain pool elevation the control dam should be able to pass the bankfull capacity of the river. Once the flood flows increase beyond the discharge the flow down the Rio Saska would be limited to bankfull capacity until the diversion design capacity is reached. This operation is associated with the rise of the water surface upstream of the dam to the diversion design elevation. If there are subsequent increases in flood flows, the water levels would be held constant at this elevation, and the additional flow would be passed down the Rio Saska, causing flooding in Portage. However, the flooding, and hence the damages in Portage, would be reduced as a result of the diversion discharge. This operational procedure is schematically indicated in Figure 28.

4-4 Other Structures Associated with Diversion

The diversion channel may contain several drop structures depending on the slope of the land and the channel gradient. These drop structures are essential to dissipate excess energy.

In addition, the entrance to the diversion may contain an elaborate control structure or it may only be a simple cut through the valley bank. It may also be controlled by a simple dike, which, when properly designed, acts as a weir.

The decision as to which structure to employ depends entirely on local circumstances. An elaborate control structure may be in order if it is essential that at certain times no water be passed by diversion: that is, if flooding perhaps were to result along Lake Manitou. Under these circumstances a structure may be justified.

A simple weir may be economically introduced if it is desirable to raise the water level behind the river dam because of municipal or recreational considerations.

Lastly, if such considerations are not relevant, the expenditures on a structure can hardly be justified, and hence, a simple cut-section through the bank may be in order.

Often the diversion is terminated by an outlet structure. The purpose of this structure is to dissipate the remaining energy which otherwise may cause serious erosion. Such a structure is particularly important when the lake levels and hence the tailwater conditions vary considerably. On occasion, when particularly favourable circumstances are present, such as the presence of sound bed rock, this structure is not necessary.

4-5 Channel Stability Considerations

When a diversion channel is to be constructed in erodible material, the short and long-term stability of the channel becomes important. However, a diversion differs considerably from a natural channel since it is an artificial channel which will discharge only over short periods during the year, and may perhaps not be used at all for several years. Consequently, diversions often are seeded to establish a grass cover. Once this has been achieved, the danger of serious erosion is greatly reduced. Generally, there are three accepted ways to assess the stability condition.

The first is generally associated with the concept of "limited velocity", which is largely emperical, based on observed velocities and flow depths in various foundation materials. Varous foundations also have different Manning's roughness values. It is generally a matter of judgement as to which values are chosen, especially when one considers the long-term condition of a channel. Often a lower value is chosen in order to calculate the velocities. A higher value is often assumed when assessing the discharge capacity of the diversion channel. When both the velocity and discharge requirements are met, a satisfactory design should evolve.

The second approach involves the consideration of the Regime Theory. Using this approach, knowing the grain size of material, a siltation factor can be computed. Also knowing the discharge, a stable channel slope can be computed. If the design slope is close to this computed slope, the channel should remain stable. This theory is also based on emperical observations of channels.

The third way utilizes the tractive force approach. At certain flow depths the weight of the flowing water exerts a normal force on the bed material tending to prevent erosion. The component of this force, parallel to the bed is the shear force causing bed movement. If the flow depth is increased this shear force can become larger before particle movement is initiated. This can be reflected by higher permissible velocities on the channel.

Since all three methods are largely based on emperical observations, it is difficult to determine the best approach. One can be reasonably certain that a stable channel has been designed if a check indicates that the results do not deviate greatly from the expected values of these three approaches.

The design of a diversion then resolves itself into an exercise of minimizing the costs associated with its construction, given the various engineering constraints, as well as maximizing the resulting benefits.

4-6 Design of the Portage Diversion

At Mile 662 an earth filled dam with a maximum height of about fifty feet, as shown in Figure 29 represents a suitable design.

Generally, it is desirable to have the diversion design elevation as high as possible in order to keep the channel excavation low. This item usually represents the major cost component. In a final design, the most economic elevation should be determined by investigating the incremental costs associated with both the Rio Saska control dam and structure and the diversion cost; a lowering of the dam would reduce its cost but increase the diversion excavation cost. Through such an investigation the minimum total cost can be found.

At Mile 662 a suitable diversion design elevation is 724.00 with a Top of Dam elevation of 727.00, allowing three feet of freeboard during flood stages of the Rio Saska.

4-7 Determination of Least Costly Diversion Channel

Initially, a particular diversion discharge was considered. Then a slope was selected, and a width calculated. A range of channel slopes was investigated and the corresponding normal depths for the channel were calculated using Manning's Equation. The elevation of the entrance of the diversion was then found by subtracting the normal depth from the diversion design elevation. At this particular slope the diversion would eventually arrive at the ground surface, on the north side of the height of land. From this point on, the channel slope would follow the ground provided the maximum permissible slope is not exceeded. If the ground slope exceeds the maximum permissible slope, a drop structure is introduced and the diversion then continues on the maximum permissible slope until it again reaches the ground surfaces requiring a further drop structure. This sequence is repeated until Lake Manitou is reached.

When the channel follows the ground slope, a permissible velocity of 4 feet per second was selected for the soils present at the diversion site. With this velocity set, and the slope known, a channel width can be calculated. Thus the diversion width will vary along the route depending on the slope.

4-8 Cost Parameters of a Diversion

During these calculations the volume of excavation from contour to contour, is also calculated. In a similar manner dike volumes are also found. The total depth of the diversion is equal to the normal depth plus one foot for freeboard. Thus, the dike height varies from zero feet at a point where the depth of cut exceeds the normal depth plus one foot, as well as to the south of the height of land, to the normal depth plus one foot, at the drop structures.

Similarly, the land required along the course of the diversion is calculated. Sufficient land must be acquired to permit the excavation of the channel, the construction of the dike along the channel, and to allow for wastage in areas of excess excavation.

The three highway bridges, the two rail crossings, as well as the drop structures and the outlet structure must also be taken into consideration.

The costs of the associated parameters were summed in order to yield the diversion cost for a channel on a certain slope and discharging a certain flow.

4-9 Steps in the Iterative Process

a) Varying the Channel Slope

In a similar manner the costs were found when the channel slope was varied to cut through the height of land, and then follows the ground slope, provided the maximum permissible slope was not exceeded.

b) Varying the Design Discharge

Subsequently, the discharge capacity was also varied to yield costs for channels at various slopes for various discharges. The costs found in this manner were then plotted as shown in Figure 30 in order to determine which particular combination of width and slope was the least costly for the particular discharge under consideration.

c) Varying the Maximum Permissible Velocity

Deciding upon the permissible velocities in an earth channel is largely a matter of judgement. Permissible velocities are a function of physical properties of the soil as well as the discharge, the flow depth and the channel width. It was felt that for the sandy clay soil found along the course of the diversion a velocity of 4 or 5 fps would be acceptable, and hence were investigated.

d) Varying the Maximum Permissible Slope

If the velocity is limited to a certain value and the slope chosen, a corresponding normal depth can be calculated using Manning's Equation. For each diversion discharge a corresponding width can also be calculated. If the slope is too steep and velocity is limited to a certain value, a wide channel with a large hydraulic radius will result. However, problems can arise if a wide section is chosen, especially with low discharges, during which meandering flows can lead to localized erosion. Thus it is desirable to avoid very wide sections.

4-10 Computer Program

In order to reduce the time required to perform such iterative calculations, the entire procedure was computerized and is presented in Appendix C-1.

Upon plotting the computed costs it became evident that slopes of about 0.00010 to 0.00030 were the ones that yielded the least costly diversion. Subsequently, this range was more closely investigated.

The least costly slope for each particular diversion capacity has been obtained from Figures 30 and 31 and have been presented in Table III. The expected maximum permissible velocity, the invert of the diversion and the location of entrance of the channel have also been shown.

4-11 Other Design Considerations

A Manning's $n = 0.025$ was used in order to calculate the velocity in the channel. The discharge is based on a value of $n = 0.030$. It is felt that these values reasonably reflect roughness to be expected in the diversion channel.

TABLE III

LEAST COSTLY DIVERSION CHANNEL

Maximum Velocity = 5 fps.

Diversion Capacity cfs	Alternative A				Alternative B			
	Diversion Invert	Station	Slope	Cost $ Million	Diversion Invert	Station	Slope	Cost $ Million
10,000	707.93	2542	0.00025	1.92	713.99	2417	0.00030	1.74
20,000	707.93	2542	0.00025	3.30	713.99	2417	0.00030	3.02
30,000	707.93	2542	0.00025	4.70	713.99	2417	0.00030	4.30
40,000	707.93	2542	0.00025	6.08	713.99	2417	0.00030	5.58
50,000	707.93	2542	0.00025	7.47	713.99	2417	0.00030	6.86
60,000	707.93	2542	0.00025	8.85	713.99	2417	0.00030	8.14
70,000	707.93	2542	0.00025	10.24	713.99	2417	0.00030	9.42
80,000	707.93	2542	0.00025	11.63	713.99	2417	0.00030	10.70

The width of drop structures introduced were arbitrarily set equal to two-thirds of the channel bottom width. In addition the drop in these structures was set equal to five feet.

Both of these assumptions should be more thoroughly investigated in a final design. It is common practice to model test structures of this nature to determine the maximum permissible convergence of flow and still retain satisfactory hydraulic performance. Usually, it is found that less costly structures result with a reduction in width. The structures will also incorporate a sill in order to assure that the flow depth upstream of the structure remains near normal depth and thus avoids excessive flow velocities. A typical drop structure is presented in Figure 32.

It was decided that a diversion entrance structure was not required for this particular situation at Portage. It may, however, be desirable to place a simple sill across the entrance, particularly when the diversion invert is at a low level. This sill would act like a weir. Its advantage lies in that it will permit the development of a higher head on the control structure without having any flow in the diversion channel. An outlet structure is envisioned at Lake Manitou.

4-12 Rio Saska Control Dam Costs

The embankment volume for the dam cross-section shown in Figure 29 was found to be 700,000 cu. yds. when located at Mile 662.

Unit Costs: Excavation = \$ .50 per cu. yd.

Placement = \$ .10 per cu. yd.

To protect this earth embankment from erosion it will be necessary to provide rock riprap protection on both the upstream and downstream faces, amounting to a total of

$$\frac{2\ (30 \times 3)\ (8{,}000)\ (1)}{27} = 53{,}400 \text{ cu. yd.}$$ when a one-foot thick blanket is placed.

Unit Cost: Rock riprap = \$6 per cu. yd.

This riprap will be placed on a six inch thick gravel bedding amounting to 26,700 cu. yds.

Unit Cost: \$2 per cu. yd.

Total Control Dam Cost

Earthwork	=	700,000 (.50 + 0.10)	=	\$420,000
Riprap	=	53,400 (6.00)	=	320,000
Bedding	=	26,700 (2.00)	=	50,000
Total Dam Control Cost				\$790,000

4-13 Control Structure

The same considerations must be applied here as when deciding upon the spillway design flood of a major reservoir. If the control dam were to be overtopped, the consequences could be catastrophic since probably the dam would fail, sending a flood wave through the City of Portage. If this were to happen, the resultant damage would be higher than if no flood protection had been provided. Thus overtopping of the control dam must be prevented.

For this particular design a river flow of 350,000 cfs, having a frequency of 0.0001, was arbitrarily chosen.

Thus, if the diversion were designed for 50,000 cfs then the structure would have to pass 300,000 cfs to meet this design criteria. However, when the diversion passes 70,000 cfs then the in-channel structure has to pass only 280,000 cfs thus permitting a narrower and less costly structure.

A structure was designed for this capacity. With a sill at elevation 690.00 and a maximum water level at 724.00 the available head is 34 feet.

Using Weir - formula

$Q = CWH^{3/2}$ where Q = discharge

C = weir coefficient = 3.9 for ogee section

H = head on weir

W = width of structure

$$W = \frac{300,000}{3.9(34)^{3/2}} = 388 \quad \text{Say 400 feet}$$

A typical design is presented in Figure 33 The volume of concrete required was estimated as 16,550 cu. yds. at a unit cost of $72 per cu. yd. This structure also requires eight gates complete with hoists at a unit cost of $160,000.

Structure Cost:

Concrete	=	16,550 (72)	=	$1,192,000
Gates	=	8 (160,000)	=	$1,280,000
Total Structure Cost				$2,472,000

The cost curve for the Channel Control Structure is presented in Figure 34.

4-14 <u>Diversion - Alternative B</u>

A diversion at Site B at approximately Mile 667 was subjected to the same analysis. The benefits associated with this diversion are the same as those found for Alternative A, since the reduction of flow is the same. Hence, if Alternative B is to be superior its cost must be lower. Hence, only a cost comparison is required to decide between Alternative A and B.

The valley at Site B is somewhat wider, but the dam is approximately the same height. The embankment volume was estimated to be about 1,100,000 cu. yds. The structure in the Rio Saska will be of the same size as in Alternative A. For Alternative B the least-cost diversion costs are tabulated in Table III.

4-15 Capital Cost Estimates

Using the estimates for the diversion excavation, the associated dike fill, the land, the railway and highway crossings, and the drop structures required along the route, as computed by the computer program in conjunction with the control dam and control structure costs, the Capital Costs for the various diversion design discharges were calculated. Similar costs were also calculated when the maximum velocity was permitted to be 5,000 fps. The Capital Costs for Diversion Alternative A and B are shown in Figure 35 and Table IV for the above mentioned conditions.

Sample Calculation

Alternative A with maximum velocity = 5.00 fps

Design discharge = 50,000 cfs.

Costs:	Diversion Cut and Structures	=	$ 7,460,000
	River Control Structure	=	2,470,000
	Rio Saska Dam	=	790,000
	Subtotal	=	$10,720,000
	30% Engineering & Contingencies	=	3,220,000
	Capital Cost	=	$13,940,000

Alternative B with maximum velocity = 4 fps.

Design discharge = 50,000 cfs.

Costs:	Diversion Cut and Structures	=	$ 6,620,000
	River Control Structure	=	2,470,000
	Rio Saska Dam	=	1,220,000
	Subtotal	=	$10,310,000
	30% Engineering & Contingencies	=	3,090,000
	Capital Cost	=	$13,400,000

TABLE IV

CAPITAL COST ESTIMATE

PORTAGE DIVERSION

Alternative A

Diversion Design Flow cfs	Capital Cost in $ Million	
	Velocity = 4 fps.	Velocity = 5 fps.
10,000	7.32	6.93
30,000	12.06	10.49
50,000	16.82	13.94
70,000	21.50	17.33

Alternative B

10,000	7.83	7.20
30,000	12.64	10.35
50,000	17.33	13.40
70,000	21.91	16.38

CHAPTER V

TOBIN RAPIDS STORAGE RESERVOIR

5-1 General Considerations

As was pointed out in the City of Portage Engineering Department report, a suitable dam site exists at Tobin Rapids. Upon study of the topographic features and bore holes in the vicinity of Tobin Rapids it was decided to locate the dam as shown in Figure 36.

5-2 Main Structures Associated with a Storage Reservoir

a) Control Dam

There are several types of dams that can be constructed to impound water in the reservoir. The type of dam chosen depends largely on local circumstances, such as foundation conditions and availability of construction materials, as well as economic considerations.

b) Spillway

Several types of spillways can be employed. The final decision is usually based on foundation and topographic and economic considerations. Their purpose is to prevent overtopping of the dam which could have serious repercussions.

c) Conduit

A conduit is particularly important in the construction of a flood control reservoir, since it permits the regulation of the river flows and also is used to drain the reservoir immediately prior to the rainy season which usually occurs during June and July. Often such conduits are also used during the dam construction for diversion purposes.

5-3 Mode of Operation

The reservoir would be operated in such a manner so that it would be empty at the beginning of the rainy season. With rainfall and the subsequent increase of river flow the conduit would be fully opened to permit maximum discharge, which is generally less than the river flow at these low heads. As a result water would be stored in the reservoir, raising the water surface, increasing the available head on the conduit and increasing the conduit discharge. This discharge would be permitted to increase until the bankfull capacity of the Rio Saska at Portage is reached. This is the maximum stage at Portage at which no damage will be caused. As the flood flows increase and the water level rises in the reservoir, the control gates on the conduit would be partially closed to keep the discharge constant at the bankfull capacity. If the flood flows are sufficiently large the water level will rise to the spillway sill and water will be discharged over the spillway. Depending on the design, the spillway may be gated or have a free overflow section.

In order to prevent overtopping of the dam, releases will be permitted to exceed the bankfull capacity of the channel, thus causing damage. The water level will continue to rise as long as the inflow exceeds the outflow. Once they are equal, the peak outflow will have been reached. At this point the water can be released at this rate, or more slowly if it is desirable to gradually drain the reservoir. The procedure is schematically illustrated in Figure 37. It should be noted that the shaded area in this Figure represents the storage in the reservoir.

5-4 Simplified Flood Reduction Analysis

It was noted that the area below the inflow curve but above the outflow curve represents the storage capacity of the reservoir. This area can readily be obtained by planimetering, which does require considerable time.

a) Perfect Flood Predictions

The situation was idealized by assuming perfect flood prediction. The result of this simplification is that the entire storage of the reservoir can be used to reduce the peak outflow. This can be represented by a horizontal line drawn at various levels as shown in Figure 38.

b) Percentage Curve

Thus, a line drawn at a certain percentage of the peak flows will contain a certain percentage of the storage. In this manner, corresponding percentage can be found for various levels, which can then be graphically represented as shown in Figure 39 . The data for this Percentage Curve is also shown in Table V .

Sample Calculations

In Figure 39 at 50% of the peak flow the area contained under the peak = 3.68 sq. in. The total area of the hydrograph contains 16.39 sq. in. The corresponding percentage of storage $= \frac{3.68}{16.39} = 22.4\%$

c) Peak Reduction Curve

Using the Percentage Curve, Figure 39, and the Stage Storage Curve, Figure 10 , the peak outflow from the reservoir can be readily calculated for each particular inflow given a certain reservoir storage.

Sample Calculations

Consider a dam with a water level at elevation 1,000.00. From the Stage-Storage Curve the storage capacity corresponds to 200,000 acre-feet.

Consider an inflow peak of 240,000 cfs. From the Basic Flood Hydrograph, Figure 15.

TABLE V

TOBIN RESERVOIR

PEAK PERCENTAGE

Percent of Peak	Area of Peak	Percent of Storage
%	sq. in.	%
12.5	0.38	2.3
25.0	1.16	7.1
37.5	2.26	13.8
50.0	3.68	22.4
62.5	5.50	33.6
75.0	7.86	48.0
87.5	11.06	67.5
100.0	16.39	100.0

Vertical Scale: 1 in.	=	30,000 cfs
Horizontal Scale: 1 in.	=	2.5 days
Thus 1 sq. in.	=	30,000 (2.5)
	=	75,000 cfs days
	=	150,000 acre feet
Total Area under hydrograph	=	16.39 sq. in.
Total Flood Volume	=	2,460,000 acre feet
Thus, Percent of Storage	=	$\frac{200,000}{2,460,000}$
	=	8.13%
From Figure 39 Corresponding percent of peak is		26.7%
26.7% of Inflow of 240,000 cfs	=	0.267 (240,000)
	=	64,000 cfs
Outflow Peak	=	240,000 - 64,000
	=	176,000 cfs

In a similar manner, the corresponding outflows are calculated for the various inflows given certain storage. The results are shown in Table VI and are graphically represented by the Peak Reduction Curve in Figure 40.

d) Flow-Damage Curve

The Flow-Damage Curve, Figure 41, was developed from the Rio Saska Rating Durve, Figure 16 and the Stage Damage Curve, Figure 17. Occasionally, it is more convenient to use this curve instead of the Stage Damage Curve.

TABLE VI

RESERVOIR PEAK REDUCTION

Peak Inflow	Flood Volume	Storage = 50,000 ac. ft. Water Level = 940				Storage = 100,000 ac. ft. Water Level = 960				Storage = 200,000 ac. ft. Water Level = 1,000			
		Storage	Peak	Peak Reduction	Peak Outflow	Storage	Peak	Peak Reduction	Peak Outflow	Storage	Peak	Peak Reduction	Peak Outflow
cfs	ac.ft.	%	%	cfs	cfs	%	%	cfs	cfs	%	%	cfs	cfs
80,000	820,000	6.10	22.3	18,000	62,000	12.2	34.0	27,000	53,000	24.4	52.0	42,000	38,000
120,000	1,230,000	4.06	17.5	21,000	99,000	8.1	27.0	32,500	87,500	16.3	41.0	48,000	72,000
150,000	1,537,000	3.25	16.0	24,000	126,000	6.5	23.0	34,500	115,500	13.0	36.0	54,000	96,000
200,000	2,045,000	2.45	12.8	25,500	174,500	4.9	20.0	40,000	160,000	9.77	30.0	60,000	140,000
240,000	2,460,000	2.04	11.4	27,500	212,500	4.1	18.0	43,000	197,000	8.13	26.7	64,000	176,000
280,000	2,870,000	1.74	10.6	29,500	250,000	3.5	16.0	45,000	235,000	6.97	22.5	63,000	217,000
320,000	3,278,000	1.53	9.4	30,000	200,000	3.0	14.5	46,500	273,500				

Peak Inflow	Flood Volume	Storage = 320,000 ac. ft. Water Level = 1025				Storage = 430,000 ac. ft. Water Level = 1050				Storage = 600,000 ac. ft. Water Level = 1075			
80,000	820,000	39.0	67.0	53,500	26,500	52.5	78.0	62,500	17,500	73.2	90.0	72,000	8,000
120,000	1,230,000	26.0	54.0	65,000	55,000	35.0	63.5	76,000	44,000	48.8	75.0	90,000	30,000
150,000	1,537,000	20.8	48.0	72,000	78,000	28.0	56.3	84,500	65,500	39.0	67.0	100,000	50,000
200,000	2,045,000	15.6	40.0	80,000	120,000	21.0	48.0	96,000	104,000	28.3	58.5	117,000	83,000
240,000	2,460,000	13.0	36.0	86,500	153,500	17.5	43.3	104,000	136,000	24.4	52.0	125,000	115,000
280,000	2,870,000	11.1	32.0	89,500	190,500	15.0	39.0	113,000	167,000	20.9	48.0	135,000	145,000
320,000	3,278,000	9.8	28.5	91,000	229,000	13.1	36.0	115,000	205,000	18.9	45.0	144,000	176,000

5-5 Cost Consideration

The Capital Cost Curve, Figure 11 , for the dam and spillway at Tobin Rapids was developed for various heights of dams at this site. This curve was accepted as presented in the preliminary report on the power development aspects of this site.

5-6 Other Considerations

The spillway capacity for Tobin Rapids was 350,000 cfs representing a frequency of 0.01 per cent. The crest width was 440 feet and the weir discharge coefficient was assumed to be 3.65 for this ungated structure located on the north abutment. Upon review of this preliminary design in the previusly mentioned power report, it was accepted as presented.

In order to be able to employ the simplified reservoir routing procedures outlined in Section 5-4, a sufficiently large conduit was assumed to be able to pass the initial flood inflows. This conduit would be at about elevation 900, such that when the reservoir is used only for flood control, no dead storage exists.

CHAPTER VI

CHANNEL IMPROVEMENTS

6-1 General Considerations

In order to reduce the water levels at Portage several measures can be initiated. Excavation at Lister Rapids either deepening, widening, or a combination thereof can have a beneficial effect. In addition, the Rio Saska channel could also be widened and deepened in the reaches upstream of Lister Rapids. Backwater calculations are necessary to determine what effect such measures will have on the water levels at Portage, referenced to Mile 662.

6-2 Excavation in Lister Rapids

a) Widening: By increasing the width of Lister Rapids, the critical depths for the various discharges will be reduced. The previously developed computer program, using an approximation of Bresse's Equation, was used to calculate the resultant water surface profile of the Rio Saska and the stages at Portage. In this manner widths of 400, 450 and 500 feet were investigated. The results are graphically shown in Figure 42 .

b) Deepening: An examination of Figure 14 indicates that the stage at Portage could be lowered by rock excavation immediately upstream of Mile 645 in order to lower the existing control point. A typical excavation as shown in Figure 43 , representing cuts of up to five feet, were investigated. The water surface profile of the Rio Saska and the stage at Portage was calculated, again using the computer program shown in Appendix A, with the resultant Rating Curve at Portage shown in Figure 44 .

6-3 Channel Improvements Upstream of Lister Rapids

a) Widening the Rio Saska Channel: This possibility was investigated in Reach A, Mile 646 to 650, which has a typical width of 500 feet. The channel width was increased to 525,550, 575 and 600 feet. Again the stage was estimated using the computer program and is shown in Figure 44 . It should be noted that widening the channel in Reach A does not have an appreciable effect on the stage at Portage.

b) Widening Reach B: In order to determine what the effect of widening Reach B will be it is again necessary to develop a Normal Depth Curve for the various widths as shown in Figure 46 . These are then again used as input for the Bossen Method Routing Program yielding the Rating Curve at Portage shown in Figure 47. It was felt that widening should not increase the channel depth beyond 700 feet in width since bank instability may result if the new channel drastically deviates from the long term equilibrium section. Minor bank failures can be tolerated in this reach since the land is primarily used for grazing.

c) Widening the City Reach: This possibility was briefly considered, however quickly abandoned because of the disruption that such a scheme would cause within the prime residential area of the city. In addition, difficulties would also be encountered at the Rio Saska bridge in Portage.

d) Deepening the Rio Saska: This was considered, however abaondoned, for the following reasons. Any excavation in the channel bed would likely be accomplished through dredging, associated perhaps with undercutting of the existing riverbanks, leading to bank failures. Another possibility that presents itself, is that with time aggragation of the channel bed will take place and thus offset any beneficial effect the initial excavation would have had. To overcome this problem annual dredging would have to be initiated in order to maintain the depth of the channel. However, such measures are costly. Consequently, it was concluded that the existing equilibrium conditions best not be upset by deepening.

6-4 Reduction of Roughness Coefficient

A reduction of the roughness coefficient can be achieved by removal of shrubs, bushes and trees from the channel and banks. This measure was, however, abandoned for the following reasons: Within the city major opposition to such a scheme can be expected from the home owners adjacent to the Rio Saska for aesthetic reasons. In the agricultural areas, trees and shrubs could be removed, but it is expected that these would rapidly grow back, eliminating any beneficial effect such a scheme would have, unless annual maintenance is undertaken. There is the danger that even if maintenance is provided, it will suffer from neglect. It is also difficult to estimate the reduction of channel roughness and hence determine the benefits of such a scheme.

6-5 Capital Cost Estimates

a) Widening of Rapids: It should be noted that since most of the excavation will be in bedrock that it would be desirable to confine cuts to one bank, thus requiring the construction of only one cofferdam to permit this excavation. A cofferdam approximately 20 feet in height with a 10 foot wide top and 2:1 side slopes should be sufficient. The excavation would be timed to coincide with the low flow period of the Rio Saska. A cofferdam of only 20 feet of height is proposed since in the event of overtopping no major losses would result.

Sample Calculations: for width of 400 ft.

Length of Excavation = $= 10{,}200$ ft.

Rock Excavation $= \dfrac{(27)\ (50)\ 10{,}200}{27} = 510{,}000$ cu. yd.

Cofferdam $= \dfrac{(10 + 30)\ (20)\, 11{,}000}{27} = 326{,}000$ cu. yd.

Earth Excavation $= \dfrac{13\ (50)\ 10{,}200}{27} = 246{,}000$ cu. yd.

Costs:

Rock Excavation	= 510,000 (4.50)	= $ 2,300,000
Cofferdam	= 326,000 (5.0 + 1.0)	= 1,957,000
Earth Excavation	= 246,000 (0.40)	= 98,000
		$ 4,355,000
30% Engineering and Contingencies		= 1,310,000
Capital Cost		= $ 5,665,000

In a similar manner the capital costs associated with the other widths considered were found and were plotted to give Figure 48 .

b) Deepening of Rapids: In order to lower the rock sill in Lister Rapids, it is necessary to construct cofferdams to enable the contractor to perform the excavation. It is proposed to halve the upstream width by placing a cofferdam near the centre of the channel. In this manner, once one side has been excavated, it is only necessary to replace the end sections and the other half of the channel is enclosed by the cofferdam. The capital costs associated with this excavation were estimated and are shown in Figure 49

c) Widening the Channel Upstream of Lister Rapids: The costs associated with widening Reach A and B were found and are presented in Figures 50 and 51 .

PART III

ECONOMIC ANALYSIS

CHAPTER VII

ECONOMIC ANALYSIS

7-1 General Considerations

In order to determine which project or combaination of projects should be implemented, it is necessary to compute economic parameters, such as the Benefit/Cost Ratio and the Net Benefits. These can then be used to justify the selection and the scope of development of the project.

7-2 Interest Rate, Life of the Project and Operation & Maintenance Charges

Upon consideration of prevailing interest rates and an appraisal of the financing of similar projects it was decided to use 6 per cent in all subsequent economic calculations. It is felt that this is approximately the real cost of borrowing when allowance is made for inflation in current market interest rates.

A life of 50 years has been assumed for all the projects. Where it was necessary to assume a shorter life, a specific explanation is provided. A 50 year life was considered realistic, even though many of the proposed projects will function for a longer period, since it is difficult to predict such things as obsolescence and future use. In any event, when discounted over a longer period the changes in the result become less and less significant.

An allowance of one percent per annum has been made for operation and maintenance unless otherwise stated.

7-3 Flood Damages

The Stage Damage Curve presented by the City of Portage Engineering Department was accepted as presented in Figure 17 .

Using this Stage Damage Curve, the Rating Curve at Portage and the Frequency Curve Figures 17 , 16 and 19, respectively, the Average Damage Curve was constructed. Figure 52 .

Sample Calculation:

At Elevation 705.00:

Flood Damage = $14.0 Million

From Figure 16 Rio Saska Discharge = 100,000 cfs.

From Figure 19 this discharge corresponds to a frequency of 9.0%.

The damage value and the frequency are then plotted as the absissa and the ordinate respectively. Similarly, other points are calculated as tabulated in Table VII and plotted to yield the Average Annual Damage Curve as shown in Figure 52.

A unique feature of this curve is that the area under the curve represents the average annual damage. The area under this curve can easily be established by mechanical integration such as measuring with a planimeter.

Sample Calculation:

Area under curve measured by planimeter = 4.10 sq. in.

Vertical scale 1 inch = 5%

Horizontal scale 1 inch = $ 20,000,000

Thus, 1 sq. in. = 20,000,000 (0.05)

= $ 1,000,000

Average Annual Damage = $ 4,000,000

7-4 Effect of Existing Dike on Average Annual Damage Curve

From the Rating Curve Figure 16 it is evident that when the water level is at elevation 703.50, the effective top of the existing dikes, the Rio Saska discharge is 80,000 cfs. This flow represents a flood frequency of 17.5%. Hence, at this frequency the damages to property in the City of Portage is zero. As soon as the water exceeds this elevation the damages are the same as when no dikes were present. This is represented on the Average Annual Damage Curve as a horizontal line at a frequency of 17.5% as shown in Figure 53.

Planimetering the remaining area under the curve indicates that the remaining average annual damage equals $4,020,000.

TABLE VII

FREQUENCY VS DAMAGE
WITHOUT PROTECTIVE WORKS

FLOW	FREQUENCY	DAMAGE
cfs	%	$ Million
70,000	25	0
80,000	17.5	2.5
83,000	15.5	4.0
90,000	12.0	8.0
100,000	9.0	14.0
120,000	4.5	30.0
136,000	2.5	47.0
150,000	1.7	58.0
185,000	0.6	80.0
220,000	0.3	94.0
250,000	0.1	102.0
Maximum	0	110.0

7-5 Determination of Diking Benefits

The benefits of the diking system are the damages that are prevented by the system.

From Figures 26 and 19 and, the Frequency Curve, Table 8 was prepared, and data was then plotted to obtain the Average Annual Damage Curves shown as Figures 53 and 54. The area below the effective elevation of the existing dikes but above the permissible water level represents the average annual benefit of this diking system.

Sample Calculation:

To illustrate calculation of average annual benefits.

In Figure 16 the permissible water level for the existing dikes is 703.50 containing a flow of 80,000 cfs. and representing a frequency of 17.5%; hence, a horizontal line at 17.5% in Figure 53. Similarly, a diking system permitting a water level of 705.00 contains a flow of 94,000 cfs having a frequency of 10.7%. This is represented by a horizontal line drawn at 10.7% in Figure 53. The area contained between the lines represents the average annual benefit of this particular diking system, and has found to be $144,000 per year. In a similar manner the annual benefits of the other diking systems were found as shown in Table IX.

The Benefit Curve as shown in Figure 55 is the graphical representation of Table IX. Within the range shown this curve makes it possible to estimate the average annual benefits of any height of dike.

7-6 Economic Analysis - Low Dike Alternative = 705.50

Capital Cost = $ 3,093,000

Annual Cost Calculation:

Consider an interest rate of 6% and a 50 year-life with annual maintenance charges equal to 1% of capital cost.

Annual Cost = $ 3,093,000 (0.0634 + 0.01)

= $ 227,000

Annual Benefits = $ 670,000

Benefit/Cost Ratio: $= \frac{670,000}{227,000} = 2.95$

TABLE VIII

DYKE PROTECTION

Permissible Water Level	Dyke Adjacent to Channel	
	Discharge	Frequency
703.50	80,000	17.5%
704.00	83,000	14.5
704.50	88,000	13.0
705.00	94,000	10.7
705.50	100,000	8.6
	Dyke 150 ft. from Channel	
703.50	80,000	17.5%
704.00	86,000	14.0
704.50	91,000	11.8
705.00	97,000	9.7
706.00	110,000	6.0
707.00	123,000	3.9
708.00	140,000	2.4
709.00	157,000	1.3
710.00	175,000	0.78

TABLE IX

DYKING BENEFITS

Dykes Adjacent to Channel

Permissible Water Level	Area sq. in.	Average Annual Benefits	Percent Excedence
703.50	0.00	0	17.5
704.00	0.10	100,000	14.5
704.50	0.21	210,000	13.0
705.00	0.44	440,000	10.7
705.50	0.67	670,000	8.6

Dykes at 150 feet from Channel

703.50	0.00	0	17.5
704.00	0.13	130,000	14.0
704.50	0.30	300,000	11.8
705.00	0.53	530,000	9.7
706.00	1.24	1,240,000	6.0
707.00	1.84	1,840,000	3.9
708.00	2.45	2,450,000	2.4

Calculation of Net Benefits:

Present Value of Benefits	= 670,000 (15.76)
	= $ 10,550,000
Present Value of Operation and Maintenance = 15.76 (0.031)	=(-) 480.000
Capital Cost	=(-) 3,093,000
Present Value of Net Benefits	= $ 6,923,000

7-7 Economic Analysis - High Diking Alternative

(a) For Dike to Elevation at 708.00

Capital Cost	= $ 31,876,000

Annual Cost Calculation:

Annual Cost	= 31,876,000 (0.0734)
	= 2,340,000
Annual Benefits	= 1,960,000
Benefit/Cost Ratio:	= $\frac{1,969,000}{2,340,000} = 0.84$

Calculation of Net Benefits:

Present Value of Benefits	= (1,960,000) (15.76)
	= $ 30,900,000
Present Value of Operation and Maintenance = 15.76 (0.32)	= 5,040,000
Capital Cost	= 31,876,000
Present Value of Net Benefits	= $ 5,964,000 (neg.)

(b) For Dike Elevation 719.00

Capital Cost	= $ 36,000,000

Annual Cost Calculations:

Annual Cost = 36,900,000 (0.0734)	= 2,710,000
Annual Benefits	= 4,020,000
Benefit/Cost Ratio:	= $\frac{4,020,000}{2,710,000} = 1.48$

Calculation of Net Benefits:

Present Value of Benefits	= 4.020,000 (15.76)
	= $ 63,400,000
Present Value of Operation and Maintenance = 15.76 (.370)	=(-) 5,810,000
Capital Cost	=(-) 36,900,000
Present Value of Net Benefits	= $ 20,600,000

7-8 Discussion of Diking Alternatives

From this economic analysis it is apparent that the Low Dike Alternative is desirable. The Benefit/Cost Ratio of 2.95 indicates that available capital will be used advantageously. The resulting Net Benefits of $6,923,000 are also substantial.

The High Dike Alternative does not appear to be attractive especially at the lower levels, where it is uneconomic. While a dike to elevation 719.00 would afford almost complete protection, it requires a large capital outlay which is not as efficiently used as the smaller capital investment in the Low Dike Alternative. Even though the Net Benefits are substantially higher for the High Diking Alternative, it is not recommended, since the implementation of this proposal would require extensive expropriation and would represent a major disruption of the community. In addition, a dike approximately twenty feet high, would in itself be an eyesore, when one considers that it would eliminate some of the prime residential areas of the city which are presently bordering the Rio Saska.

Higher water levels would result upstream of the City of Portage. However, the valley flats which are primarily used for cattle grazing are in any event periodically flooded. The somewhat higher stages will cause no additional damages.

7-9 Economic Analysis of the Portage Diversion

The diversion benefits were assessed for diversion flows ranging from 10,000 cfs to 80,000 cfs.

A certain flood flow having a certain frequency is reduced by the diversion flow, thus causing less damage at the City of Portage. The appropriate reductions are shown in Table X and are graphically represented in Figures 56 and 57. The average annual benefits are represented by the reduction of average annual damages, and can be found by planimetering the appropriated areas under the curves. The benefits as well as their present values are presented in Table XI and Figure 58.

TABLE X

DIVERSION DAMAGE CURVE

DIVERSION CAPACITY = 10,000 cfs

Flood Flow cfs	Flood Freq. %	Diversion Flow cfs	Rio Saska Flow cfs	Damage $ Million
90,000	12.0		80,000	0.0
100,000	8.7		90,000	8.0
110,000	6.1		100,000	14.0
120,000	4.5		110,000	21.7
130,000	3.2		120,000	30.0
140,000	2.4		130,000	40.0
150,000	1.7		140,000	49.5
160,000	1.2		150,000	59.0
170,000	0.9		160,000	65.5
200,000	0.4		190,000	82.2
220,000	0.23		210,000	91.3

DIVERSION CAPACITY = 20,000 cfs

Flood Flow cfs	Flood Freq. %	Diversion Flow cfs	Rio Saska Flow cfs	Damage $ Million
100,000	8.7		80,000	0.0
110,000	6.1		90,000	8.0
120,000	4.5		100,000	14.0
130,000	3.2		110,000	21.7
140,000	2.4		120,000	30.0
150,000	1.7		130,000	40.0
160,000	1.2		140,000	49.5
170,000	0.9		150,000	59.0
200,000	0.4		180,000	77.5
220,000	0.23		200,000	87.0

DIVERSION CAPACITY = 30,000 cfs

Flood Flow cfs	Flood Freq. %	Diversion Flow cfs	Rio Saska Flow cfs	Damage $ Million
110,000	6.1		80,000	0.0
120,000	4.5		90,000	8.0
130,000	3.2		100,000	14.0
140,000	2.4		110,000	21.7
150,000	1.7		120,000	30.0
160,000	1.2		130,000	40.0
170,000	0.9		140,000	49.5
200,000	0.4		170,000	71.7
220,000	0.23		190,000	82.2

DIVERSION CAPACITY = 40,000 cfs

Flood Flow cfs	Flood Freq. %	Diversion Flow cfs	Rio Saska Flow cfs	Damage $ Million
120,000	4.5		80,000	0.0
130,000	3.2		90,000	8.0
140,000	2.4		100,000	14.0
150,000	1.7		110,000	21.7
160,000	1.2		120,000	30.0
170,000	0.9		130,000	40.0
200,000	0.4		160,000	65.5
220,000	0.23		180,000	77.5

DIVERSION CAPACITY = 50,000 cfs

Flood Flow cfs	Flood Freq. %	Diversion Flow cfs	Rio Saska Flow cfs	Damage $ Million
130,000	3.2		80,000	0.0
140,000	2.4		90,000	8.0
150,000	1.7		100,000	14.0
160,000	1.2		110,000	21.7
170,000	0.9		120,000	30.0
180,000	0.68		130,000	40.0
210,000	0.30		160,000	65.5
230,000	0.17		180,000	77.5

DIVERSION CAPACITY = 60,000 cfs

Flood Flow cfs	Flood Freq. %	Diversion Flow cfs	Rio Saska Flow cfs	Damage $ Million
140,000	2.4		80,000	0.0
150,000	1.7		90,000	8.0
160,000	1.2		100,000	14.0
170,000	0.9		110,000	21.7
180,000	0.68		120,000	30.0
190,000	0.50		130,000	40.0
220,000	0.23		160,000	65.5
240,000	0.13		180,000	77.5

DIVERSION CAPACITY = 70,000 cfs

Flood Flow cfs	Flood Freq. %	Diversion Flow cfs	Rio Saska Flow cfs	Damage $ Million
150,000	1.7		80,000	0.0
160,000	1.2		90,000	8.0
170,000	0.9		100,000	14.0
180,000	0.68		110,000	21.7
190,000	0.50		120,000	30.0
200,000	0.40		130,000	40.0
230,000	0.17		160,000	65.5

DIVERSION CAPACITY = 80,000 cfs

Flood Flow cfs	Flood Freq. %	Diversion Flow cfs	Rio Saska Flow cfs	Damage $ Million
160,000	1.2		80,000	0.0
170,000	0.9		90,000	8.0
180,000	0.68		100,000	14.0
190,000	0.50		110,000	21.7
200,000	0.40		120,000	30.0
210,000	0.30		130,000	40.0
220,000	0.23		140,000	49.5
230,000	0.17		150,000	59.0

TABLE XI

PORTAGE DIVERSION - SUMMARY OF BENEFITS

Diversion Capacity cfs	Average Annual Benefits $ Million	Present Value of Benefits $ Million
10,000	1.22	19.20
20,000	1.98	31.15
30,000	2.50	39.35
40,000	2.95	46.50
50,000	3.17	50.00
60,000	3.37	53.10
70,000	3.65	57.80
80,000	3.74	58.90

It was felt that velocities of 5.00 fps would be acceptable. The Capital Savings that result by using 5 fps instead of 4 fps are substantial, especially at the higher flows as is evident from Figure 35.

The appropriate economic parameters were computed as follows:

Sample Calculation for Alternative B with a Diversion Capacity = 50,000 cfs.

Capital Cost		=	\$13,400,000
Annual Cost	= 13,400,000	=	973,000
Annual Benefits from Table		=	3,170,000
Benefit/Cost Ratio	$= \frac{3,170,000}{1,025}$	=	3.26

Present Value of Net Benefits:

Present value of benefits from Table	=	\$50,000,000
Operation & Maintenance = .134 (15.76)	=	− 2,110,000
Capital Cost	=	−13,400,000
Net Benefits	=	\$34,490,000

In a similar manner the other diversion capacities were analized, the results of which are to be found in Table and graphically illustrated in Figure 59.

7-10 Discussion of the Portage Diversion

From Figures 30 and 31 it appears that the channel at Site B is less costly for most discharges. However, the dam costs at this site are higher partly offsetting this advantage, to the extent that there is practically no difference in the Capital Costs. As shown in Figure 59, Diversion B has a maximum Benefit Cost Ratio of 3.31 at diversion capacity of 40,000 cfs. Similarly, the maximum Net Benefits are \$38.12 million occurring at a diversion design capacity of 70,000 cfs.

From these observations it is apparent that Diversion B is more beneficial and thus should be constructed.

TABLE XII

PORTAGE DIVERSION

ECONOMIC SUMMARY

Diversion Capacity	Capital Cost	Annual Cost	Annual Benefits	PV of Benefits	PV of O & M	Benefit Cost Ratio	Net Benefits
cfs	$ Million	$ Million	$ Million	$ Million			$ Million
10,000	7.25	0.532	1.22	19.20	1.14	2.29	10.81
20,000	8.80	0.650	1.98	31.15	1.38	3.07	20.97
30,000	10.35	0.760	2.50	39.35	1.63	3.29	27.37
40,000	12.00	0.880	2.90	45.70	1.89	3.31	31.81
50,000	13.40	0.970	3.17	50.00	2.11	3.26	34.49
60,000	15.00	1.100	3.40	53.60	2.36	3.09	36.24
70,000	16.40	1.200	3.63	57.10	2.58	3.02	38.12
80,000	18.60	1.370	3.74	58.90	2.93	2.74	37.37

If capital were scarce, it would be desirable to design the diversion for 40,000 cfs in order to use the available money most intensively.

If money were readily available, it would be desirable to construct a diversion for 70,000 cfs yielding the maximum Net Benefits.

7-11 Benefit Determination of Tobin Reservoir

Using the Peak Reduction Curve, Figure 40, in conjunction with the Frequency Curve, Figure 19, and the Flow Damage Curve, Figure 41, Table 13 was prepared to relate inflow, outflow, frequency and resulting damages for the various reservoir sizes considered. The graphical representation of this table is the Average Annual Damage Curve in Figures 60 and 61.

It should be noted that the reduction of average annual damages are the annual benefits of the storage reservoir. These can be determined by planimetering the appropriate areas in Figures 60 and 61. The results of this exercise are illustrated as Figure 62, the Benefit-Storage Curve.

7-12 Economic Analysis of Tobin Reservoir

Using the benefit and cost curves previously defined for Tobin Reservoir, the economic analysis was proceeded with.

Sample Calculation:

A reservoir with a water level of 1,000.00 has a live storage for flood control purposes of 200,000 acre-feet.

Capital Cost, from Figure 11	= \$18,700,000
Annual Cost	= 18,700,000 (0.0734)
	= 1,370,000
Annual Benefits	= 3,360,000
Benefit/Cost Ratio	$= \frac{3,360,000}{1,370,000} = \underline{\underline{2.45}}$

TABLE XIII

TOBIN RESERVOIR - AVERAGE ANNUAL DAMAGE CURVE

Storage ac.-ft.	Inflow cfs	Frequency %	Outflow cfs	Damage $ Million
50,000				
	100,000	8.7	80,000	0.0
	112,000	5.6	91,000	8.0
	122,000	4.0	100,000	14.0
	136,000	2.7	113,000	24.0
	150,000	1.7	126,000	36.0
	167,000	1.0	142,000	51.0
	180,000	0.7	155,000	62.0
	200,000	0.4	175,000	74.5
	240,000	0.15	212,500	91.5
100,000				
	112,000	5.8	80,000	0.0
	120,000	4.5	87,500	7.0
	130,000	3.2	96,000	11.0
	150,000	1.7	115,500	26.0
	167,000	1.0	130,000	40.0
	200,000	0.4	160,000	65.0
	240,000	0.14	187,000	86.0
	280,000	0.05	235,000	99.0
	320,000	0.02	273,000	106.0
200,000				
	132,000	3.0	80,000	0.0
	150,000	1.7	96,000	11.0
	179,000	0.7	120,000	30.0
	200,000	0.4	140,000	50.0
	240,000	0.15	176,000	75.0
320,000				
	152,000	1.7	80,000	0.0
	180,000	0.7	103,000	17.0
	200,000	0.4	120,000	30.0
	240,000	0.15	154,000	62.0
430,000				
	170,000	1.0	80,000	0.0
	180,000	0.7	88,000	7.0
	200,000	0.4	104,000	17.5
	240,000	0.15	135,000	44.0
600,000				
	194,000	0.45	80,000	0.0
	220,000	0.23	100,000	14.0
	274,000	0.055	142,000	51.5
	318,000	0.020	175,000	74.0

Net Benefits:

Present Value of Benefits	= $53,250,000
Present Value of Operation and Maintenance = 15.76 (.187)	= - 2,950,000
Capital Cost	= -18,700,000
Net Benefits	= $31,600,000

In a similar manner the economic parameters for the other reservoir storages were computed and are shown in Table XIV and illustrated in Figure 62.

7-13 Discussion of Tobin Reservoir

An examination of Figure 63 reveals that the most intensive use of the available capital can be made at a reservoir capacity of about 130,000 acre-feet and a corresponding water level of 975. The Benefit/Cost Ratio is about 2.47.

The maximum Net Benefits result with a storage capacity of approximately 450,000 acre-feet, with a corresponding water level of about 1055. The Benefit/Cost Ratio at this level of development is approximately 2.33.

Since the benefits are maximized and the capital is efficiently used, a storage reservoir of 450,000 acre-feet should be constructed.

7-14 Economic Analysis of Channel Improvement

The channel improvements outlined in Chapter VI were subjected to an economic analysis.

a) Widening Lister Rapids: Figure 48 was used to estimate the Capital Costs associated with widening Lister Rapids.

To determine the benefits it is necessary to determine what effect the widening of the rapids has on the river stage at Portage. The Rating Curve was obtained as explained in Section 6-2 (a) and is shown in Figure 42. Using the Rating Curve, the Frequency Curve, Figure 19, and

TABLE XIV

ECONOMIC SUMMARY

TOBIN RESERVOIR

		BENEFITS		COSTS			ECONOMIC PARAMETER	
Water Level	Reservoir Storage	Annual Benefits	Present Value of Benefits	Capital Cost	Annual Cost	Present Value O & M	Benefit/ Cost Ratio	Net Benefits
	ac. ft.	$ Mill.	$ Mill.	$ Mill.	$ Mill.	$ Mill.		$ Mill.
940.00	50,000	2.14	33.75	14.5	1.06	0.23	2.01	19.02
963.00	100,000	2.83	44.65	16.5	1.21	0.26	2.34	27.89
1000.00	200,000	3.36	53.00	18.7	1.37	0.29	2.45	34.00
1025.00	320,000	3.68	58.00	20.8	1.53	0.33	2.40	36.87
1055.00	430,000	3.84	60.55	22.3	1.64	0.35	2.34	37.90
1075.00	600,000	3.93	62.00	24.5	1.80	0.39	2.18	37.11

the Stage Damage Curve, Figure 17 , Table XV was prepared which was then used to plot the damage curve shown on Figure 64 . By planimetering the appropriate areas, the benefits were found. The Annual Costs, the Benefit/Cost Ratio and Net Benefits were calculated and were summarized in Table XVI .

b) Deepening Lister Rapids: Figure 49 was used to estimate the capital costs associated with deepening of Lister Rapids.

To determine the benefits of deepening the rapids it was again necessary to develop a Rating Curve at Portage. This was done as outlined in Section 6-2 (b) and is shown in Figure 44 . Using this Rating Curve in conjunction with the Frequency Curve and the Stage-Damage Curve, Table XVII was prepared. This Table was then used to plot the Damage Curve shown in Figure 66 from which the average annual benefits were determined by planimetering. The remainder of the economic analysis were proceeded with as outlined in Section 7-14 (a) and the results are tabulated in Table XVI .

c) Widening Reach A: Again the Rating at Portage, as shown on Figure 45 had to be determined and then used to prepare a table similar to Table XVII. This table is not presented but was used to plot the Damage Curve shown on Figure 68 from which the benefits were determined. The remainder of the economic analysis were proceeded with as outlined in Section 7-14 (a), the results of which are presented in Table XVI and Figure 70 .

d) Widening Reach B: The analysis followed the methodology outlined in Section 7-14 (c). The Rating Curve, the tabulated damage estimates, the Damage Curve, and a Summary of the economic analysis for this proposal are presented in Figure 47 , Table 18 , Figure 71 and Table 16 respectively.

e) Comments on Channel Improvements: From the foregoing analysis it is apparent that widening of Lister Rapids is uneconomical while deepening of Lister Rapids is

TABLE XV

Channel Improvements

Widen Lister Rapids

Width = 400 ft.

Elevation	Discharge 1,000 cfs	Frequency %	Damage $ Million
703.5	82.0	16.3	0.0
703.5	82.0	16.3	3.0
703.8	85.0	14.6	5.0
704.0	88.0	13.2	6.0
705.0	101.0	8.5	14.0
706.0	115.0	5.3	25.5
707.0	132.0	3.0	40.0
708.0	150.0	1.7	55.0
710.0	188.0	.55	79.0

Width = 450 ft.

Elevation	Discharge 1,000 cfs	Frequency %	Damage $ Million
703.0	83.5	15.4	0.0
703.5	83.5	15.4	3.0
704.0	89.5	12.8	6.0
705.0	103.0	8.0	14.0
706.0	117.0	4.8	25.5
707.0	135.0	2.7	40.0
708.0	154.0	1.5	55.0
710.0	193.0	.47	79.0

Width = 500 ft.

Elevation	Discharge 1,000 cfs	Frequency %	Damage $ Million
703.5	85.0	14.5	0.0
703.5	85.0	14.5	3.0
703.8	88.0	13.2	5.0
704.0	91.0	11.8	6.0
705.0	106.0	7.1	14.0
706.0	121.0	4.3	25.5
707.0	137.5	2.5	40.0
708.0	157.5	1.3	55.0
710.0	198.0	0.4	79.0

TABLE XVI

CHANNEL IMPROVEMENT

ECONOMIC SUMMARY

Width (ft.)	Depth of Excav. (ft)	Capital Cost	Annual Cost	Annual Benefits	PV of Benefits	PV of O & M	Benefit Cost Ratio	Net Benefits
		$ Million						
Widening Lister Rapids								
400		5.67	0.36	0.30	4.72		0.83	Neg.
450		8.78	0.56	0.57	8.98		1.02	0.20
500		11.90	0.75	0.86	13.70		1.15	2.00
Deepening Lister Rapids								
	1	2.56	0.19	0.06	0.95	0.40	0.32	Neg.
	2	3.23	0.24	0.13	2.05	0.51	0.54	"
	3	4.20	0.31	0.235	3.70	0.66	0.76	"
	4	4.93	0.36	0.29	4.56	0.78	0.80	"
	5	5.85	0.43	0.32	5.04	0.02	0.75	"
Widening Reach A.								
525		.183	0.015	0.085	1.34	0.06	5.66	1.10
550		.375	0.031	0.13	2.05	0.12	4.20	1.65
575		.57	0.048	0.155	2.44	0.18	3.23	1.69
600		.775	0.065	0.17	2.68	0.25	2.61	1.66
Widening Reach B.								
625		0.34	0.025	0.49	7.72	0.077	18.5	7.30
650		0.68	0.050	0.87	13.70	0.14	17.4	12.88
675		1.02	0.075	1.16	18.30	0.18	15.5	17.10
700		1.36	0.100	1.33	21.00	0.21	13.3	19.43

TABLE XVII

Channel Improvements

Deepen Lister Rapids

Excavation 2 ft.

Elevation	Discharge 1,000 cfs	Frequency %	Damage $ Million
703.5	85.0	14.5	0
703.5	85.0	14.5	3.0
704.0	90.0	12.0	8.0
705.0	102.5	8.0	16.0
706.0	115.0	5.4	25.7
707.0	131.0	3.0	40.0
708.0	148.0	1.8	57.0
710.0	189.0	0.5	82.3

Excavation 4 ft.

Elevation	Discharge 1,000 cfs	Frequency %	Damage $ Million
703.5	90.0	12.0	0
703.5	90.0	12.0	3.0
704.0	95.0	10.2	10.8
705.0	108.0	6.5	21.0
706.0	123.0	4.0	33.0
707.0	138.0	2.5	47.5
708.0	157.0	1.3	63.0
710.0	195.0	0.45	84.7

Excavation 5 ft.

Elevation	Discharge 1,000 cfs	Frequency %	Damage $ Million
703.5	91.5	11.7	0
703.5	91.5	11.7	3.0
704.0	97.5	9.5	12.3
705.0	110.0	6.1	22.0
706.0	125.0	3.8	34.0
707.0	140.0	2.3	50.0
708.0	160.0	1.2	65.0
710.0	202.0	0.37	88.0

TABLE XVIII

CHANNEL IMPROVEMENTS

WIDEN REACH B

Elevation	Damage	Width of Reach B: 650 ft. Discharge 1,000 cfs.	650 ft. Frequency %	700 ft. Discharge 1,000 cfs.	700 ft. Frequency %
703.50	0.0	85.0	14.5	90.0	12.2
703.50	3.0	85.0	14.5	90.0	12.2
703.75	4.0	88.0	13.0	92.5	11.2
704.00	7.5	92.0	11.5	95.5	10.0
705.00	12.5	104.0	7.5	108.0	6.4
706.00	22.7	117.5	4.8	123.0	4.0
707.00	33.0	132.0	3.0	138.0	2.5
708.00	50.0	148.0	1.8	155.0	1.4
709.00	73.0	167.0	1.0	173.0	0.85
710.00	78.0	184.0	0.6	191.0	0.49

economical. However, the Net Benefits that can be achieved are small. This is also true for widening of Reach A to 600 feet. Widening Reach B to 700 feet is a very economic undertaking as can be seen from Table XVI. The Benefit/Cost Ratio is high and the Net Benefits achieved are also substantial. From these observations it can be surmized that a combination involving widening of Reach B will likely represent the optimum development of flood protection.

7-15 Analysis of Combinations of Projects

Using the previously developed Cost Curves for the individual projects, it is possible to estimate the cost associated with a combination of projects. Using the basic approach employed for the single projects, the benefits associated with a combaination of projects can also be determined.

7-16 Combination - Reservoir and Diversion

In this manner various reservoir sizes were combined with various diversion capacities. The analysis of a combination of a reservoir at Tobin Rapids having a storage capacity of 50,000 acre-feet and diversion near the City of Portage, having a capacity of 30,000 cfs will be presented to illustrate the methodology employed.

Sample Calculation:

a) Capital Cost Estimate of Combination:

Reservoir Cost (Figure 11)	$14.50 Million
Diversion Cost (Figure 35)	10.30
Total Capital Cost	$24.80 Million

b) Annual Cost Estimate of Combination:

In a similar manner the Annual Costs are:

Reservoir	$ 1.05 Million
Diversion	0.75
Total Annual Cost	$ 1.80 Million

c) Benefit Determination:

It should be noted the bankfull capacity of the existing diking system at Mile 662 is 80,000 cfs. This can be found from the Rating Curve at Portage as shown in Figure 16 and a water level of 703.50 at this location. If the diversion capacity is 30,000 cfs, the permissible outflow from Tobin Rapids is 80,000 + 30,000 = 110,000 cfs. This outflow is equivalent to an inflow of 132,000 cfs into 50,000 acre-foot reservoir at Tobin Rapids as can be determined from Peak Reduction Curve shown in Figure 40. From Figure 19, it is found that a flood flow of 132,000 cfs has a frequency of 3.0%.

This particular combination of reservoir and diversion thus reduces a flood flow of 132,000 cfs sufficiently so that the damages in the City of Portage are zero.

If the flow was fractionally larger, the existing dikes would be overtopped causing an estimated damage of \$3.0 Million as indicated by the Stage Damage Curve, Figure 17. Similarly, when a flood of 144,000 cfs having a 2% frequency occurs, the outflow from Tobin Reservoir will be 120,000 cfs, which when reduced by the 30,000 cfs diversion, yields a river flow at Portage of 90,000 cfs, causing an estimated \$8.0 Million damage. In this manner, other larger flood flows were treated, the results of which are presented in Table XIX. The frequency and damage values thus tabulated are plotted to yield the Damage Curve for this combination shown in Figure 74.

As was the case for the previously illustrated single projects, the benefits of this combination are the damages prevented. The benefits were determined by planimetering the area under the curve and were found to be \$3.40 Million annually.

d) Economic Analysis of Combination:

The Annual Costs and Annual Benefits thus found were \$1.80 Million and \$3.40 Million respectively.

$$\text{Benefit Cost Ratio:} = \frac{\text{Annual Benefits}}{\text{Annual Costs}} = \frac{3.40}{1.80} = \underline{\underline{1.89}}$$

TABLE XIX

COMBINATION - RESERVOIR AND DIVERSION

DAMAGE CURVE

Frequency %	Flood Flow 1000 cfs	Reservoir Outflow 1000 cfs	Diversion Flow 1000 cfs	River Flow 1000 cfs	Stage	Damage $ Million
3.0	132	110	30	80	703.5	0.0
3.0	132	110	30	80	703.5	3.0
2.0	144	120	30	90	704.3	8.0
1.0	168	143	30	113	705.2	14.5
0.5	190	165	30	135	707.6	42.0
0.25	215	188	30	158	708.9	65.0

Present Value of Net Benefits:

Present Value of Benefit = 15.76 (3.40)	= $53.50 Million
Present Value of Operation and Maintenance = 15.76 (0.0195 + 0.103)	= -1.86
Capital Cost	= -24.80
Net Benefits	= $26.84 Million

The foregoing calculations representing Combination 2 were summarized in Table XX .

e) Other Reservoir-Diversion Combinations:

In a similar manner, other diversion capacities, 10,000, 50,000 and 70,000 cfs, were combined with the 50,000 acre-foot reservoir to determine what degree of flood protection they would provide at what cost. The tables required to plot the Damage Curve for these combinations, similar to Table 19 were computed but are not presented since little is gained from them. However, the Damage Curve for these combinations, required for the determination of the benefits are also presented in Figure 74. The results of these four combinations 1 to 4, are summarized in Table XX .

In a similar manner, larger reservoir sizes, 100,000, 200,000 and 400,000 acre-feet in combinations with previous mentioned diversion capacities were examined. The corresponding Damage Curves are shown in Figures 75, 76, and 77 and are summarized in Table XX under Combinations 5 to 16 inclusive.

f) Comments on Reservoir-Diversion Combination:

An inspection of the summary of combinations 1 to 16 reveals that the maximum Net Benefits are obtained on combination 13 amounting to $30.82 million. This is considerably less than the maximum for the individual reservoir, equal to $37.90 million and individual diversion of $38.12 million as presented in Tables XIV and XII. The graphical summary as presented in Figure 78 confirms this trend. A single diversion or reservoir is indicated by the increase of Net Benefits as the diversion capacity is decreased.

TABLE XX

COMBINATION RESERVOIR AND DIVERSION

ECONOMIC SUMMARY

Combination	SIZE Res. 1000 ac.ft.	SIZE Div. 1000 cfs	CAPITAL COST Res. ($ Million)	CAPITAL COST Div. ($ Million)	CAPITAL COST Total ($ Million)	ANNUAL COST Res. ($ Million)	ANNUAL COST Div. ($ Million)	ANNUAL COST Total ($ Million)	Annual Benefits ($ Million)	P.V. of Benefits ($ Million)	P.V. of O & M ($ Million)	Benefit Cost Ratio	Net Benefits ($ Million)
1	50	10	14.50	7.20	21.70	1.05	0.54	1.59	2.70	42.50	1.37	1.70	19.43
2		30		10.30	24.80		0.75	1.80	3.40	53.50	1.86	1.89	26.84
3		50		13.50	28.00		0.97	2.02	3.70	58.25	2.36	1.83	27.89
4		70		16.50	31.00		1.23	2.28	3.86	60.60	2.84	1.69	26.76
5	100	10	16.50	7.20	23.70	1.21	0.54	1.75	3.14	45.45	1.40	1.80	24.35
6		30		10.30	26.80		0.75	1.96	3.57	56.20	1.89	1.82	27.51
7		50		13.50	30.00		0.97	2.18	3.81	60.00	2.40	1.75	27.60
8		70		16.50	33.00		1.23	2.44	3.92	61.75	2.87	1.60	25.88
9	200	10	18.80	7.20	26.00	1.37	0.54	1.91	3.59	56.50	1.43	1.88	29.07
10		30		10.30	29.10		0.75	2.12	3.83	60.40	1.92	1.81	29.38
11		50		13.50	32.30		0.97	2.34	3.97	62.50	2.43	1.70	27.77
12		70		16.50	35.30		1.23	2.60	4.03	63.50	2.90	1.55	25.30
13	400	10	21.60	7.20	28.80	1.60	0.54	2.14	3.84	60.50	1.48	1.80	30.22
14		30		10.30	31.90		0.75	2.35	3.98	62.70	1.97	1.69	28.83
15		50		13.50	35.10		0.97	2.57	4.05	63.80	2.44	1.58	26.26
16		70		16.50	38.10		1.23	2.83	4.10	64.60	2.90	1.45	23.60

It thus is evident that a combination of reservoir and diversion will not represent the optimum development. If channel improvements and/or diking were considered in conjunction with this combination, slightly higher Net Benefits could perhaps be obtained. However, such combinations would not be competetive with the individual reservoir or diversion, because of the large initial capital expenditures required for such a combination. Consequently such combinations did not warrant further consideration.

7-17 Combination - Reservoir and Channel Improvement

In Combinations 17 to 24 inclusive, the previously stated reservoir sizes were analysed in conjunction with channel improvements upstream of Lister Rapids.

a) Reservoir and Reach A and B: Reach A and B were widened to 600 and 700 feet respectively.

The Capital and Annual Costs for these combinations were again obtained by summing the respective costs of the individual projects as illustrated in Section 7-16 (a) and (b)

In order to determine the annual benefits for these combinations it was necessary to obtain a new Rating Curve at Portage. The previously developed computer program was again used to yield the Rating Curve shown in Figure 79. It should be noted, that at impending overtopping of the existing diking system, equivalent to an elevation of 703.50 at Mile 662, the discharge capacity of the Rio Saska under these improved conditions has increased from 80,000 to 90,000 cfs.

Following the methodology established in Section 7-16 (c), a table similar to Table XIX was prepared. The data from this table was then used to prepare the Damage Curve for these combinations. By planimetering the appropriate areas the average annual benefits were found.

Subsequently the Benefit/Cost Ratio and the Net Benefits were calculated using the methodology presented in Section 7-16 (d). The results of the analysis of combinations 17 to 20 are summarized in Table XXI .

b) Reservoir and Reach B: In a similar manner, the reservoir sizes in combination with widening of Reach B to 700 feet but leaving Reach A in its natural condition was considered. The entire analysis is as outlined in Section 7-17 except that in Figure 47, the Rating Curve reflecting the channel improvements in Reach B, was used to yield the Damage Curve shown in Figure 81. The results of these investigations are presented in Table 21 as Combinations 21 to 24 inclusive .

7-18 Combination - Reservoir Channel Improvement and Dike

Combinations 25 to 28 inclusive, represent developments encompassing the previously stated reservoir sizes in conjunction with a widening of Reach B to 700 feet and the low diking system presented in Chapter III.

The analysis of these combinations follows that presented in Section 7-17 (b). It differs, however, since no damage will be done until the Rio Saska water levels are in excess of elevation 705.50 at Mile 662. The resultant Damage Curve is Figure 82 and these combinations are summarized in Table XXI .

7-19 Channel Improvement and Dike

a) Reach B and Low Dikes: Even though it was realized that with engineering constraints imposed on the level of channel improvement and diking as outlined in Chapters III and VI, the maximum Net Benefits could not be achieved, it was felt that this combination would likely yield considerable protection at moderate costs. The analysis followed the methodology developed previously and utilized the Damage Curve in Figure 83. Combination 29 is summarized in Table XXI .

TABLE XXI

COMBINATION

ECONOMIC SUMMARY

Combination	SIZE: Res.	SIZE: Chann. Reach A=600 Reach B=700	CAPITAL COST ($ Million): Res.	CAPITAL COST: Chann.	CAPITAL COST: Total	ANNUAL COST ($ Million): Res.	ANNUAL COST: Chann.	ANNUAL COST: Total	Annual Benefit ($ Million)	P.V. of Benefit	P.V. of O & M	Benefit Cost Ratio	Net Benefit $Million
17	50	"	14.50	2.10	16.60	1.05	0.16	1.21	2.73	43.00	0.56	2.25	25.84
18	100	"	16.50		18.60	1.21		1.37	3.21	50.60	0.60	2.35	31.40
19	200	"	18.80		20.90	1.37		1.53	3.65	57.50	0.64	2.39	35.96
20	400	"	21.60		23.70	1.60		1.76	3.92	61.80	0.68	2.23	37.42
	Res.	Reach B	Res.	Reach B		Res.	Reach B						
21	50	700	14.50	1.36	15.86	1.05	0.10	1.15	2.70	42.60	0.44	2.35	26.30
22	100		16.50		17.86	1.21		1.31	3.15	49.70	0.47	2.40	31.37
23	200		18.80		20.16	1.37		1.47	3.59	56.60	0.51	2.44	35.95
24	400		21.60		22.96	1.60		1.70	3.87	61.00	0.57	2.28	37.47
	Res.	Reach B L. Dike	Res.	Reach B L. Dike		Res.	Reach B L. Dike						
25	50	700	14.50	4.45	18.95	1.05	0.35	1.40	3.04	47.95	0.93	2.17	28.07
26	100	705.50	16.50		20.95	1.21		1.56	3.43	54.00	0.96	2.20	32.09
27	200		18.80		23.25	1.37		1.72	3.76	59.20	1.00	2.19	34.95
28	400		21.60			1.60		1.95	3.97	62.55	1.04	2.04	35.46
	Reach B	L. Dike	Reach B	L. Dike		Reach B	L. Dike						
29	700	705.50	1.36	3.09	4.45	0.10	0.23	0.33	1.90	29.90	0.70	5.76	24.75
	Reach A & B	L. Dike	Reach A & B										
30	700 & 600	705.50	2.14	3.09	5.23	0.16	0.23	0.39	1.91	30.10	0.83	4.99	24.04

b) Reach A and B and Low Dike: In a similar manner, a project consisting of Reaches A and B and the Low Diking system was analyzed. Reach A and B were widened to 600 and 700 feet respectively. The Damage Curve for combination 30 is shown in Figure 84 and the results are also summarized in Table XXI.

7-20 Combination - Diversion and Channel Improvements

Diversion capacities of 10, 30, 50 and 70 thousand cfs in conjunction with widening of Reach B to 700 feet were considered.

The basic approach to the analysis of these combinations was similar to that presented in Section 7-17 involving the reservoir and channel improvements. Only the graphs used differed.

Using the Rating Curve in Figure 47, the damages were tabulated for floods having various frequencies. These results were then used to plot the Damage Curve shown in Figure 85 from which the average annual benefits of these combinations were determined. The results of the analysis can be found in Table XXII under Combinations 31 to 34 inclusive.

7-21 Combination - Diversion and Dike:

Combinations 35 to 38 involve the diversion capacities outlined in Section 7-20 and the Low Dike alternative.

This dike affords complete protection until the river stage exceeds elevation 705.50, and corresponds to a discharge of 105,000 cfs as was determined from Figure 26. To this flow the diversion capacity was added to yield the largest flood flow causing no damage. Larger floods were examined and estimates of the resultant damages were made using the Stage Damage Curve, Figure 17. This data was then used to draw the Damage Curve, Figure 86, from which the average annual benefits were determined by planimetering.

The remainder of the economic analysis was proceeded with as previously outlined, the results of which are presented in Table XXII.

TABLE XXII

COMBINATIONS

ECONOMIC SUMMARY

Combination	SIZE Div. 1000 cfs	SIZE Chann. Imp. Reach B	CAPITAL COST Div.	CAPITAL COST Chann.	CAPITAL COST Total	ANNUAL COST Div.	ANNUAL COST Chann.	ANNUAL COST Total	Annual Benefits	P.V. of Benefit	P.V. of O & M	Benefit Cost Ratio	Net Benefit
			$ Million			$ Million			$ Million				$ Million
31	10	700	7.20	1.36	8.56	0.54	0.10	0.64	1.95	30.70	1.35	3.05	20.79
32	30		10.30		11.66	0.75		0.85	2.95	46.50	1.83	3.47	33.01
33	50		13.50		14.86	0.97		1.07	3.54	55.80	2.34	3.31	38.60
34	70		16.50		17.86	1.23		1.33	3.83	60.40	2.81	2.88	39.73
	Div.	Low Dike	Div.	Low Dike		Div.	Low Dike						
35	10	705.50	7.20	3.09	10.29	0.54	0.23	0.77	1.82	28.70	1.62	2.37	16.79
36	30		10.30		13.39	0.75		0.98	2.87	45.20	2.11	2.93	29.70
37	50		13.50		16.59	0.97		1.20	3.46	54.50	2.61	2.88	35.30
38	70		16.50		19.59	1.23		1.46	3.72	58.60	3.09	2.55	35.92
	Div.	Reach B L. Dike	Div.	Reach B L. Dike		Div.	Reach B L. Dike						
39		700	7.20	4.45	11.65	0.54	0.33	0.87	2.52	39.70	1.84	2.90	26.21
40		705.50	10.30		14.75	0.75		1.08	3.30	52.00	2.32	3.05	34.93
41			13.50		17.95	0.97		1.30	3.68	58.00	2.83	2.83	37.22
42			16.50		20.95	1.23		1.46	3.98	62.00	3.30	2.69	37.75

7-22 Combination - Diversion, Channel Improvement and Dike

The previously indicated diversion capacities were combined with widening Reach B to 700 feet and the Low Dike adjacent to the channel affording protection up to elevation 705.50.

Using the approach previously employed, the Damage Curve shown on Figure 87 was drawn. The summary of the economic analysis for combinations 39 to 42 are shown in Table XXII .

7-23 Other Possible Combinations

While quite a number of other combinations are possible, it is felt that the 42 combinations investigated indicate the results that may be expected. It is felt that introducing such projects as excavation in Lister Rapids into the combinations would not increase the Net Benefits, since these projects in themselves are marginal propositions. A similar observation may be made regarding the reservoir, the diversion, the low dike and widening of Reach B to 700 feet. However, all of these projects were highly beneficial when considered individually. Consequently, it is reasonable to expect that a combination involving all or a few of these four projects would comprise the optimum development

The annual costs and benefits associated with the 42 combinations have been summarized in graphical form in Figure 88 .

CHAPTER VIII

CONCLUSIONS AND RECOMMENDATIONS

8-1 General Considerations

This study of possible flood control measures for the city of Portage has established several individual projects and combinations thereof can provide flood protection of various degrees at reasonable costs.

8-2 Limitations of the Present Study

a) Cost Estimates: The estimates of the components for the various projects are based on preliminary designs accurate enough to obtain a reasonable cost estimate.

b) Unit Prices: The unit prices used in estimating were obtained from the appropriate graphs presented in Appendix . It is felt that these graphs reflect current material and construction costs in the Province of Assiniboia.

c) Approximations:

(i) The Rating Curves at Portage were developed using an approximation of Bresse's Equation as proposed by Bossen. The cross-section of the Rio Saska for the three reaches outlined were idealized.

(ii) In the design of the diversion simplifying assumptions were made in sizing the control structures association with the design. The drop structures required, for example, were arbitrarily narrowed to two-thirds the width of the channel. Similarly, no designs were prepared for the required railway and highway crossings.

(iii) In the design of Tobin Reservoir, an approximate routing procedure was employed. Because of the use of this method, reduction of flood flows by the reservoir is somewhat larger than could be expected.

(iv) When the individual channel improvements were considered, it became evident that a major portion of costs for these projects were associated with the construction of appropriate cofferdams. Little design work was done to evolve suitable cofferdams.

8-3 Conclusions

From the study conducted, it may be concluded that the order of preference based on Maximum Net Benefits would be:

TABLE XXIII

SUMMARY OF MOST ECONOMIC PROJECTS

Order of Preference	Project	Net Benefits Dollars/Millions
1.	Combination 34	39.73
2.	Combination 33	38.60
3.	Diversion of 70,000 cfs	38.12
4.	Tobin Reservoir of 430,000 ac-ft	37.90
5.	Combination 42	37.75
6.	Combination 24	37.47
7.	Combination 38	35.92
8.	Combination 28	35.46

If it is desirable to limit the field of possible projects further, only the first four projects as listed in Table XXIII need by considered. Even if projects 5 to 8 are not eliminated, a future study program should commence with an analysis of the projects that appear to be most promising as determined in this study.

8-4 Recommendations of Projects for Further Study

Based on the analyses presented on this study, several projects merit further study.

a) Investigate a Reservoir at Tobin Rapids having a storage capacity of approximately 450,000 acre-feet.

The structures associated with this reservoir, such as the spillway and the conduit, should be designed in detail. The flood flows should be routed through the reservoir by a more accurate method to determine the actual inflow-outflow relationships.

b) Investigate a project similar to Combination 27 as outlined in Table XXI. This involves a reservoir at Tobin Rapids having a storage capacity of approximately 400,000 acre-feet in conjunction with widening of Reach B to about 700 feet. A thorough study should be undertaken to determine the practicability of widening Reach B to this extent. Since the potential benefits are considerable, the possibility of widening Reach B beyond the 700 foot limit previously set should be thoroughly examined. This may require model studies, perhaps incorporating river training works such as riprap bank protection to minimize erosion.

c) Investigate a project similar to Combination 28 as outlined in Table XXI. This involves a reservoir of about 400,000 acre-feet, widening of Reach B to about 700 feet and the construction of a low dike about 5 feet high adjacent to the Rio Saska within the city of Portage. In this proposal a thorough study should again be made of Reach B and the possibility of raising the low dike to a height of 6 or 7 feet should be studied. Significant benefits could accrue if this were to be possible.

d) Investigate a storage reservoir at Tobin Rapids of about 400,000 acre-feet in conjunction with a low dike adjacent to the river in Portage and about 5 feet in height.

e) Investigate a diversion at Site B having a design capacity of about 70,000 cfs.

All structures associated with this diversion should be designed in detail to obtain more precise cost estimates. This design may involve the model studies of control structure on the Rio Saska, the inlet weir to the diversion and the drop structures along the diversion channel and the diversion outlet structure at Lake Manitou. A review should also be made of the proposed diversion section, particularly the side slopes and maximum permissible channel slope and maximum velocity.

The sites of the railway crossings should also be surveyed and the crossings designed in detail to ascertain that realistic costs are used on the cost estimate. While it is thought that a maximum water level of 727 is suitable,optimum level should be found in the manner indicated in Section 4-6.

f) Investigate a project similar to Combinations 33 and 34. These involve a diversion design capacity of 50,000 and 70,000 cfs in conjunction with widening Reach B to 700 feet.

g) Investigate a project similar to Combination 38, which involves a diversion of about 70,000 cfs with a dike about 5 feet in height adjacent to the Rio Saska channel within the city of Portage.

h) Investigate a project similar to combination 42 involving a diversion having a capacity of approximately 70,000 cfs, widening of Reach B to 700 feet and the construction of a low dike of about 5 feet in height adjacent to the river within the city of Portage.

8-5 Socio-Economic Considerations

This preliminary study has dealt little with the socio-economic aspects of the proposed flood control measures, apart from recognizing a few obvious problems that may arise. It was primarily because of social considerations that the High Dike alternatives as proposed in Chapter II were rejected.

It is recommended that the socio-economic implications of these flood control measures be assessed before any decision is made as to which scheme is to be implemented.

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LIST OF REFERENCES

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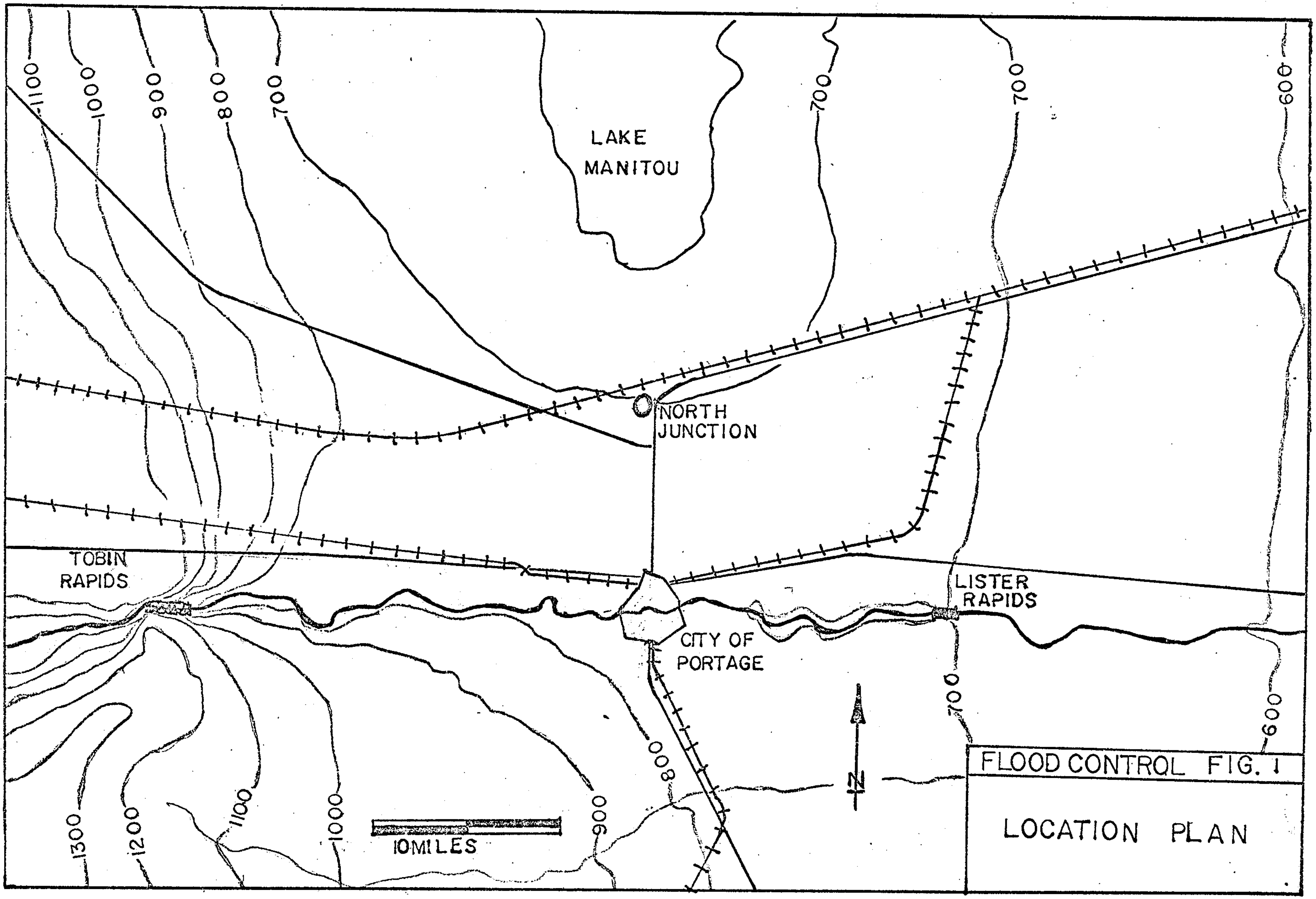
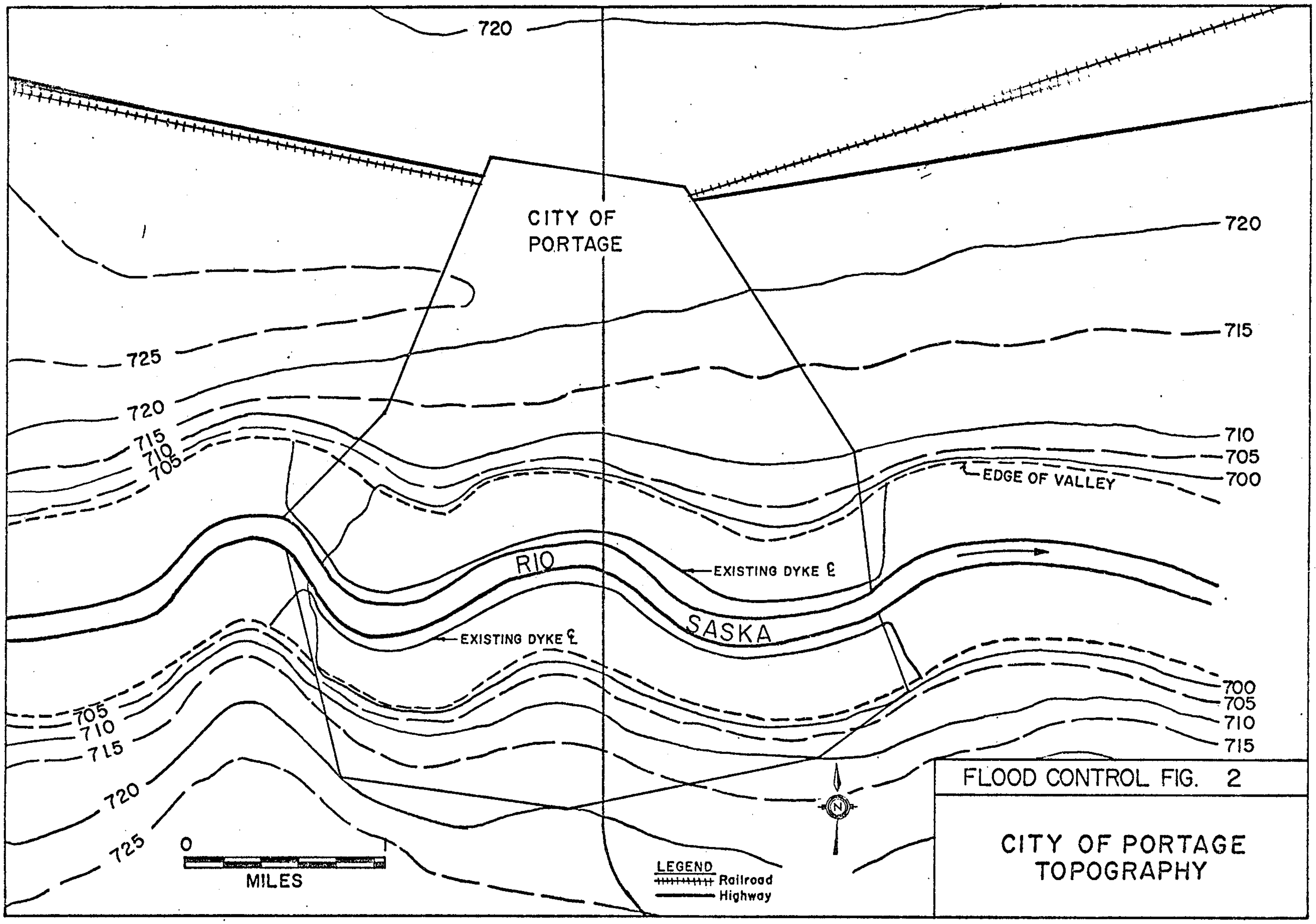
FIGURES



FLOOD CONTROL FIG. 1

LOCATION PLAN



FLOOD CONTROL FIG. 2

CITY OF PORTAGE TOPOGRAPHY

CITY OF PORTAGE
DIKE PROPOSAL

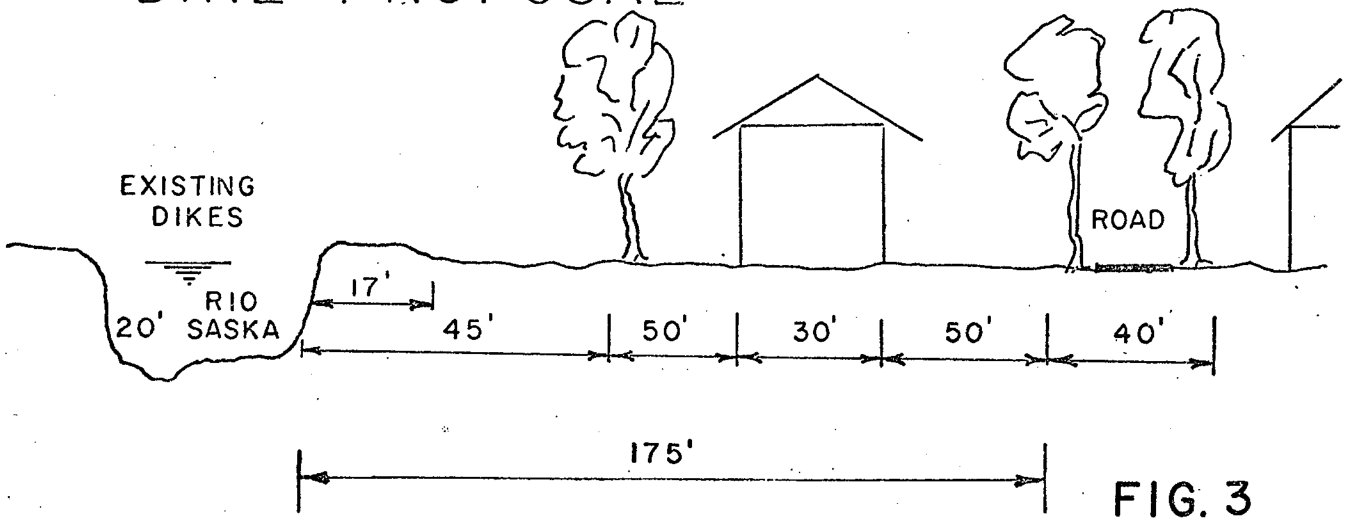


FIG. 3

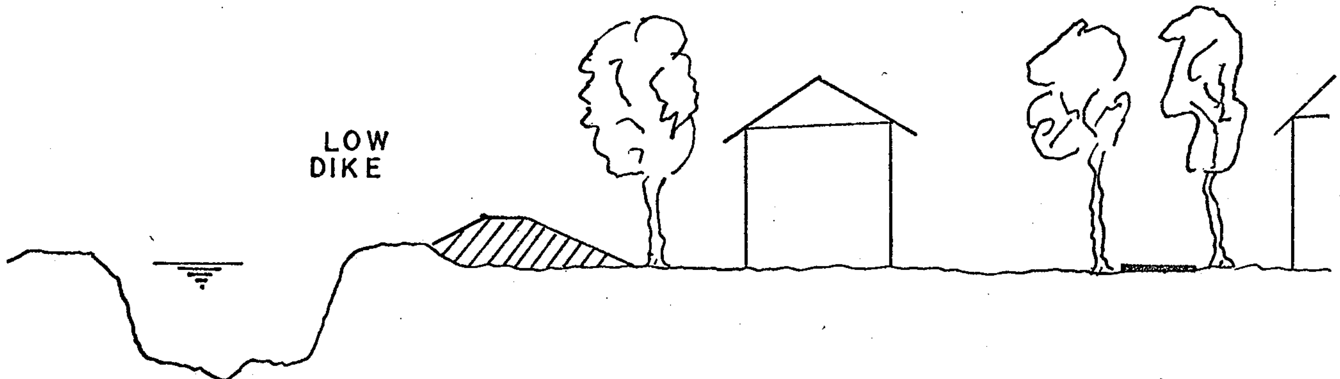
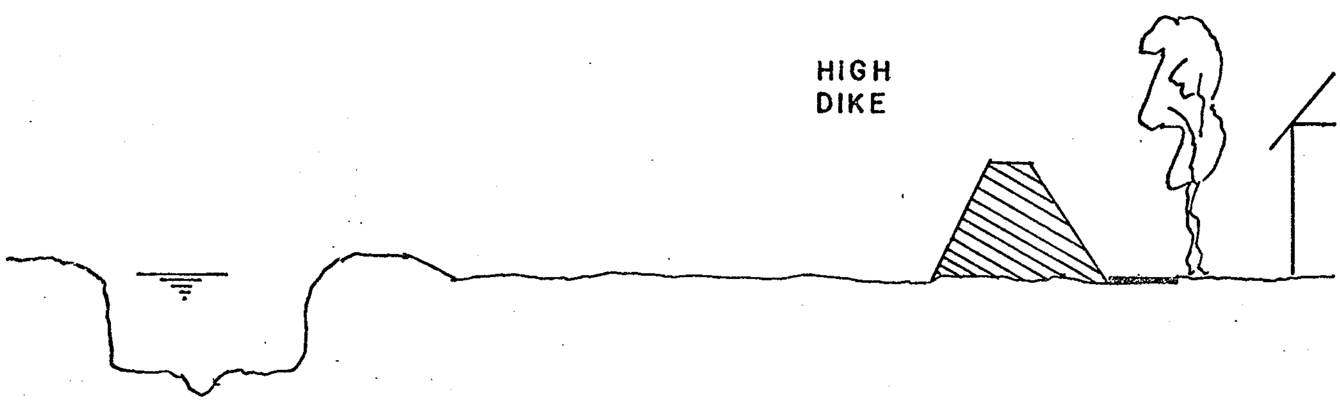
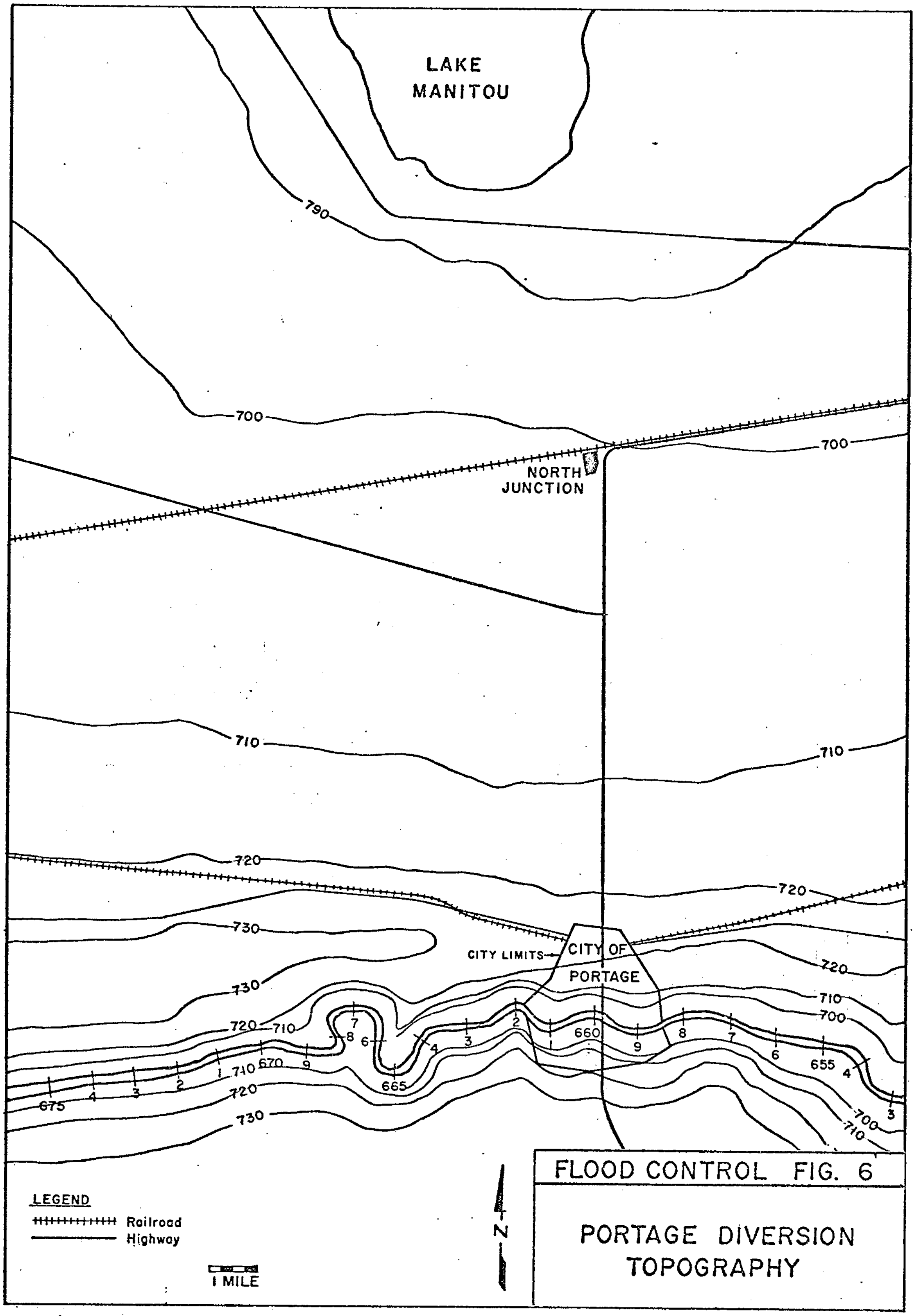


FIG. 4



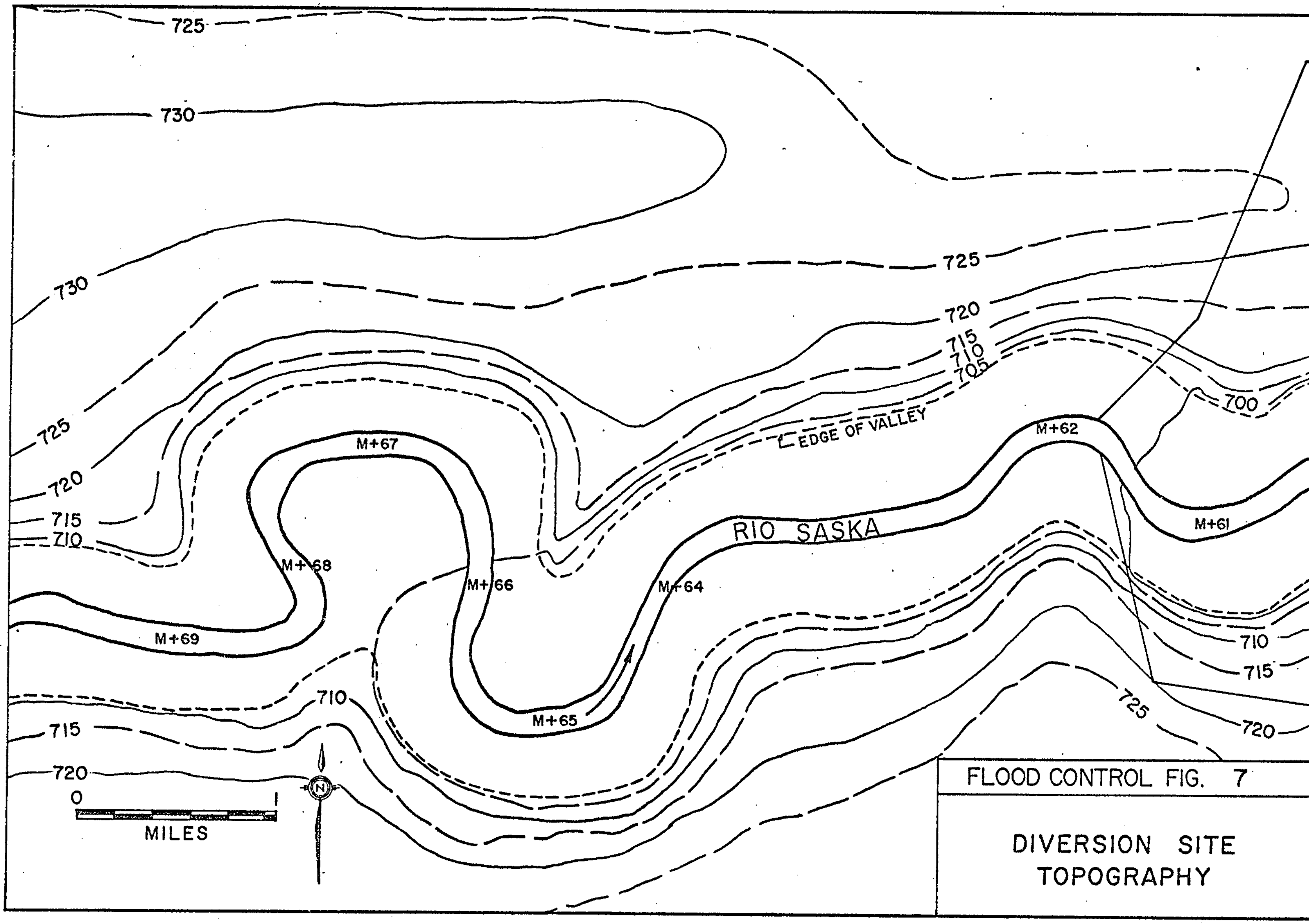
NOTE : SYMETRICAL ABOUT RIO SASKA

FIG. 5



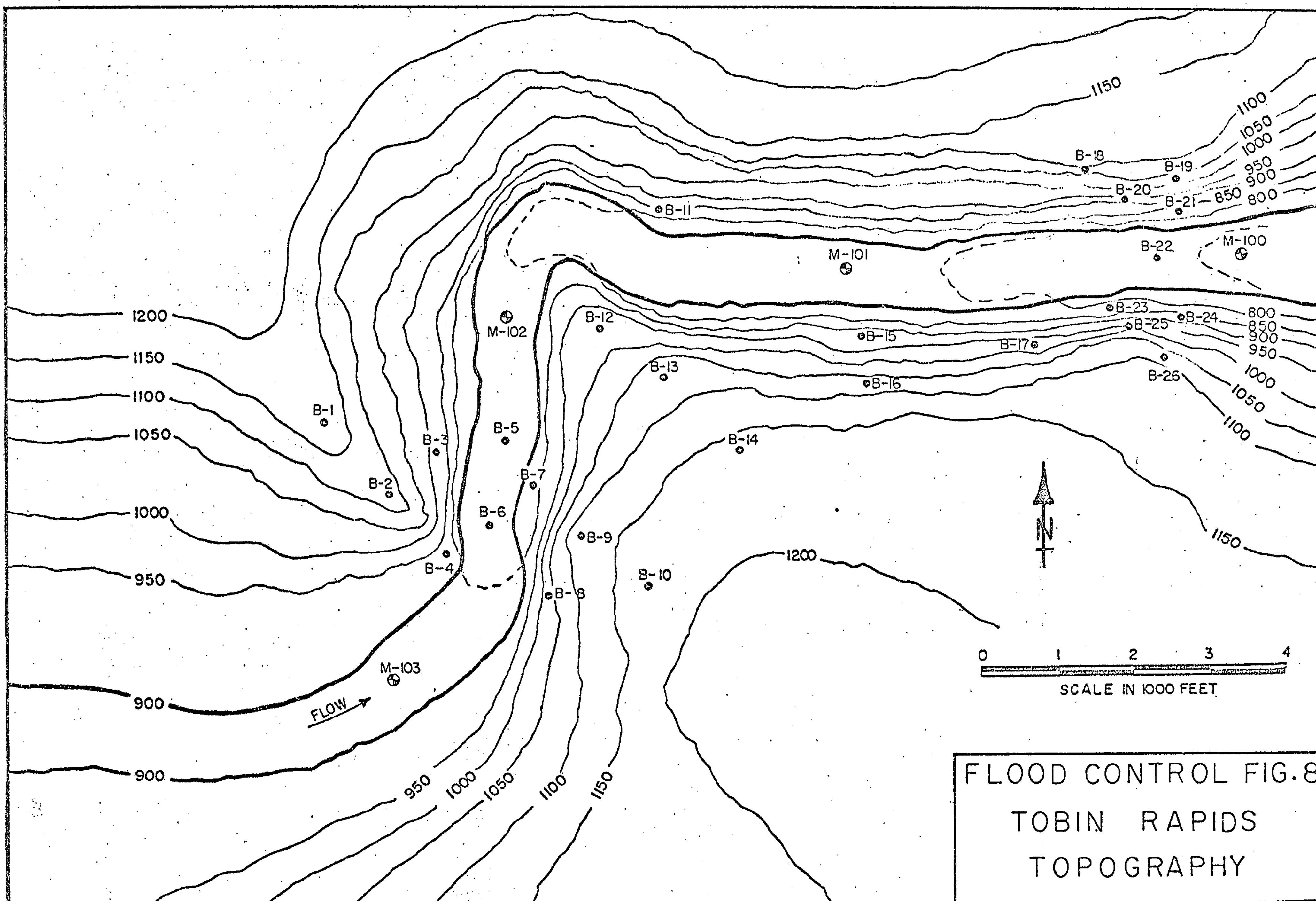
FLOOD CONTROL FIG. 6

PORTAGE DIVERSION TOPOGRAPHY

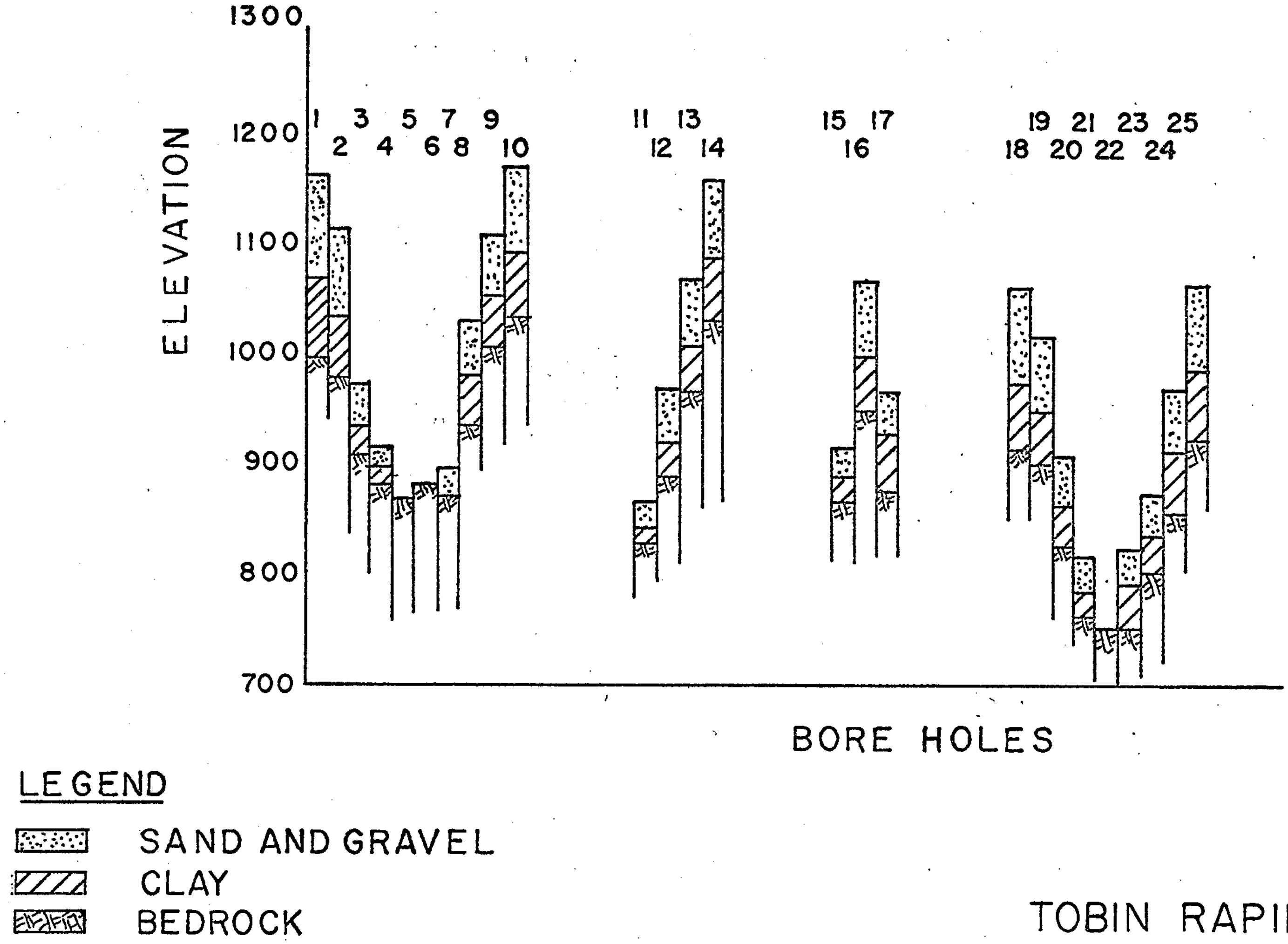


FLOOD CONTROL FIG. 7

DIVERSION SITE TOPOGRAPHY

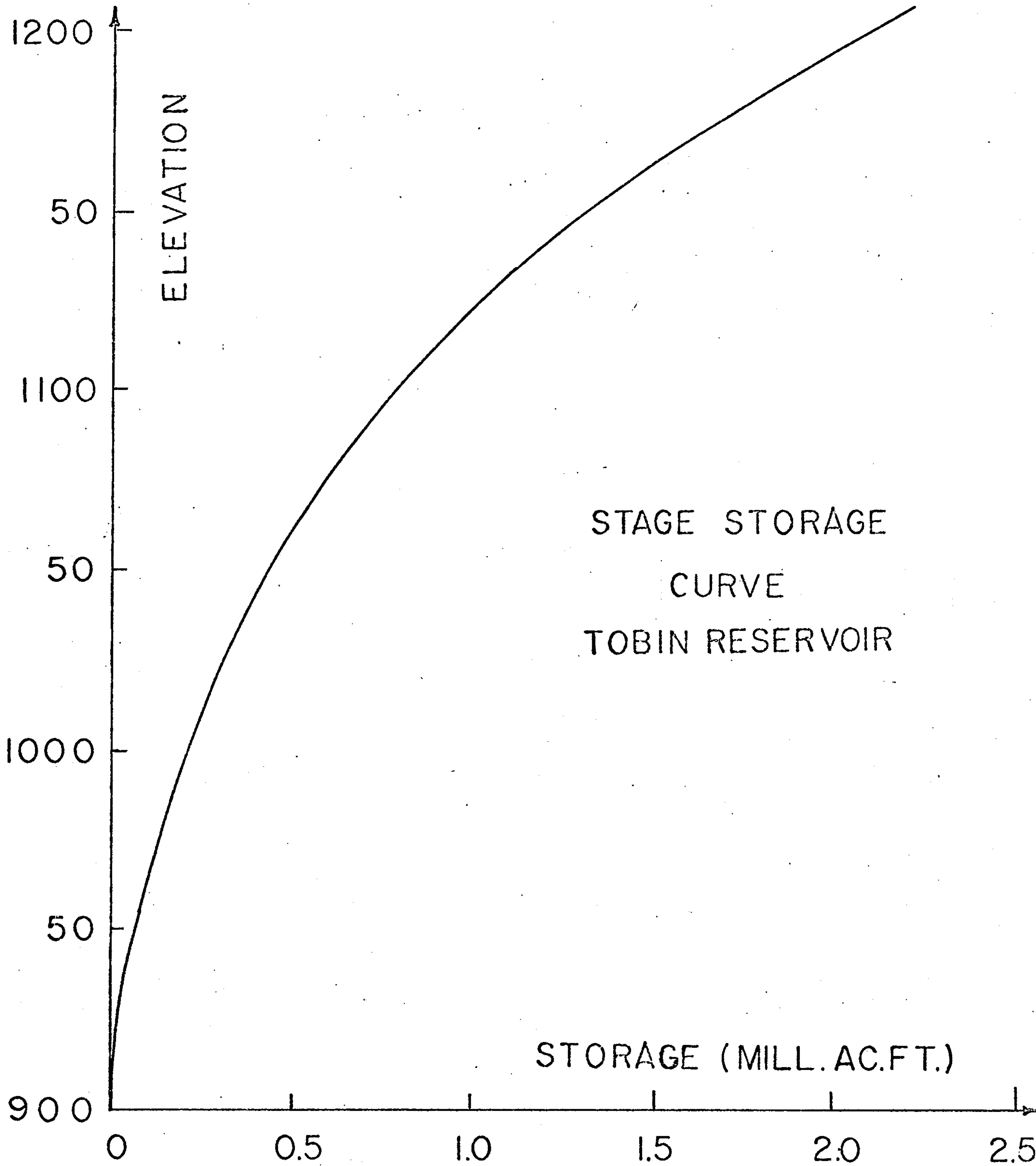


FLOOD CONTROL FIG. 8
TOBIN RAPIDS
TOPOGRAPHY



TOBIN RAPIDS
SOILS LOG

FIG. 9



1200
50
1100
50
1000
50
900
ELEVATION
0
0.5
1.0
1.5
2.0
2.5
STORAGE (MILL. AC.FT.)
STAGE STORAGE
CURVE
TOBIN RESERVOIR

FIG. 10

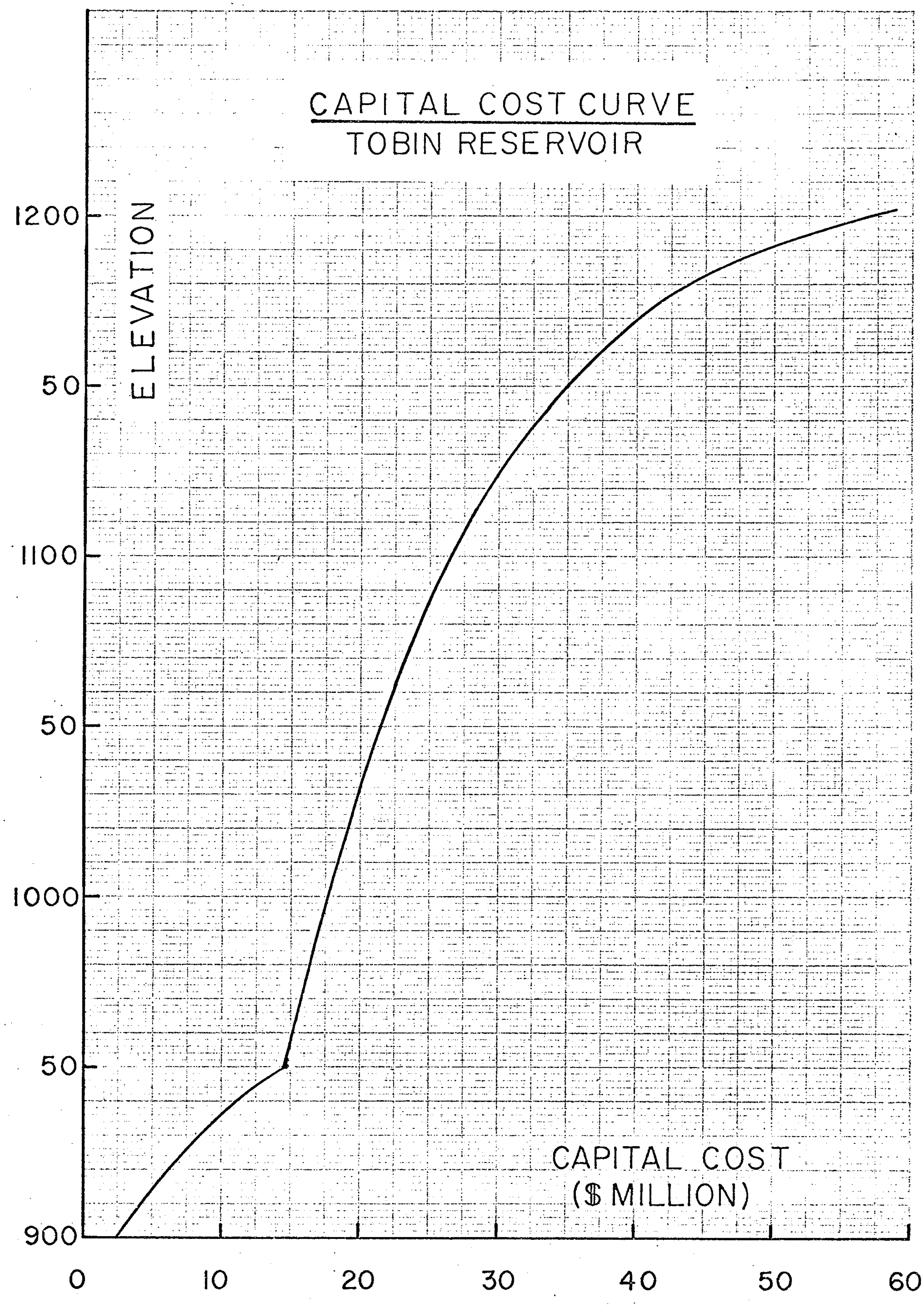
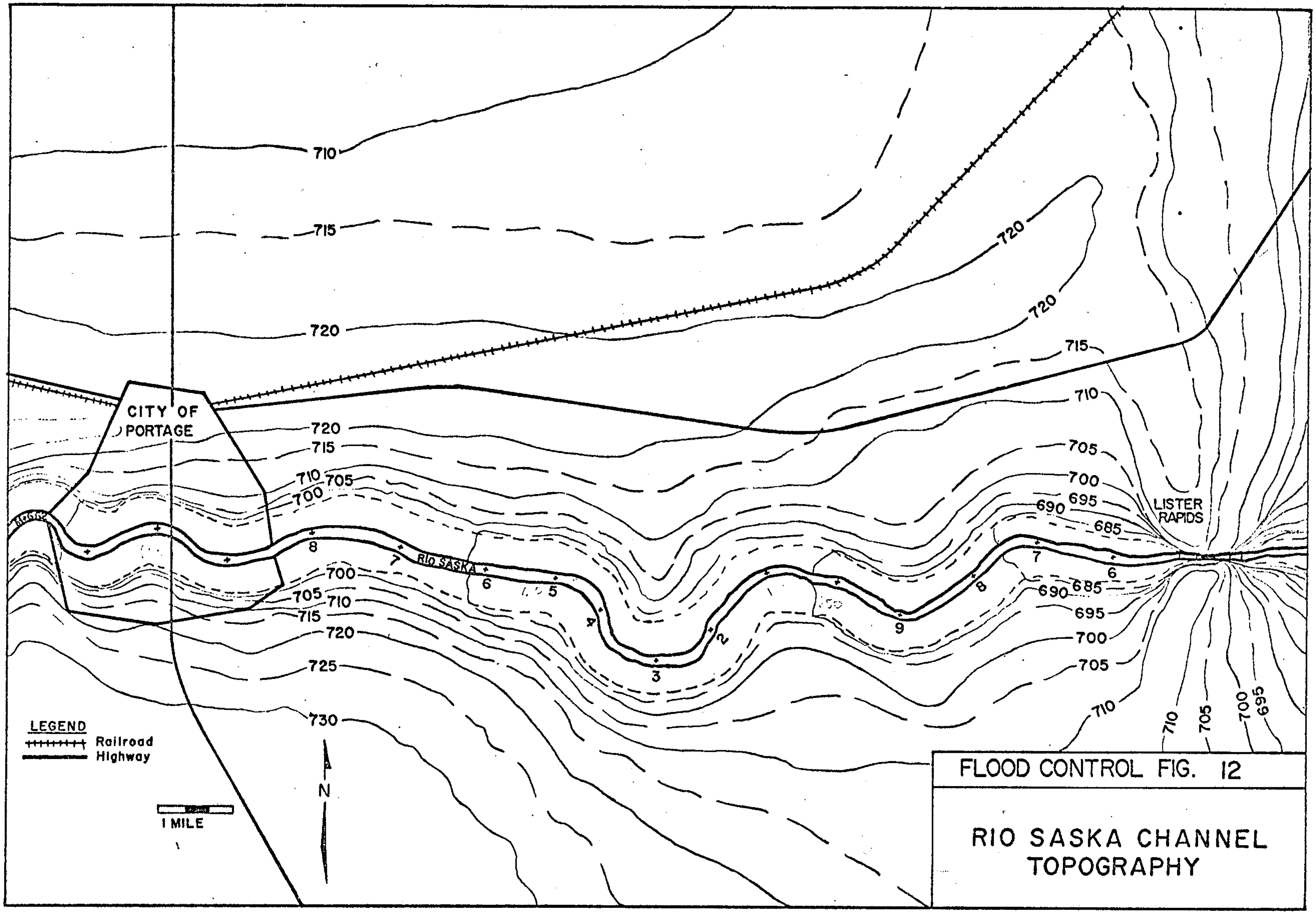
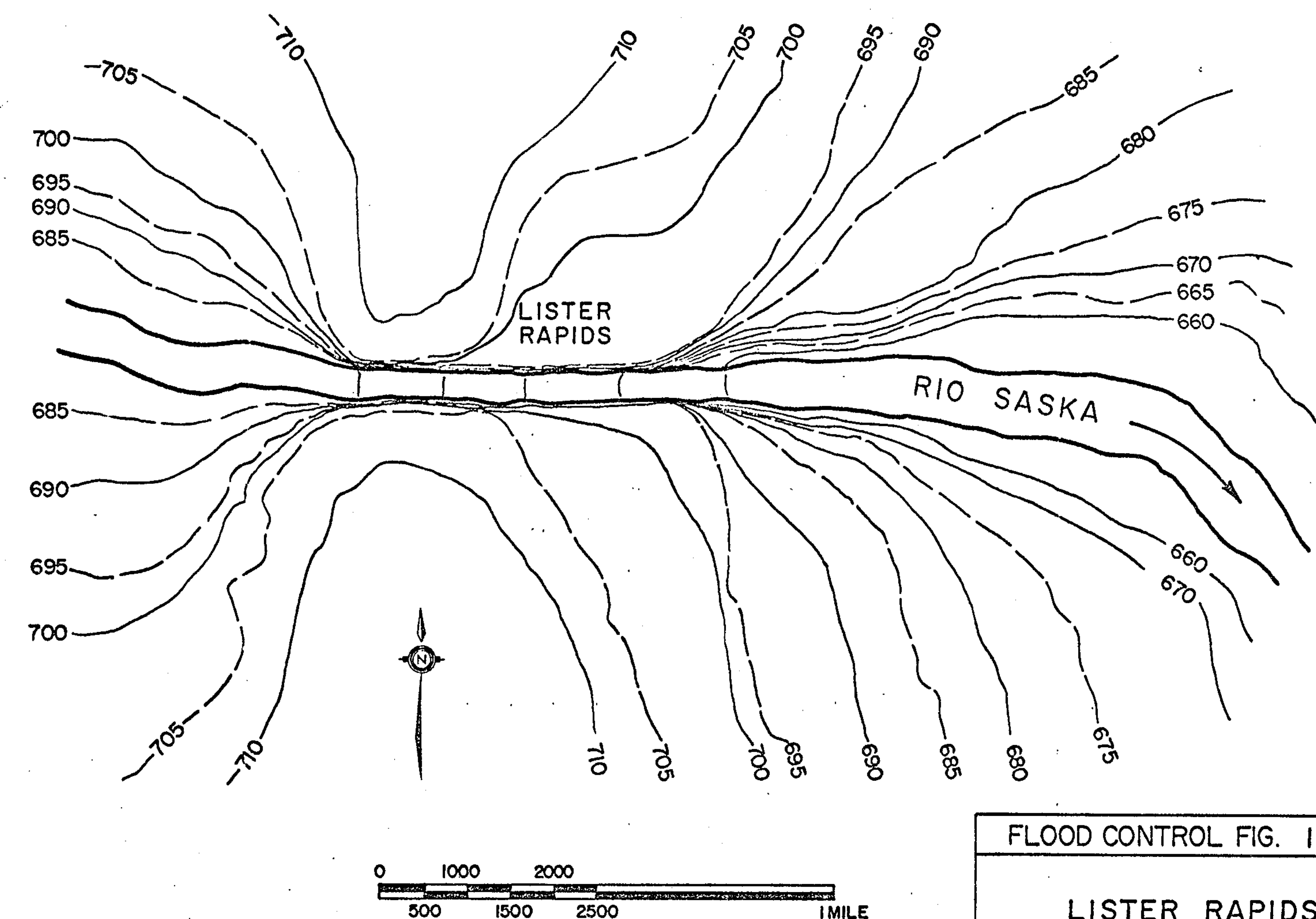

CAPITAL COST CURVE
TOBIN RESERVOIR
ELEVATION
1200
50
1100
50
1000
50
900
CAPITAL COST
($ MILLION)
0
10
20
30
40
50
60

FIG. 11



FLOOD CONTROL FIG. 12

RIO SASKA CHANNEL TOPOGRAPHY



FLOOD CONTROL FIG. 13

LISTER RAPIDS
TOPOGRAPHY

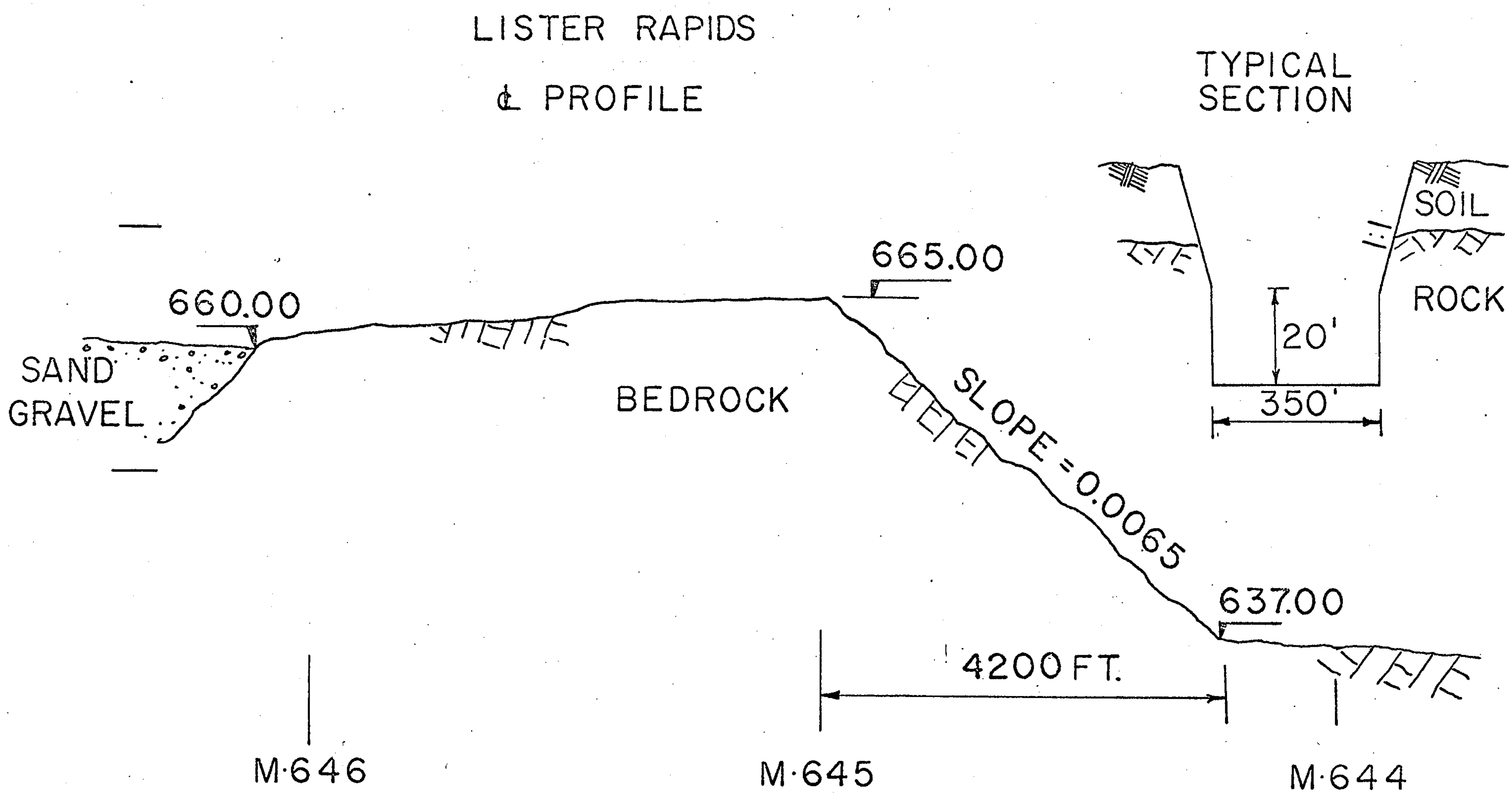

LISTER RAPIDS
℄ PROFILE
TYPICAL
SECTION
SOIL
ROCK
20'
350'
665.00
660.00
SAND
GRAVEL
BEDROCK
SLOPE = 0.0065
637.00
4200 FT.
M·646
M·645
M·644

FIG. 14

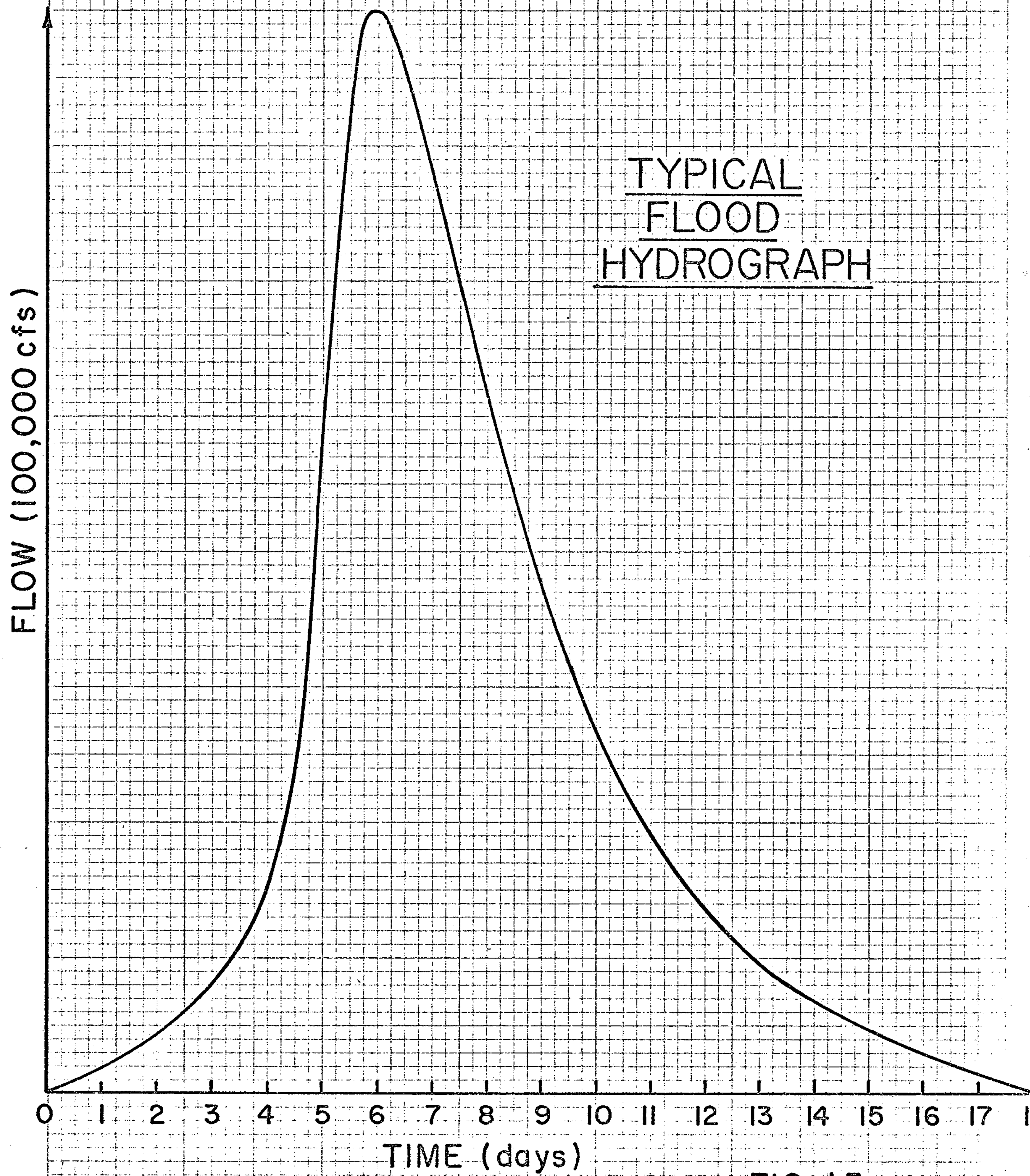

TYPICAL
FLOOD
HYDROGRAPH
FLOW (100,000 cfs)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
TIME (days)

FIG. 15

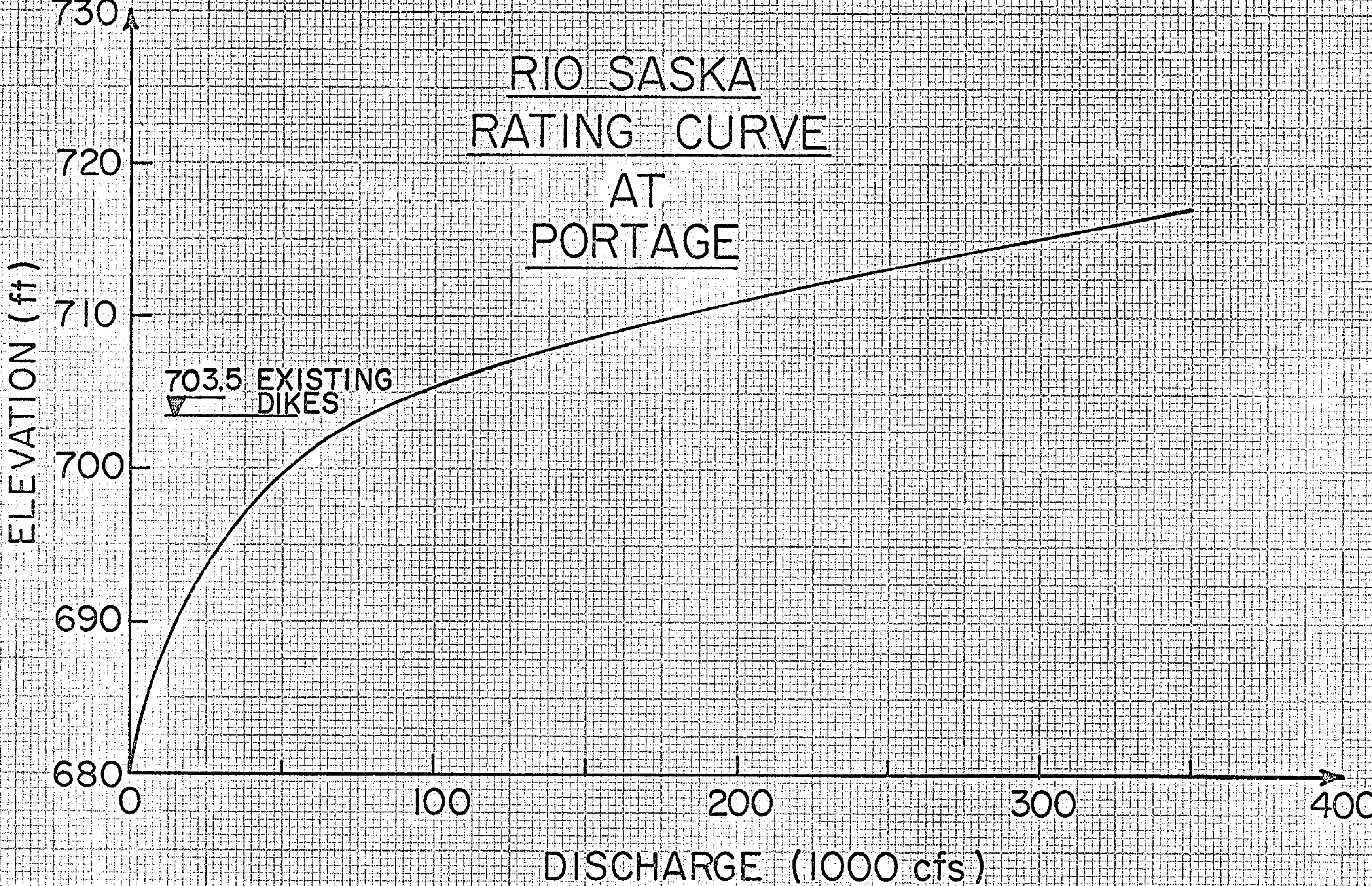

RIO SASKA
RATING CURVE
AT
PORTAGE
ELEVATION (ft)
730
720
710
700
690
680
703.5 EXISTING
DIKES
0
100
200
300
400
DISCHARGE (1000 cfs)

FIG. 16

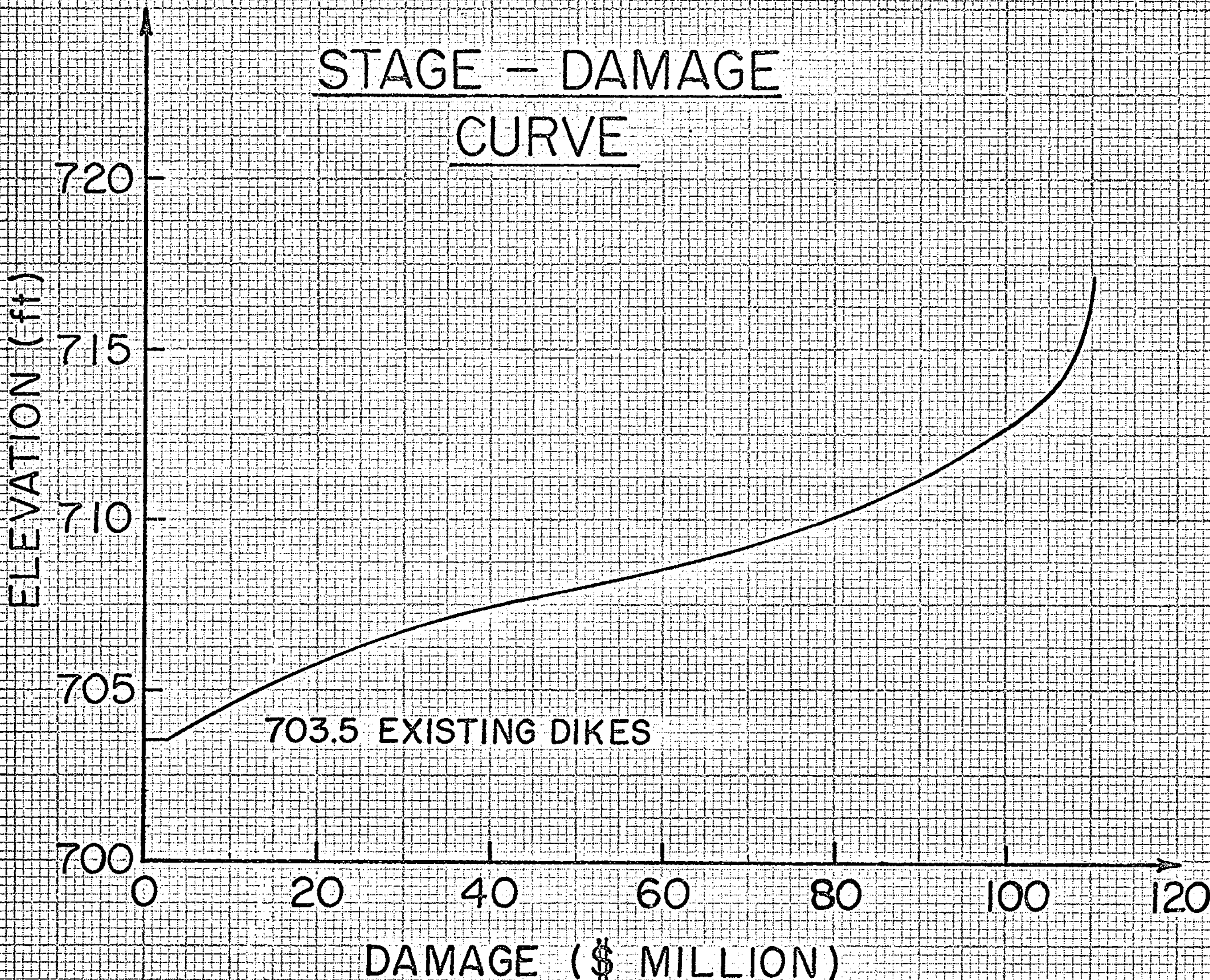

STAGE - DAMAGE
CURVE
ELEVATION (ft)
720
715
710
705
700
703.5 EXISTING DIKES
0
20
40
60
80
100
120
DAMAGE ($ MILLION)

FIG. 17

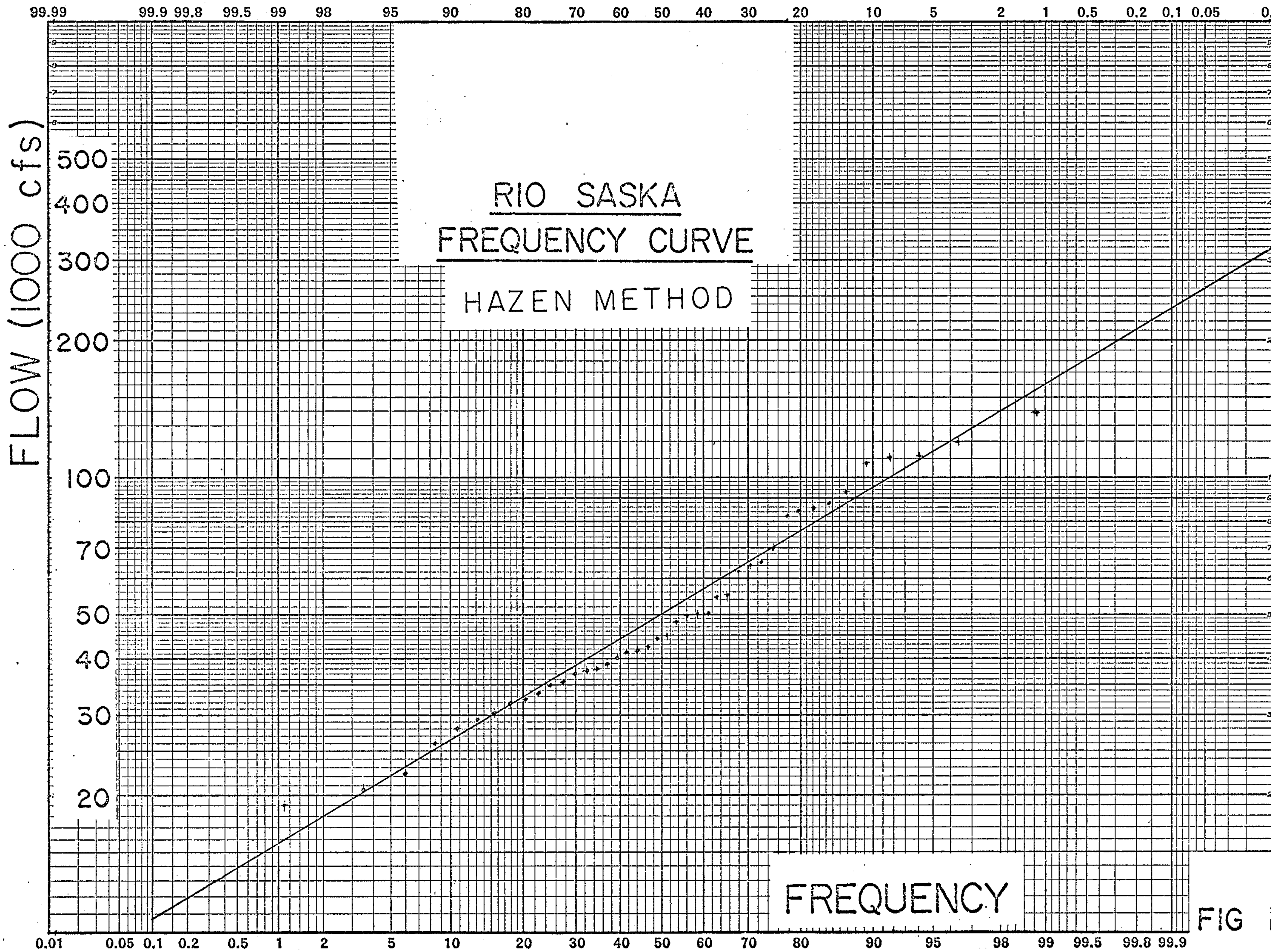

RIO SASKA
FREQUENCY CURVE
HAZEN METHOD
FLOW (1000 cfs)
FREQUENCY
99.99 99.9 99.8 99.5 99 98 95 90 80 70 60 50 40 30 20 10 5 2 1 0.5 0.2 0.1 0.05 0.01
500 400 300 200 100 70 50 40 30 20
0.01 0.05 0.1 0.2 0.5 1 2 5 10 20 30 40 50 60 70 80 90 95 98 99 99.5 99.8 99.9

FIG 18

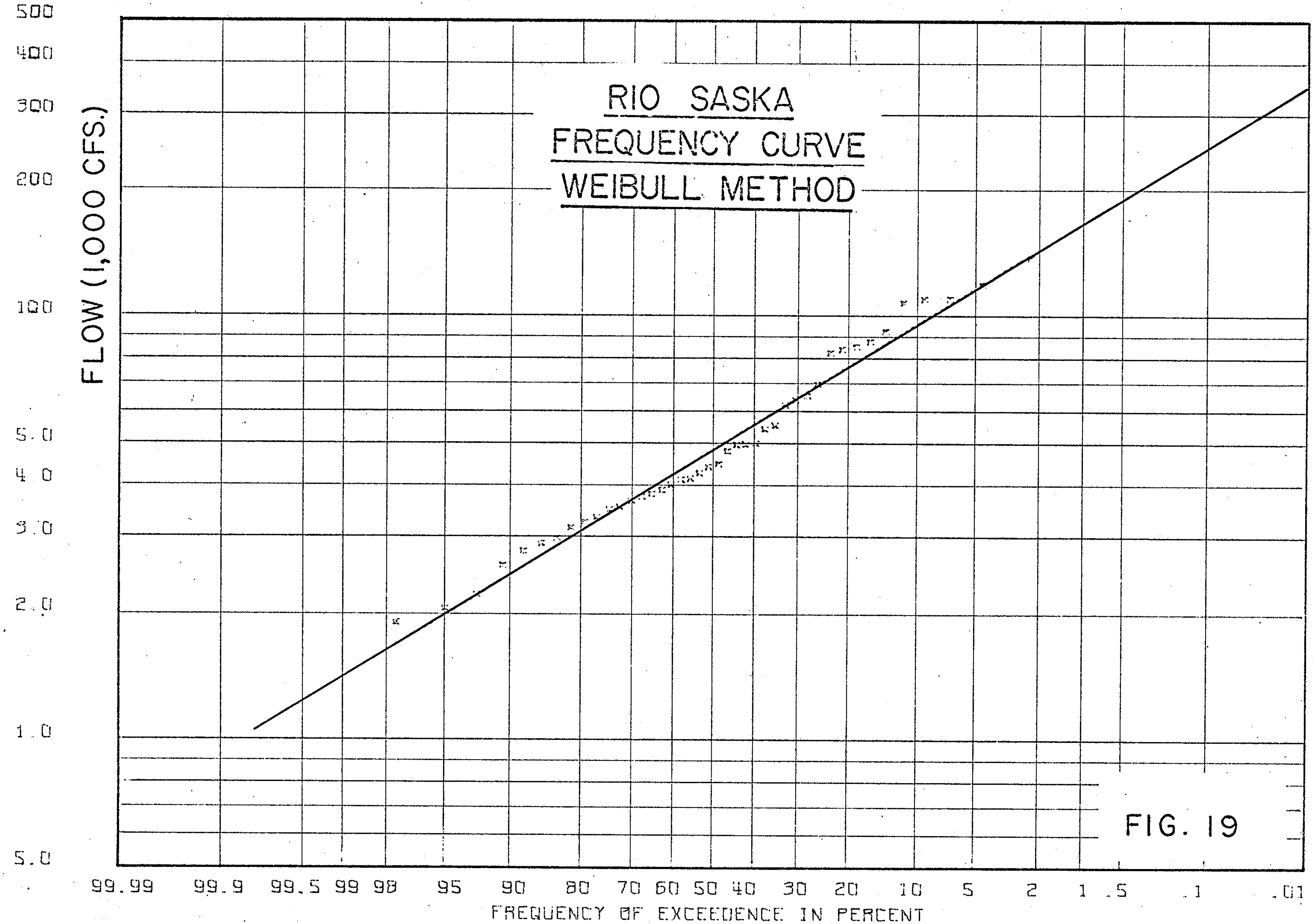

RIO SASKA
FREQUENCY CURVE
WEIBULL METHOD
FLOW (1,000 CFS.)
500
400
300
200
100
5.0
4.0
3.0
2.0
1.0
5.0
99.99
99.9
99.5
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
.5
.1
.01
FREQUENCY OF EXCEEDENCE IN PERCENT

FIG. 19

RIO SASKA
℄ PROFILE

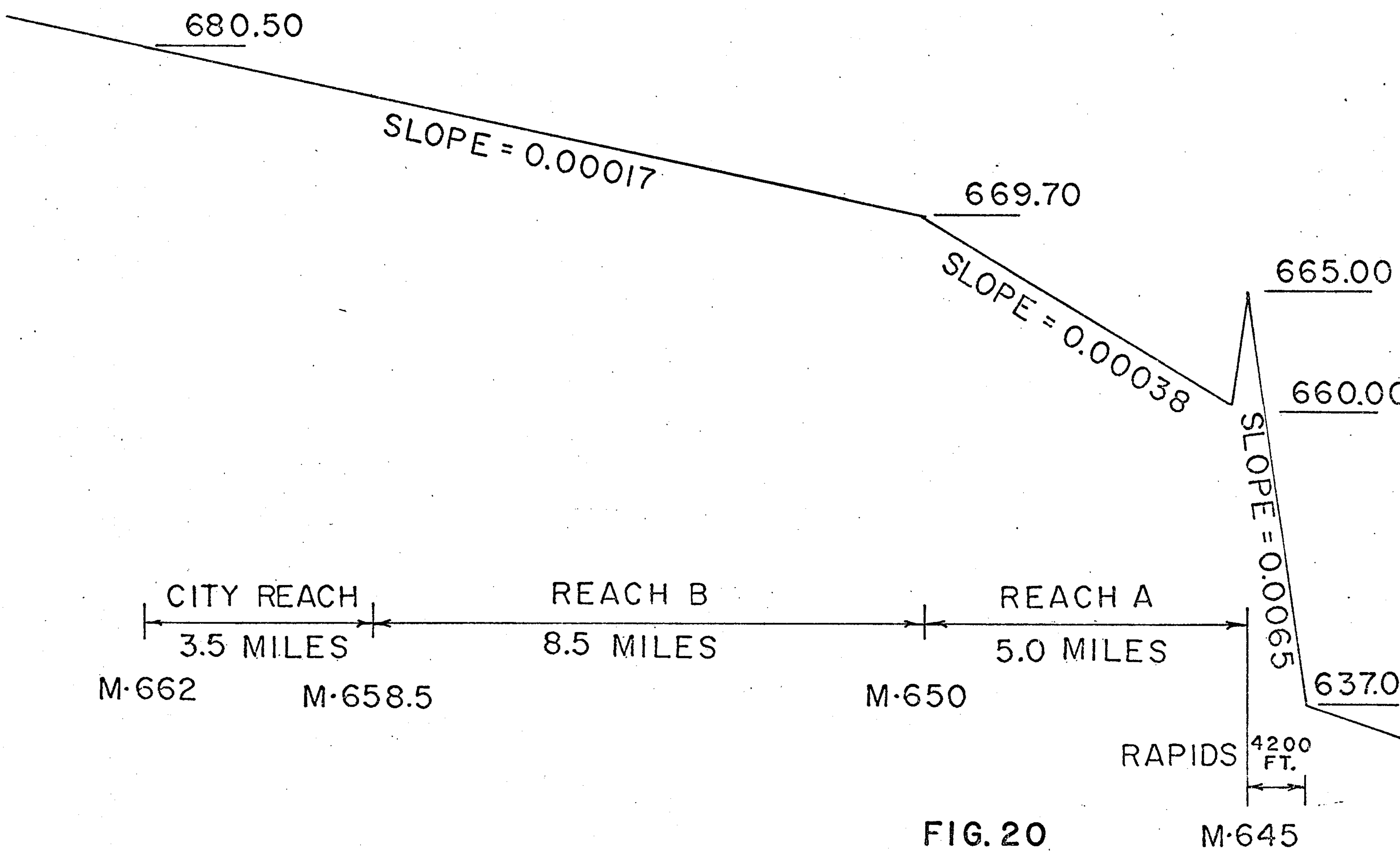
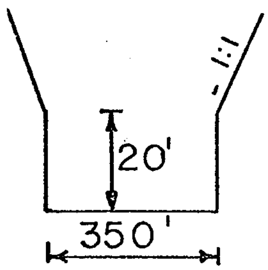
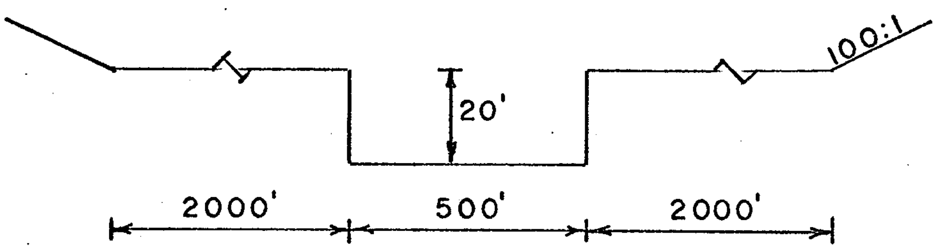


FIG. 20

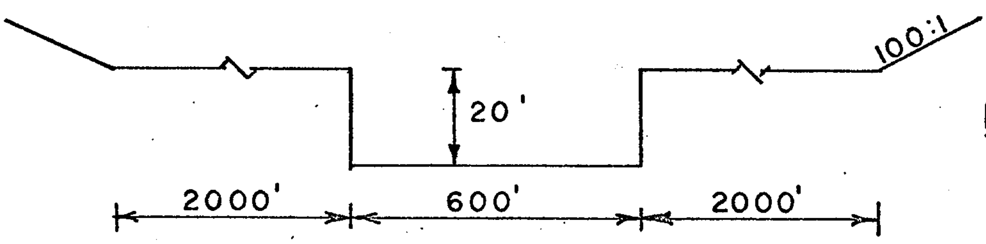
RIO SASKA
TYPICAL CROSS-SECTIONS



LISTER RAPIDS



REACH A



REACH B&C

FIG. 21

NORMAL DEPTH CURVE
REACH A

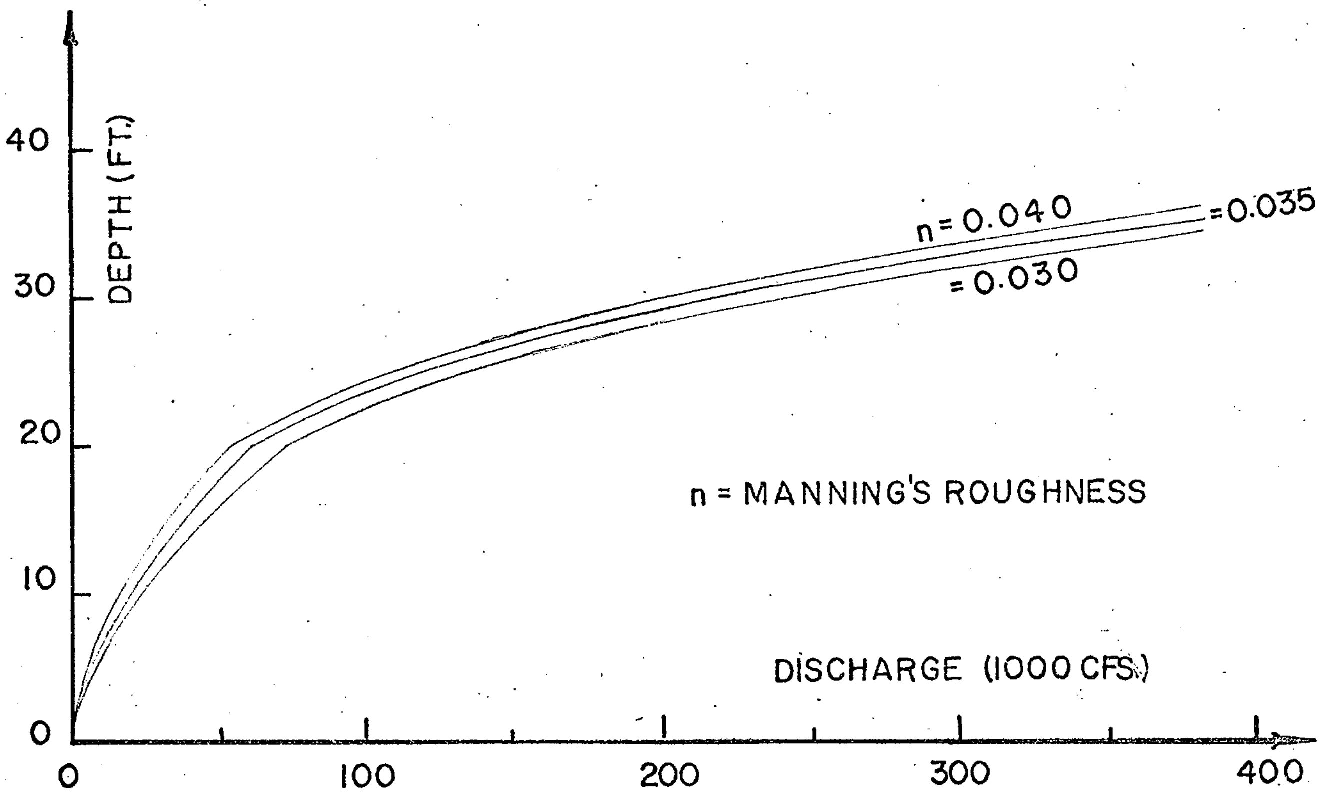


FIG. 22

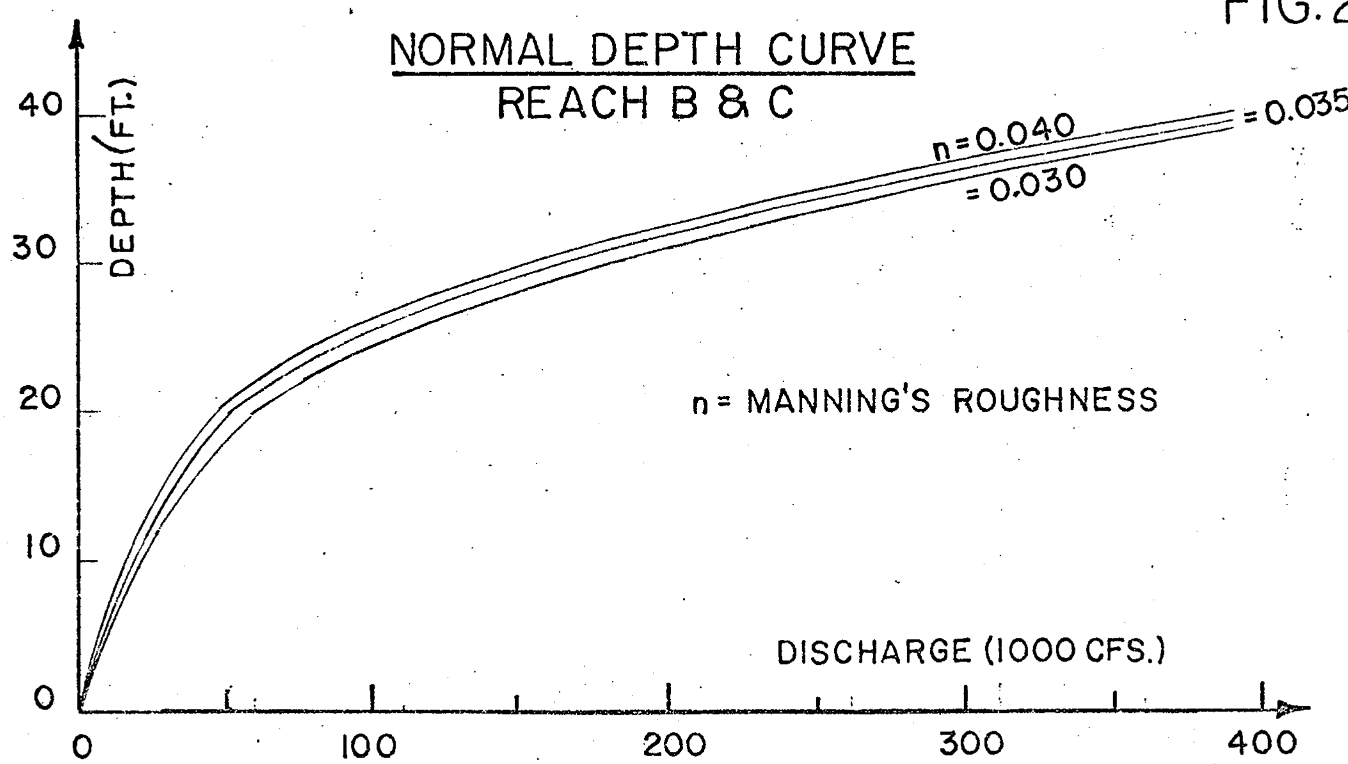


FIG. 23

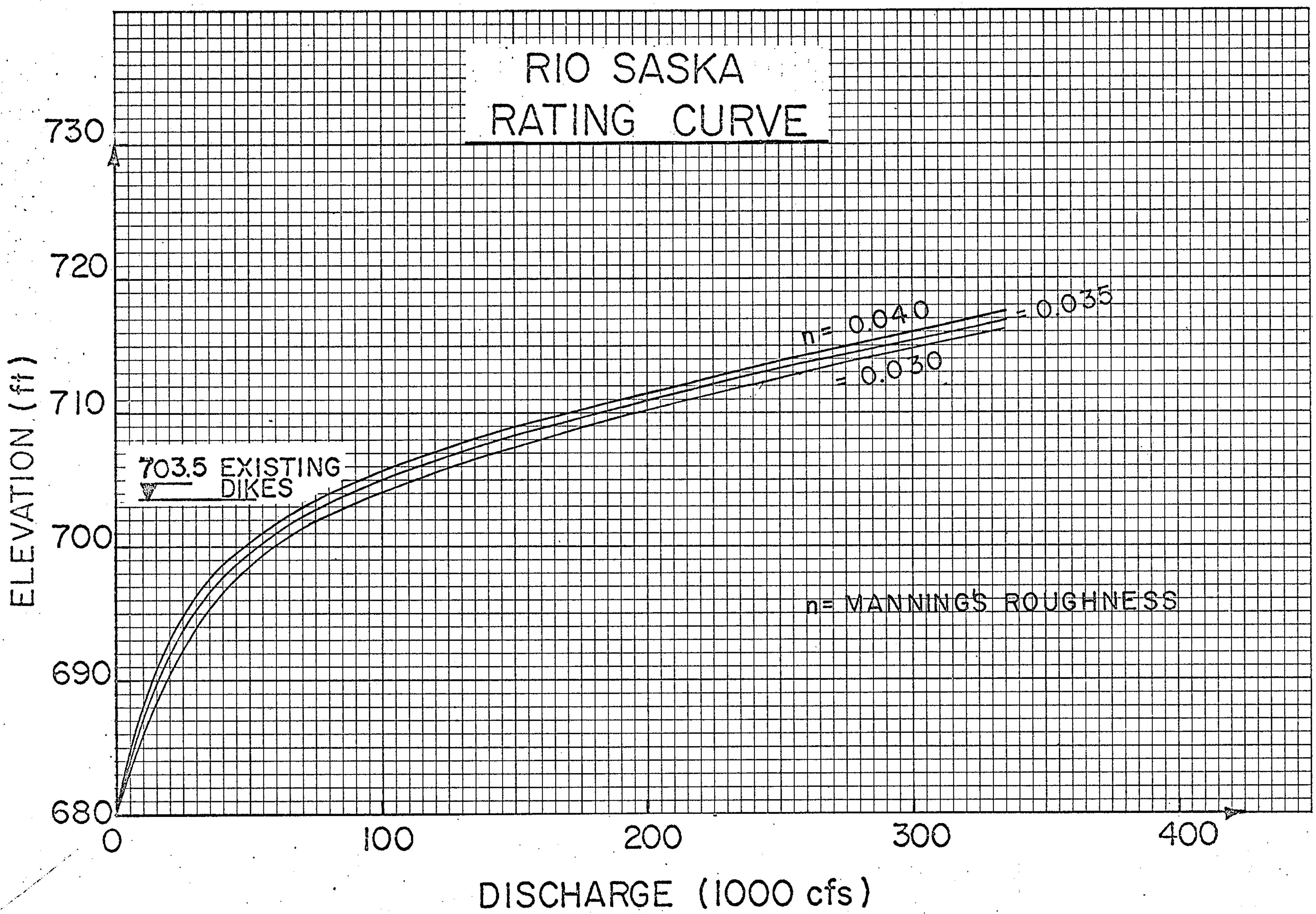

RIO SASKA
RATING CURVE
730
720
710
700
690
680
ELEVATION (ft)
703.5 EXISTING DIKES
n= 0.040
= 0.035
= 0.030
n= MANNINGS ROUGHNESS
0
100
200
300
400
DISCHARGE (1000 cfs)

FIG. 24

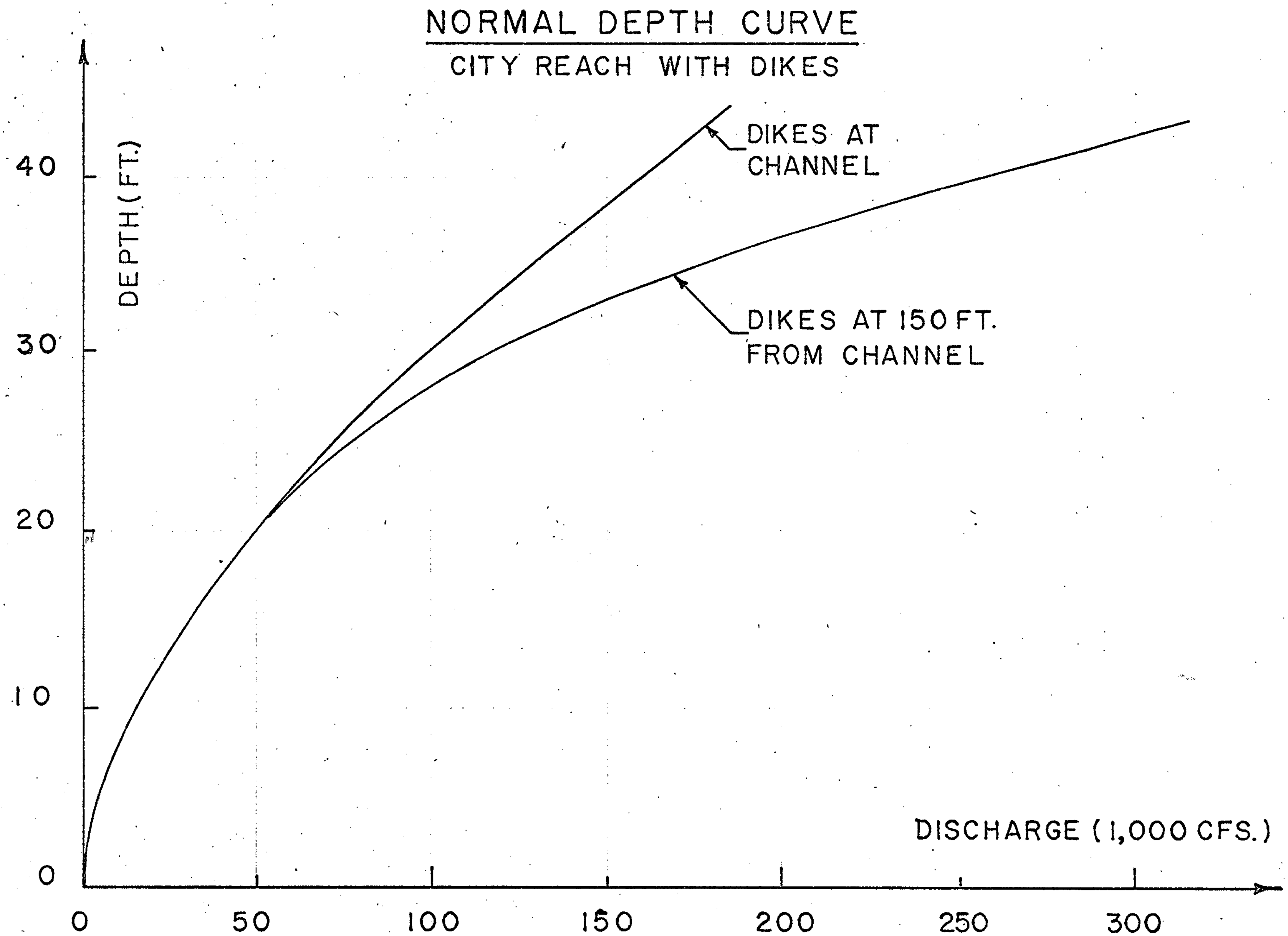

NORMAL DEPTH CURVE
CITY REACH WITH DIKES
DEPTH (FT.)
40
30
20
10
0
DIKES AT
CHANNEL
DIKES AT 150 FT.
FROM CHANNEL
DISCHARGE (1,000 CFS.)
0
50
100
150
200
250
300

FIG. 25

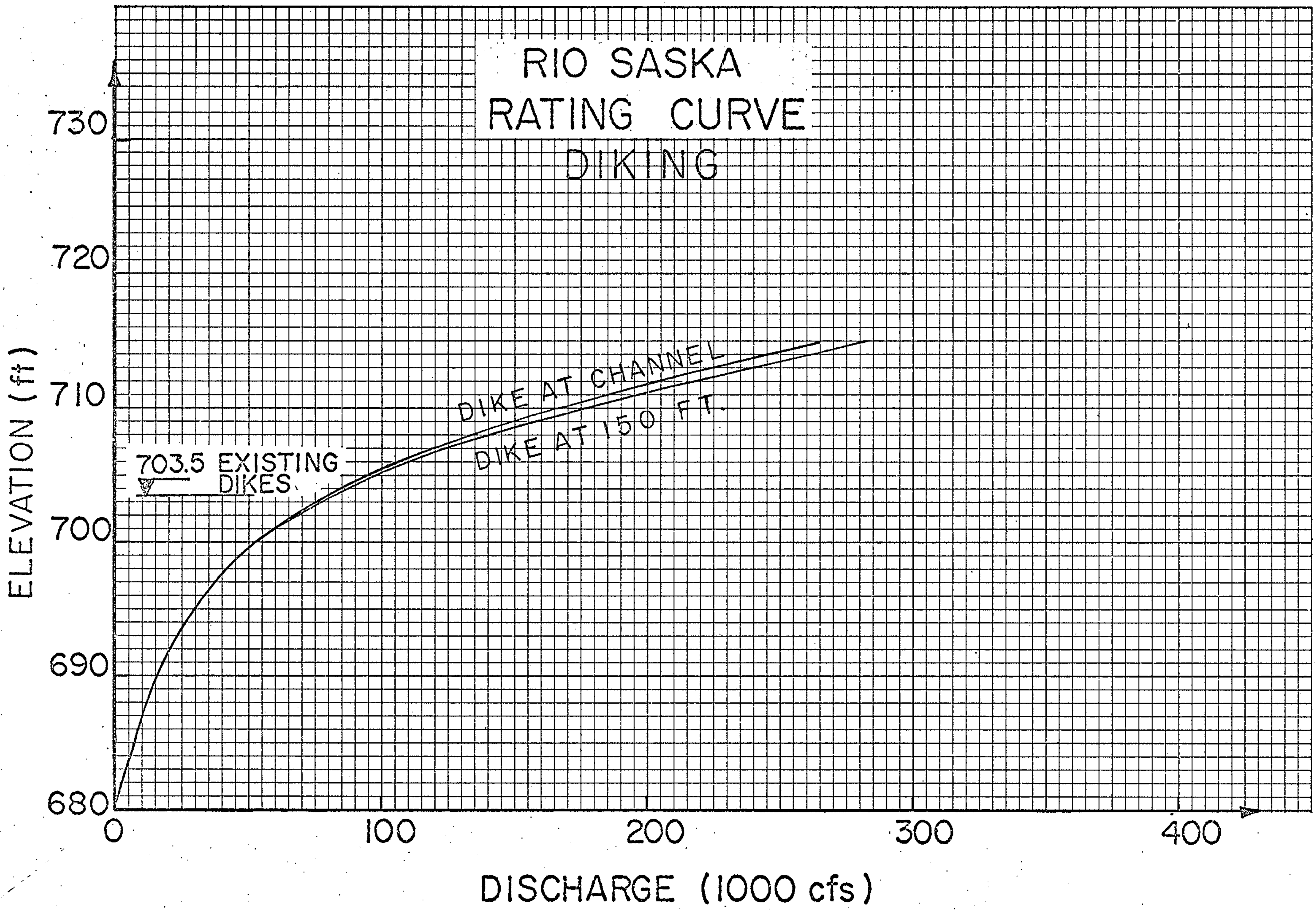
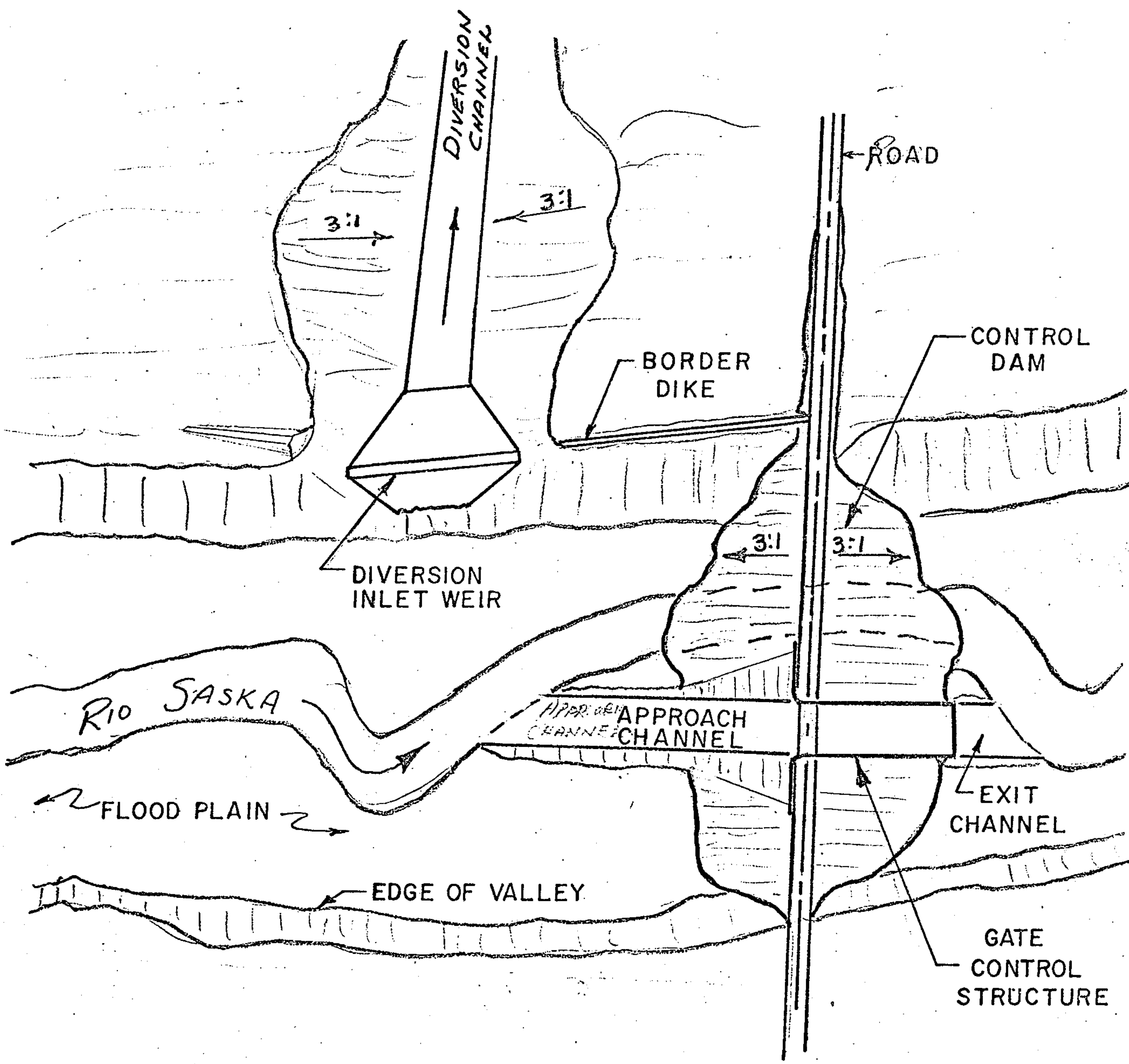
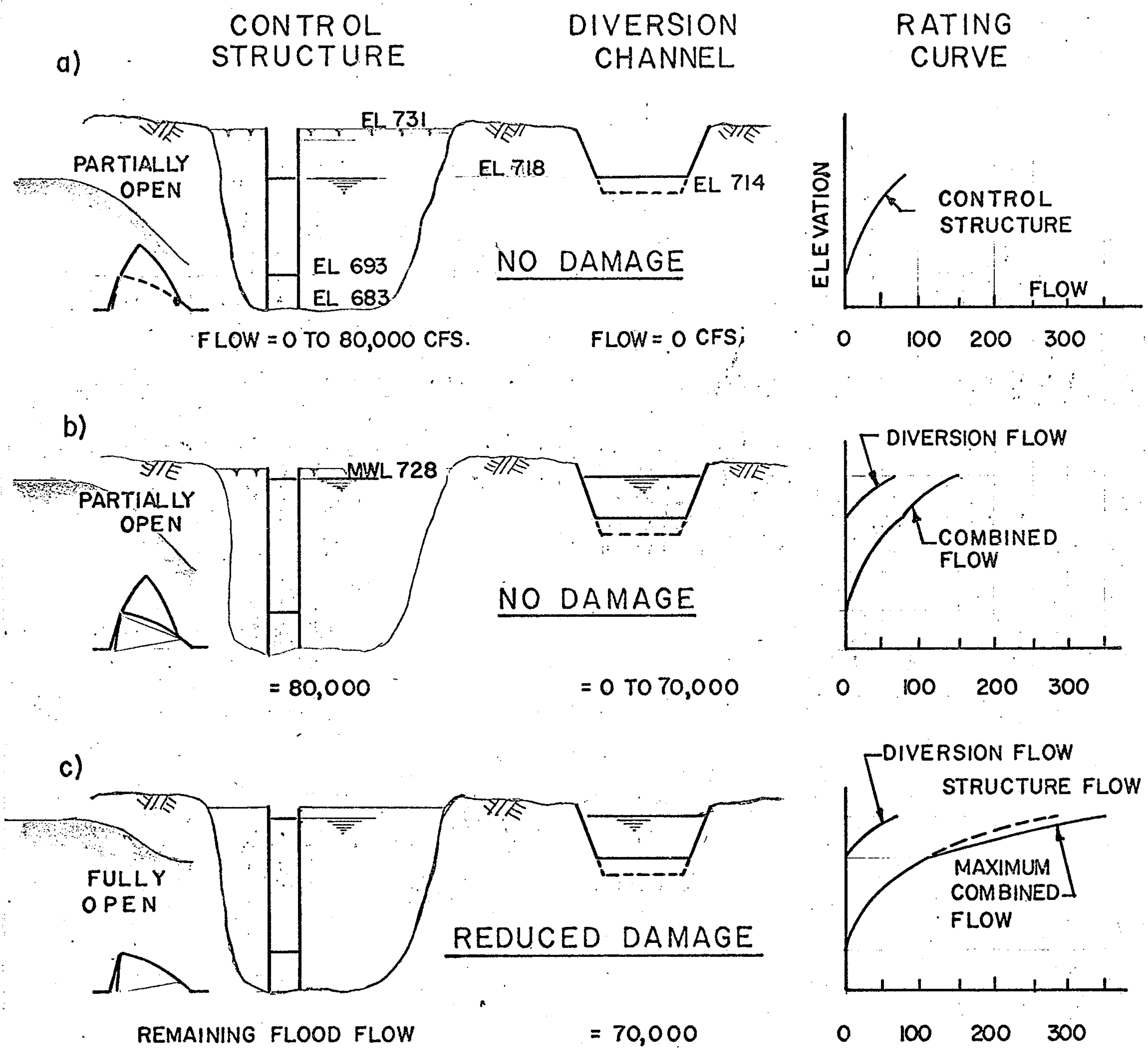

RIO SASKA
RATING CURVE
DIKING
730
720
710
700
690
680
0
100
200
300
400
ELEVATION (ft)
DISCHARGE (1000 cfs)
703.5 EXISTING DIKES
DIKE AT CHANNEL
DIKE AT 150 FT.

FIG. 26



PORTAGE DIVERSION
TYPICAL LAYOUT

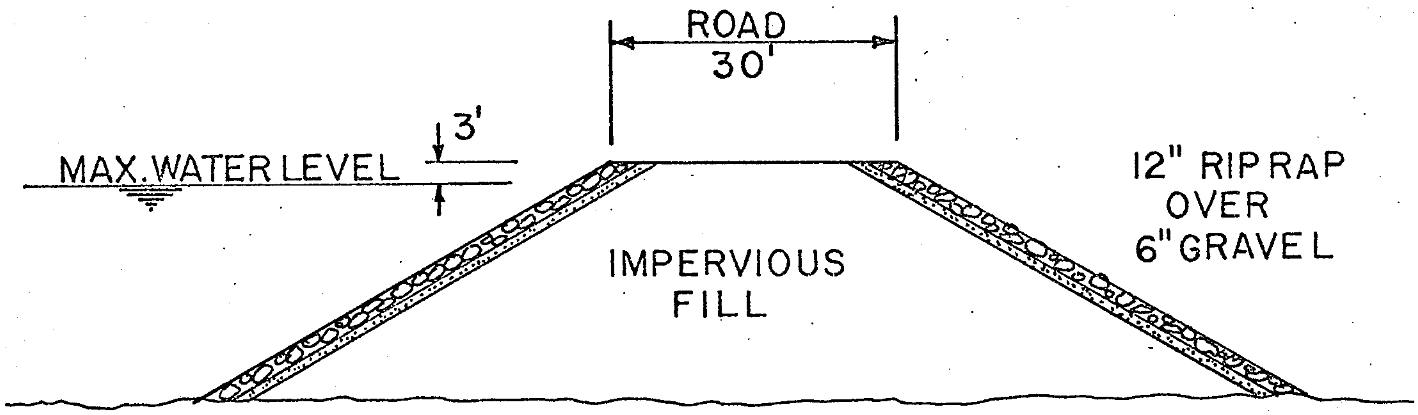
FIG. 27



PORTAGE DIVERSION

SCHEMATIC MODE OF OPERATION

FIG. 28



PORTAGE DIVERSION
CONTROL DAM
TYPICAL CROSS-SECTION

FIG. 29

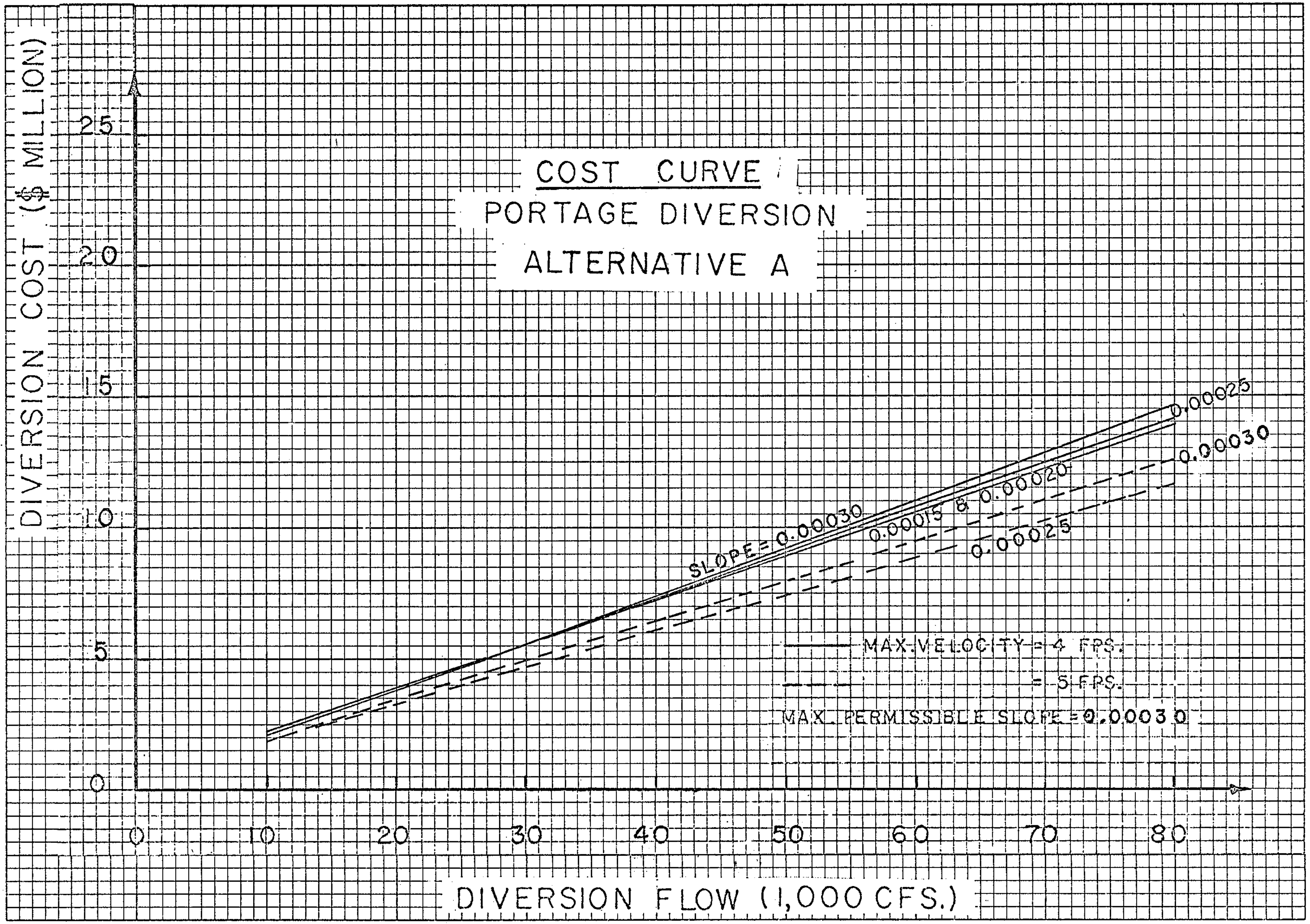

COST CURVE
PORTAGE DIVERSION
ALTERNATIVE A
DIVERSION COST ($ MILLION)
25
20
15
10
5
0
0
10
20
30
40
50
60
70
80
DIVERSION FLOW (1,000 CFS.)
SLOPE = 0.00030
0.00015 & 0.00020
0.00025
0.00030
0.00025
MAX. VELOCITY = 4 FPS.
= 5 FPS.
MAX. PERMISSIBLE SLOPE = 0.0003 0

FIG.30

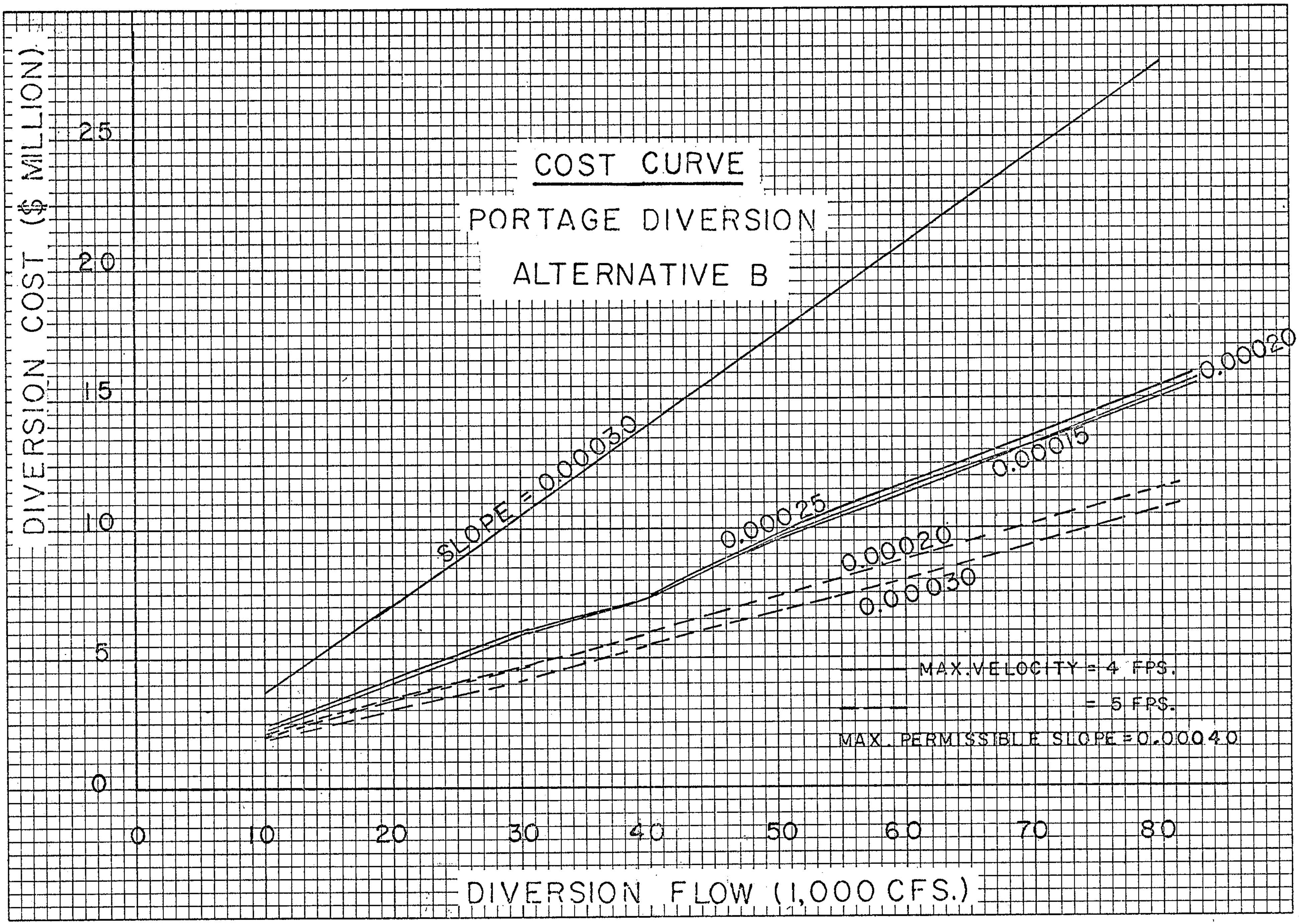

COST CURVE
PORTAGE DIVERSION
ALTERNATIVE B
DIVERSION COST ($ MILLION)
DIVERSION FLOW (1,000 CFS.)
0
5
10
15
20
25
0
10
20
30
40
50
60
70
80
SLOPE = 0.00030
0.00025
0.00020
0.00015
0.00020
0.00030
MAX. VELOCITY = 4 FPS.
= 5 FPS.
MAX. PERMISSIBLE SLOPE = 0.00040

FIG. 31

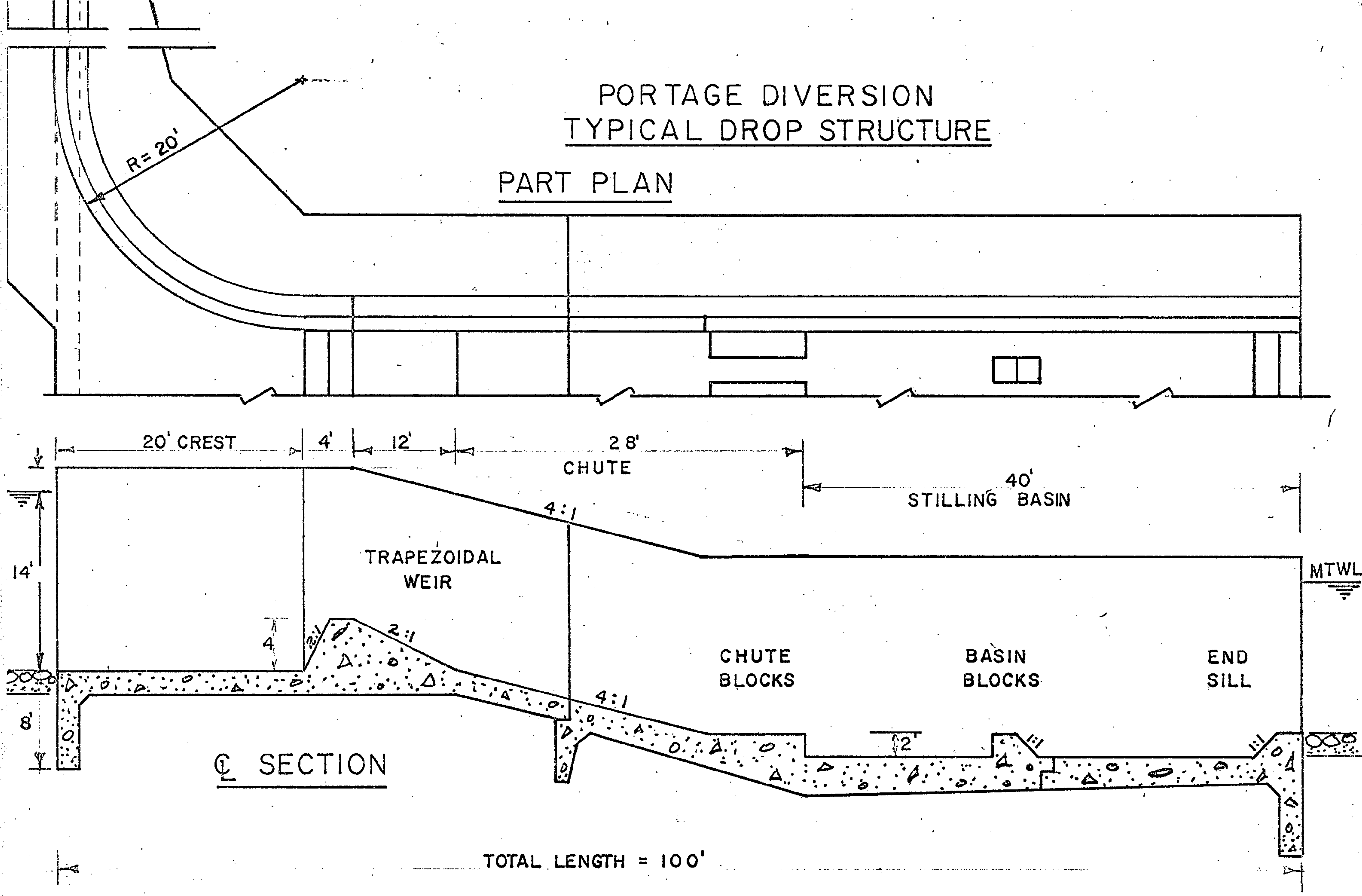

PORTAGE DIVERSION
TYPICAL DROP STRUCTURE
PART PLAN
R= 20'
20' CREST
4'
12'
28'
CHUTE
40'
STILLING BASIN
4:1
14'
TRAPEZOIDAL
WEIR
MTWL
4
2:1
2:1
CHUTE
BLOCKS
BASIN
BLOCKS
END
SILL
8'
4:1
2
1:1
1:1
℄ SECTION
TOTAL LENGTH = 100'

FIG. 32

PORTAGE DIVERSION
TYPICAL CONTROL STRUCTURE

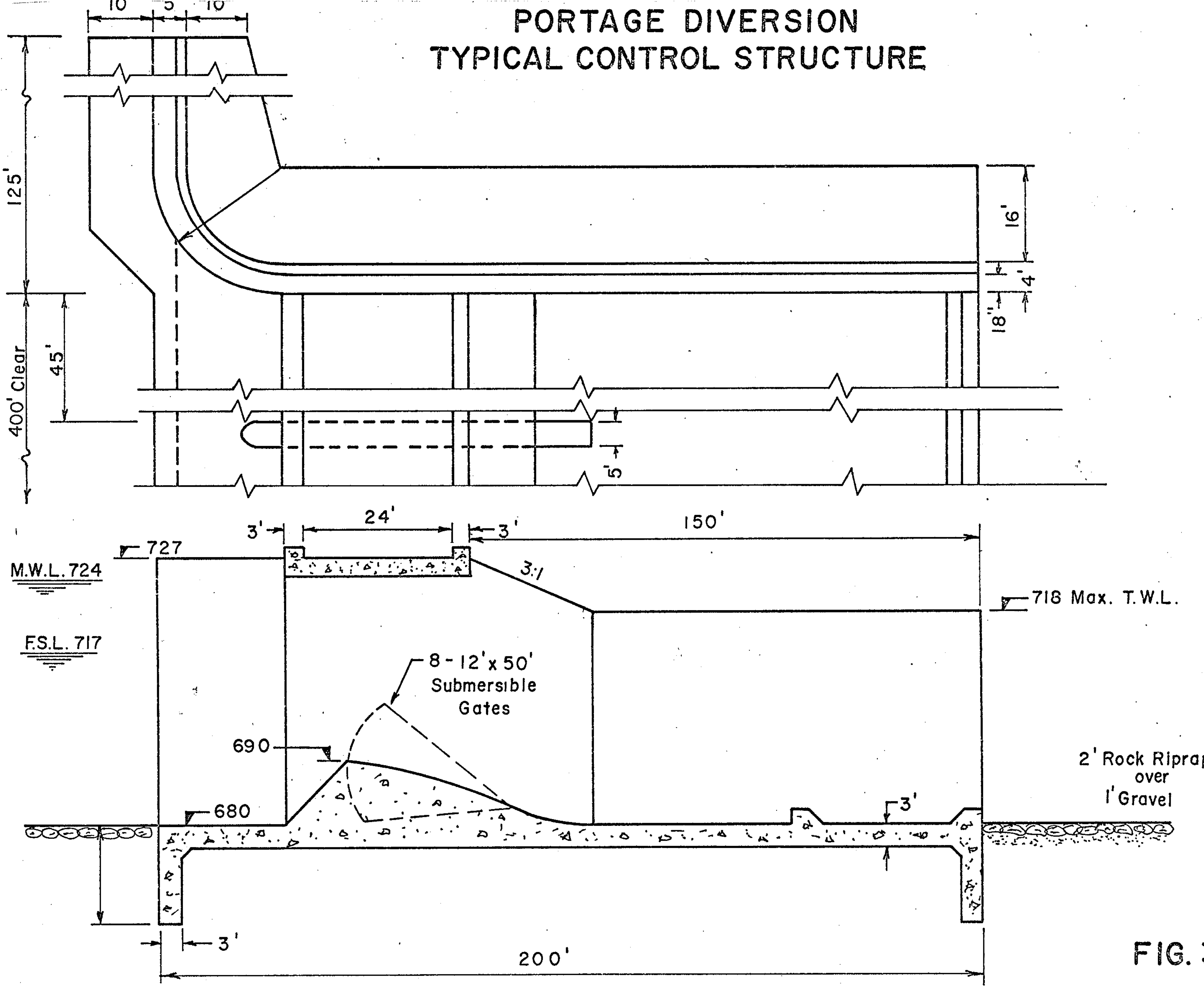


FIG. 33

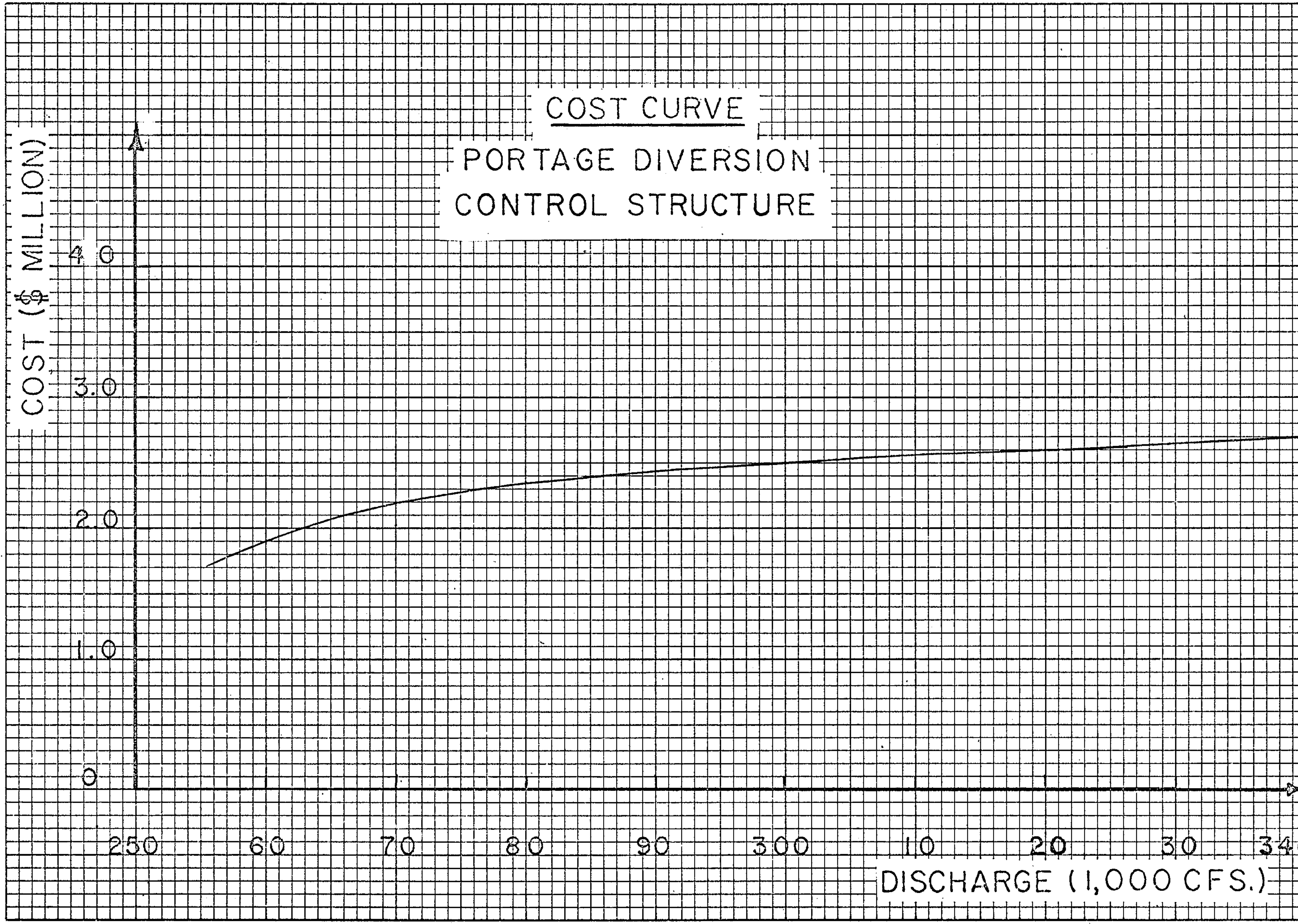

COST CURVE
PORTAGE DIVERSION
CONTROL STRUCTURE
COST ($ MILLION)
4.0
3.0
2.0
1.0
0
250
60
70
80
90
300
10
20
30
340
DISCHARGE (1,000 CFS.)

FIG. 34

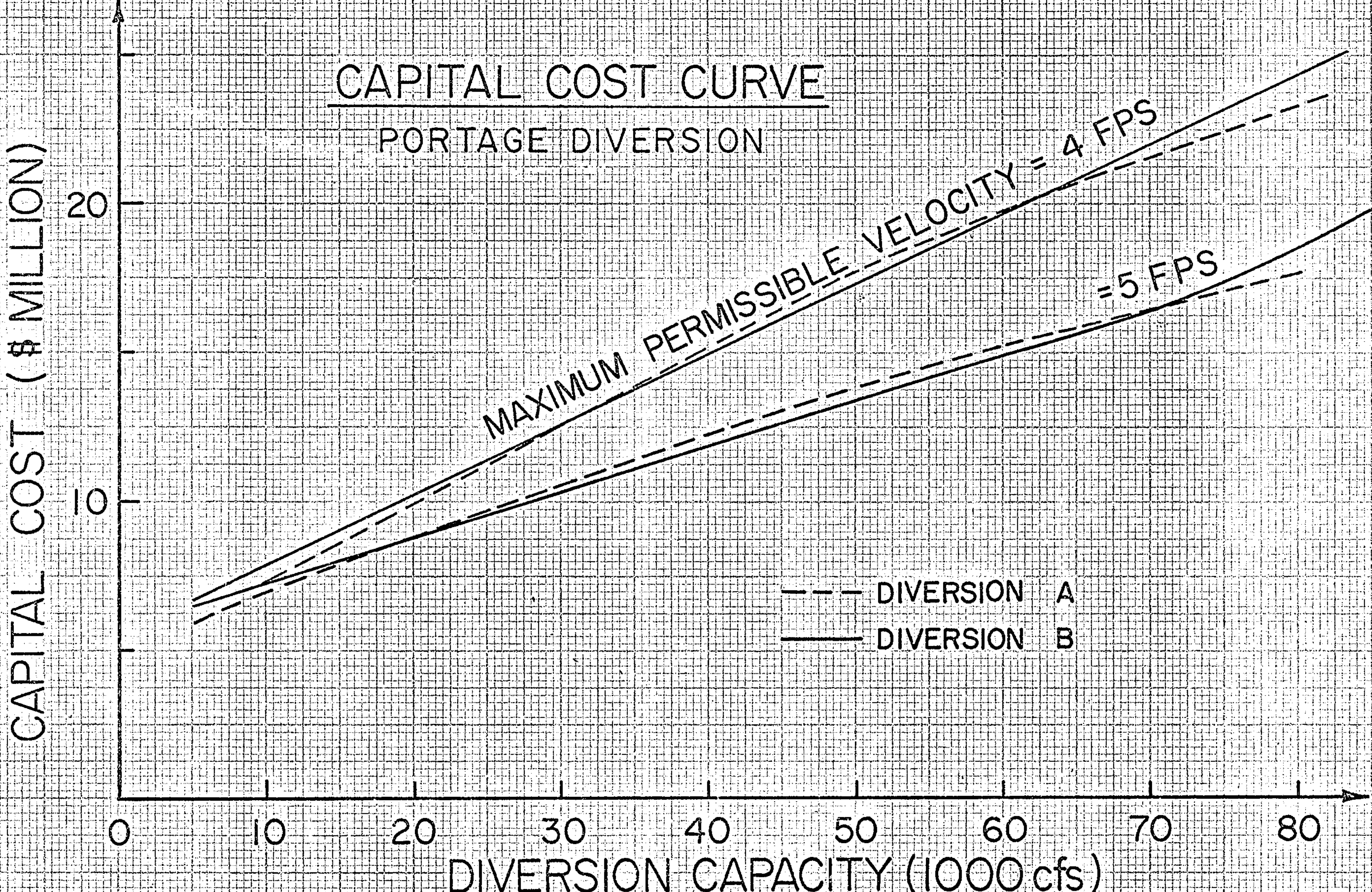
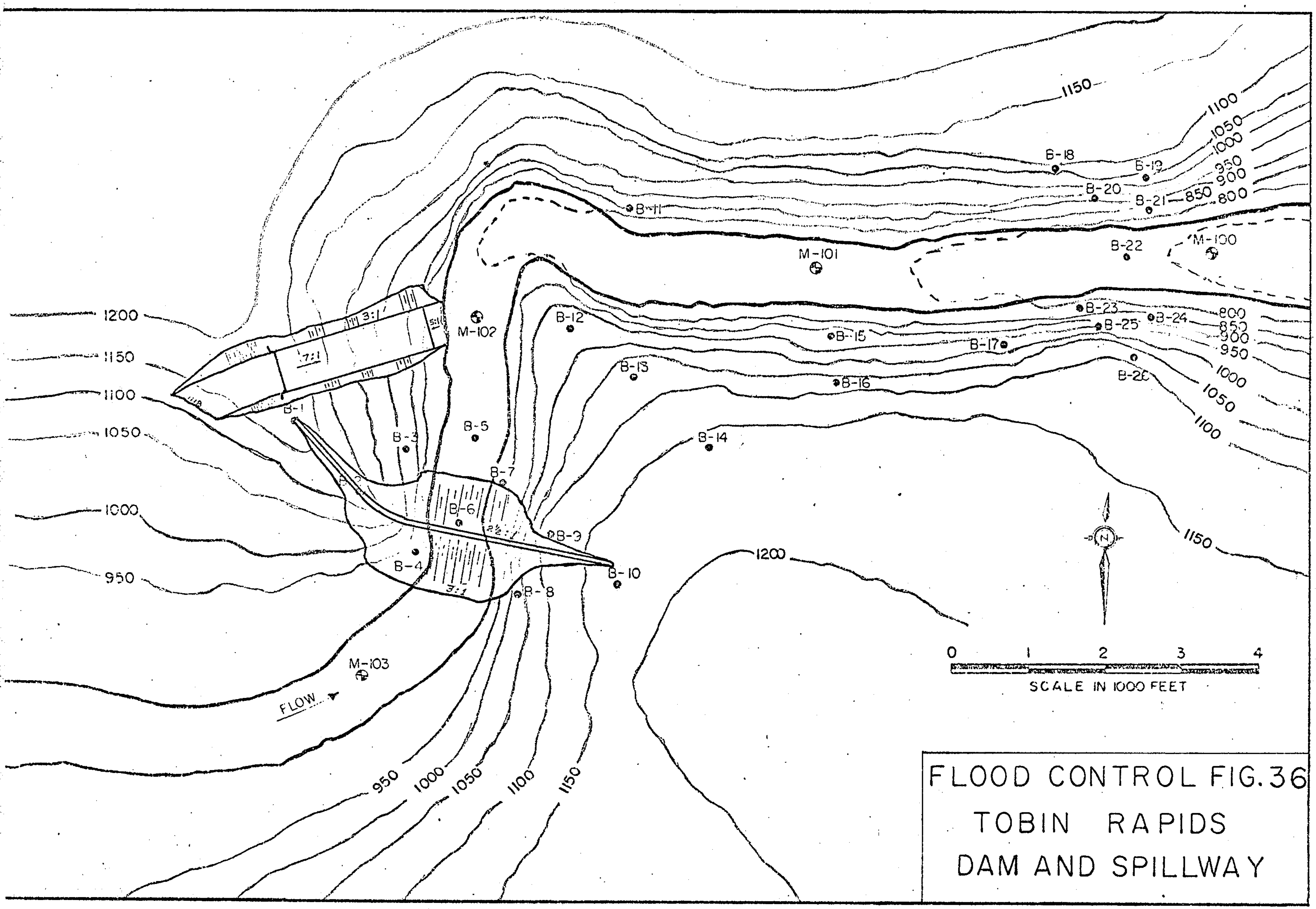
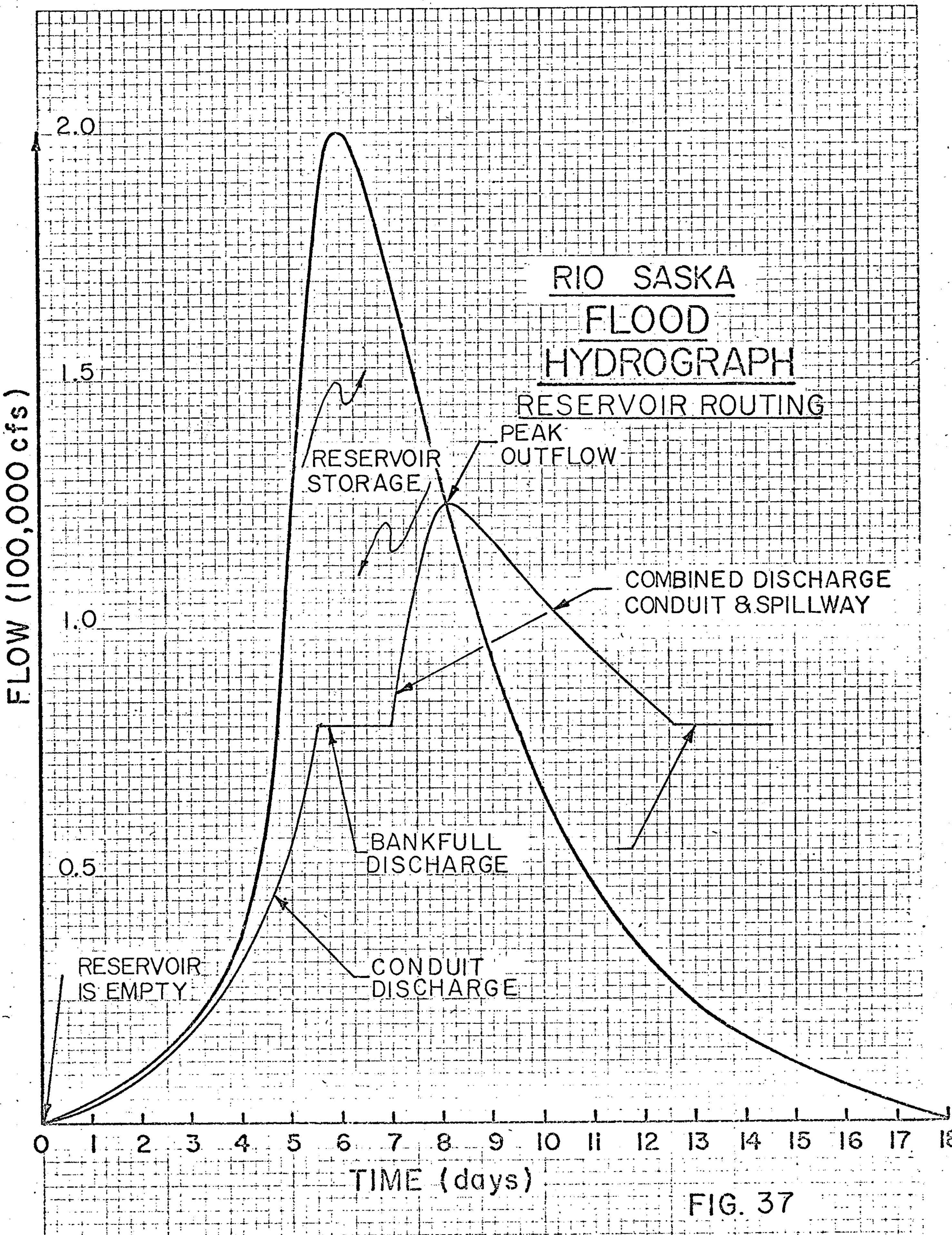

CAPITAL COST CURVE
PORTAGE DIVERSION
MAXIMUM PERMISSIBLE VELOCITY = 4 FPS
= 5 FPS
DIVERSION A
DIVERSION B
CAPITAL COST ($ MILLION)
20
10
0
10
20
30
40
50
60
70
80
DIVERSION CAPACITY (1000 cfs)

FIG. 35



FLOOD CONTROL FIG. 36
TOBIN RAPIDS
DAM AND SPILLWAY



RIO SASKA
FLOOD
HYDROGRAPH
RESERVOIR ROUTING
FLOW (100,000 cfs)
2.0
1.5
1.0
0.5
RESERVOIR STORAGE
PEAK OUTFLOW
COMBINED DISCHARGE CONDUIT & SPILLWAY
BANKFULL DISCHARGE
CONDUIT DISCHARGE
RESERVOIR IS EMPTY
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18
TIME (days)

FIG. 37

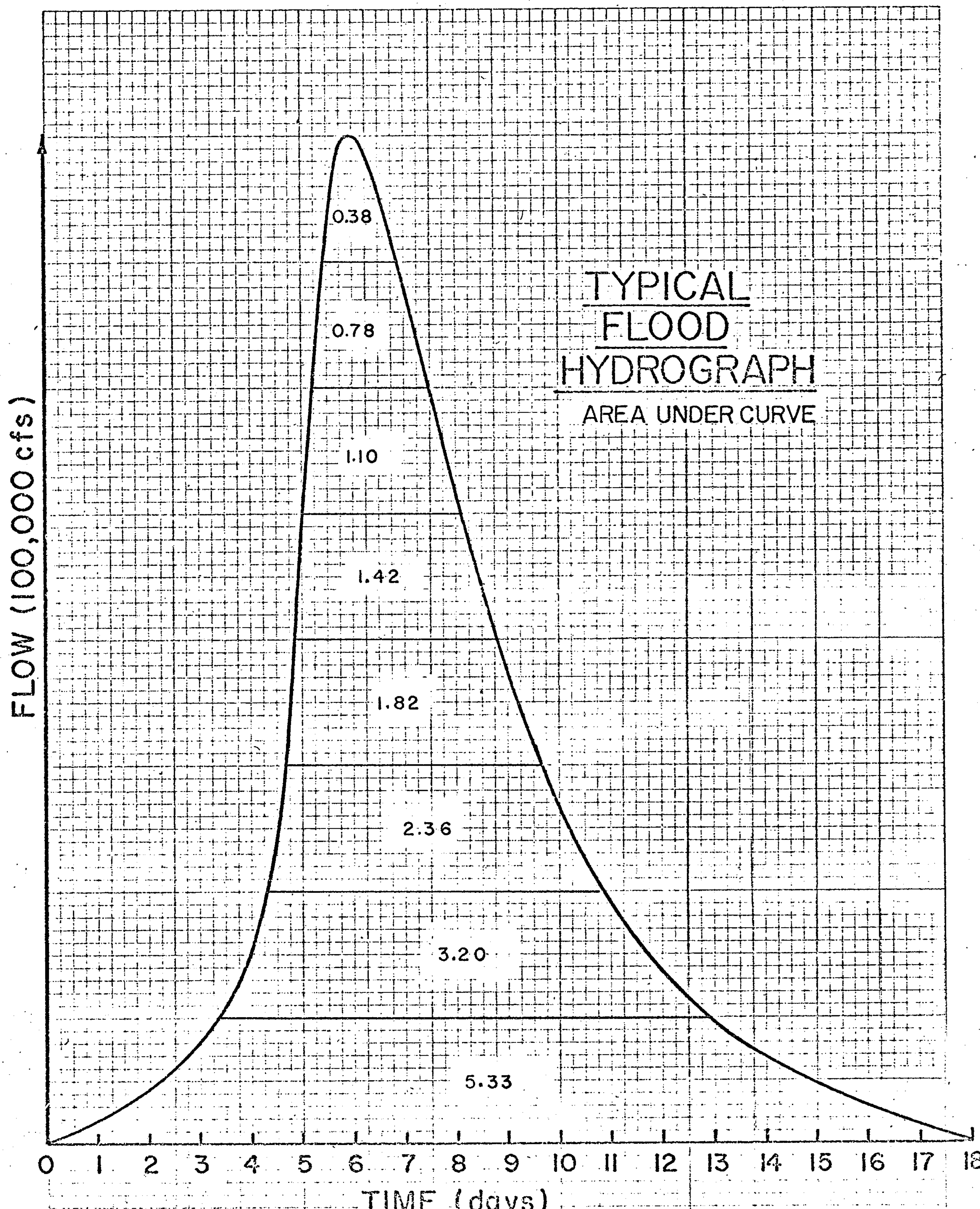

TYPICAL FLOOD HYDROGRAPH
AREA UNDER CURVE
FLOW (100,000 cfs)
TIME (days)
0.38
0.78
1.10
1.42
1.82
2.36
3.20
5.33
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18

FIG. 38

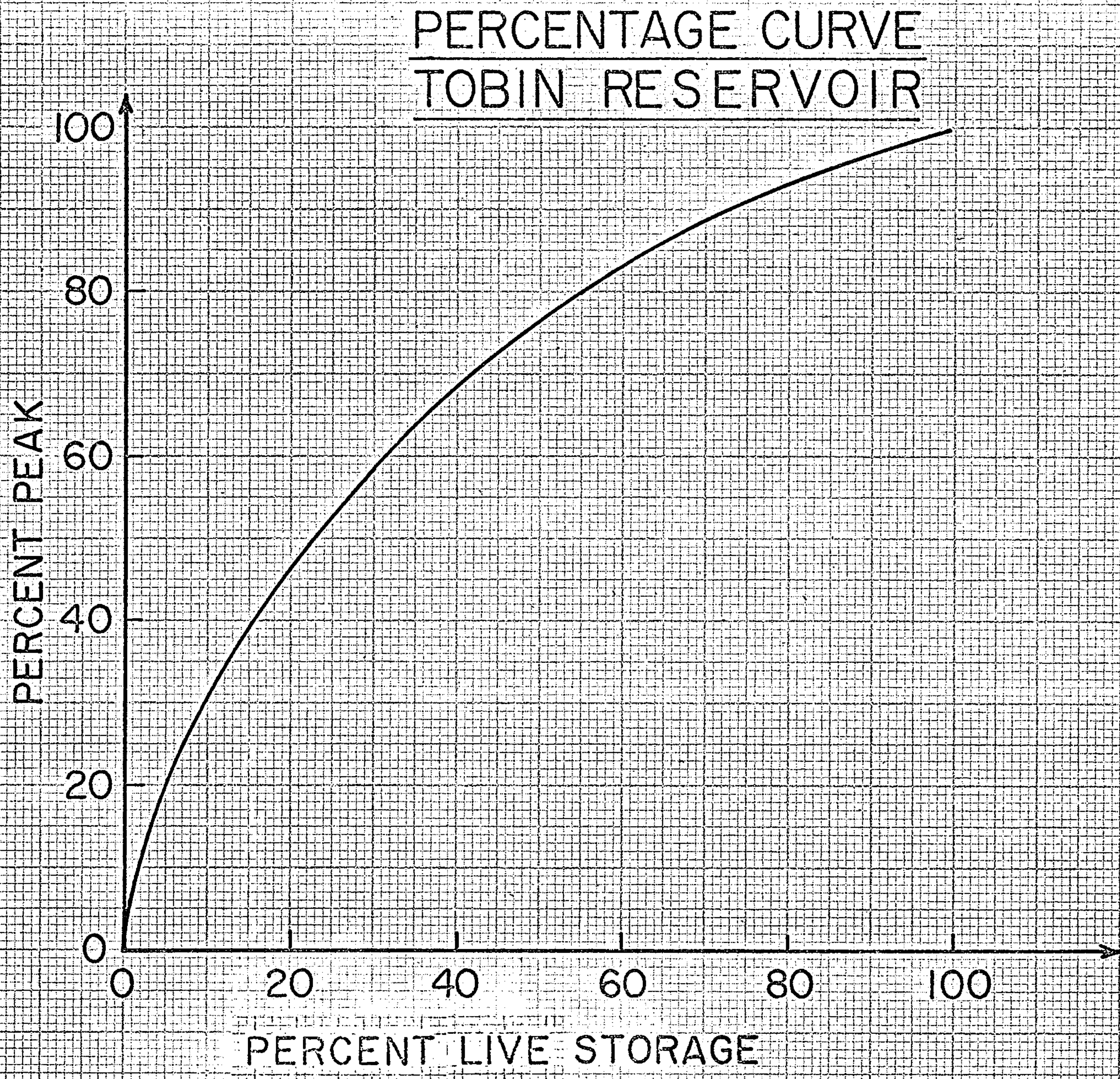
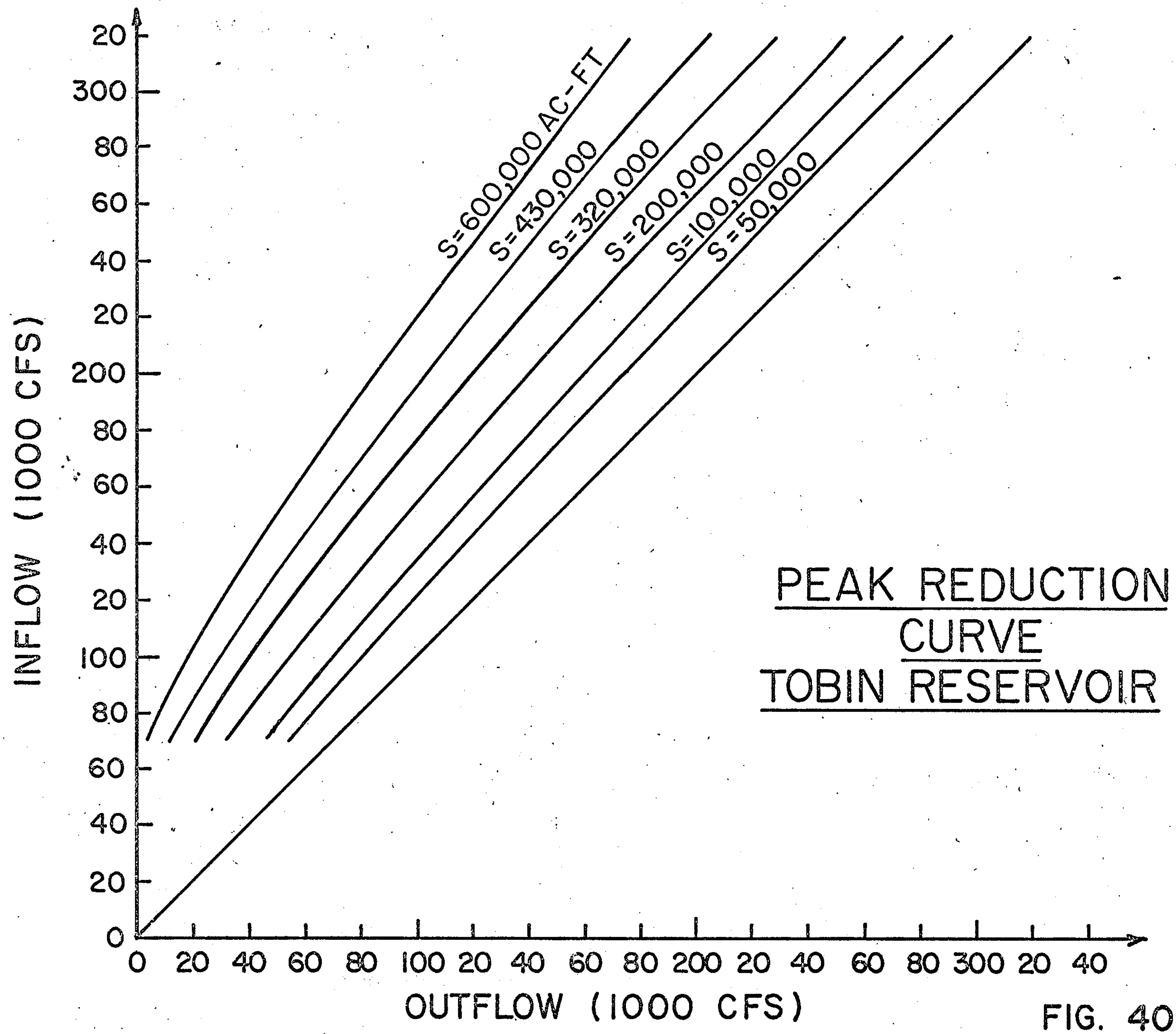

PERCENTAGE CURVE
TOBIN RESERVOIR
PERCENT PEAK
0
20
40
60
80
100
PERCENT LIVE STORAGE

FIG. 39



PEAK REDUCTION CURVE TOBIN RESERVOIR
S=600,000 AC-FT
S=430,000
S=320,000
S=200,000
S=100,000
S=50,000
INFLOW (1000 CFS)
0
20
40
60
80
100
20
40
60
80
200
20
40
60
80
300
20
OUTFLOW (1000 CFS)
0
20
40
60
80
100
20
40
60
80
200
20
40
60
80
300
20
40

FIG. 40

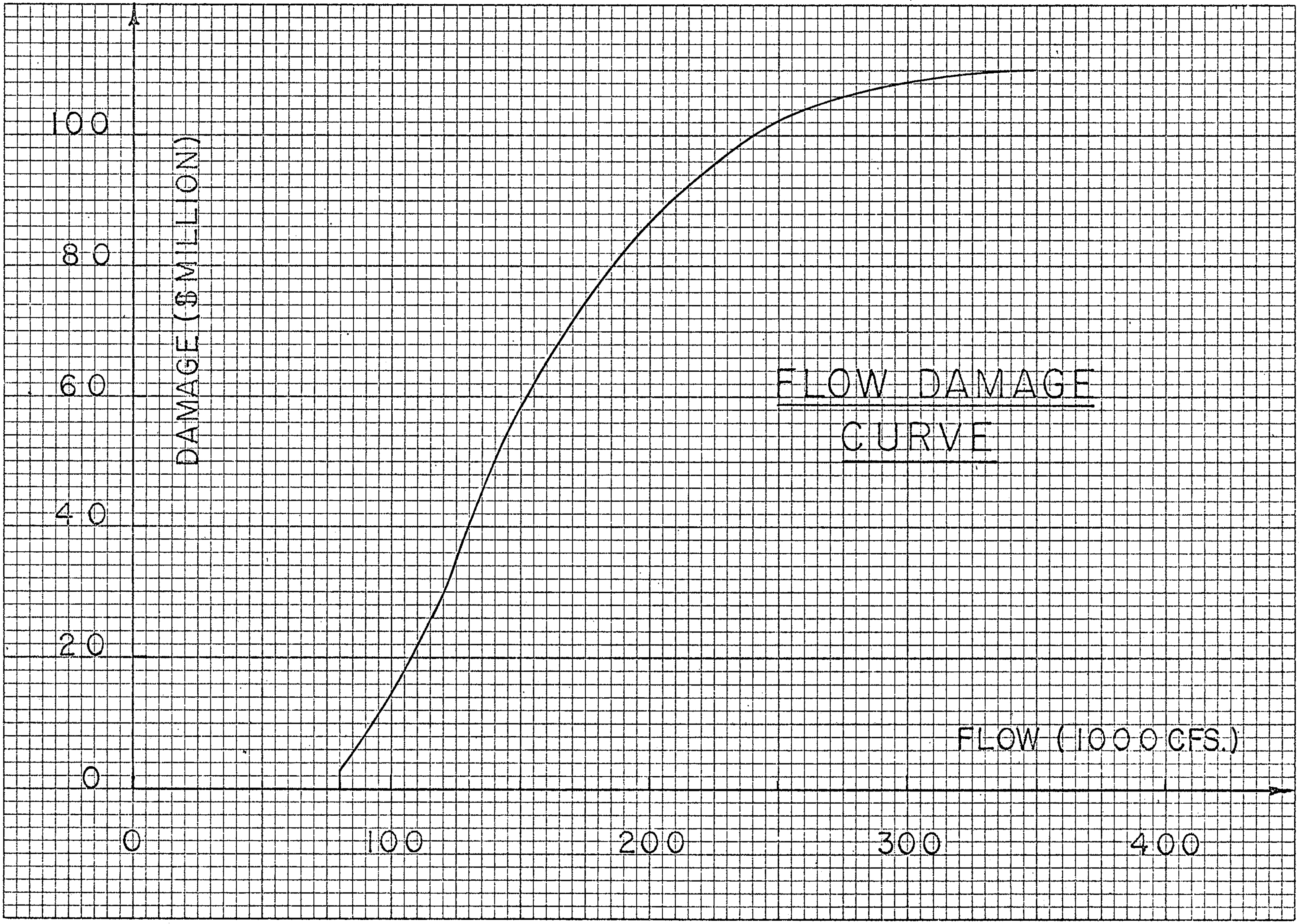

FLOW DAMAGE CURVE
DAMAGE ($ MILLION)
FLOW (1000 CFS.)
100
80
60
40
20
0
0
100
200
300
400

FIG. 41

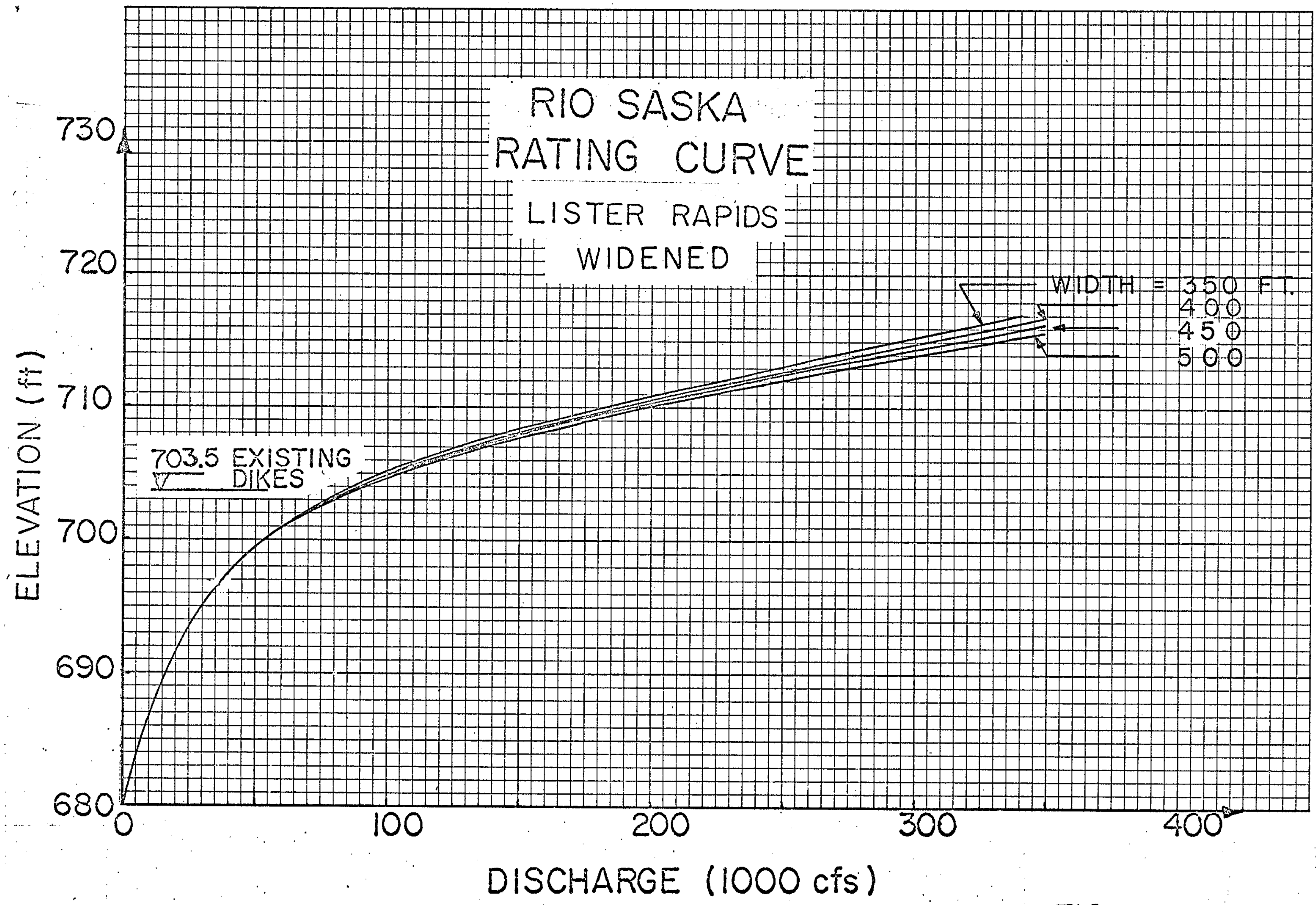

RIO SASKA
RATING CURVE
LISTER RAPIDS
WIDENED
WIDTH = 350 FT.
400
450
500
703.5 EXISTING DIKES
ELEVATION (ft)
730
720
710
700
690
680
0
100
200
300
400
DISCHARGE (1000 cfs)

FIG. 42

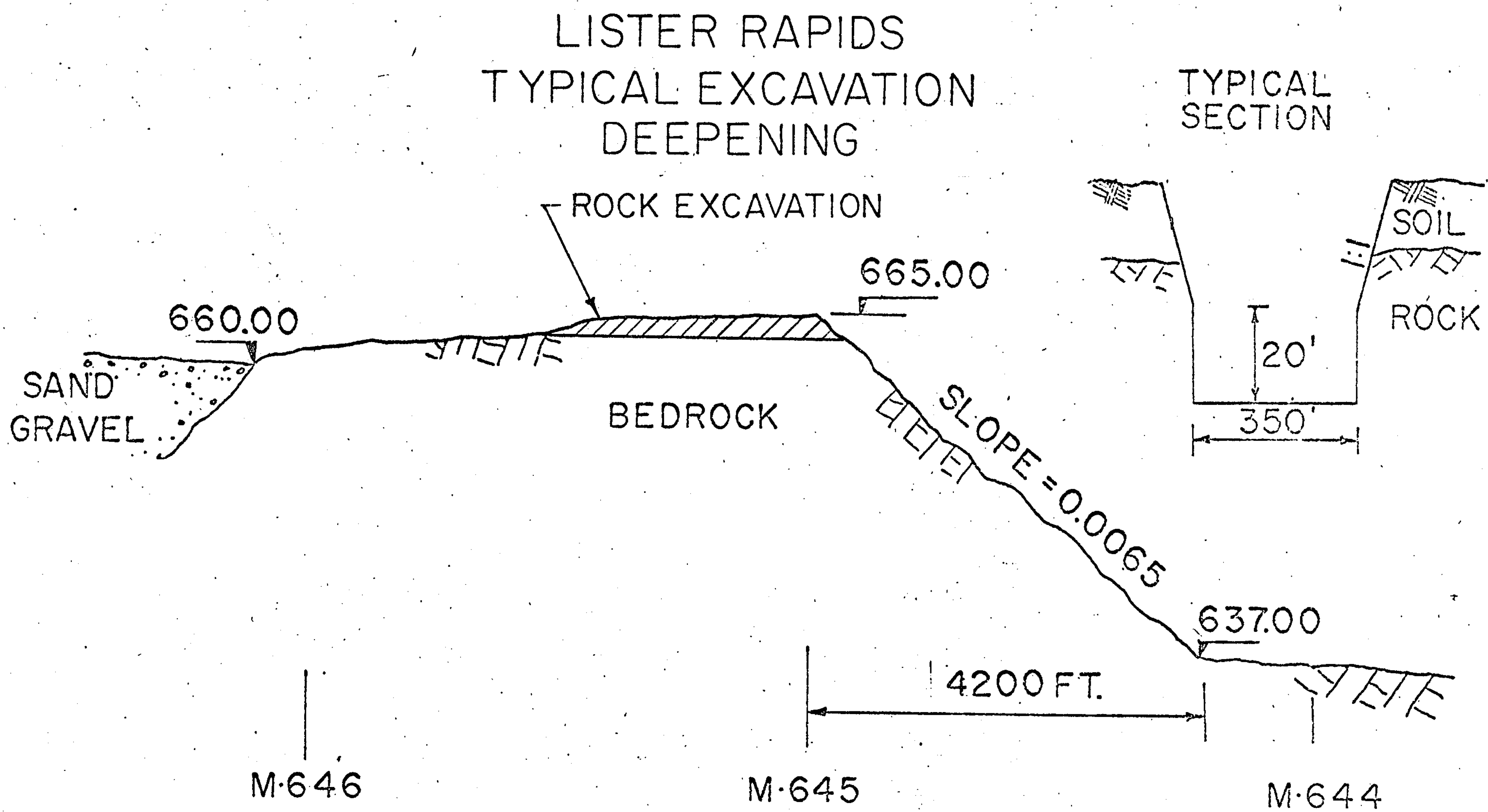

LISTER RAPIDS
TYPICAL EXCAVATION
DEEPENING
TYPICAL
SECTION
ROCK EXCAVATION
665.00
660.00
SOIL
SAND
GRAVEL
BEDROCK
ROCK
20'
350'
1:1
SLOPE = 0.0065
637.00
4200 FT.
M·646
M·645
M·644

FIG. 43

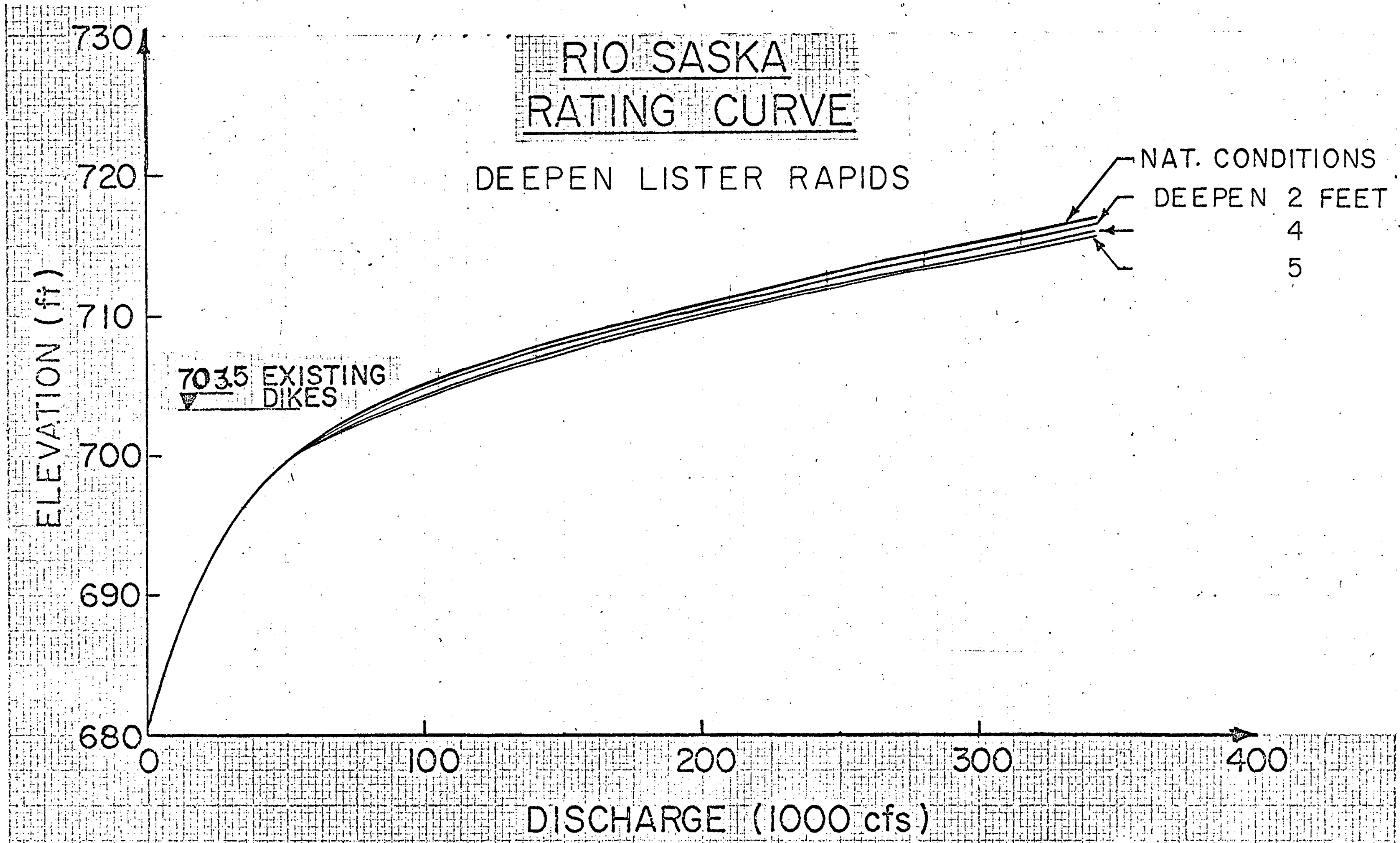
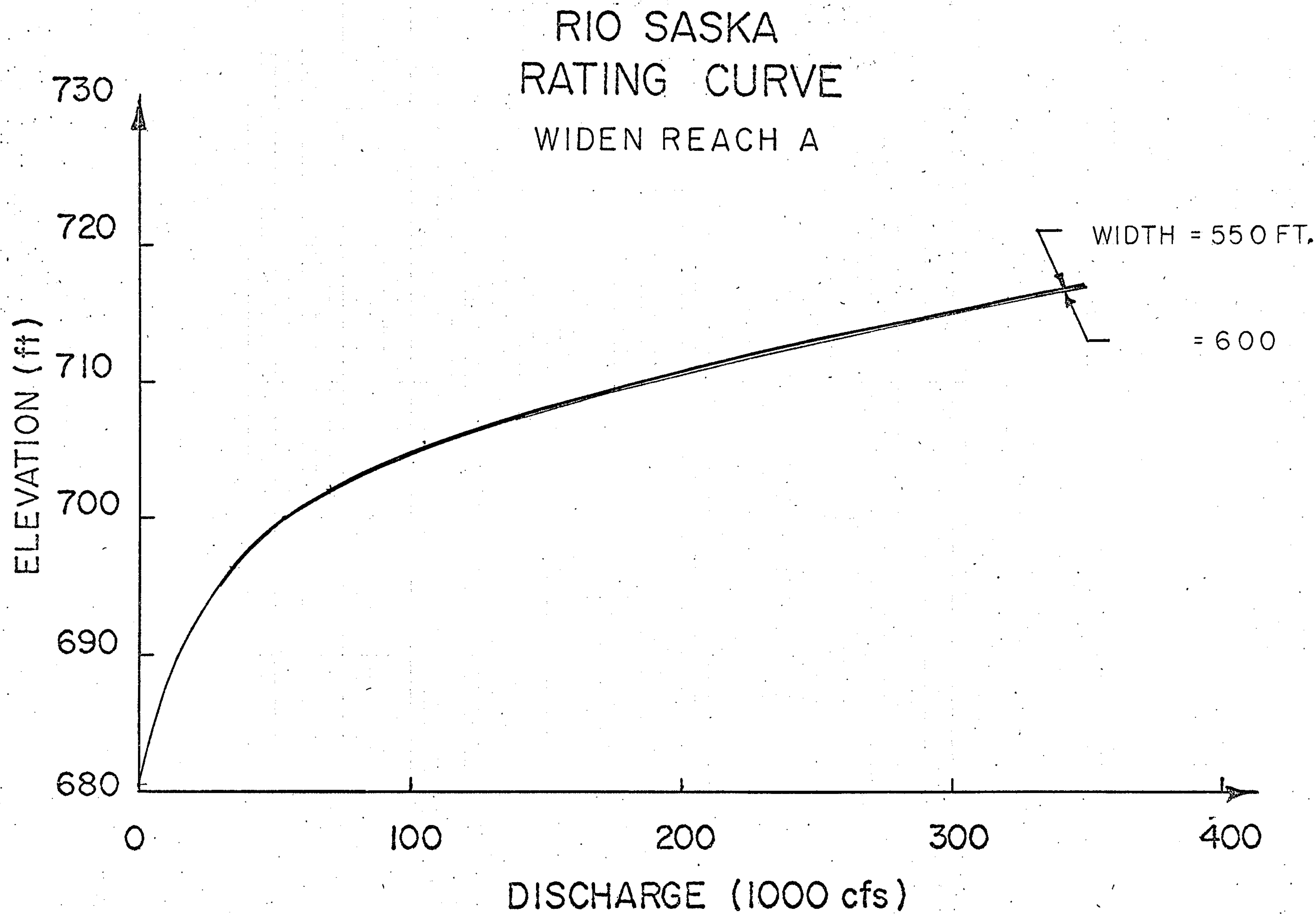

RIO SASKA
RATING CURVE
DEEPEN LISTER RAPIDS
NAT. CONDITIONS
DEEPEN 2 FEET
4
5
703.5 EXISTING DIKES
730
720
710
700
690
680
ELEVATION (ft)
0
100
200
300
400
DISCHARGE (1000 cfs)

FIG. 44



RIO SASKA
RATING CURVE
WIDEN REACH A
WIDTH = 550 FT.
= 600
ELEVATION (ft)
730
720
710
700
690
680
0
100
200
300
400
DISCHARGE (1000 cfs)

FIG. 45

NORMAL DEPTH CURVE

WIDEN REACH B

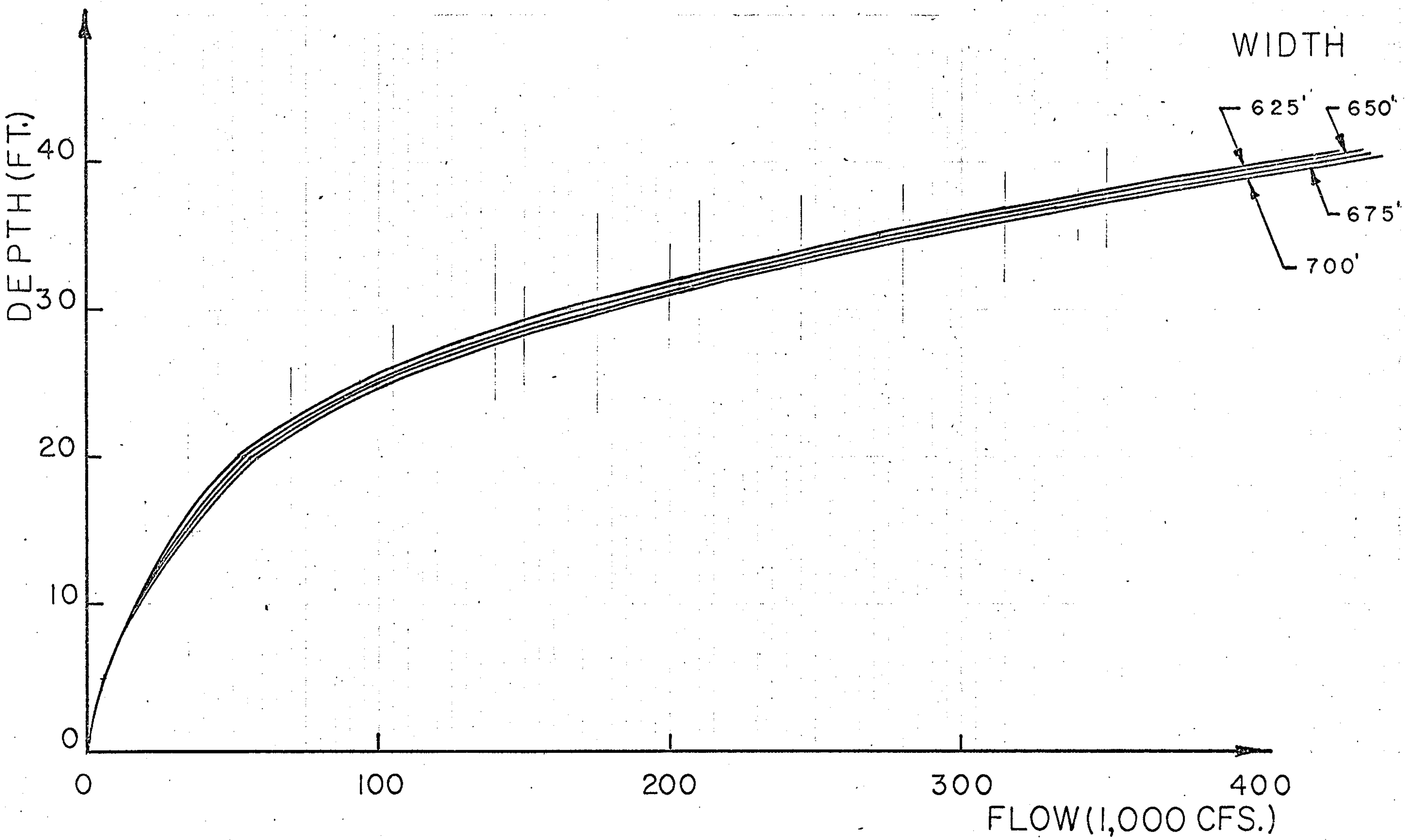


FIG. 46

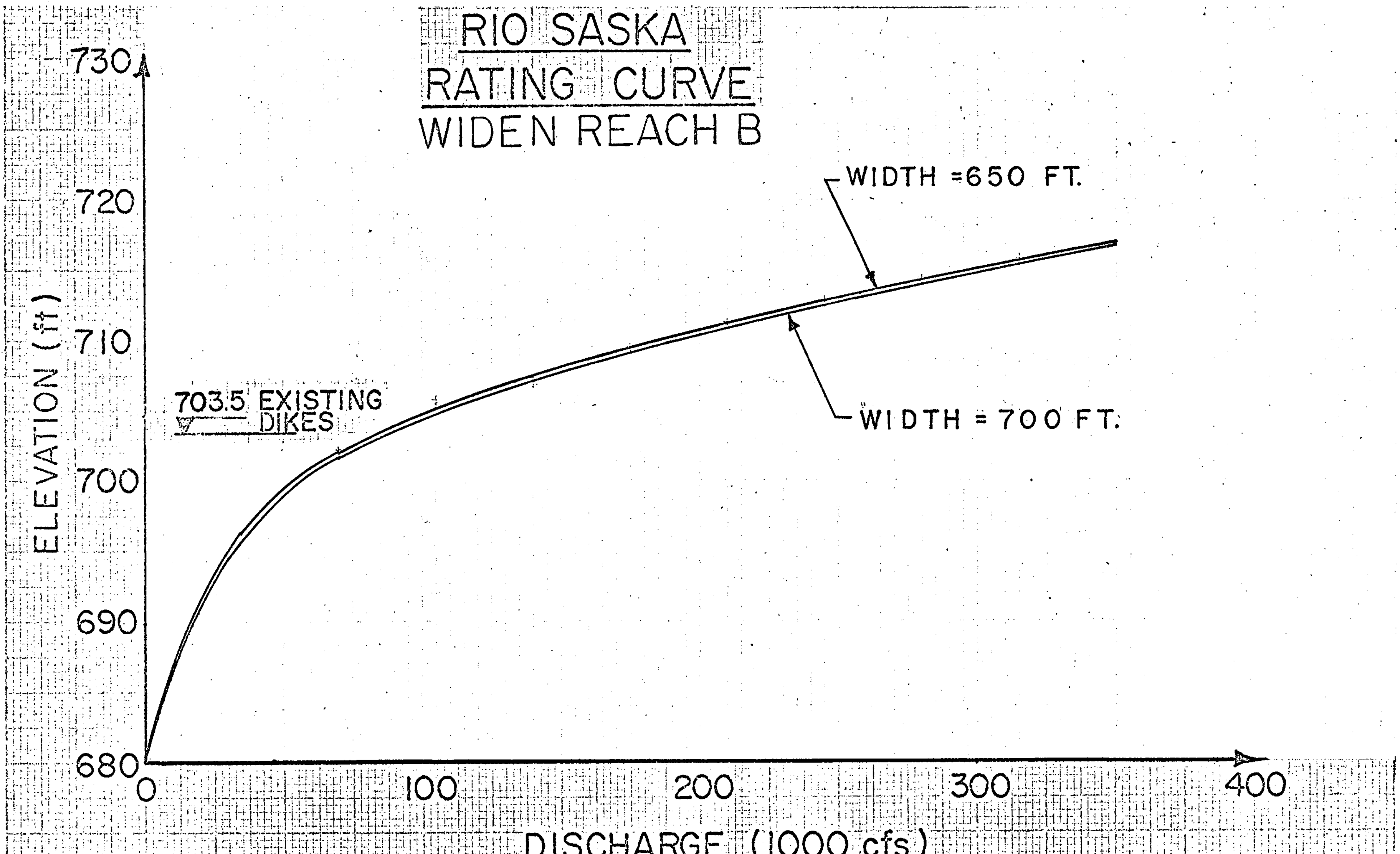
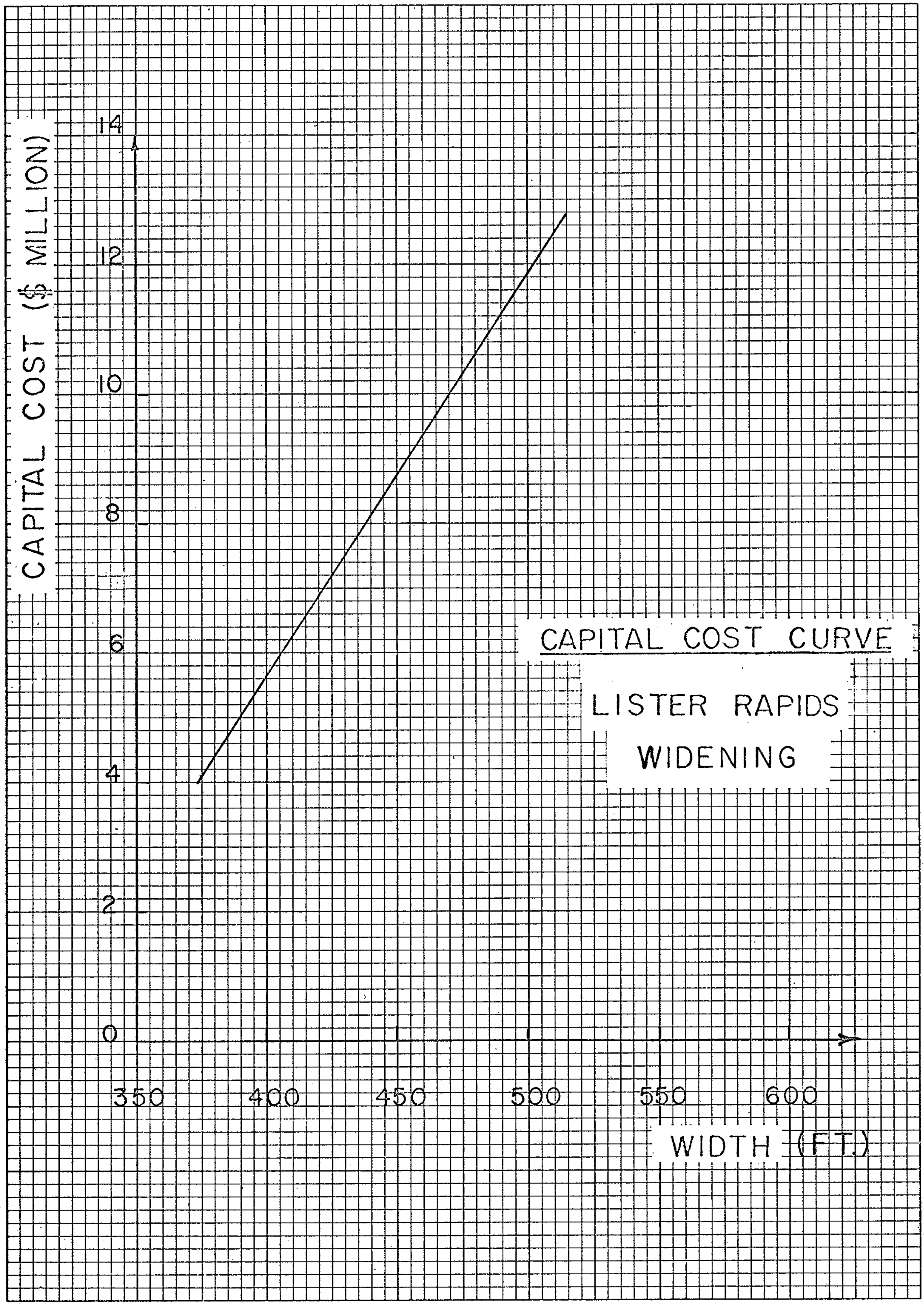

RIO SASKA
RATING CURVE
WIDEN REACH B
WIDTH =650 FT.
WIDTH = 700 FT.
703.5 EXISTING DIKES
ELEVATION (ft)
730
720
710
700
690
680
0
100
200
300
400
DISCHARGE (1000 cfs)

FIG. 47



CAPITAL COST ($ MILLION)
14
12
10
8
6
4
2
0
350
400
450
500
550
600
WIDTH (FT.)
CAPITAL COST CURVE
LISTER RAPIDS
WIDENING

FIG. 48

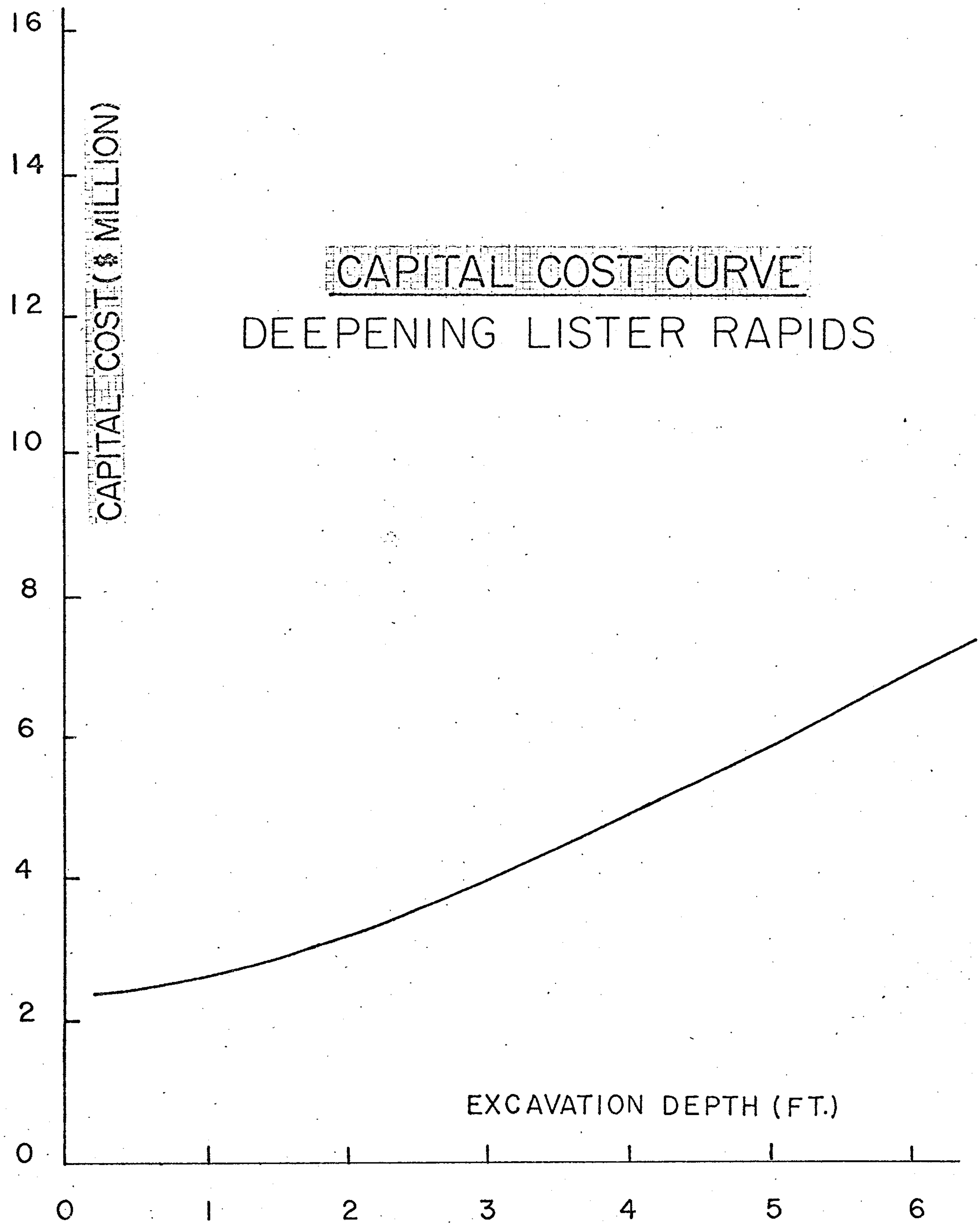

CAPITAL COST CURVE
DEEPENING LISTER RAPIDS
CAPITAL COST ($ MILLION)
EXCAVATION DEPTH (FT.)
0
2
4
6
8
10
12
14
16
0
1
2
3
4
5
6

FIG. 49

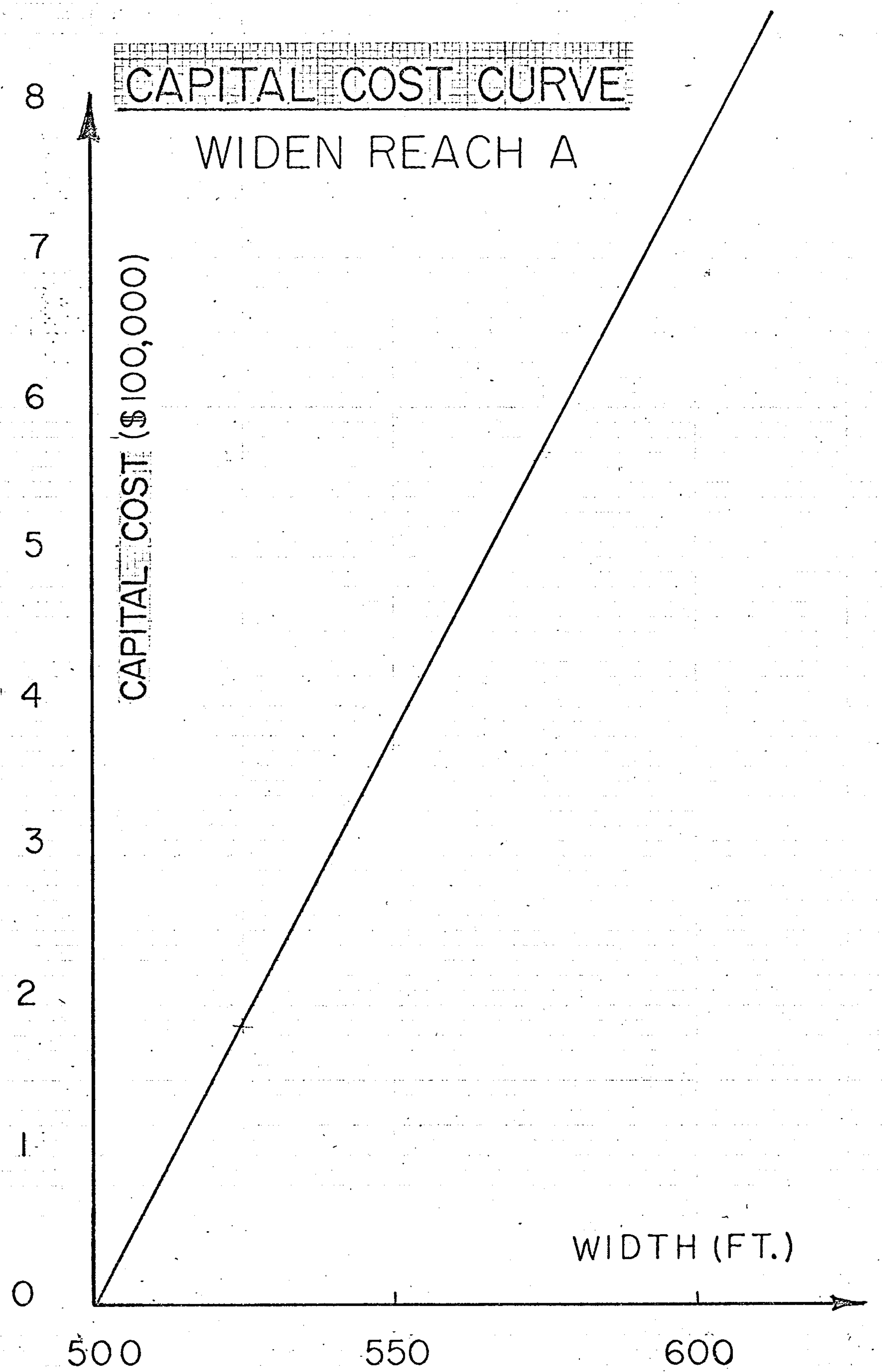

CAPITAL COST CURVE
WIDEN REACH A
CAPITAL COST ($100,000)
WIDTH (FT.)
0
1
2
3
4
5
6
7
8
500
550
600

FIG. 50

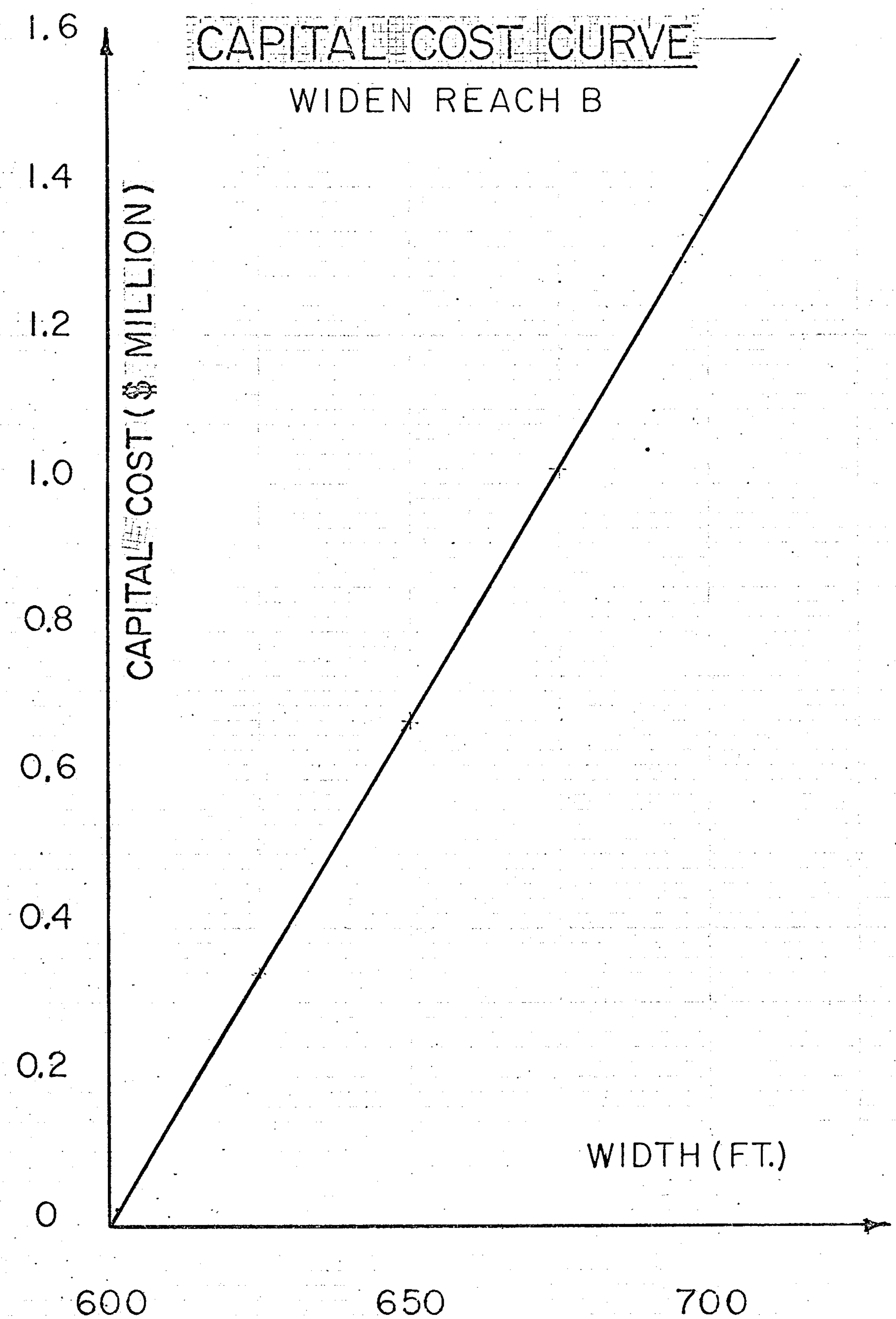
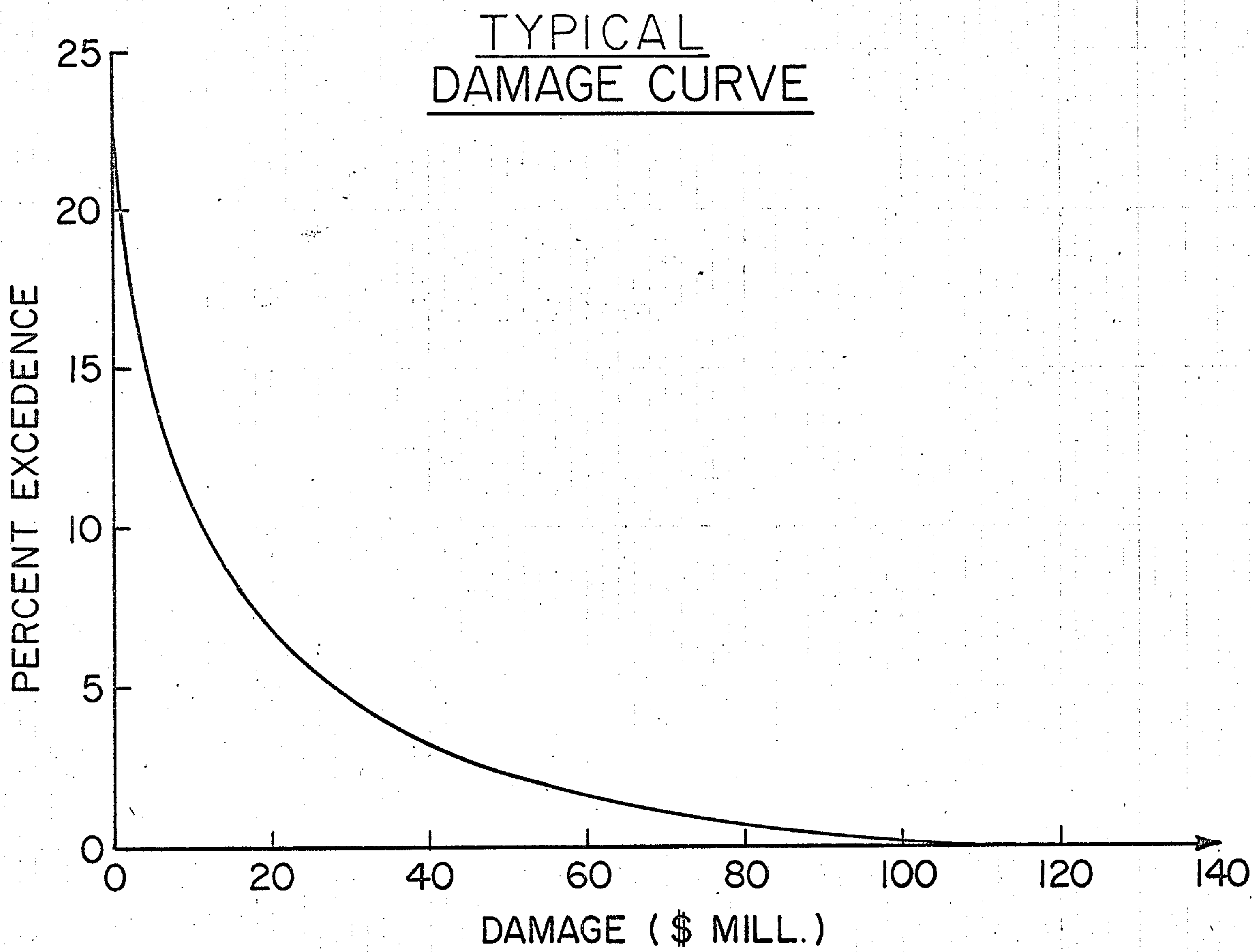

CAPITAL COST CURVE
WIDEN REACH B
CAPITAL COST ($ MILLION)
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
WIDTH (FT.)
600
650
700

FIG. 51



TYPICAL
DAMAGE CURVE
PERCENT EXCEDENCE
25
20
15
10
5
0
0
20
40
60
80
100
120
140
DAMAGE ($ MILL.)

FIG. 52

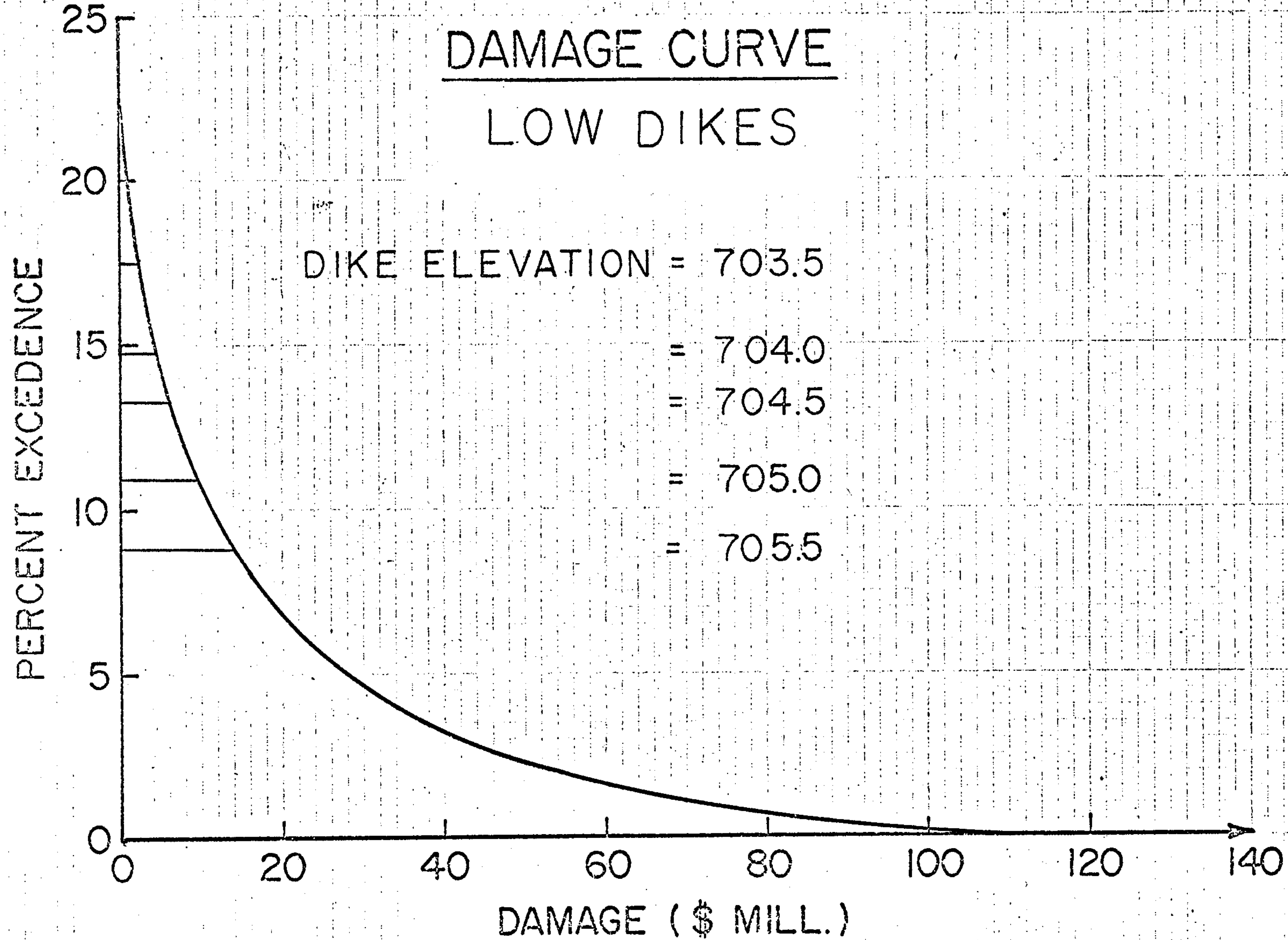

DAMAGE CURVE
LOW DIKES
DIKE ELEVATION = 703.5
= 704.0
= 704.5
= 705.0
= 705.5
PERCENT EXCEDENCE
25
20
15
10
5
0
0
20
40
60
80
100
120
140
DAMAGE ($ MILL.)

FIG. 53

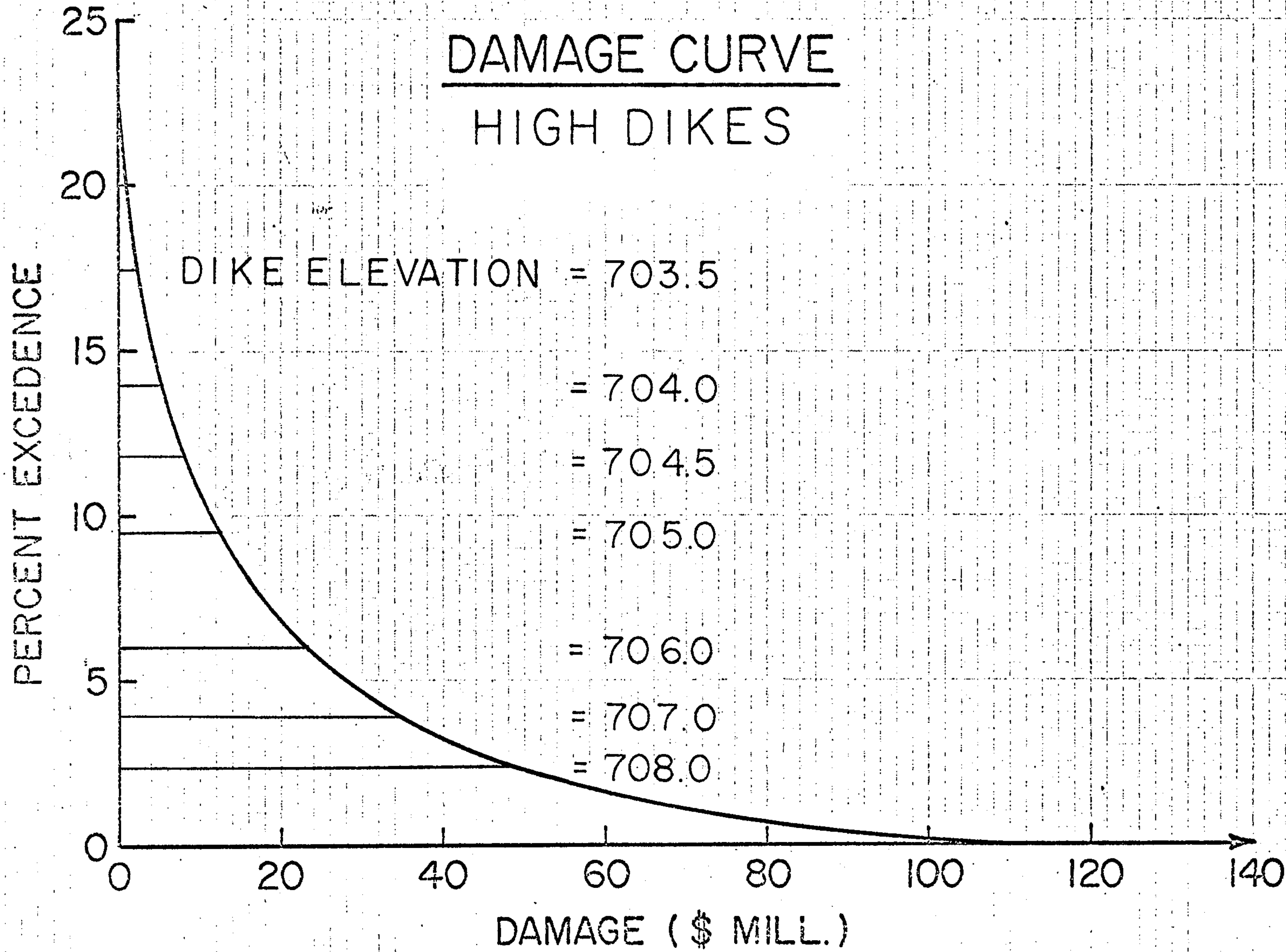
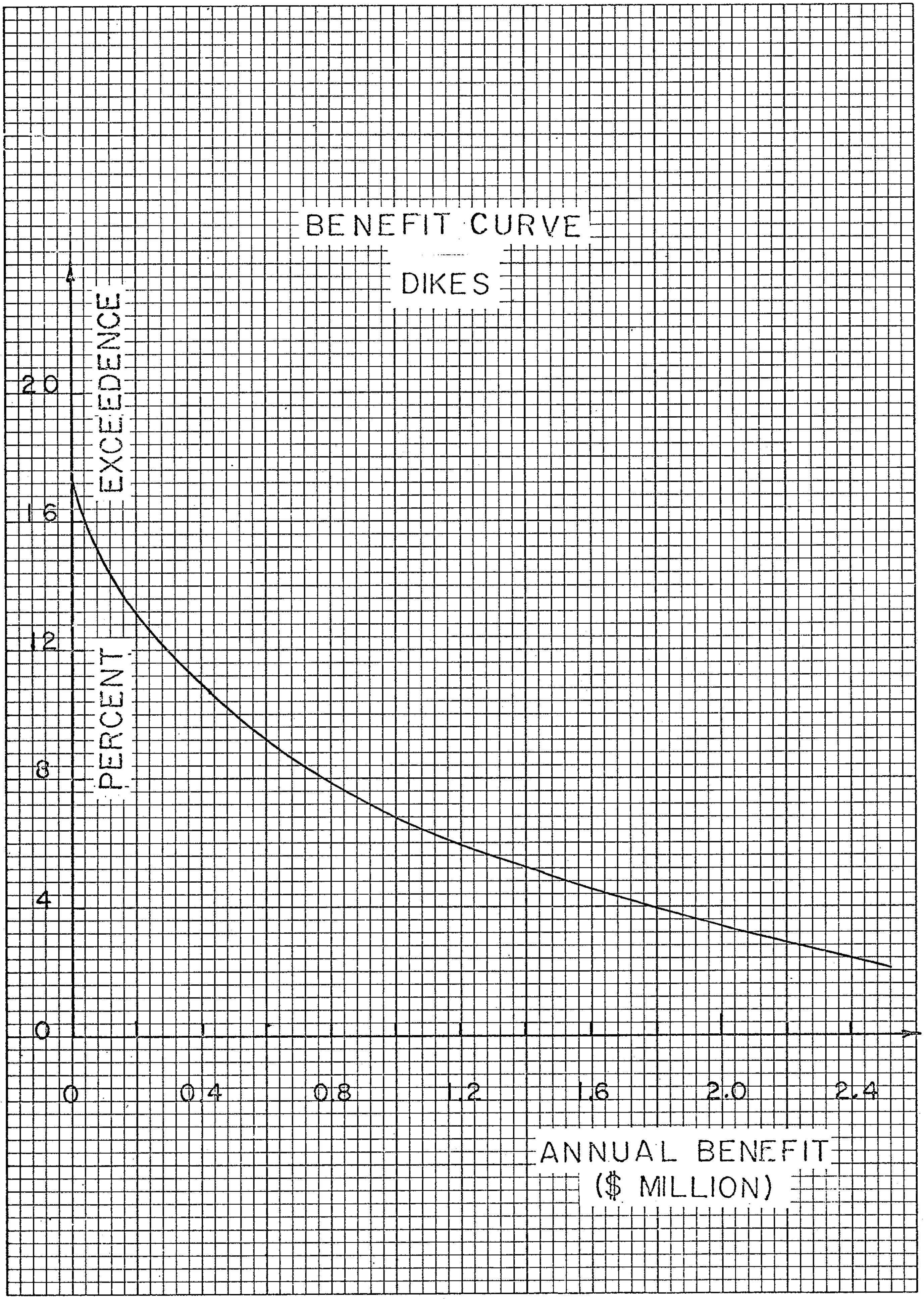

DAMAGE CURVE
HIGH DIKES
DIKE ELEVATION = 703.5
= 704.0
= 704.5
= 705.0
= 706.0
= 707.0
= 708.0
PERCENT EXCEDENCE
25
20
15
10
5
0
0
20
40
60
80
100
120
140
DAMAGE ($ MILL.)

FIG. 54



BENEFIT CURVE
DIKES
EXCEEDENCE PERCENT
20
16
12
8
4
0
0
0.4
0.8
1.2
1.6
2.0
2.4
ANNUAL BENEFIT ($ MILLION)

FIG. 55

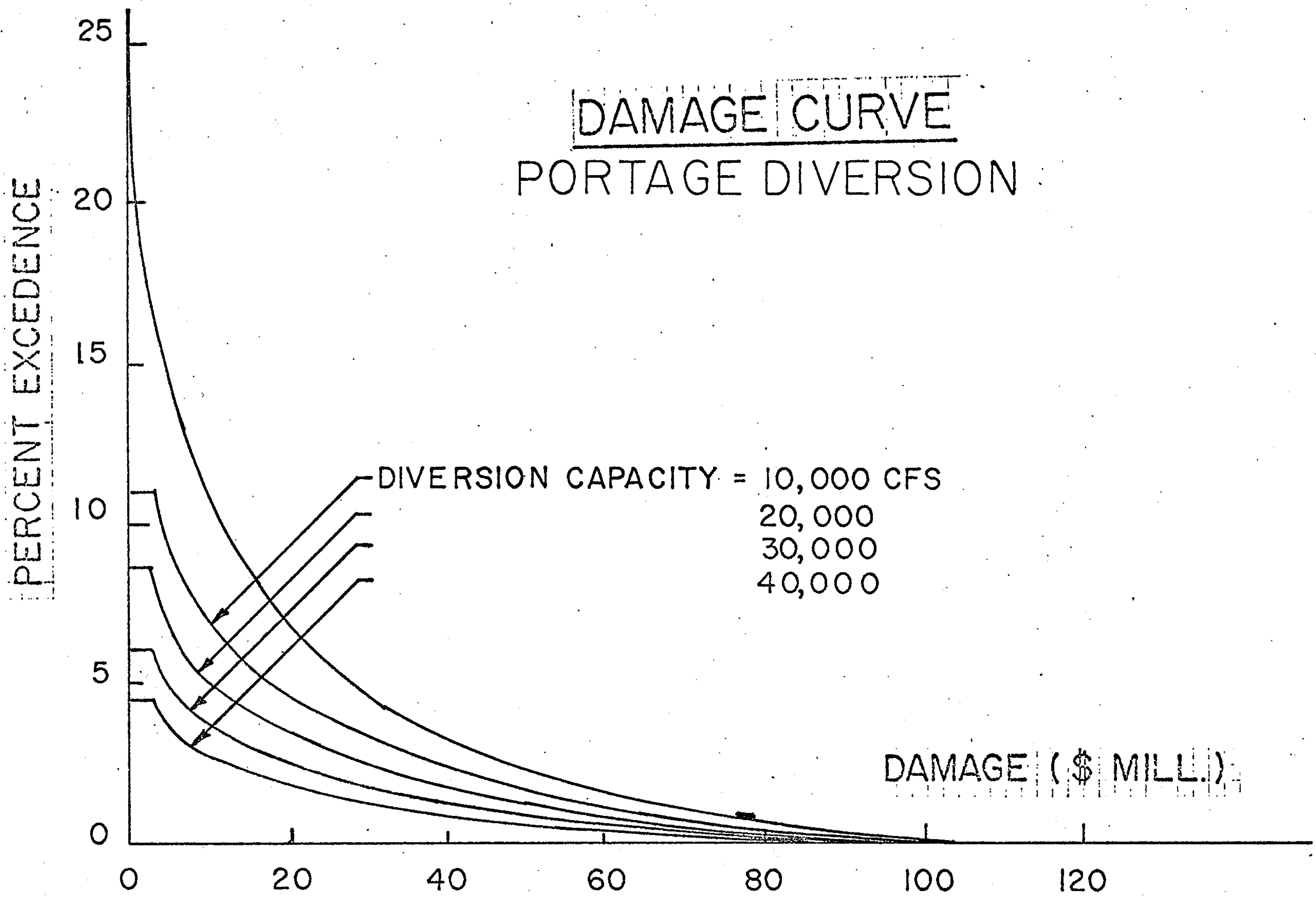

DAMAGE CURVE
PORTAGE DIVERSION
PERCENT EXCEDENCE
25
20
15
10
5
0
DIVERSION CAPACITY = 10,000 CFS
20,000
30,000
40,000
DAMAGE ($ MILL.)
0
20
40
60
80
100
120

FIG. 56

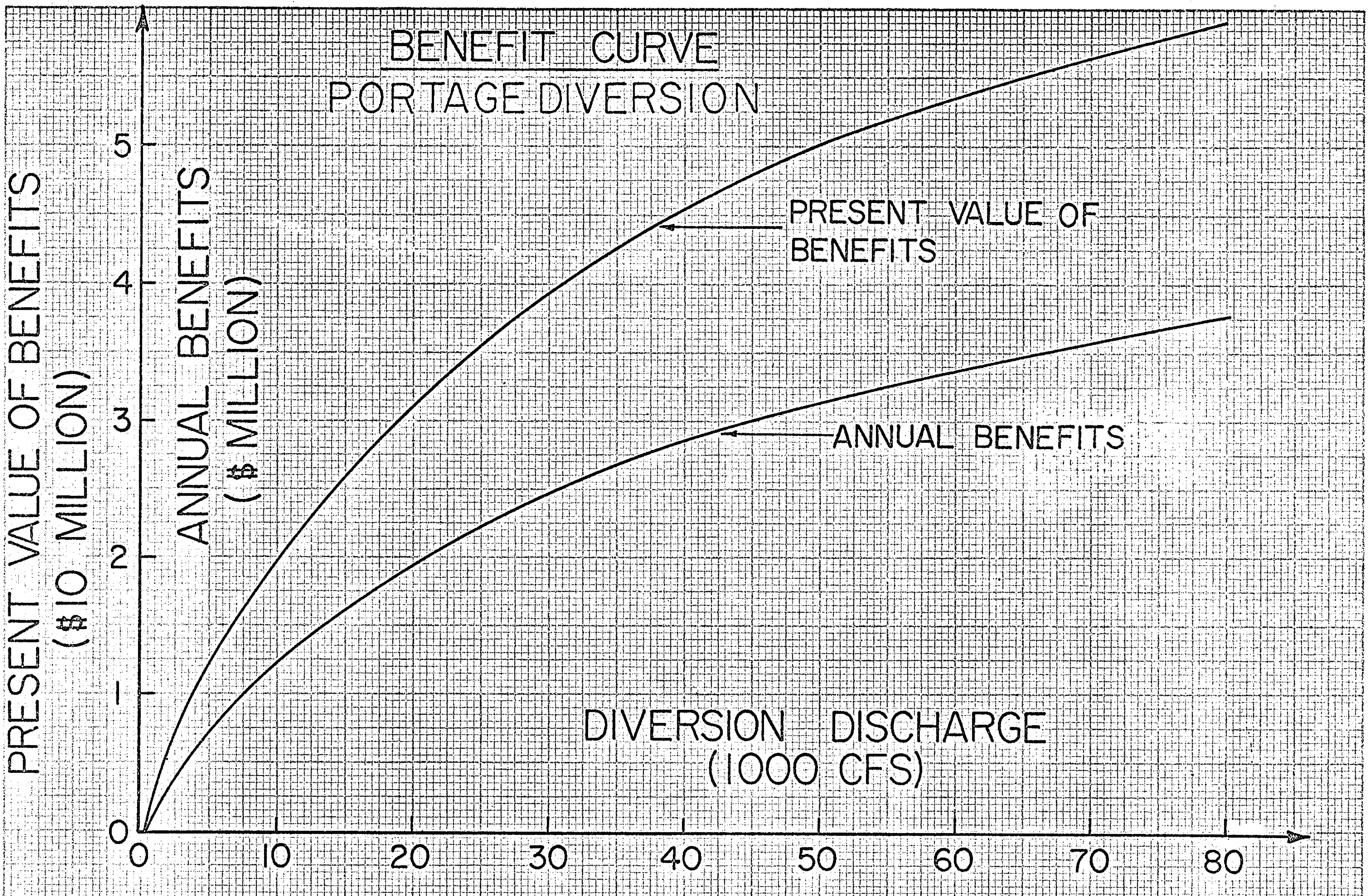

BENEFIT CURVE
PORTAGE DIVERSION
PRESENT VALUE OF BENEFITS ($10 MILLION)
ANNUAL BENEFITS ($ MILLION)
PRESENT VALUE OF BENEFITS
ANNUAL BENEFITS
DIVERSION DISCHARGE (1000 CFS)
0
1
2
3
4
5
0
10
20
30
40
50
60
70
80

FIG 58

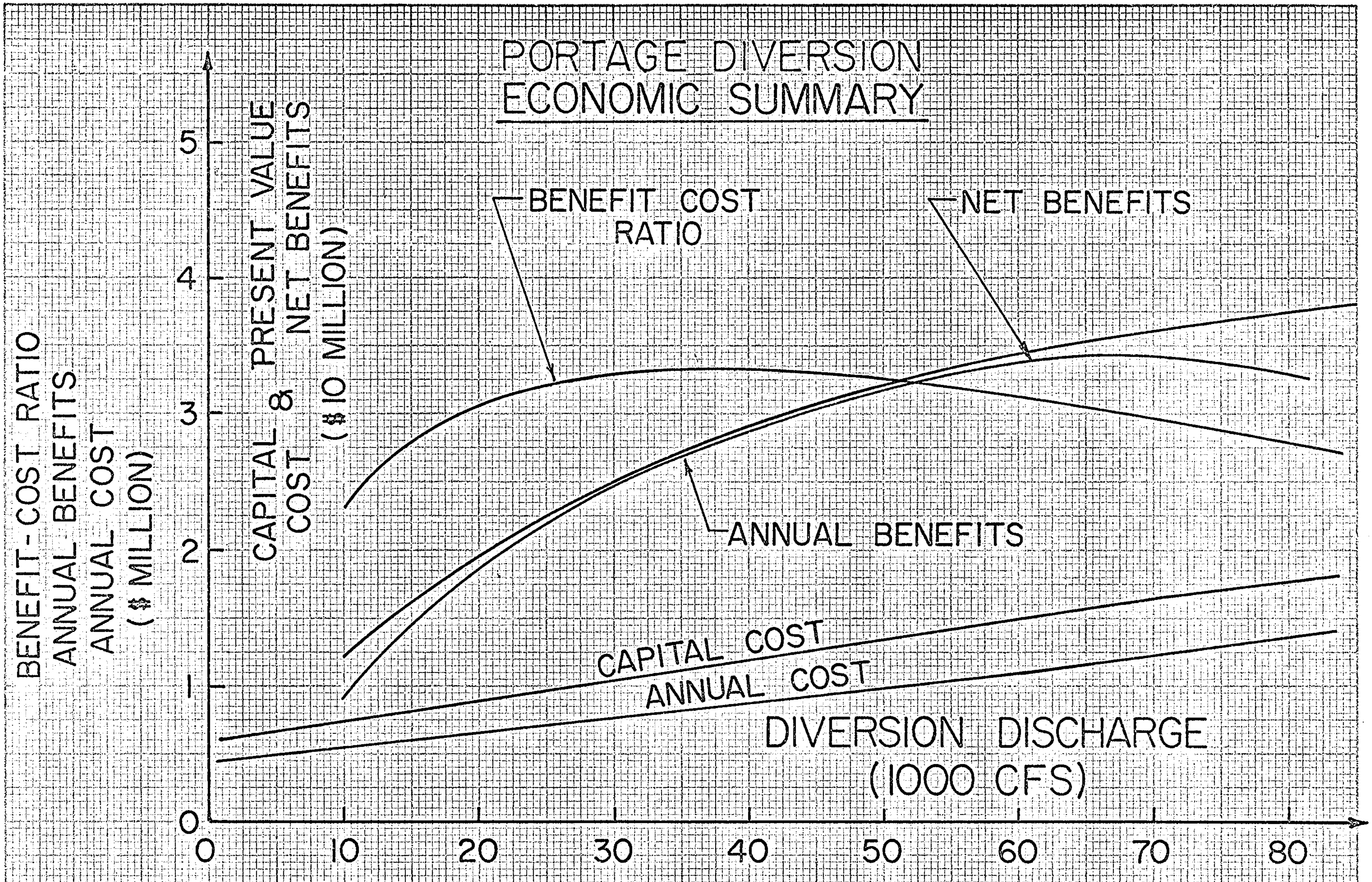
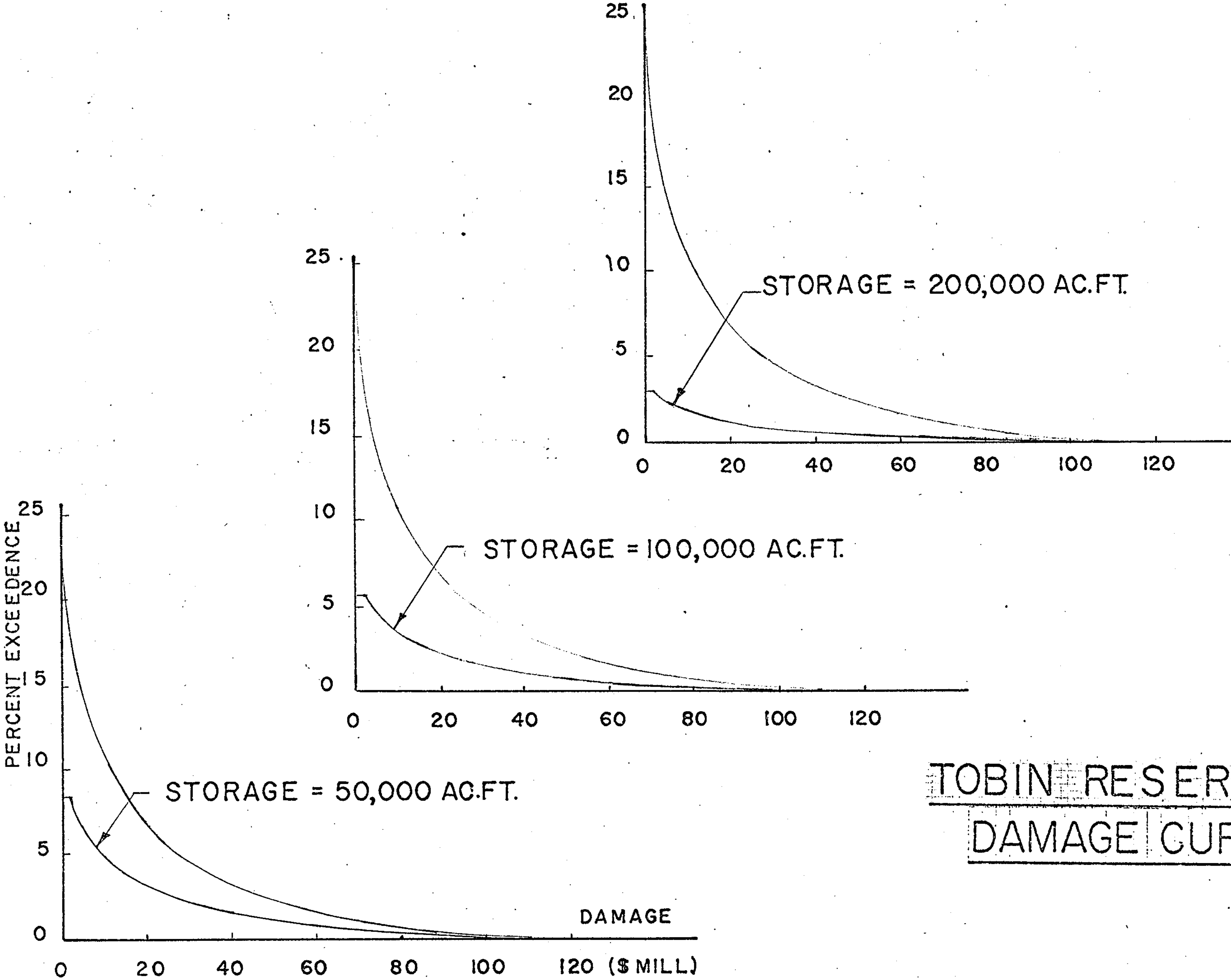
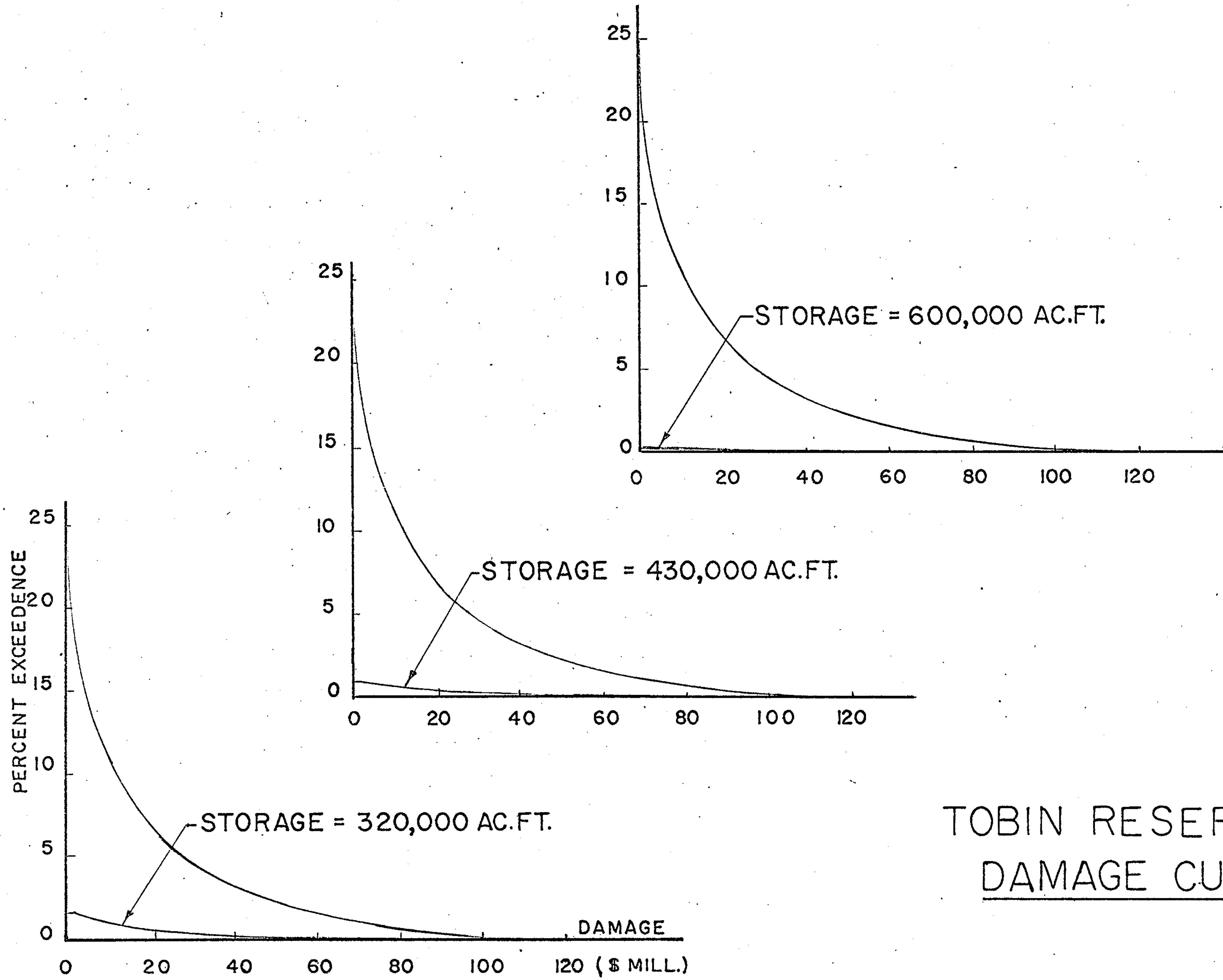

PORTAGE DIVERSION
ECONOMIC SUMMARY
BENEFIT-COST RATIO
ANNUAL BENEFITS
ANNUAL COST
($ MILLION)
CAPITAL & PRESENT VALUE
COST NET BENEFITS
($10 MILLION)
BENEFIT COST RATIO
NET BENEFITS
ANNUAL BENEFITS
CAPITAL COST
ANNUAL COST
DIVERSION DISCHARGE
(1000 CFS)
0
1
2
3
4
5
0
10
20
30
40
50
60
70
80

FIG 59



TOBIN RESERVOIR
DAMAGE CURVE

FIG. 60



TOBIN RESERVOIR DAMAGE CURVE

FIG. 61

TOBIN RESERVOIR
BENEFIT CURVE

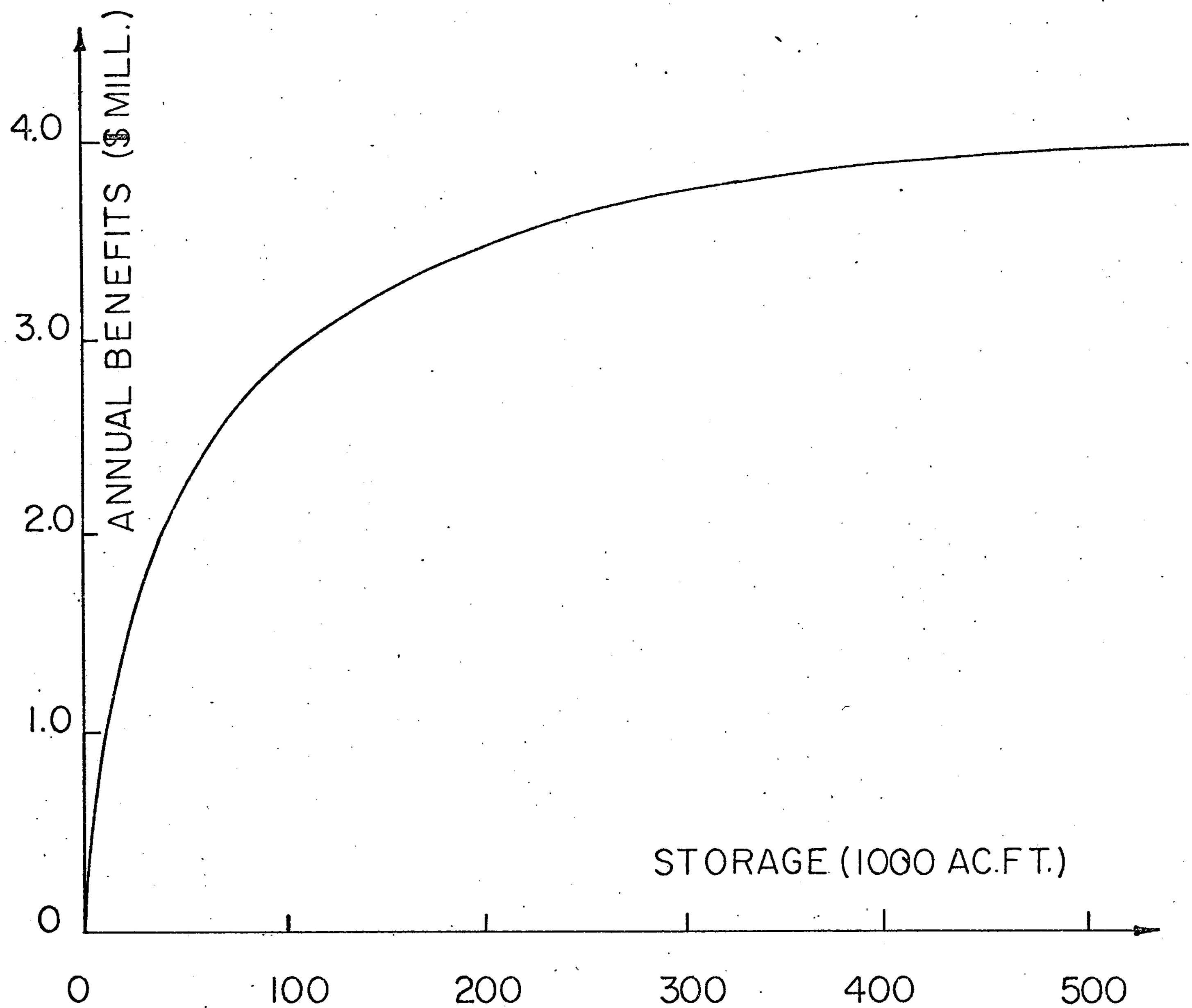


FIG. 62

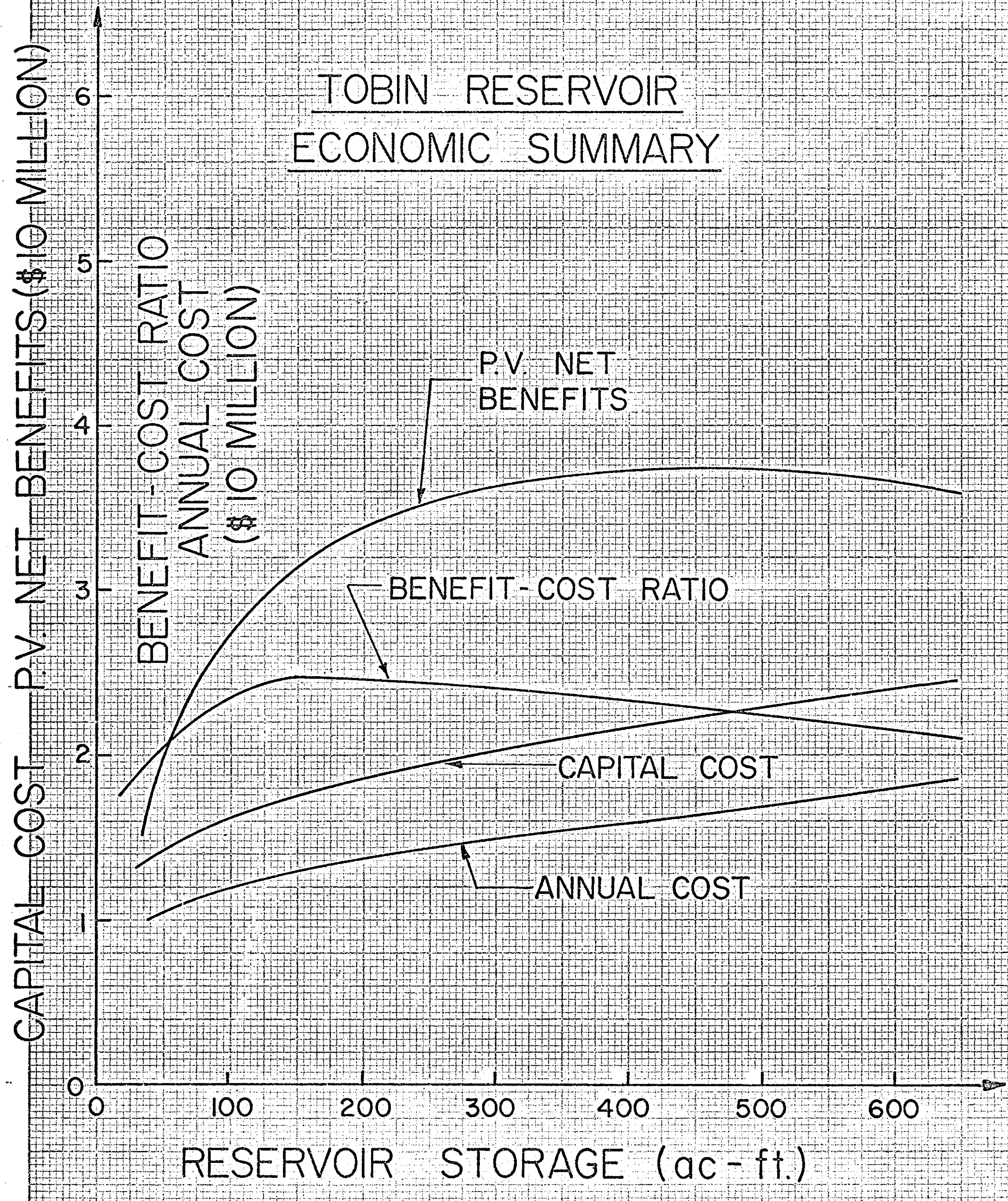

TOBIN RESERVOIR
ECONOMIC SUMMARY
CAPITAL COST P.V. NET BENEFITS ($10 MILLION)
BENEFIT-COST RATIO
ANNUAL COST
($10 MILLION)
P.V. NET
BENEFITS
BENEFIT-COST RATIO
CAPITAL COST
ANNUAL COST
0
1
2
3
4
5
6
0
100
200
300
400
500
600
RESERVOIR STORAGE (ac-ft.)

FIG 63

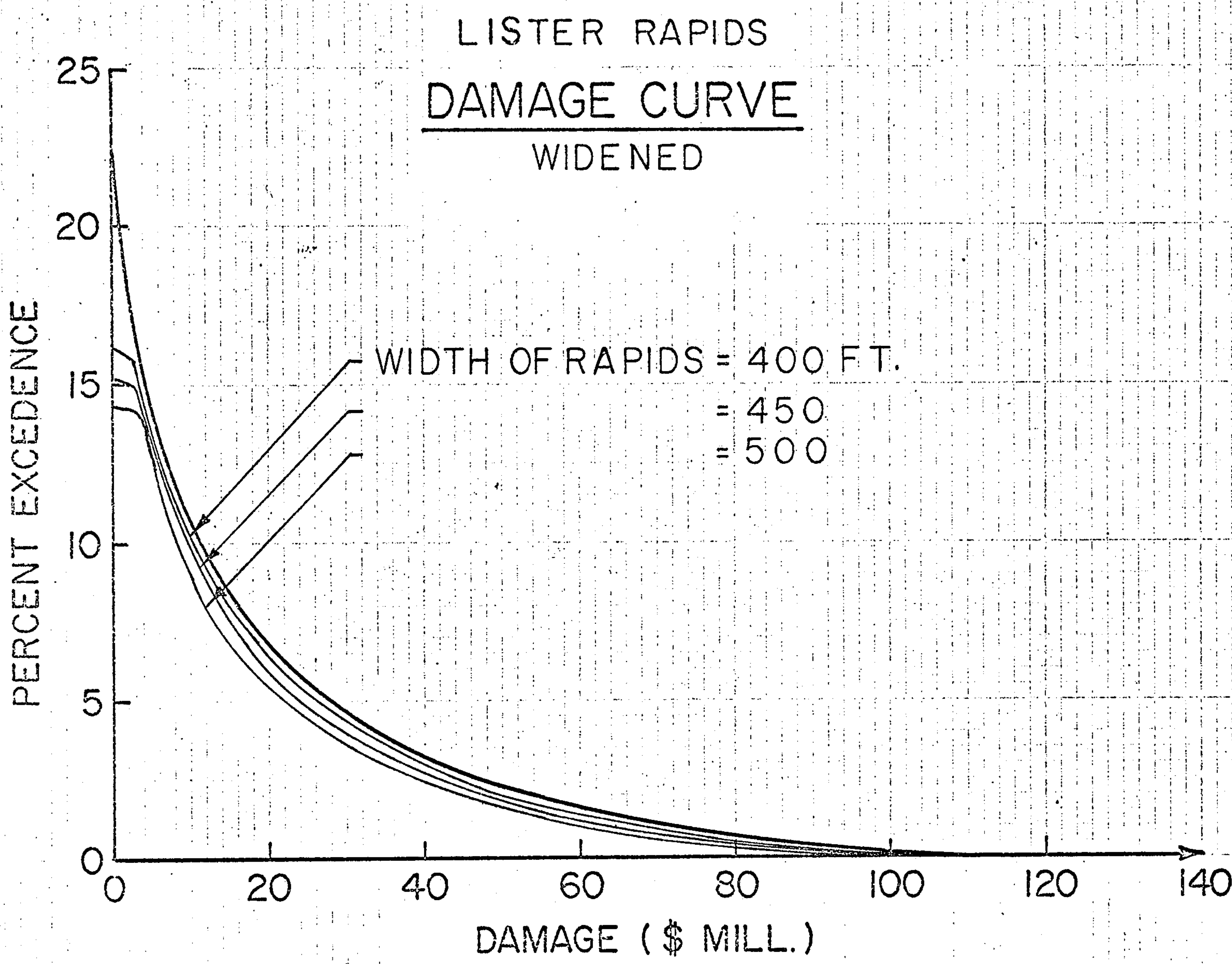
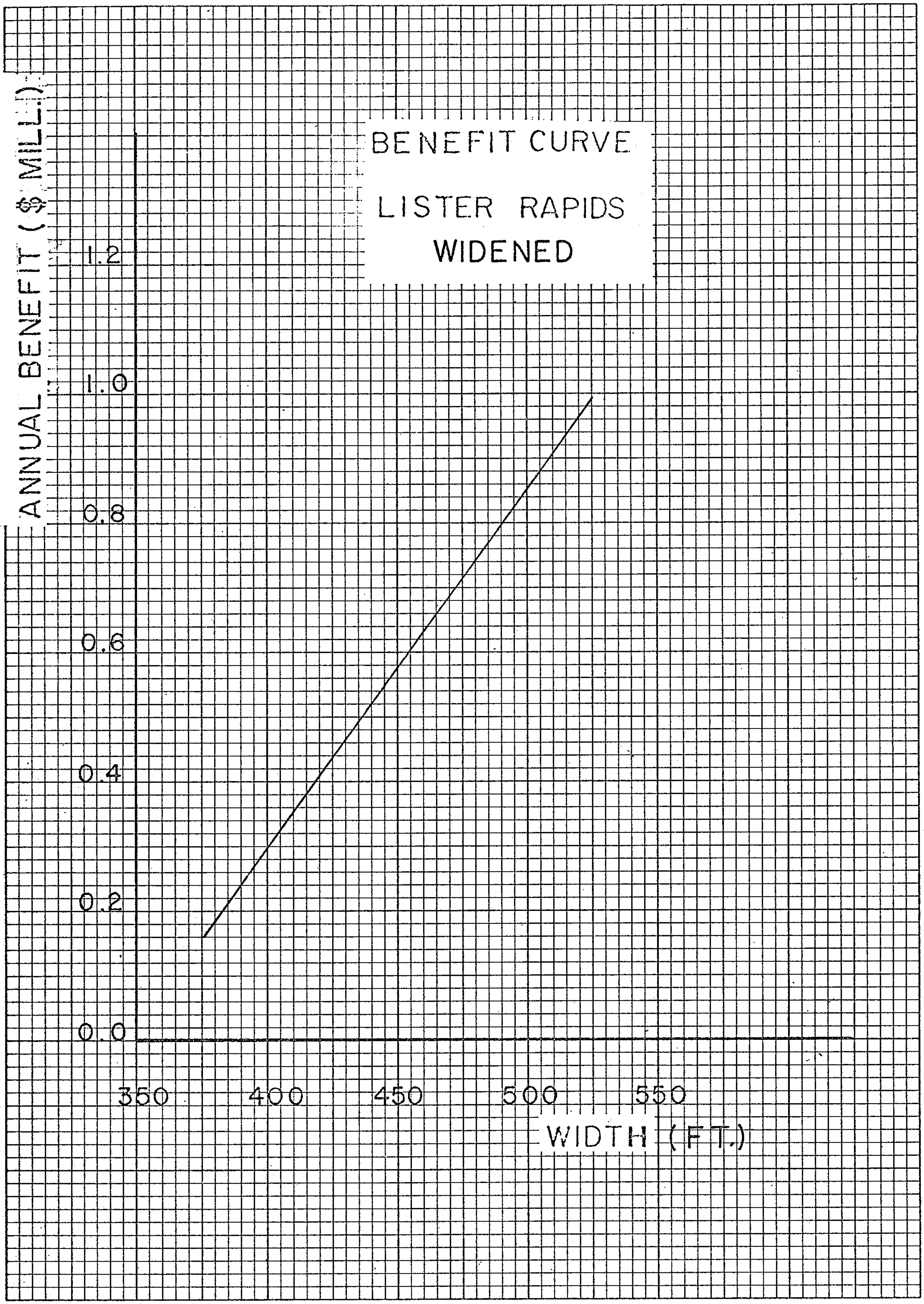

LISTER RAPIDS
DAMAGE CURVE
WIDENED
WIDTH OF RAPIDS = 400 FT.
= 450
= 500
PERCENT EXCEDENCE
25
20
15
10
5
0
0
20
40
60
80
100
120
140
DAMAGE ($ MILL.)

FIG. 64



BENEFIT CURVE
LISTER RAPIDS
WIDENED
ANNUAL BENEFIT ($ MILL.)
1.2
1.0
0.8
0.6
0.4
0.2
0.0
350
400
450
500
550
WIDTH (FT.)

FIG. 65

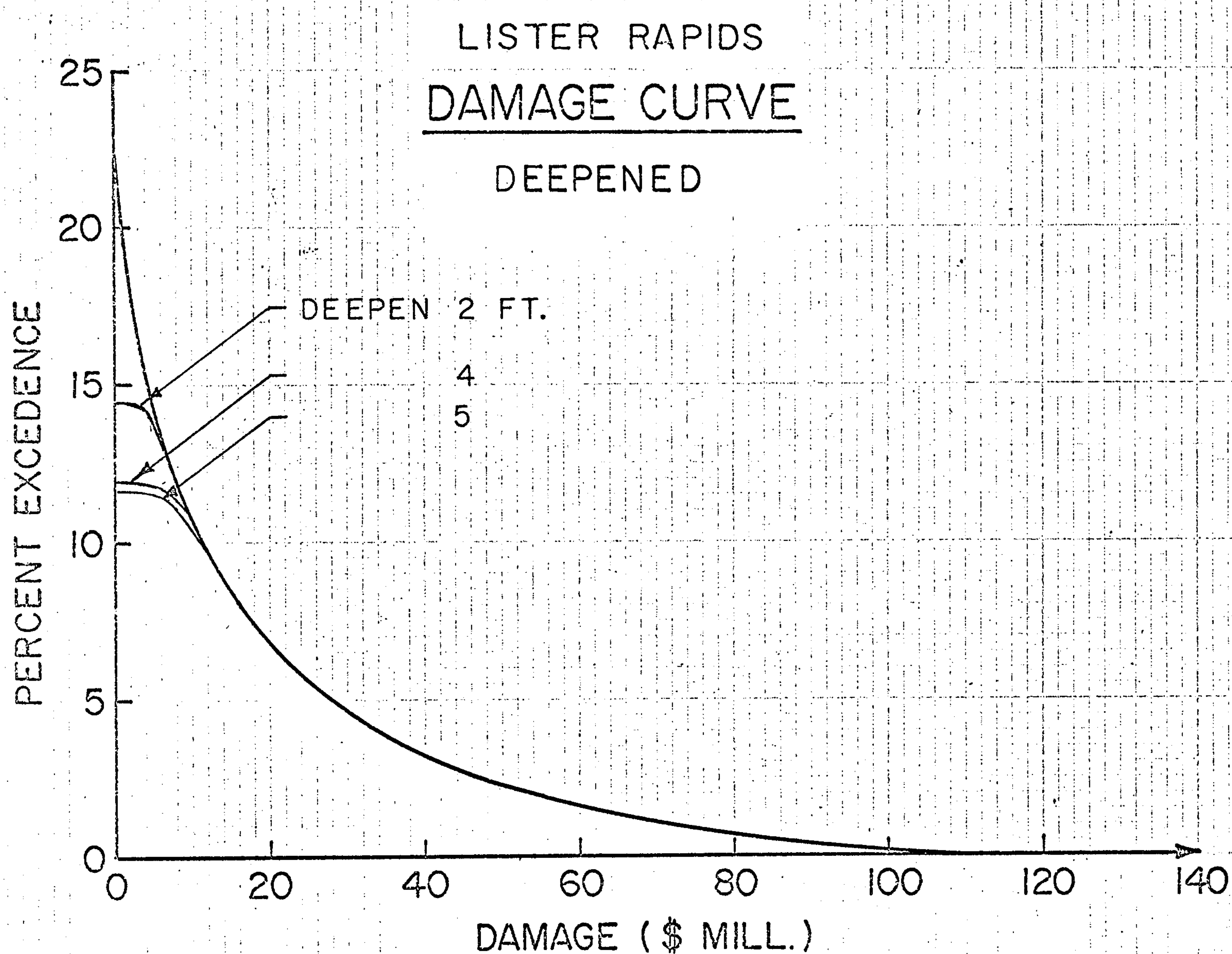

LISTER RAPIDS
DAMAGE CURVE
DEEPENED
DEEPEN 2 FT.
4
5
PERCENT EXCEDENCE
25
20
15
10
5
0
0
20
40
60
80
100
120
140
DAMAGE ($ MILL.)

FIG. 66

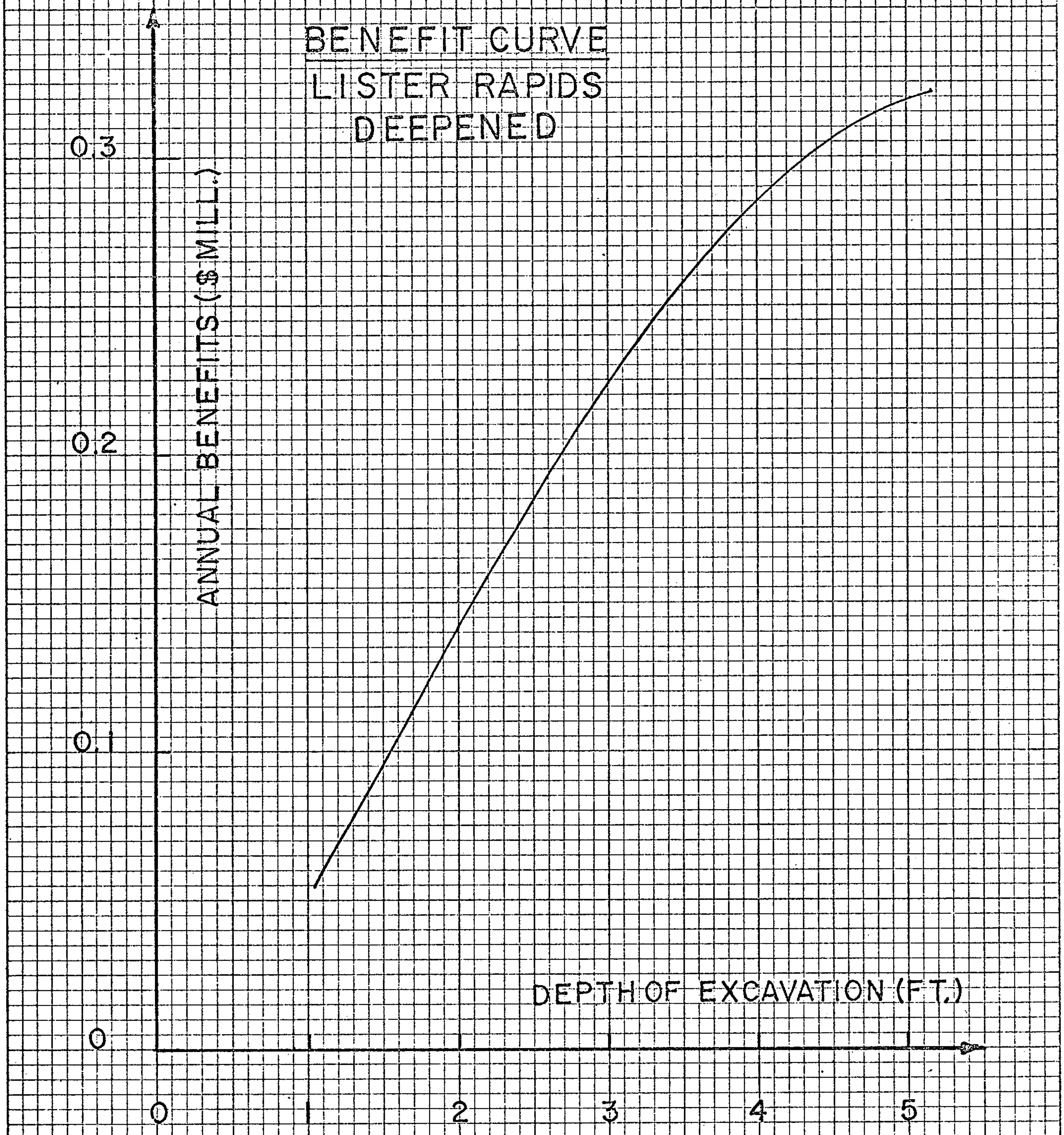

BENEFIT CURVE
LISTER RAPIDS
DEEPENED
ANNUAL BENEFITS ($ MILL.)
0.3
0.2
0.1
0
DEPTH OF EXCAVATION (FT.)
0
1
2
3
4
5

FIG. 67

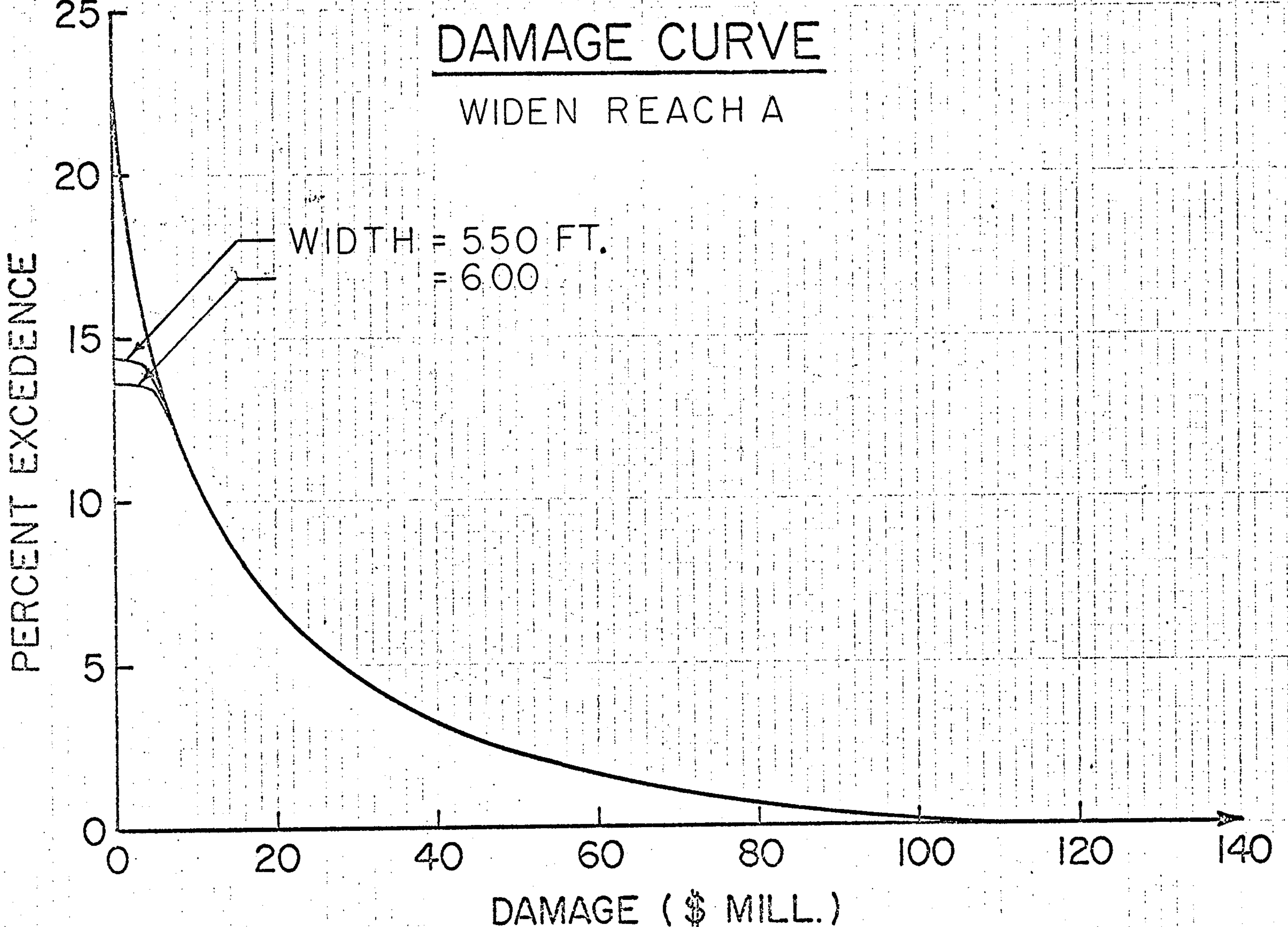
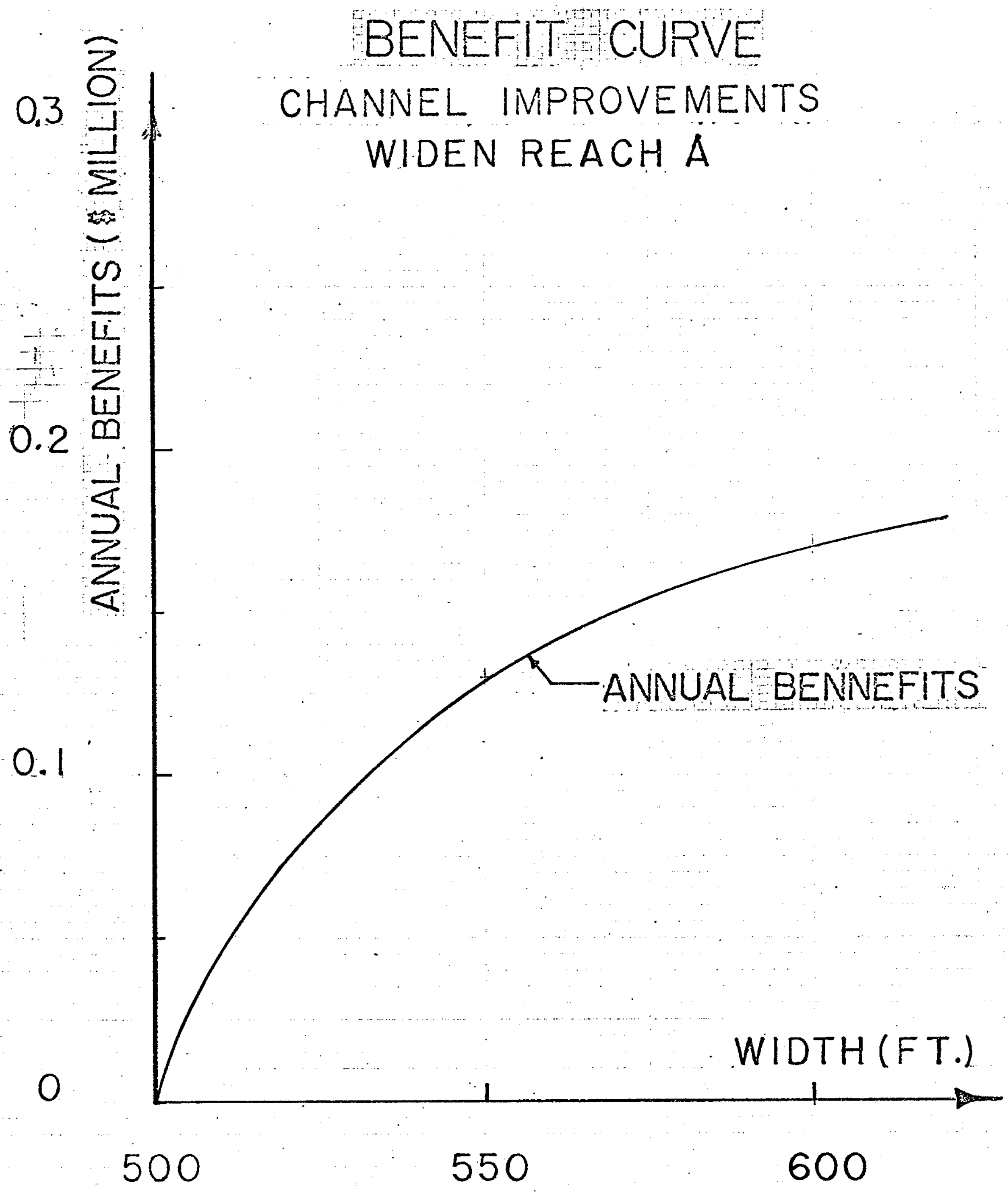

DAMAGE CURVE
WIDEN REACH A
WIDTH = 550 FT.
= 600
PERCENT EXCEDENCE
25
20
15
10
5
0
0
20
40
60
80
100
120
140
DAMAGE ($ MILL.)

FIG. 68



BENEFIT CURVE
CHANNEL IMPROVEMENTS
WIDEN REACH A
ANNUAL BENEFITS ($ MILLION)
0.3
0.2
0.1
0
ANNUAL BENNEFITS
WIDTH (FT.)
500
550
600

FIG. 69

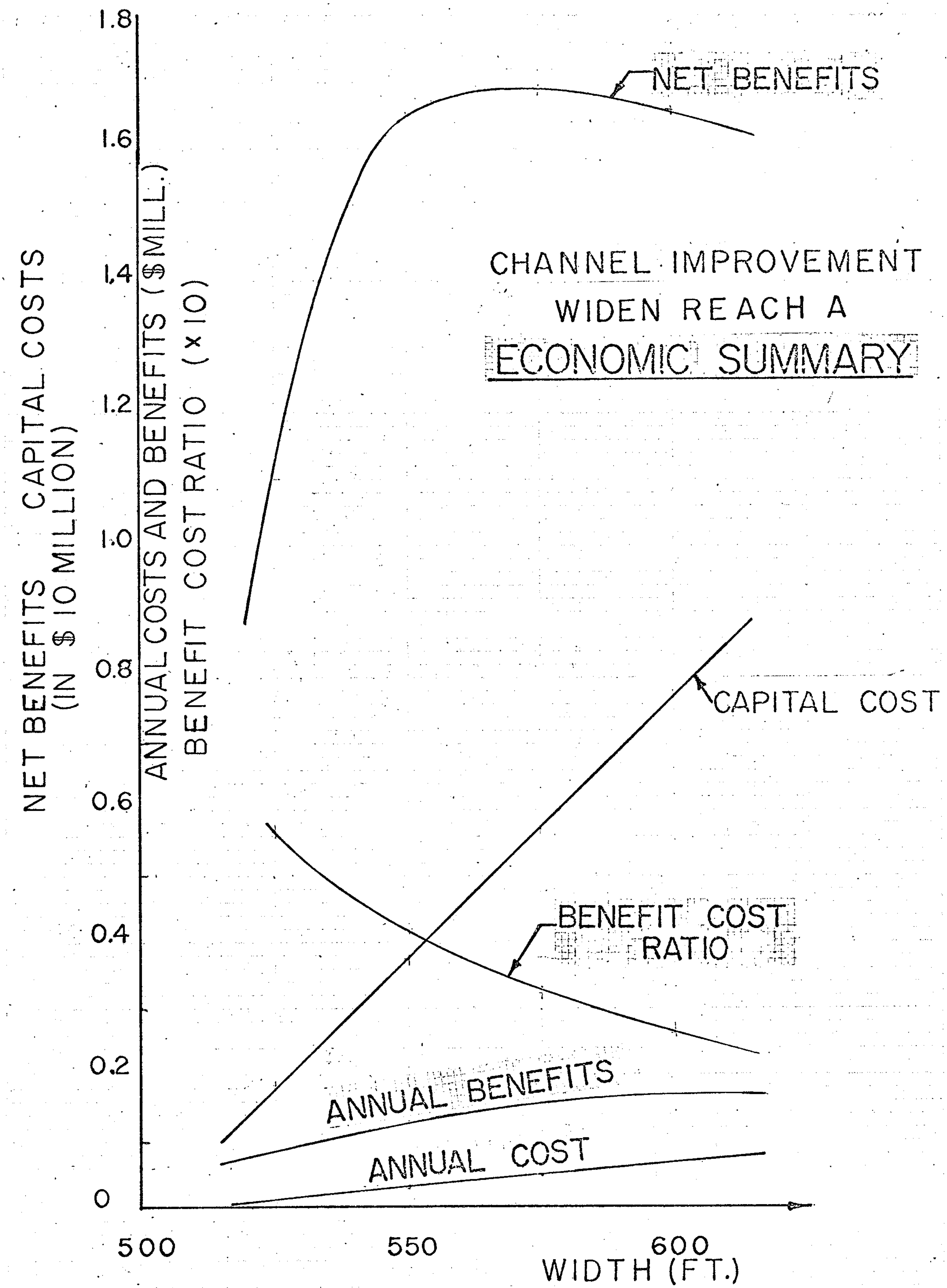

NET BENEFITS
CHANNEL IMPROVEMENT
WIDEN REACH A
ECONOMIC SUMMARY
CAPITAL COST
BENEFIT COST RATIO
ANNUAL BENEFITS
ANNUAL COST
NET BENEFITS CAPITAL COSTS (IN $ 10 MILLION)
ANNUAL COSTS AND BENEFITS ($ MILL.)
BENEFIT COST RATIO (x 10)
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
500
550
600
WIDTH (FT.)

FIG. 70

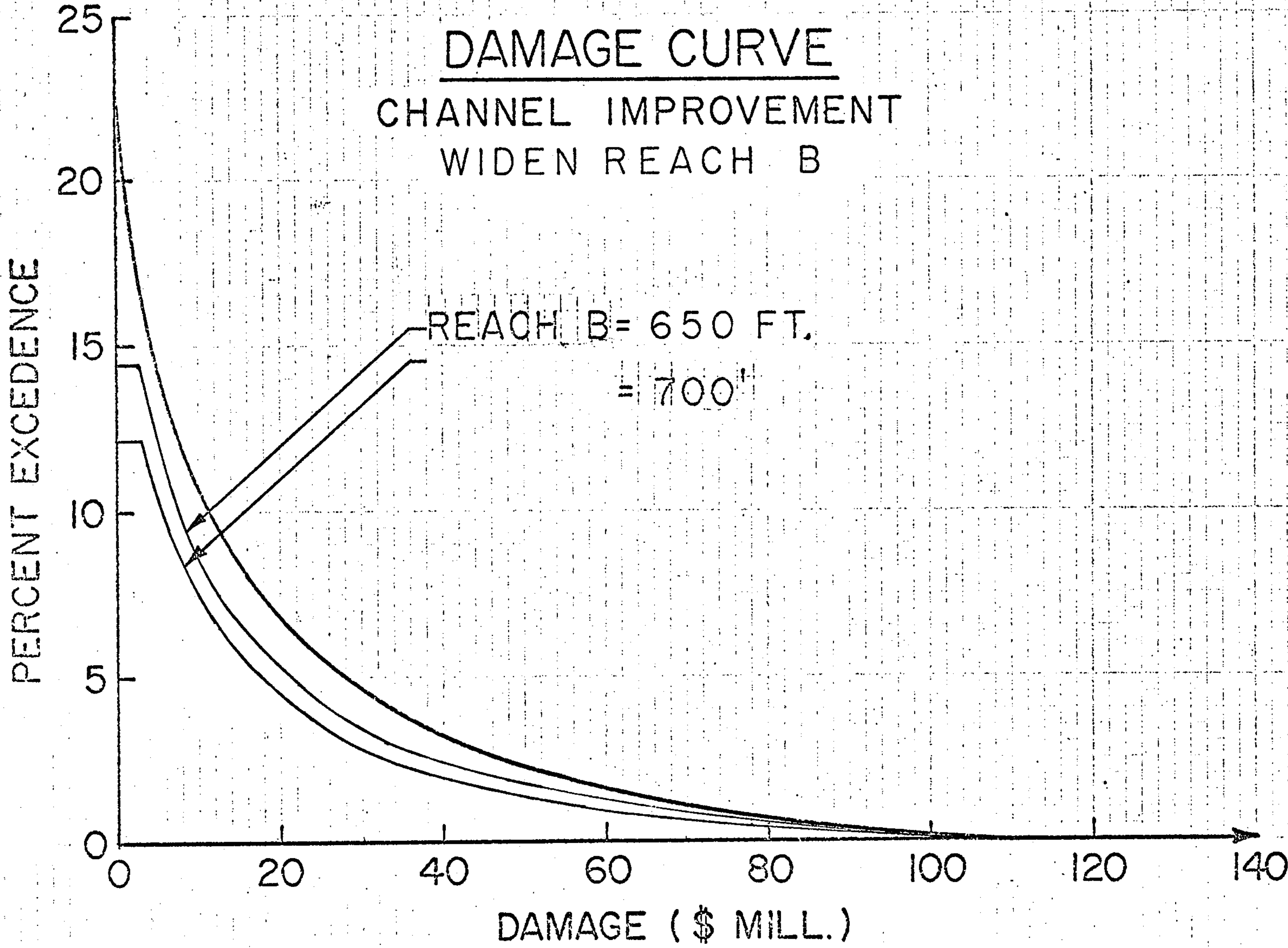

DAMAGE CURVE
CHANNEL IMPROVEMENT
WIDEN REACH B
REACH B= 650 FT.
= 700'
PERCENT EXCEDENCE
25
20
15
10
5
0
0
20
40
60
80
100
120
140
DAMAGE ($ MILL.)

FIG. 71

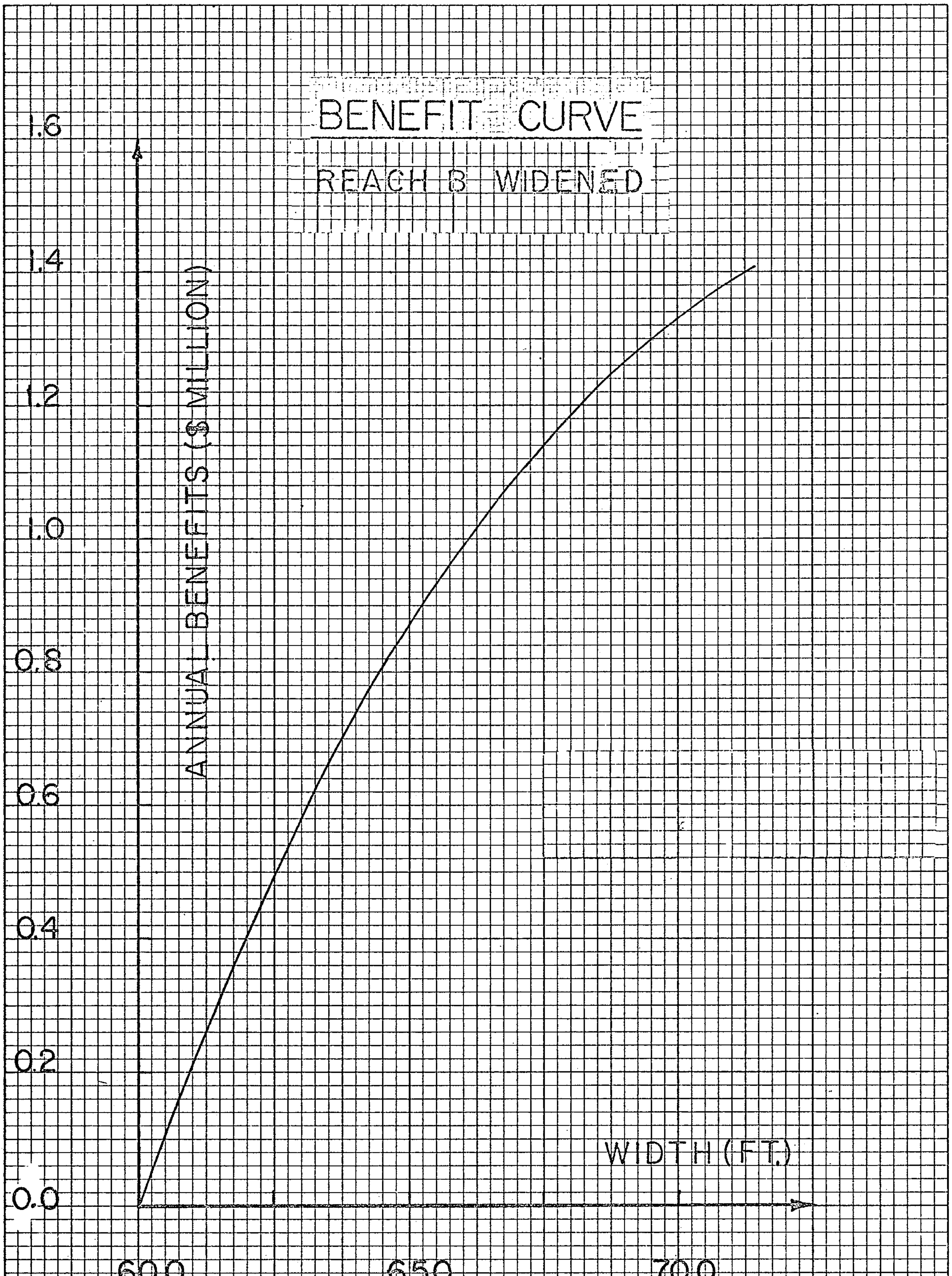
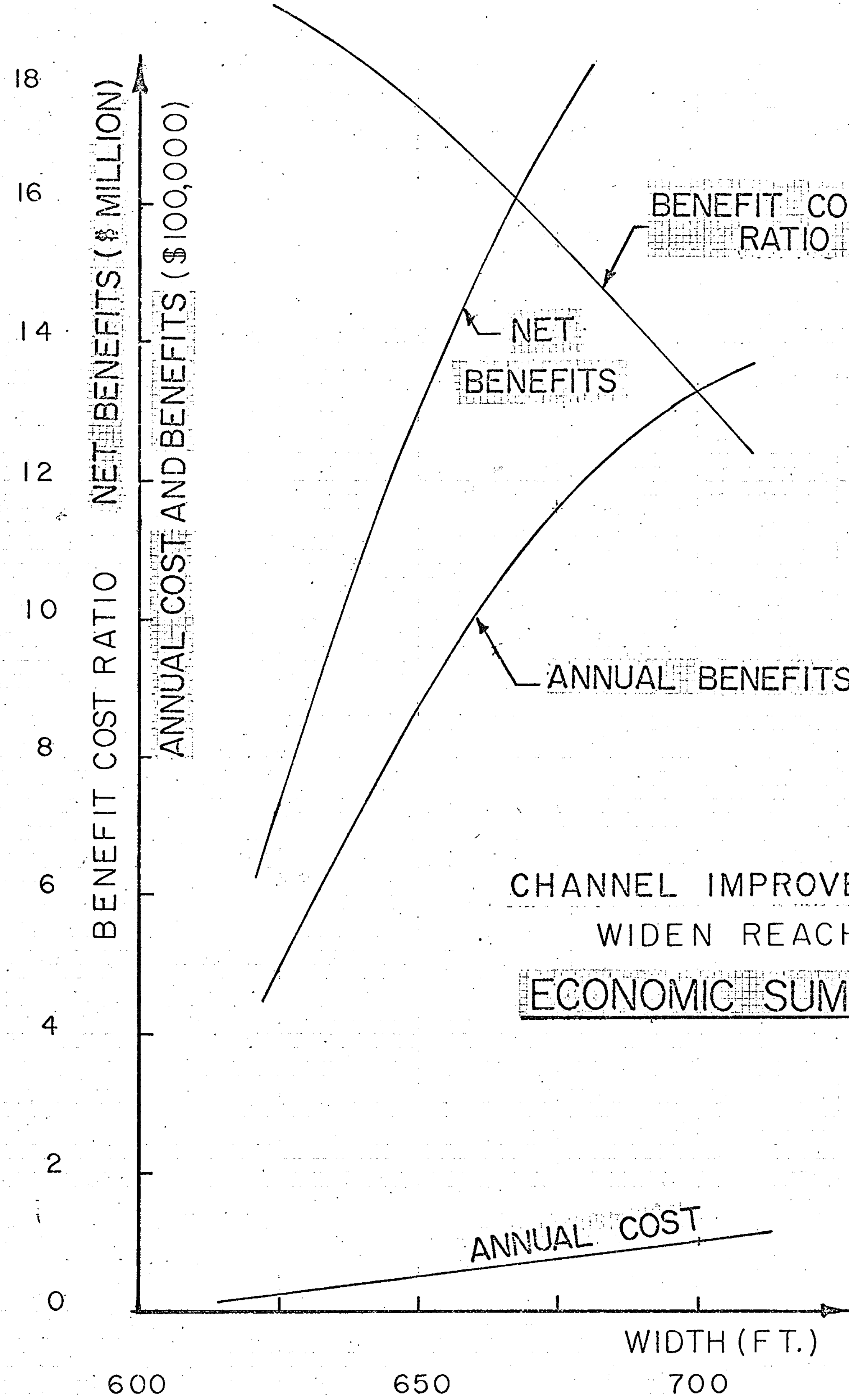

BENEFIT CURVE
REACH B WIDENED
ANNUAL BENEFITS ($ MILLION)
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
WIDTH (FT.)
600
650
700

FIG. 72



CHANNEL IMPROVEMENT
WIDEN REACH B
ECONOMIC SUMMARY

FIG. 73

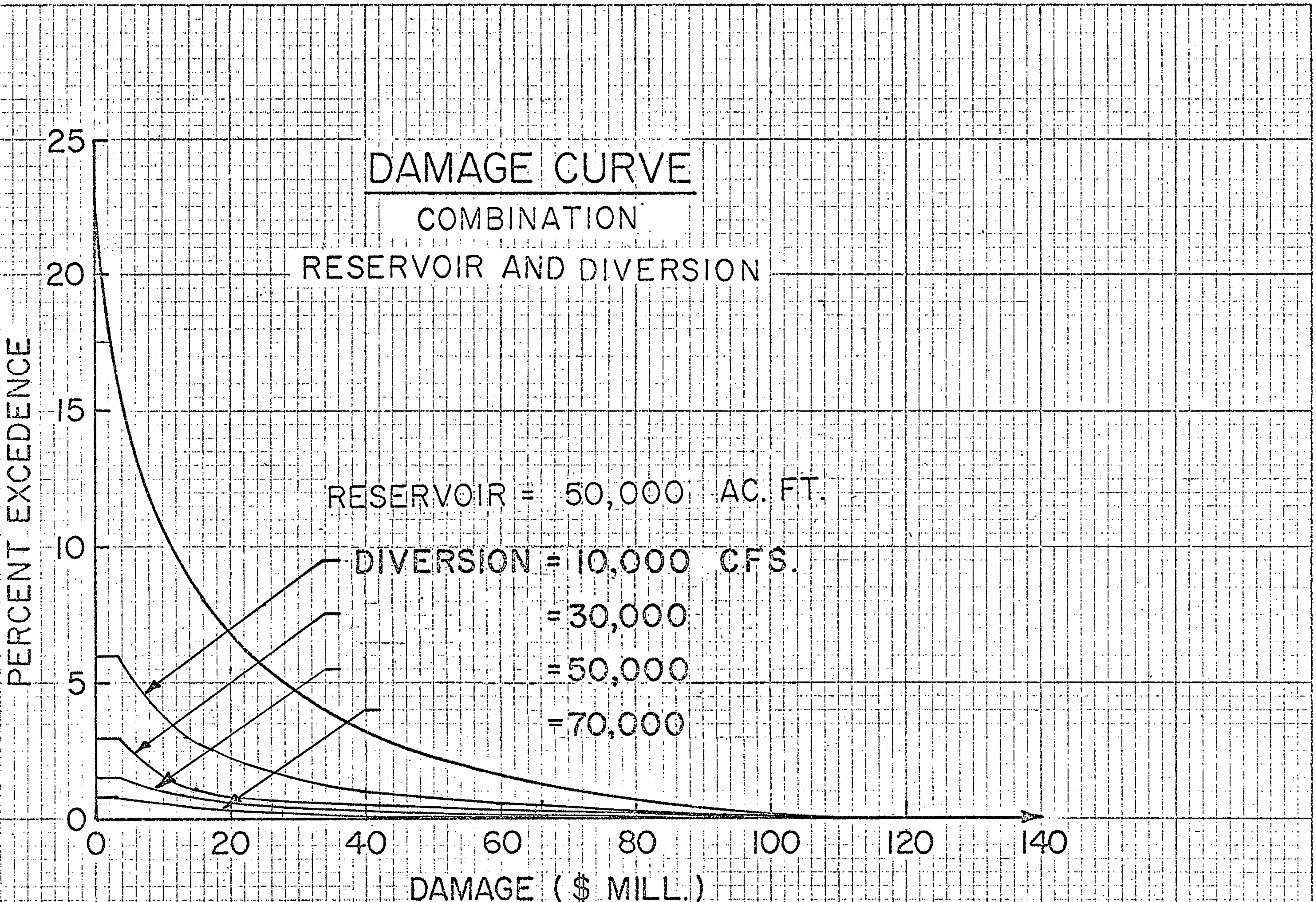

DAMAGE CURVE
COMBINATION
RESERVOIR AND DIVERSION
RESERVOIR = 50,000 AC. FT.
DIVERSION = 10,000 CFS.
= 30,000
= 50,000
= 70,000
PERCENT EXCEDENCE
25
20
15
10
5
0
0
20
40
60
80
100
120
140
DAMAGE ($ MILL.)

FIG. 74

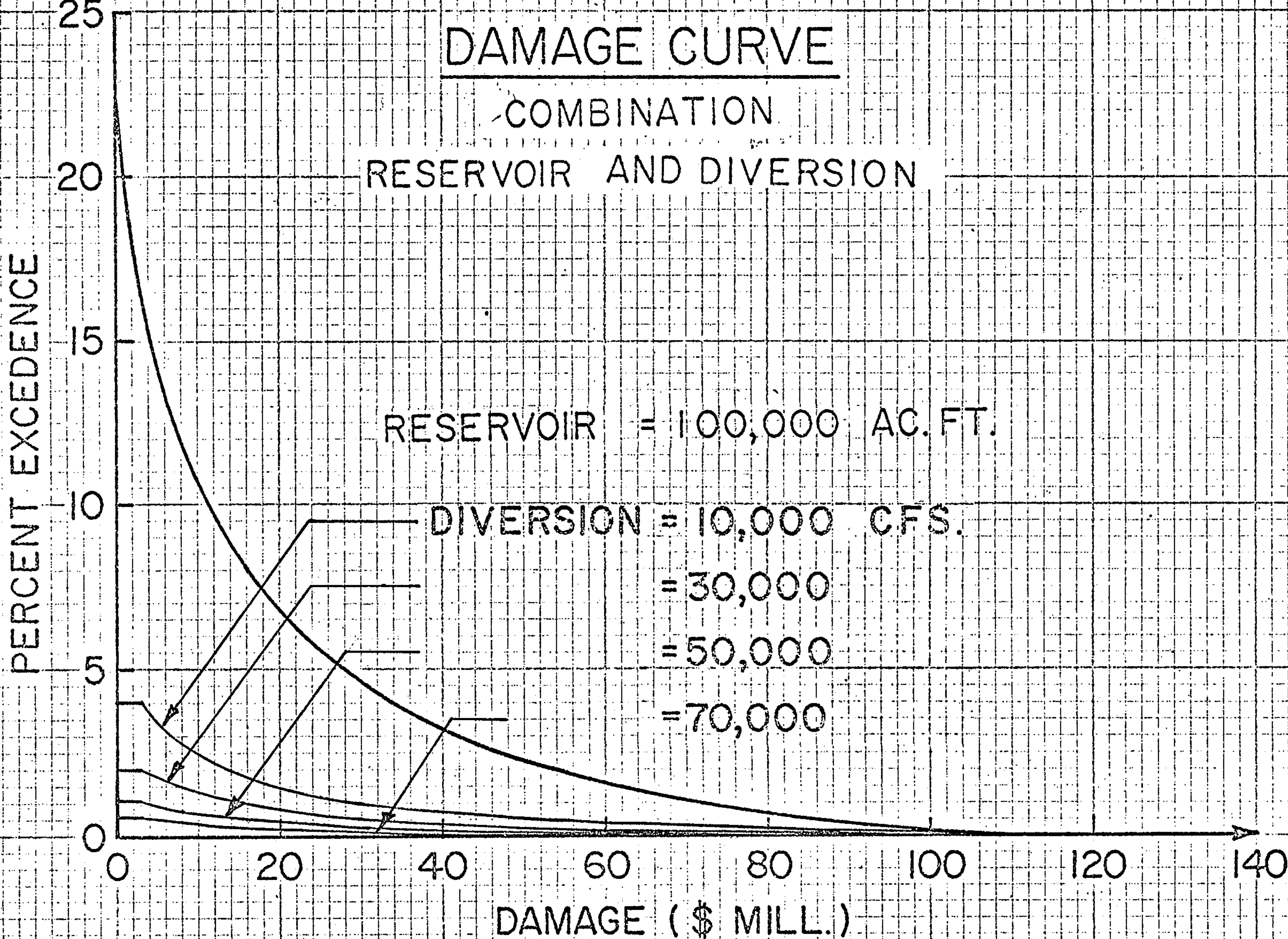

DAMAGE CURVE
COMBINATION
RESERVOIR AND DIVERSION
RESERVOIR = 100,000 AC. FT.
DIVERSION = 10,000 CFS.
=30,000
=50,000
=70,000
PERCENT EXCEDENCE
0
5
10
15
20
25
0
20
40
60
80
100
120
140
DAMAGE ($ MILL.)

FIG. 75

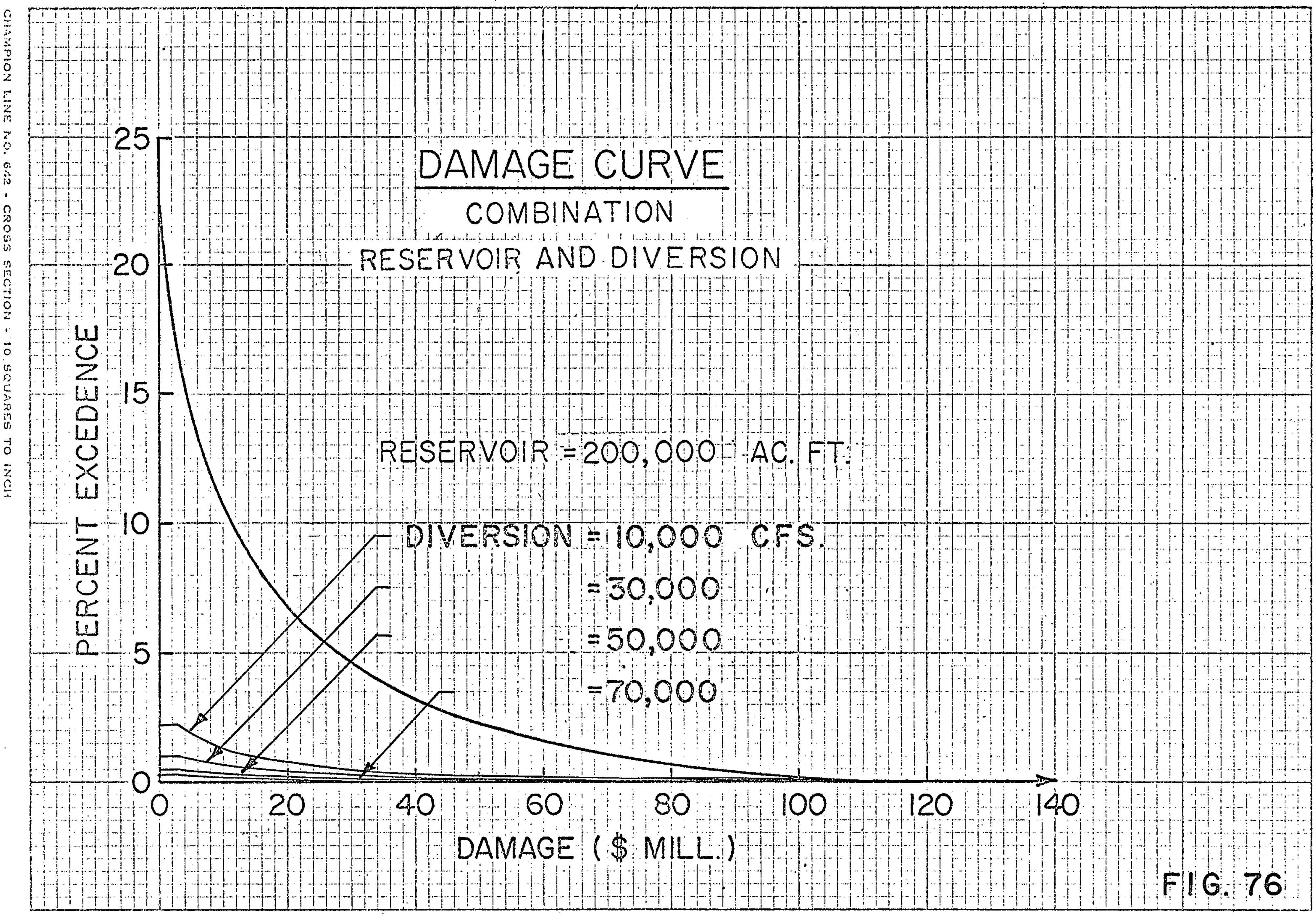

DAMAGE CURVE
COMBINATION
RESERVOIR AND DIVERSION
RESERVOIR = 200,000 AC. FT.
DIVERSION = 10,000 CFS.
= 30,000
= 50,000
= 70,000
PERCENT EXCEDENCE
DAMAGE ($ MILL.)
0
5
10
15
20
25
0
20
40
60
80
100
120
140
CHAMPION LINE NO. 642 - CROSS SECTION - 10 SQUARES TO INCH

FIG. 76

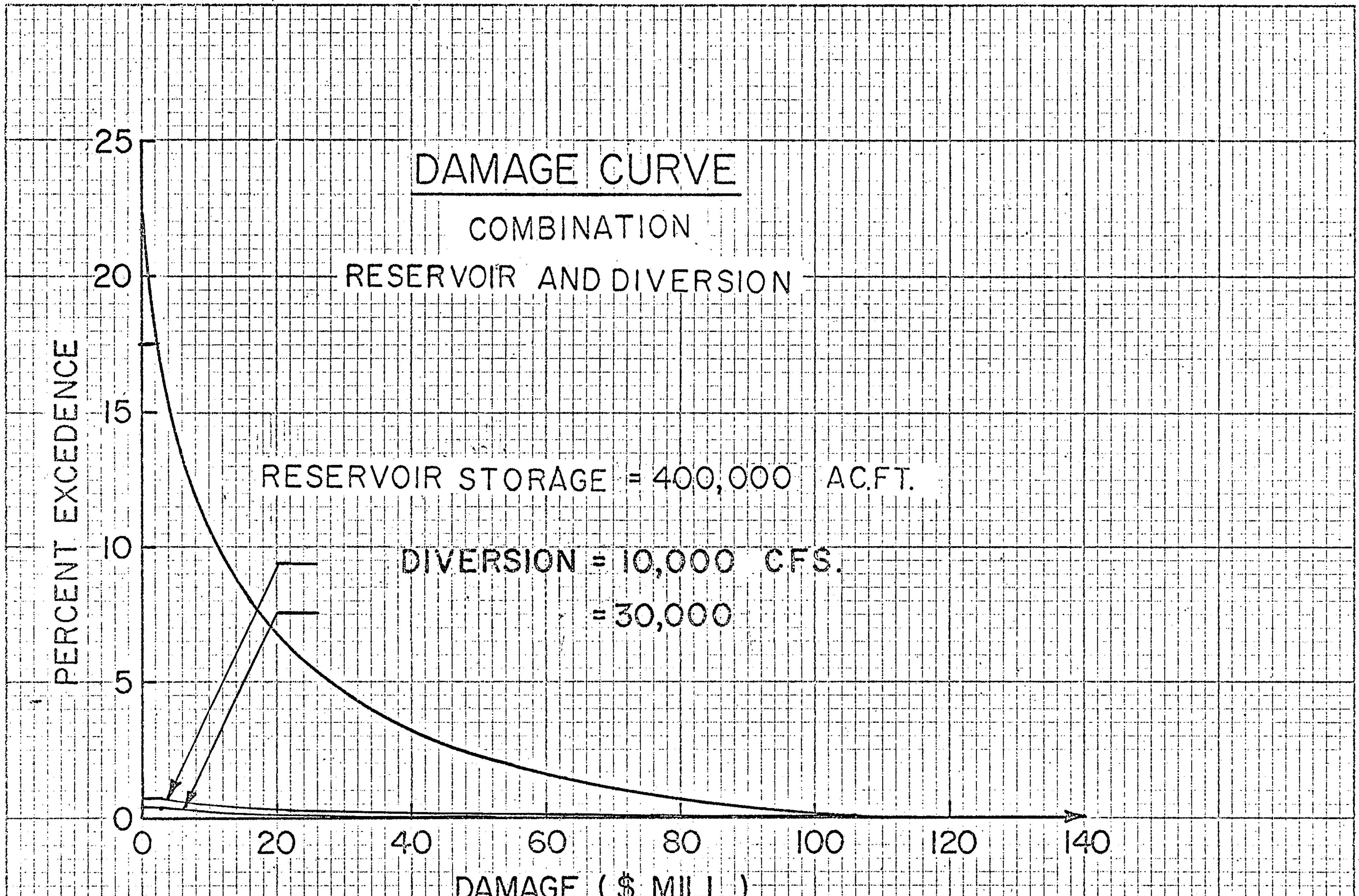

DAMAGE CURVE
COMBINATION
RESERVOIR AND DIVERSION
RESERVOIR STORAGE = 400,000 AC.FT.
DIVERSION = 10,000 CFS.
= 30,000
PERCENT EXCEDENCE
DAMAGE ($ MILL.)
0
5
10
15
20
25
0
20
40
60
80
100
120
140

FIG. 77

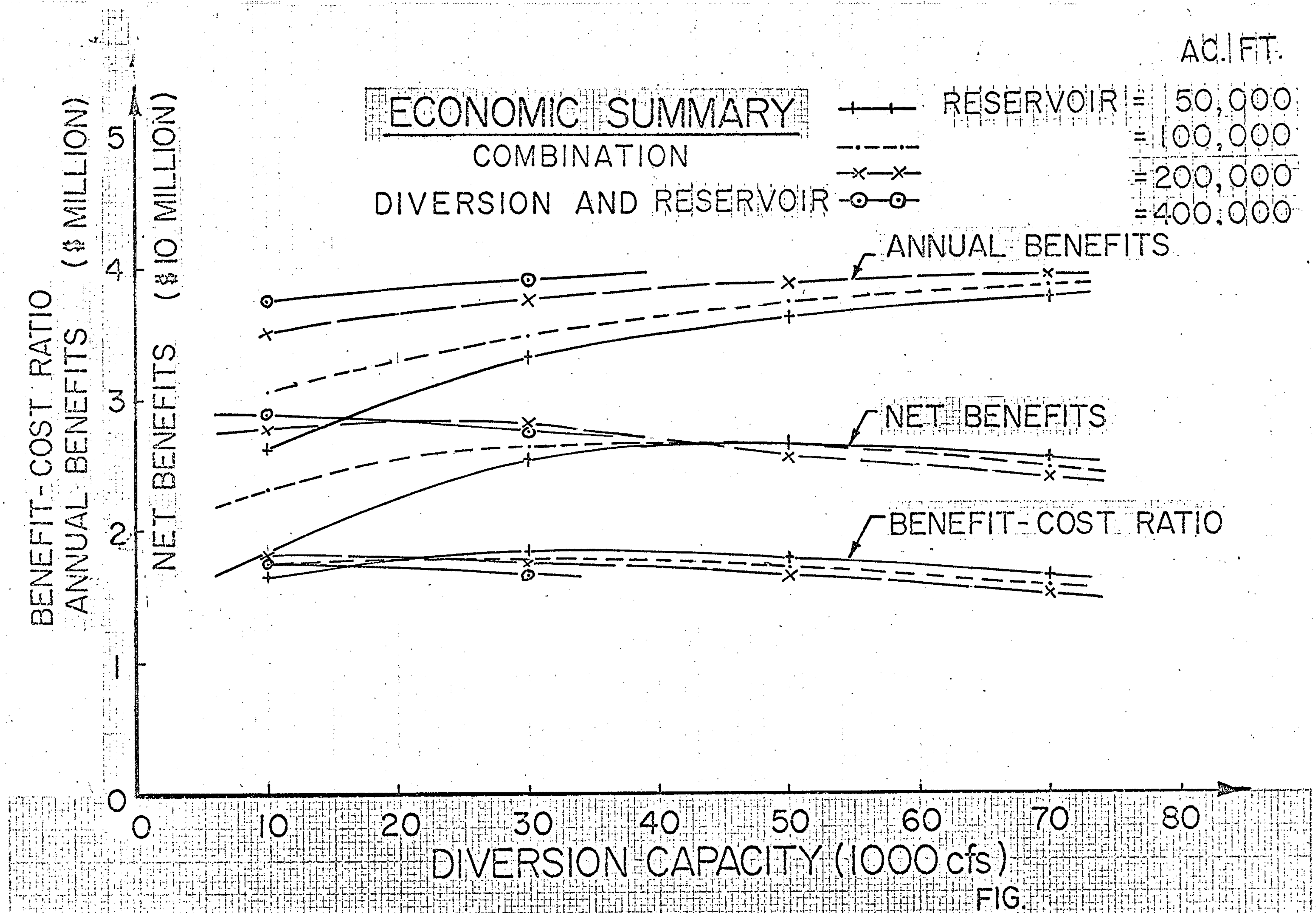

ECONOMIC SUMMARY
COMBINATION
DIVERSION AND RESERVOIR
AC. FT.
RESERVOIR = 50,000
= 100,000
= 200,000
= 400,000
ANNUAL BENEFITS
NET BENEFITS
BENEFIT-COST RATIO
BENEFIT-COST RATIO
ANNUAL BENEFITS ($ MILLION)
NET BENEFITS ($10 MILLION)
0
1
2
3
4
5
0
10
20
30
40
50
60
70
80
DIVERSION CAPACITY (1000 cfs)
FIG.

FIG. 78

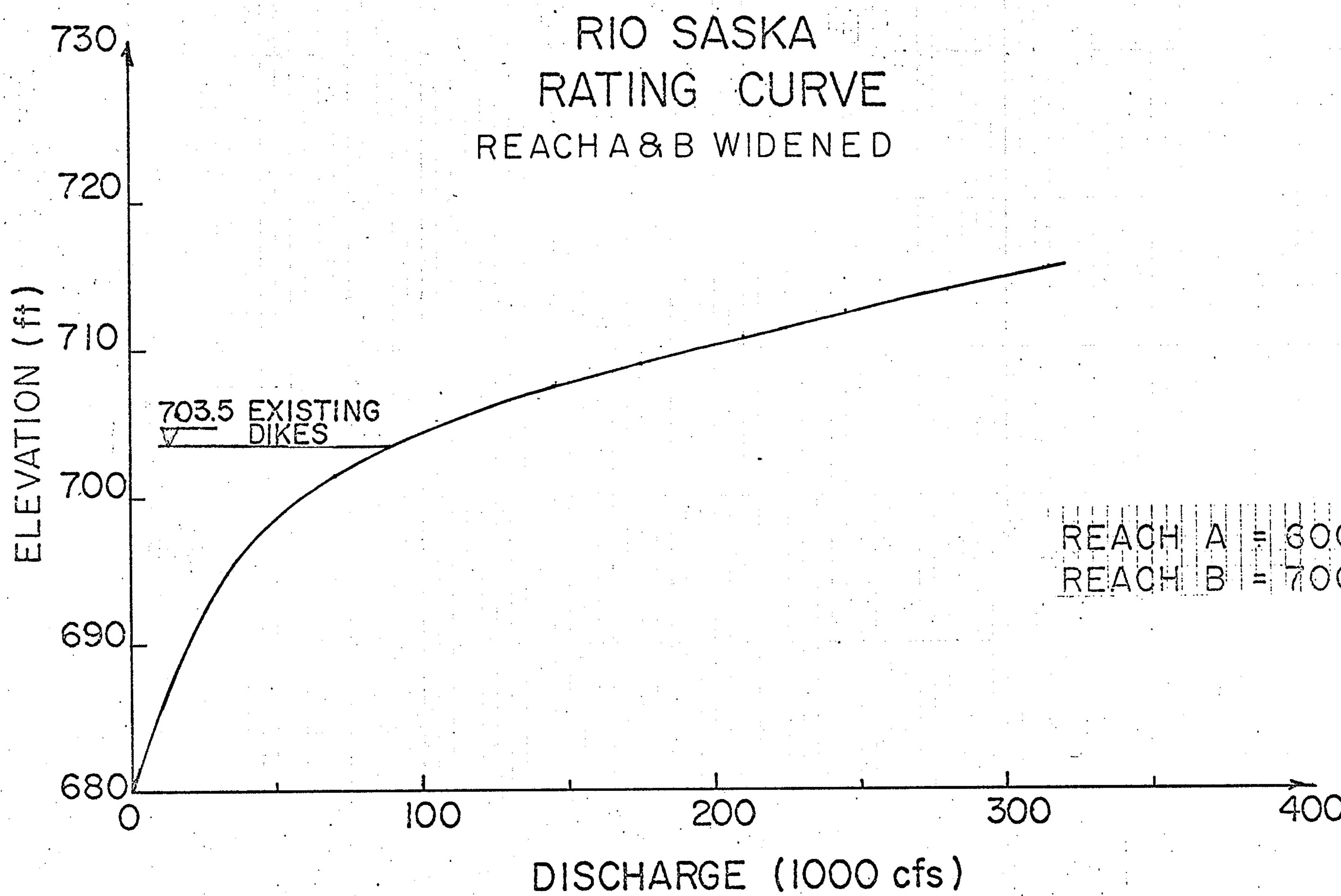
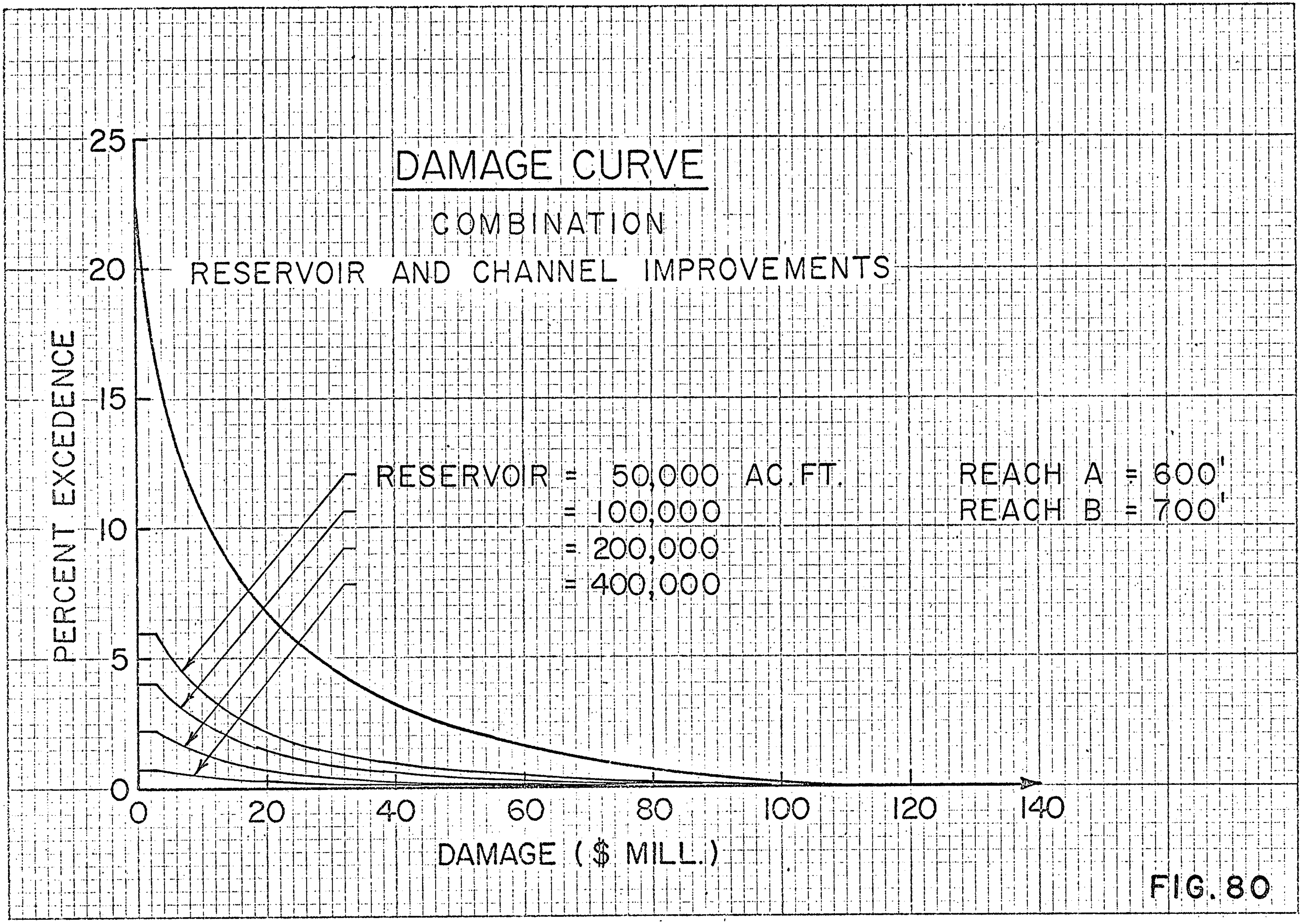

RIO SASKA
RATING CURVE
REACH A & B WIDENED
ELEVATION (ft)
730
720
710
700
690
680
703.5 EXISTING DIKES
REACH A = 600'
REACH B = 700'
0
100
200
300
400
DISCHARGE (1000 cfs)

FIG. 79


DAMAGE CURVE
COMBINATION
RESERVOIR AND CHANNEL IMPROVEMENTS
RESERVOIR = 50,000 AC. FT.
= 100,000
= 200,000
= 400,000
REACH A = 600'
REACH B = 700'
PERCENT EXCEDENCE
0
5
10
15
20
25
0
20
40
60
80
100
120
140
DAMAGE ($ MILL.)
FIG. 80

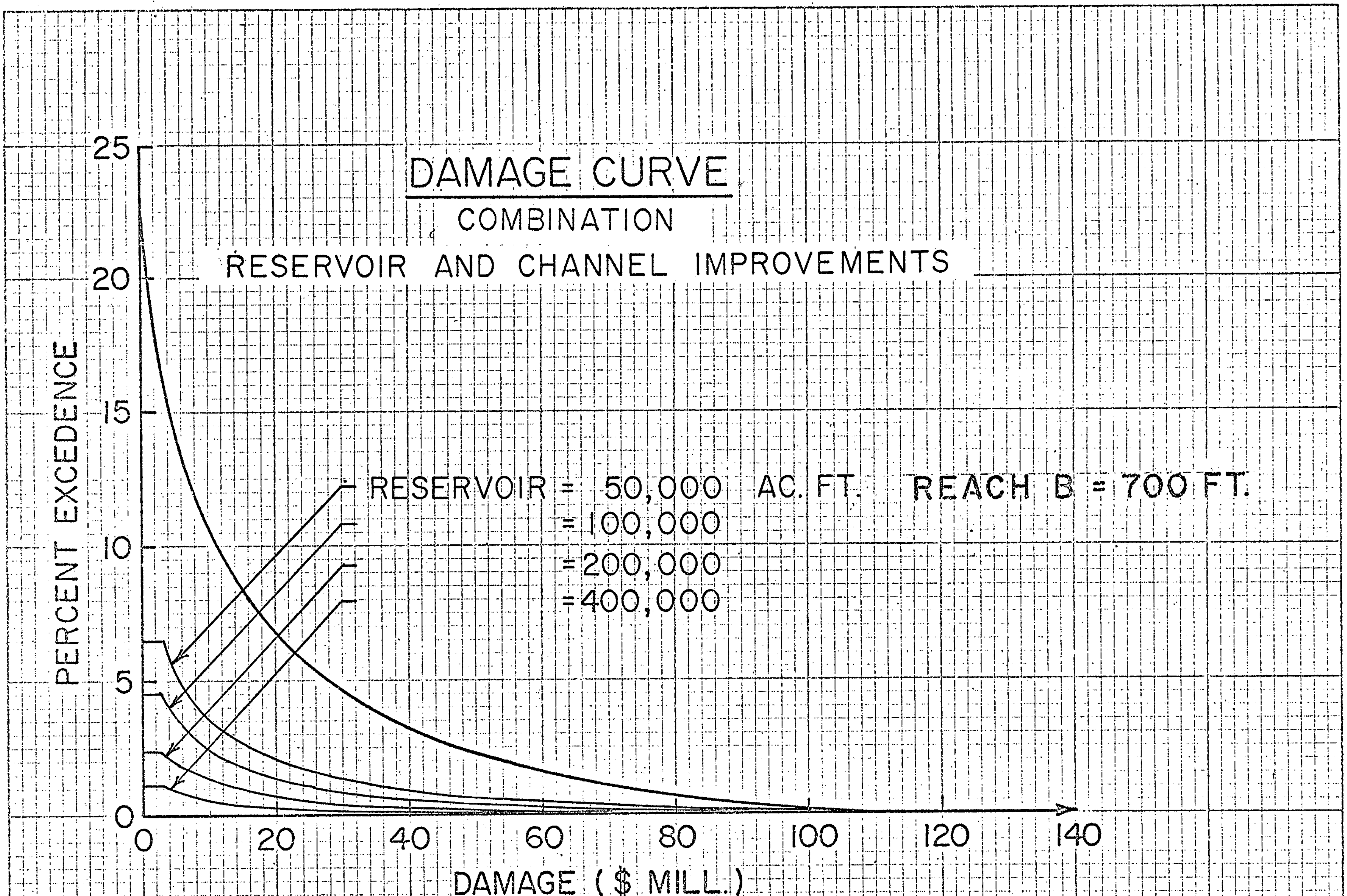
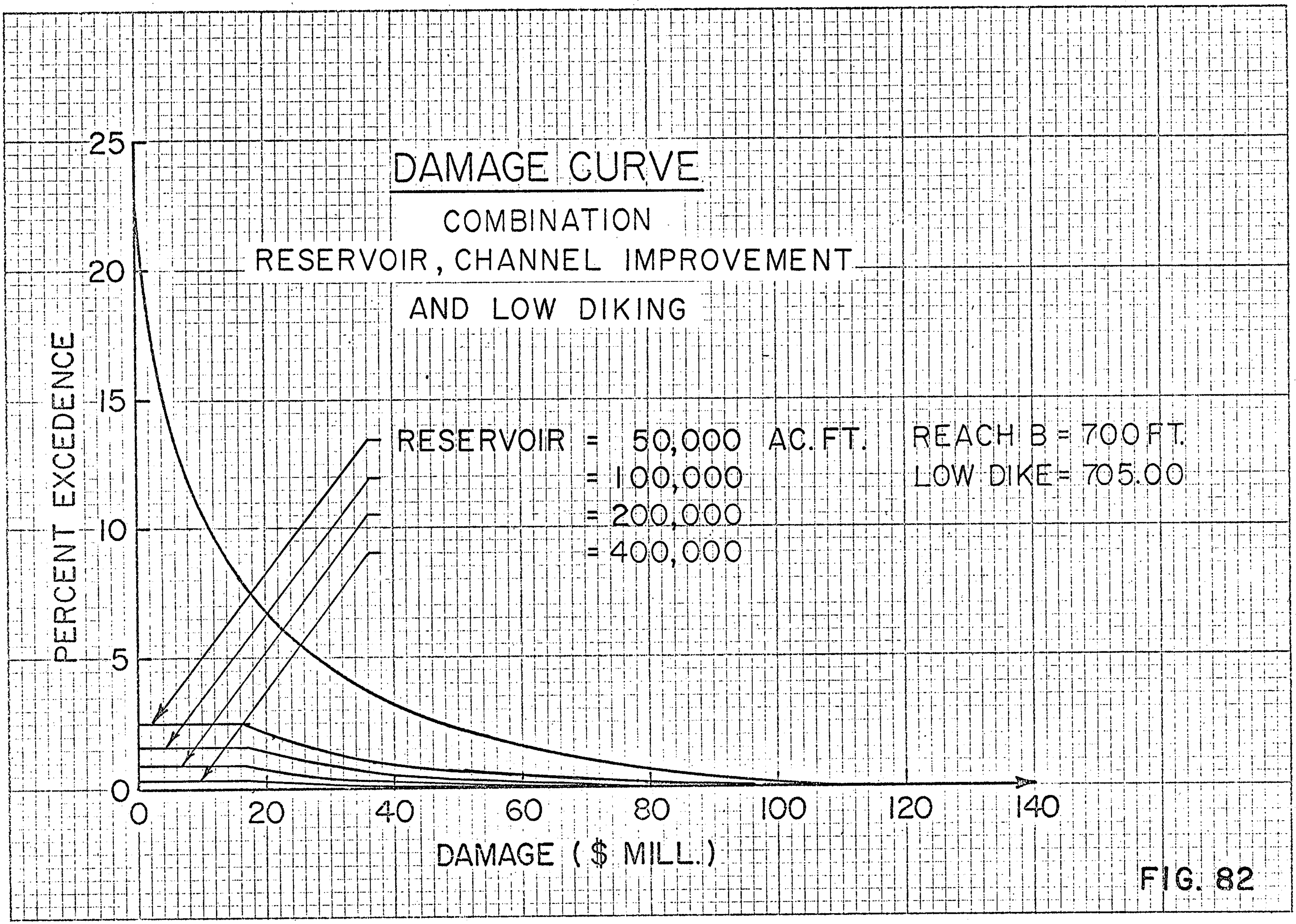

DAMAGE CURVE
COMBINATION
RESERVOIR AND CHANNEL IMPROVEMENTS
RESERVOIR = 50,000 AC. FT.
= 100,000
= 200,000
= 400,000
REACH B = 700 FT.
PERCENT EXCEDENCE
0
5
10
15
20
25
0
20
40
60
80
100
120
140
DAMAGE ($ MILL.)

FIG. 81



DAMAGE CURVE
COMBINATION
RESERVOIR, CHANNEL IMPROVEMENT
AND LOW DIKING
RESERVOIR = 50,000 AC. FT.
= 100,000
= 200,000
= 400,000
REACH B = 700 FT.
LOW DIKE = 705.00
PERCENT EXCEDENCE
25
20
15
10
5
0
0
20
40
60
80
100
120
140
DAMAGE ($ MILL.)

FIG. 82

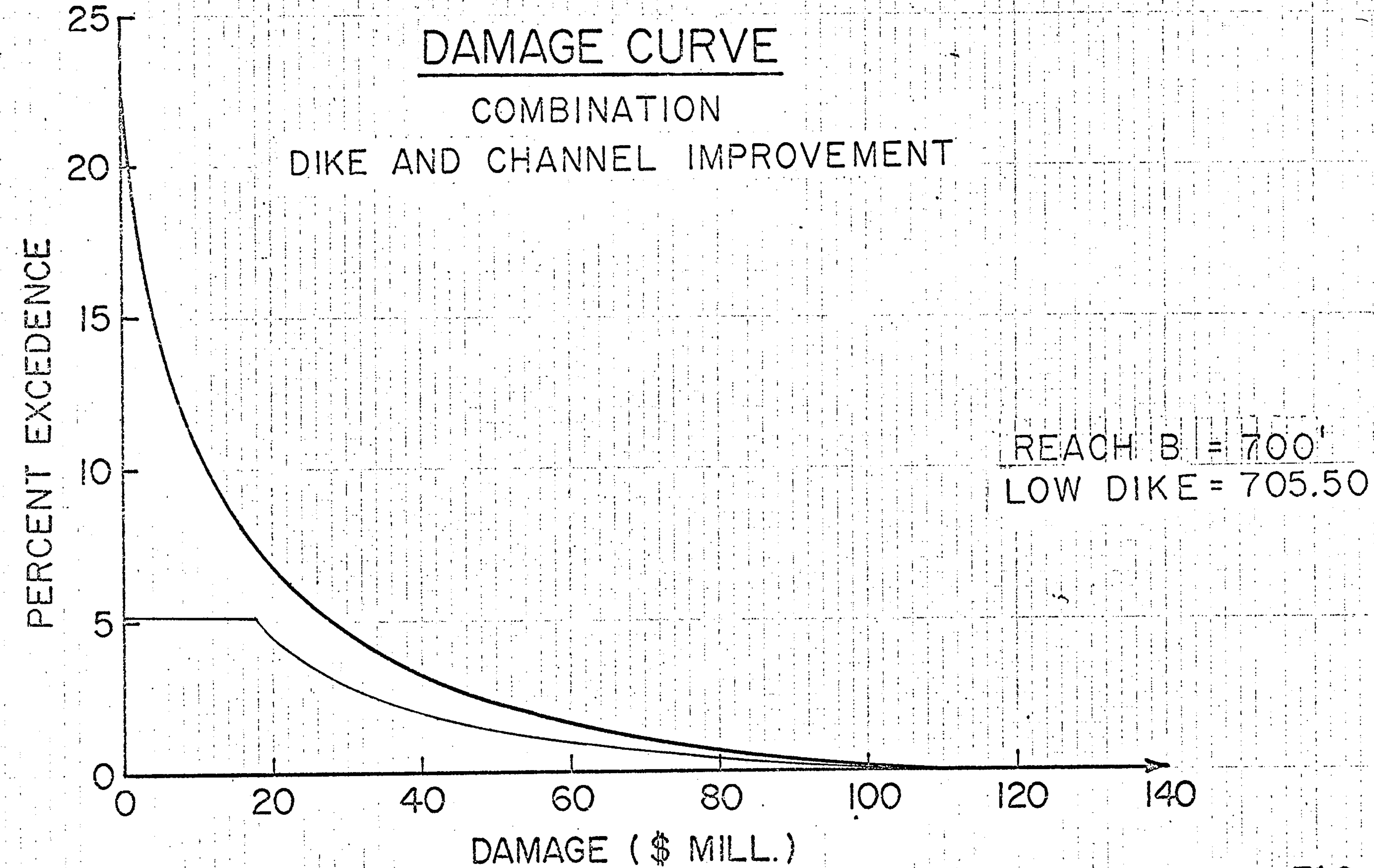

DAMAGE CURVE
COMBINATION
DIKE AND CHANNEL IMPROVEMENT
REACH B = 700'
LOW DIKE = 705.50
PERCENT EXCEDENCE
25
20
15
10
5
0
0
20
40
60
80
100
120
140
DAMAGE ($ MILL.)

FIG. 83

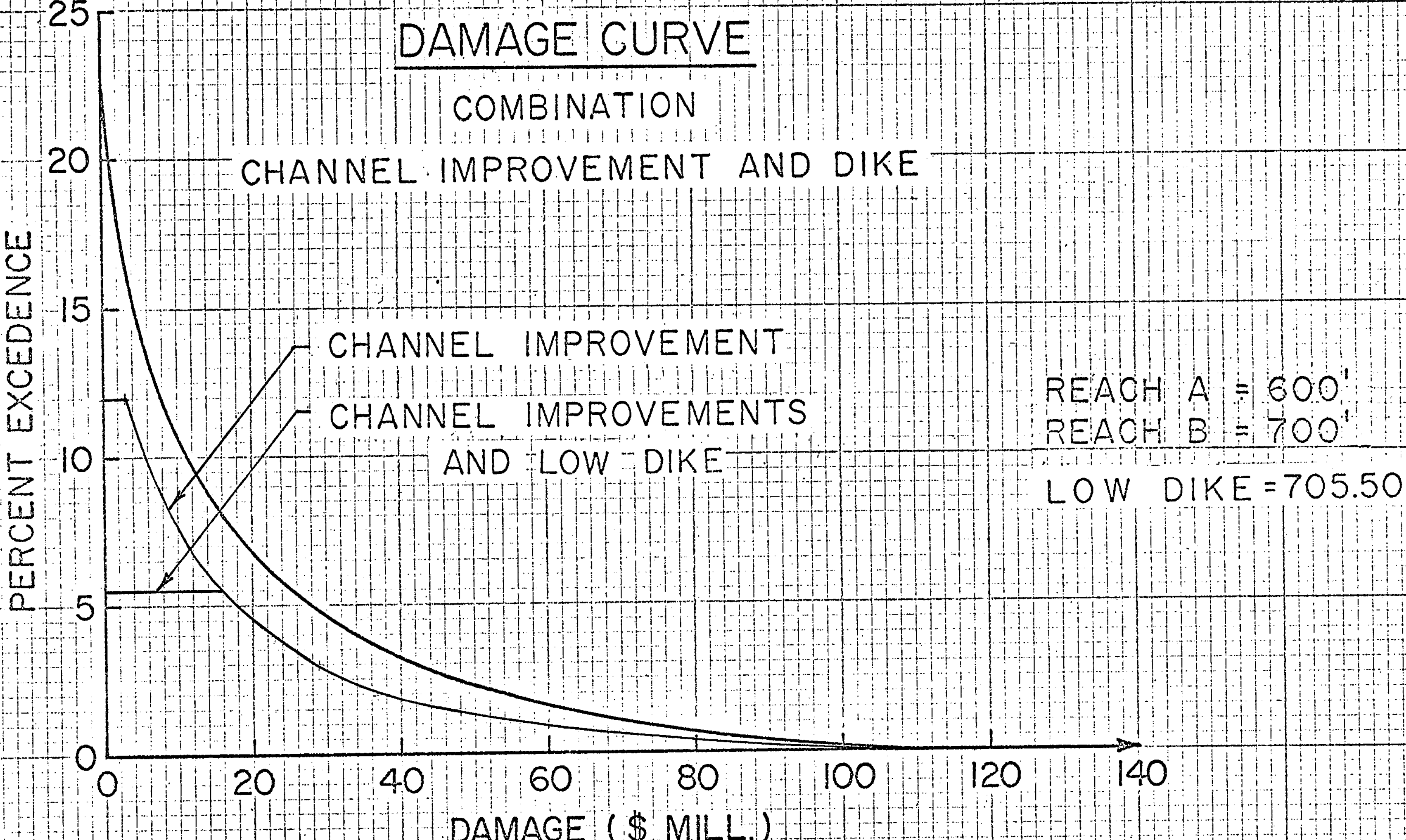

DAMAGE CURVE
COMBINATION
CHANNEL IMPROVEMENT AND DIKE
CHANNEL IMPROVEMENT
CHANNEL IMPROVEMENTS
AND LOW DIKE
REACH A = 600'
REACH B = 700'
LOW DIKE = 705.50
PERCENT EXCEDENCE
0
5
10
15
20
25
0
20
40
60
80
100
120
140
DAMAGE ($ MILL.)

FIG. 84

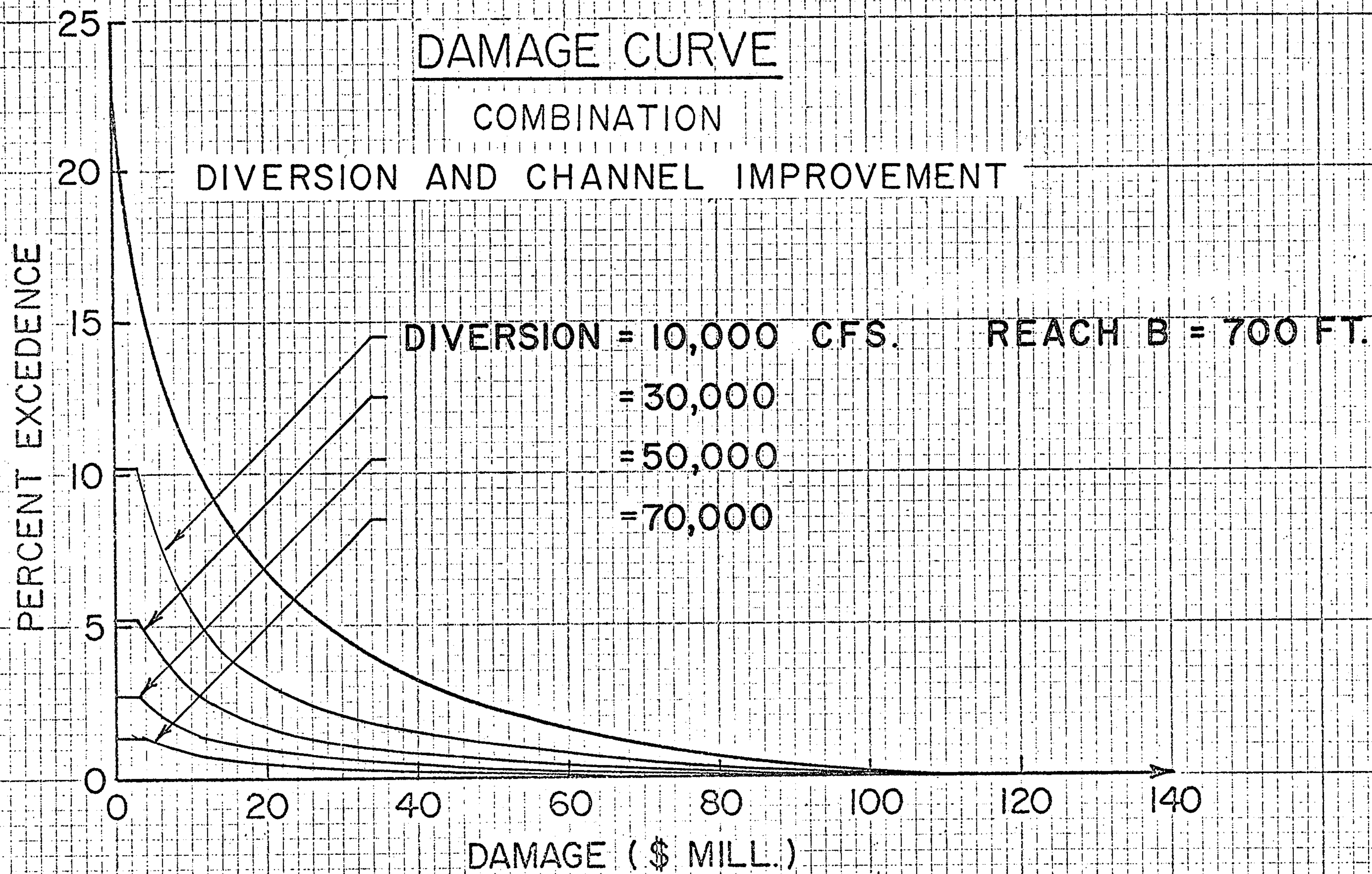

DAMAGE CURVE
COMBINATION
DIVERSION AND CHANNEL IMPROVEMENT
DIVERSION = 10,000 CFS.
= 30,000
= 50,000
= 70,000
REACH B = 700 FT.
PERCENT EXCEDENCE
25
20
15
10
5
0
0
20
40
60
80
100
120
140
DAMAGE ($ MILL.)

FIG. 85

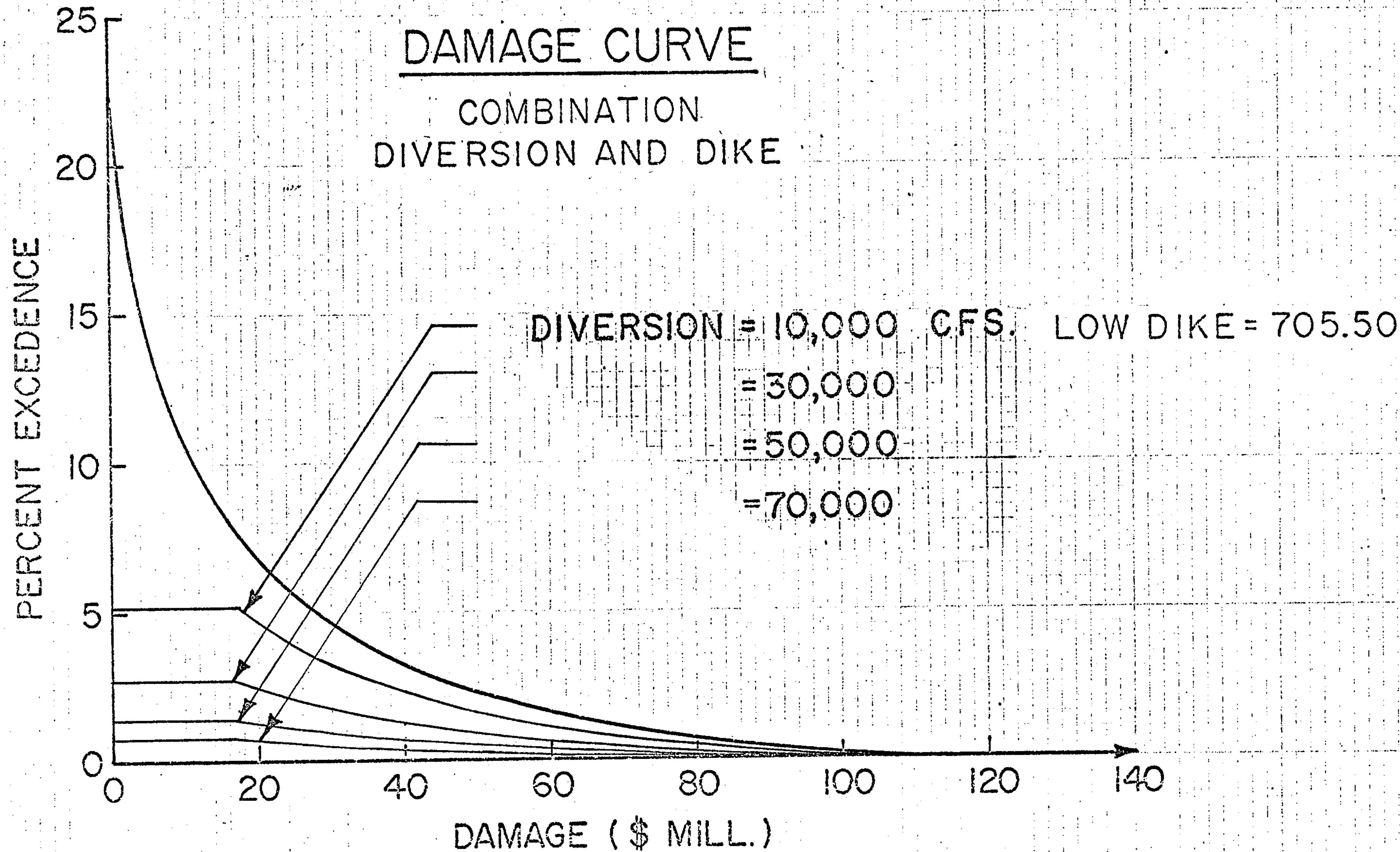

DAMAGE CURVE
COMBINATION
DIVERSION AND DIKE
DIVERSION = 10,000 CFS.
= 30,000
= 50,000
= 70,000
LOW DIKE = 705.50
PERCENT EXCEDENCE
25
20
15
10
5
0
0
20
40
60
80
100
120
140
DAMAGE ($ MILL.)

FIG. 86

CHAMPION LINE NO. 642 - CROSS SECTION - 10 SQUARES TO INCH

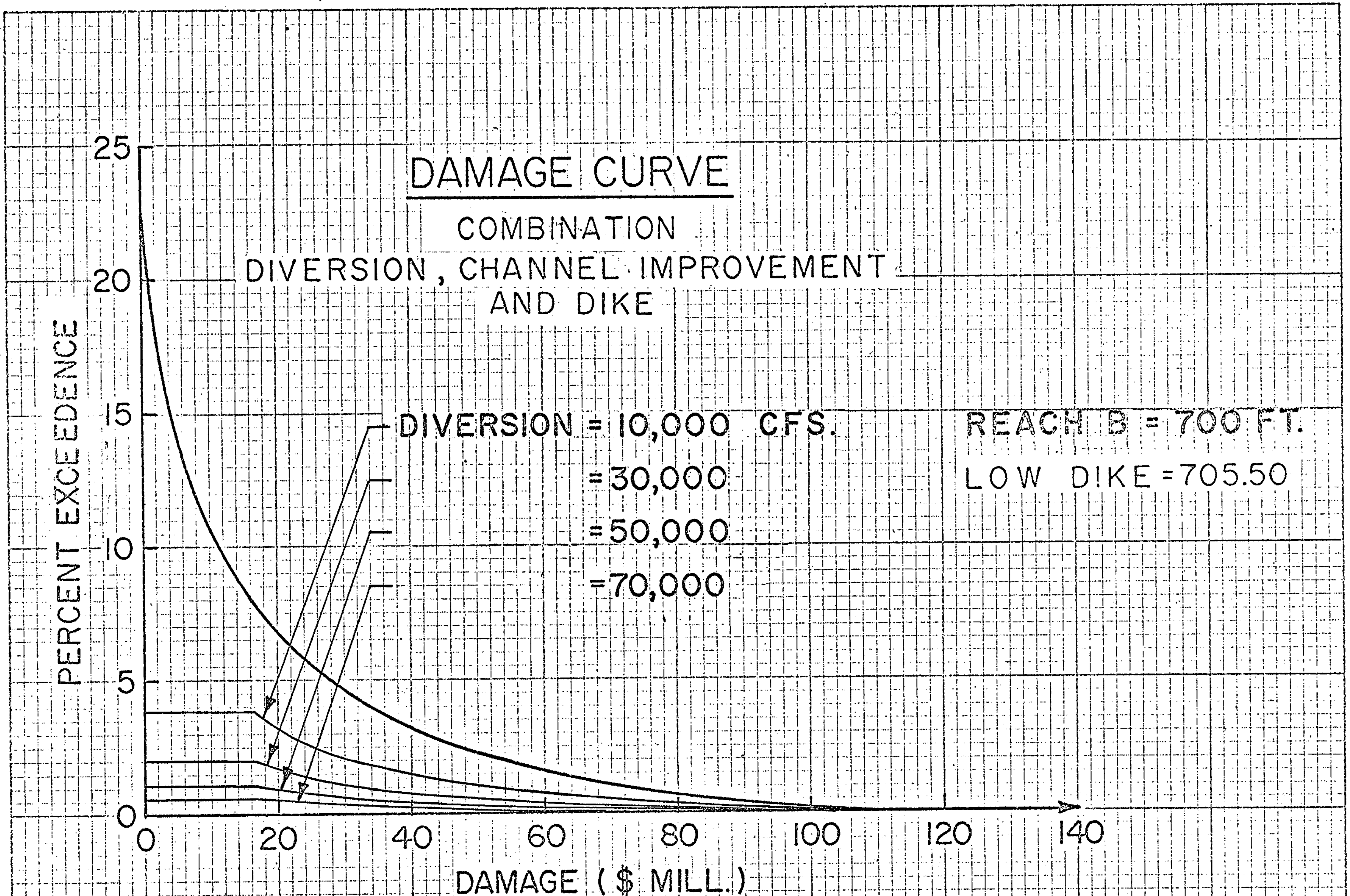


FIG. 87

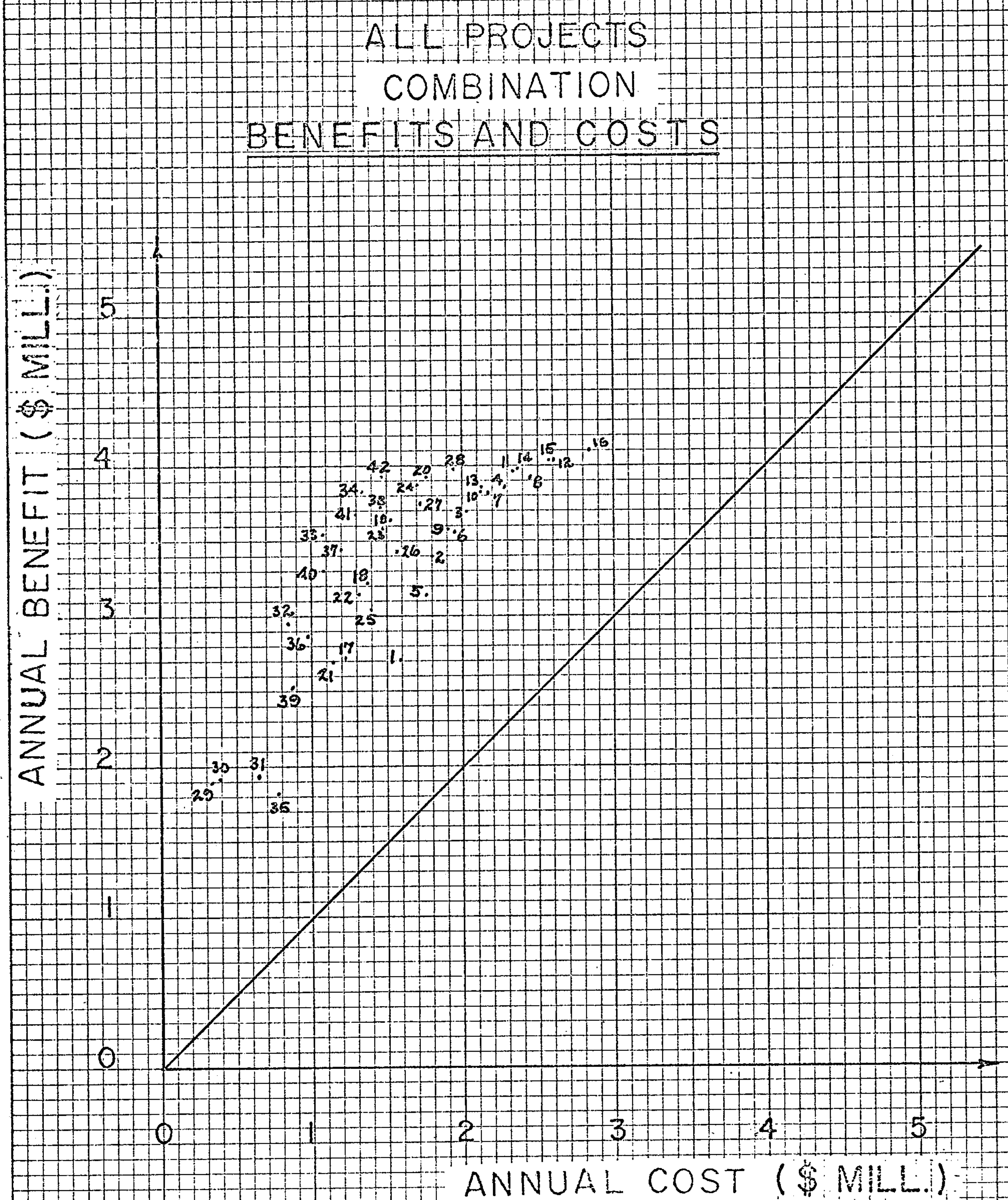

ALL PROJECTS
COMBINATION
BENEFITS AND COSTS
ANNUAL BENEFIT ($ MILL.)
ANNUAL COST ($ MILL.)
0
1
2
3
4
5

FIG. 88

APPENDICES

APPENDIX A

Appendix A-1 consists of a brief computer program that can be used to calculate Normal Depths for any channel on any slope, given the discharge as outlined in Chapter II.

This program was introduced in order to minimize the work associated with calculating the normal depths for various flows, especially when both channel and overbank flows are present. The appropriate Normal Depth Curves were plotted and then used for obtaining input for the Bossen Routing Program, Appendix B-1.

In Appendix A-1 the computer program is presented, while in A-2 typical output is shown.

APPENDIX B

Appendix B-1 consists of a computer program of the Bossen Routing Method, a simplification of the Bresse Equation, as outlined in Chapter II.

This routing program was produced to reduce the computational time required to calculate the Rating Curve at the City of Portage when changes were introduced in the channel downstream of the city. This program used the Normal Depths which were calculated with the program in Appendix A.

In Appendix B-1 the program is presented while in Appendix B-2 typical output is shown.

APPENDIX C

Appendix C-1 consists of a computer program which was used to calculate costs associated with various diversion channels as outlined in Chapter IV. The topographic data along the route of the channel is read into the computer. Then given the maximum permissible slope and velocity, the excavation quantities as well as side dike volumes were calculated. Similarly, the diversion intake elevation, its location and the channel widths were calculated. Costs associated with these items, as well as land acquisition and the costs of highway and railway crossings and the costs of the required drop structures were calculated.

In Appendix C-1 the Bossen Routing Program, described above and in Chapter IV, is presented, while C-2 shows typical output.

APPENDIX D

In Appendix D the cost curves presented were used to determine the unit costs of various construction items encountered in the investigation of the projects.

Appendix A-1

NORMAL DEPTH CURVE PROGRAM

```
      WRITE (6,10)
   10 FORMAT (10X, ' NORMAL RATING CURVE', / / )
C READ DATA
   20 READ (5,30) SLOPE, W
   30 FORMAT (F10.6, F10.0)
      DO 100 L = 500, 600, 25
      XL = L
      W = XL
      WRITE (6,40)
   40 FORMAT ( / / )
C INITIALIZATION
      QC = 0.0
      QOB = 0.0
      QTOT = 0.0
      TQC = 0.0
      TQOB = 0.0
C CHANNEL CONSTANTS
      SS = 100.0
      K = 35
      CN = K * 0.001
      CC = (1.49 * SLOPE ** 0.5) / CN
      BN = 0.070
      COB = (1.49 * SLOPE ** 0.5) / BN
      WRITE (6,50) W, CN, BN, SLOPE
   50 FORMAT (5X, ' CHANNEL WIDTH =', F5.0, 5X, ' CHANNEL N =', F5.3, 5X
     X, ' OVERBANK N =', F5.3, ' SLOPE =', F8.6, / / )
      WRITE (6,60)
   60 FORMAT (9X, ' DEPTH', 6X, ' TOTAL QC', 7X, ' DEPTH OB', 5X, ' TOTA
     XL QOB', 9X, ' TOTAL FLOW', / )
C VARY DEPTH
      DO 100 I = 5, 40, 5
      DEPTH = I
      DOB = 0.0
      QC = DEPTH ** 1.67 * CC
      TQC = W * QC
C OVERBANK FLOW CALCULATIONS
      IF (DEPTH - 20.0) 80,80,70
   70 DOB = DEPTH - 20.0
      QOB = COB * DOB ** 1.67
      TQOB = QOB * (DOB * SS + 4000.0)
   80 QTOT = TQC + TQOB
      WRITE (6,90) DEPTH, TQC, DOB, TQOB, QTOT
   90 FORMAT (5X, F10.2, ' FT.', 1X, F10.0, 5X, F10.2, 5X, F10.0, 5X,
     XF10.0, ' CFS')
  100 CONTINUE
      GO TO 20
  110 CALL EXIT
      END
```

APPENDIX A-2

NORMAL DEPTH CURVE

TYPICAL OUTPUT

DYKING AT PORTAGE

TOE OF DIKES AT 150 FT. FROM CHANNELS EDGE

CHANNEL WIDTH = 600.   CHANNEL N =0.030   OVERBANK N =0.070

DEPTH	TOTAL QC	DEPTH OB	TOTAL QOB	TOTAL FLO
5.00 FT.	5711.	0.0	0.	5711. CFS
10.00 FT.	18174.	0.0	0.	18174. CFS
15.00 FT.	35769.	0.0	0.	35769. CFS
20.00 FT.	57831.	0.0	0.	57831. CFS
25.00 FT.	83946.	5.00	3263.	87209. CFS
30.00 FT.	113823.	10.00	16875.	130699. CFS
35.00 FT.	147242.	15.00	45989.	193232. CFS
40.00 FT.	184026.	20.00	95008.	279034. CFS

CHANNEL WIDTH = 600.   CHANNEL N =0.035   OVERBANK N =0.070

DEPTH	TOTAL QC	DEPTH OB	TOTAL QOB	TOTAL FLO
5.00 FT.	4895.	0.0	0.	4895. CFS
10.00 FT.	15577.	0.0	0.	15577. CFS
15.00 FT.	30659.	0.0	0.	30659. CFS
20.00 FT.	49569.	0.0	0.	49569. CFS
25.00 FT.	71954.	5.00	3263.	75217. CFS
30.00 FT.	97563.	10.00	16875.	114438. CFS
35.00 FT.	126208.	15.00	45989.	172197. CFS
40.00 FT.	157736.	20.00	95008.	252744. CFS

CHANNEL WIDTH = 600.   CHANNEL N =0.040   OVERBANK N =0.070

DEPTH	TOTAL QC	DEPTH OB	TOTAL QOB	TOTAL FLO
5.00 FT.	4283.	0.0	0.	4283. CFS
10.00 FT.	13630.	0.0	0.	13630. CFS
15.00 FT.	26827.	0.0	0.	26827. CFS
20.00 FT.	43373.	0.0	0.	43373. CFS
25.00 FT.	62959.	5.00	3263.	66223. CFS
30.00 FT.	85368.	10.00	16875.	102243. CFS
35.00 FT.	110432.	15.00	45989.	156421. CFS
40.00 FT.	138019.	20.00	95008.	233027. CFS

Appendix B-1

BOSSEN ROUTING PROGRAM

using Simplified Bresse Method

```
      WRITE (6,10)
   10 FORMAT (' RATING CURVE AT PORTAGE USING BRESSE METHOD', / / )
   20 WRITE (6,30)
   30 FORMAT ( / )
C     INITIALIZATION
      DEPTH = 0.0
      DELH = 0.0
      DELX = 0.0
      HINT = 0.0
      HADD = 0.0
      DRC = 0.0
      DISX = 0.0
      TLEN = 0.0
      N = 1
      I = 1
      NI = 1
      XN = N
      XI = I
      XNI = NI
      ELEVEL = 0.0
      ELCR = 690.50
C     READ DATA
      READ (5,40) WR, DNA, DNB, QTOT, SLOPEA, SLOPEB, CN, DNC
   40 FORMAT (F4.0, F4.1, F4.1, F8.0, F8.7, F8.7, F5.3, F5.1)
      WR = 350.
      DRC = (((QTOT / WR) ** 2.0) / 32.2) ** 0.333
C     REACH A
      DEPTH = DRC + 5.0
      WRITE (6,50) WR, DNA, SLOPEA, CN, DRC, QTOT
   50 FORMAT (5X, ' RAPID W ', F5.0, 5X, ' DNA =', F4.1, 5X, ' SLOPE A =
     X', F8.7, 5X, ' CN =', F5.3, 5X, ' DRC =', F4.1, 5X, ' QTOT =',
     XF10.0, ' CFS.', / )
      WRITE (6,60)
   60 FORMAT (6X, ' NO.', 4X, ' DEPTH', 5X, ' DELX', 2X, 'TOTAL L', / )
      DELH = DNA - DEPTH
      DELX = 0.25 * DNA / SLOPEA
      DISX = DELX * XN
      WRITE (6,70) N, DEPTH, DELX, TLEN
   70 FORMAT (5X, I5, F10.1, 2F10.0)
      TLEN = DISX
   80 IF (DISX - 26400.0) 90,110,110
   90 DEPTH = DEPTH + DELH / 2.0
      DELH = DNA - DEPTH
      N = N + 1
      XN = N
      WRITE (6,100) N, DEPTH, DISX, TLEN
  100 FORMAT (5X, I5, F10.1, 2F10.0)
      DISX = DELX * XN
      TLEN = DISX
      GO TO 80
C     LINEAR INTERPOLATION
  110 HINT = (DELH / 2.0) * (DISX - 26400.0) / (DISX / XN)
      HADD = DELH / 2.0 - HINT
      DEPTH = DEPTH + HADD
      TLEN = 26400.
      WRITE (6,120) DEPTH, TLEN
  120 FORMAT (10X, F10.1, 10X, F10.0)
      WRITE (6,130)
  130 FORMAT (5X, ' END OF REACH A', / )
C     REACH B
```

```
      WRITE (6,140) DNB, SLOPEB
  140 FORMAT (5X, ' DNB =', F4.1, 5X, ' SLOPE B =', F8.7)
      DELH = DNB - DEPTH
      DELX = 0.25 * DNB / SLOPEB
      DISX = DELX * XI
      WRITE (6,150) I, DEPTH, DELX, TLEN
  150 FORMAT (5X, I5, F10.1, 2F10.0)
      TLEN = 26400. + DISX
  160 IF (DISX - 44880.0) 170,190,190
  170 DEPTH = DEPTH + DELH / 2.0
      DELH = DNB - DEPTH
      I = I + 1
      XI = I
      WRITE (6,180) I, DEPTH, DISX, TLEN
  180 FORMAT (5X, I5, F10.1, 2F10.0)
      DISX = DELX * XI
      TLEN = 26400. + DISX
      GO TO 160
C     LINEAR INTERPOLATION
  190 HINT = (DELH / 2.0) * (DISX - 44880.0) / (DISX / XI)
      HADD = DELH / 2.0 - HINT
      DEPTH = DEPTH + HADD
      TLEN = 26400. + 44880.
      WRITE (6,200) DEPTH, TLEN
  200 FORMAT (10X, F10.1, 10X, F10.0)
      WRITE (6,210)
  210 FORMAT (5X, ' END OF REACH B', / )
C     CITY REACH
      WRITE (6,220) DNC, SLOPEB
  220 FORMAT (5X, ' DNC =', F4.1, 5X, ' SLOPE B =', F8.7)
      DELH = DNC - DEPTH
      DELX = 0.25 * DNC / SLOPEB
      DISX = DELX * XNI
      WRITE (6,230) NI, DEPTH, DELX, TLEN
  230 FORMAT (5X, I5, F10.1, 2F10.0)
      TLEN = 71280. + DISX
  240 IF (DISX - 18480.) 250,270,270
  250 DEPTH = DEPTH + DELH / 2.0
      DELH = DNC - DEPTH
      NI = NI + 1
      XNI = NI
      WRITE (6,260) NI, DEPTH, DISX, TLEN
  260 FORMAT (5X, I5, F10.1, 2F10.0)
      DISX = DELX * XNI
      TLEN = 79200. + DISX
      GO TO 240
C     LINEAR INTERPOLATION
  270 HINT = (DELH / 2.0) * (DISX - 18480.) / (DISX / XNI)
      HADD = DELH / 2.0 - HINT
      DEPTH = DEPTH + HADD
      TLEN = 89760.
      WRITE (6,280) DEPTH, TLEN
  280 FORMAT (10X, F10.1, 10X, F10.0)
      WRITE (6,290)
  290 FORMAT (5X, ' END OF CITY REACH', / )
      ELEVWL = ELCB + DEPTH
      WRITE (6,300) ELEVWL
  300 FORMAT (10X, ' WATER LEVEL =', F10.2, / / )
      GO TO 20
  310 CALL EXIT
      END
```

APPENDIX B-2

BOSSEN ROUTING

TYPICAL OUTPUT

RATING CURVE AT PORTAGE USING BRESSE METHOD

CN =0 030 DRC = 6.8 QTOT = 35000. CFS.

RAPID W 350 DNA =12 9 SLOPE A = 0003800

NO	DEPTH	DELX	TOTAL L
1	11 8	8487	0
2	12 3	8487	8487.
3	12 6	16974	16974
4	12 8	25461	25461.
	12 8		26400.

END OF REACH A

DNB =15 0 SLOPE B = 0001700

NO	DEPTH	DELX	TOTAL L
1	12 8	22059	26400
2	13 9	22059	48459.
3	14 4	44118	70518.
	14 5		79200

END OF REACH B

DNC =15 0 SLOPE B = 0001700

NO	DEPTH	DELX	TOTAL L
1	14 5	22059	79200.
	14 7		89760.

END OF CITY REACH

WATER LEVEL = 695.16

CN =0 030 DRC =10.7 QTOT = 70000. CFS.

RAPID W 350 DNA =19 5 SLOPE A = 0003800

NO	DEPTH	DELX	TOTAL L
1	15 7	12829	0
2	17 5	12829	12829.
3	18 5	25658	25658.
	18 5		26400.

END OF REACH A

DNB =21 7 SLOPE B = 0001700

NO	DEPTH	DELX	TOTAL L
1	18 5	31912	26400
2	20 1	31912.	58312.
	20 7		79200

END OF REACH B

DNC =22 1 SLOPE B = 0001700

NO	DEPTH	DELX	TOTAL L
1	20 7	22500	79200.
	20 9		89760.

END OF CITY REACH

WATER LEVEL = 701.39

Appendix C-1

Portage Diversion

Program

```
     C PORTAGE DIVERSION,EXCAVATION,LAND,DYKE AND COSTS
0001       DIMENSION A(100), L(100), ELG(100), ELC(100), VOL(100), H(100),
          XLTOT(99), VOLTOT(100), XELG(100), XLTOT(100), TOTW(100), AVTOTW(
          X100), XLAND(100), TLAND(100), DYKE(100), TDYKE(100), DYKA(100), W(
          X100)
0002       REAL L, LTOT, LINT, LTOTIN
0003       N = 19
     C READ DATA
0004       READ (5,10) (XELG(I), XLTOT(I), I = 2, N)
0005    10 FORMAT (2F10.2)
     C VARY DIVERSION FLOW
0006       DO 600 NK = 10, 90, 10
0007       XNK = NK
0008       QTOT = XNK * 1000.
     C VARY SLOPE
0009       DO 600 KK = 10, 30, 5
0010       XKK = KK
0011       SLOPE = XKK / 100000.
0012       SOORG = SLOPE
0013       K = 1
0014       DO 20 I = 2, N
0015       ELG(I) = XELG(I)
0016       LTOT(I) = XLTOT(I)
0017    20 CONTINUE
0018       CN = 0.030
0019       VEL = 4.00
0020       DN = ((VEL * CN) / (1.49 * SLOPE ** 0.50)) ** 1.5
0021       W(1) = QTOT / (VEL * DN)
0022       FSL = 728.00
0023       ELG(1) = FSL - DN
0024       ELC(1) = ELG(1)
     C INITIALIZATION
0025       A(1) = 0.0
0026       VOL(1) = 0.0
0027       VOLTOT(1) = 0.0
0028       L(1) = 0.0
0029       LINT = 0.0
0030       TOTW(1) = W(1) + 2.0 * (6.0 * (DN + 1.0) + 10.0 + 10.0)
0031       AVTOTW(1) = 0.0
0032       XLAND(1) = 0.0
0033       TLAND(1) = 0.0
0034       DYKA(1) = 0.0
0035       DYKE(1) = 0.0
0036       TDYKE(1) = 0.0
0037       TWDROP = 0.0
0038       WDROP = 0.0
0039       SOMAX = 0.00040
     C UNIT PRICES
0040       COSTEU = 0.30
0041       COSTLU = 100.00
0042       COSTDU = 0.10
0043       COSTSU = 50000.00
0044       COSTHU = 500.00
0045       COSTPU = 300.00
0046       WRITE (6,30)
0047    30 FORMAT ( / / )
     C TITLE
0048       WRITE (6,40) W(1), SLOPE, QTOT
0049    40 FORMAT ( /, ' PORTAGE DIVERSION CALCULATIONS                WIDTH
          X=', F10.0, ' SLOPE =', F10.6, ' QTOT =', F10.0, ' CFS ', / )
```

```
0050          WRITE (6,50) FSL, DN, VEL
0051       50 FORMAT (' ', ' FSL =', F7.2, ' DN =', F6.2, ' VELOCITY =', F6.2, '
             X FPS', / )
        C EXCAVATION TABLE
0052          WRITE (6,60)
0053       60 FORMAT (2X, ' NUMBER', 2X, ' ELINT', 4X, ' ELG', 4X, ' ELC', 3X, '
             X LINT', 8X, ' L', 5X, ' LTOT', 3X, ' VOLINT', 6X, ' VOL', 3X, ' VO
             XLTOT', 3X, ' DYKE A', 5X, ' DYKE', 4X, ' TDYKE', / )
0054          WRITE (6,70)
0055       70 FORMAT (13X, ' FT.', 5X, 'FT.', 4X, ' FT.', 4X, ' FT.', 6X, ' FT.'
             X, 6X, ' FT.', 3X, ' CU.YD.', 3X, ' CU.YD.', 3X, ' CU.YD.', 3X, ' S
             XQ.FT.', 3X, ' CU.YD.', 3X, ' CU.YD.', / )
0056          WRITE (6,80) ELG(1), ELC(1)
0057       80 FORMAT (7X, ' 1'9X, 2F8.2, 16X, F8.0)
        C STATION TO STATION CALCULATIONS
0058          DO 470 I = 2, N
        C TO FIND STARTING POINT
0059          IF (7 - I) 140,90,90
0060       90 IF (706.00 - ELG(I)) 100,580,580
0061      100 IF (ELG(I) - ELG(1)) 120,110,110
0062      110 A(I) = A(I - 1)
0063          VOL(I) = VOL(I - 1)
0064          VOLTOT(I) = VOLTOT(I - 1)
0065          L(I) = L(I - 1)
0066          ELC(I) = ELC(I - 1)
0067          TOTW(I) = TOTW(I - 1)
0068          XLAND(I) = XLAND(I - 1)
0069          TLAND(I) = TLAND(I - 1)
0070          DYKA(I) = DYKA(I - 1)
0071          DYKE(I) = DYKE(I - 1)
0072          TDYKE(I) = TDYKE(I - 1)
0073          W(I) = W(I - 1)
0074          GO TO 470
0075      120 IF (5.0 - (ELG(I) - ELG(1))) 140,140,130
0076      130 LTOT(I - 1) = LTOT(I - 1) + (LTOT(I) - LTOT(I - 1)) * (ELG(1) -
             XELG(I - 1)) / (ELG(I) - ELG(I - 1))
0077      140 L(I) = LTOT(I) - LTOT(I - 1)
0078          W(I) = W(I - 1)
0079          IF (1.0 - LINT) 150,180,180
0080      150 IF (ELG(I) - ELC(I)) 160,160,180
0081      160 SLOPE = (ELG(I - 1) - ELG(I)) / L(I)
0082          IF (SOMAX - SLOPE) 170,190,180
0083      170 SLOPE = SOMAX
0084      180 ELC(I) = ELC(I - 1) - L(I) * SLOPE
        C LINEAR INTERPOLATION
0085      190 IF (ELG(I) - ELC(I)) 200,340,340
0086      200 LINT = (ELG(I - 1) - ELC(I - 1)) / ((ELG(I - 1) - ELG(I)) / L(I) -
             XSLOPE)
0087          LTOTIN = LTOT(I - 1) + LINT
0088          WINT = TOTW(1)
0089          AVTOTW(I - 1) = (TOTW(1) + TOTW(I - 1)) / 2.0
0090          XLAND(I - 1) = AVTOTW(I - 1) * LINT * 640.0 / (5280.0 ** 2.0)
0091          TLAND(I) = TLAND(I - 1) + XLAND(I - 1)
0092          DYKAIN = ((10.0 + 3.0 * DN) * DN) * 2.0
0093          DYKINT = (DYKA(I - 1) + DYKAIN) * LINT / (2.0 * 27.0)
0094          TDYKE(I) = TDYKE(I - 1) + DYKINT
0095          WRITE (6,210) DYKAIN, DYKINT, TDYKE(I)
0096      210 FORMAT (90X, 3F10.0)
0097          ELINT = ELC(I - 1) - LINT * SLOPE
0098          AINT = 0.0
```

```
0099         VOLINT = ((A(I - 1) + AINT) / 2.0) * LINT / 27.0
0100         VOLTOT(I) = VOLTOT(I - 1) + VOLINT
0101         VOLTOT(I - 1) = VOLTOT(I)
0102     220 WRITE (6,230)
0103     230 FORMAT (5X'FOLLOW GROUND-SLOPE', 1X)
0104         SLOPE = (ELG(I - 1) - ELG(I)) / L(I)
0105         IF (SOMAX - SLOPE) 260,240,240
0106     240 DN = ((VEL * CN) / (1.49 * SLOPE ** 0.50)) ** 1.5
0107         W(I) = QTOT / (VEL * DN)
0108         WRITE (6,250) ELINT, LINT, LTOTIN, VOLINT, VOLTOT(I - 1)
0109     250 FORMAT (9X, F8.2, 16X, F8.0, 10X, 2F10.0, 10X, F10.0)
0110         ELC(I) = ELG(I) - 0.2 * DN
0111         AINT = W(I) * 0.20 * DN + 3.0 * (0.2 * DN) ** 2.0
0112         GO TO 310
0113     260 WRITE (6,270)
0114     270 FORMAT (5X, ' GROUND SLOPE TOO STEEP', / )
0115         SLOPE = SOMAX
0116         DN = ((VEL * CN) / (1.49 * SLOPE ** 0.50)) ** 1.5
0117         W(I) = QTOT / (VEL * DN)
0118     280 WRITE (6,290)
0119     290 FORMAT (24H DROP STRUCTURE REQUIRED)
0120         WDROP = 0.67 * W(I)
0121         K = K + 1
0122         WRITE (6,300) ELINT, LINT, LTOTIN, VOLINT, VOLTOT(I - 1)
0123     300 FORMAT (9X, F8.2, 16X, F8.0, 10X, 2F10.0, 10X, F10.0)
0124         ELINT = ELINT - 5.0
0125         ELC(I) = ELINT - (L(I) - LINT) * SLOPE
0126         AINT = W(I) * 5.0 + 3.0 * 5.0 ** 2.0
0127     310 A(I - 1) = AINT
0128         L(I) = L(I) - LINT
0129         H(I) = ELG(I) - ELC(I)
0130         AINTO = W(I) * H(I) + 3.0 * H(I) ** 2.0
0131         A(I) = AINTO
0132         TOTW(I - 1) = W(I) + 2.0 * (10.0 + 3.0 * (DN + 1.0) + 3.0 * (DN +
            X1.0 - 5.0) + 10.0)
0133         TLAND(I - 1) = TLAND(I)
0134     320 DYKA(I - 1) = ((0.8 * DN + 1.0) * (10.0 + 3.0 * 0.8 * DN + 1.0)) *
            X2.0
0135     330 TDYKE(I - 1) = TDYKE(I)
0136         GO TO 360
       C END OF INTERPOLATION
       C EXCAVATION
0137     340 H(I) = ELG(I) - ELC(I)
0138         A(I) = W(I) * H(I) + 3.0 * H(I) ** 2.0
0139         IF (1.0 - LINT) 350,360,360
0140     350 VEL = 4.00
0141         DN = ((VEL * CN) / (1.49 * SLOPE ** 0.50)) ** 1.5
0142         W(I) = QTOT / (VEL * DN)
0143     360 VOL(I) = ((A(I) + A(I - 1)) / 2.0) * L(I) / 27.0
0144         VOLTOT(I) = VOLTOT(I - 1) + VOL(I)
0145         WRITE (6,370) SLOPE, W(I)
0146     370 FORMAT (5X, ' SLOPE=', F8.6, 5X, ' WIDTH=', F7.0, ' FT.', 1X)
0147         WRITE (6,380) I, ELG(I), ELC(I), L(I), LTOT(I), VOL(I), VOLTOT(I)
0148     380 FORMAT (I9, 8X, 2F8.2, 8X, 2F10.0, 10X, 2F10.0)
       C LAND ACQUISITION
0149         IF (H(I) - (DN + 1.0)) 390,400,400
0150     390 TOTW(I) = W(I) + 2.0 * (10.0 + 3.0 * (DN + 1.0) + 3.0 * (DN + 1.0
            X - H(I)) + 10.0)
0151         GO TO 410
0152     400 TOTW(I) = W(I) + 2.0 * (H(I) * 3.0 + 50.0)
```

```
  410 AVTOTW(I) = (TOTW(I - 1) + TOTW(I)) / 2.0
      XLAND(I) = AVTOTW(I) * L(I) * 640.0 / (5280.0 ** 2.0)
      TLAND(I) = TLAND(I - 1) + XLAND(I)
C DYKE QUANTITIES
      IF (8 - I) 420,430,430
  420 IF (H(I) - (DN - 1.0)) 440,430,430
  430 DYKA(I) = 0.0
      GO TO 450
  440 DYKA(I) = (DN + 1.0 - H(I)) * (3.0 * (DN + 1.0 - H(I)) + 10.0) *
     X2.0
  450 DYKE(I) = (DYKA(I - 1) + DYKA(I)) * L(I) / (2.0 * 27.0)
      TDYKE(I) = TDYKE(I - 1) + DYKE(I)
      WRITE (6,460) DYKA(I), DYKE(I), TDYKE(I)
  460 FORMAT (90X, 3F10.0)
C DROP STRUCTURES
      TWDROP = TWDROP + WDROP
      XK = K
C HIGHWAY BRIDGES
      WBRIDG = W(11) + W(15) + W(18)
C RAILWAY CROSSINGS
      WRAIL = W(11) + W(15)
C COST CALCULATIONS
      COSTE = VOLTOT(I) * COSTEU
      COSTL = TLAND(I) * COSTLU
      COSTD = TDYKE(I) * COSTDU
      COSTS = (TWDROP / 100.00) * COSTSU
      COSTH = WBRIDG * COSTHU
      COSTR = WRAIL * COSTRU
      TCOST = COSTE + COSTL + COSTD + COSTS + COSTH + COSTR
  470 CONTINUE
      WRITE (6,480)
  480 FORMAT ( / / )
      WRITE (6,490)
  490 FORMAT ( /, ' COST CALCULATIONS', / )
      WRITE (6,500)
  500 FORMAT (5X, ' ITEM', 26X, ' WIDTH', 11X, ' QUANTITY', 3X, ' UNIT C
     XOST', 10X, ' COST', / )
      WRITE (6,510) W(I), VOLTOT(I), COSTEU, COSTE
  510 FORMAT (5X, ' DIVERSION EXCAVATION', 3X, F10.0, 1X, ' FT.', F10.0,
     X4X, ' CU.YD.', F10.2, 5X, F10.0)
      WRITE (6,520) TLAND(I), COSTLU, COSTL
  520 FORMAT (5X, ' LAND ACQUISITION', 22X, F10.0, 5X, ' ACRES', F10.2,
     X5X, F10.0)
      WRITE (6,530) TDYKE(I), COSTDU, COSTD
  530 FORMAT (5X, ' DIVERSION DYKES', 23X, F10.0, 4X, ' CU.YD.', F10.2,
     X5X, F10.0)
      WRITE (6,540) TWDROP, K, COSTSU, COSTS
  540 FORMAT (5X, ' DROP STRUCTURES', 8X, F10.0, 1X, ' FT.', I10, 11X,
     XF10.2, 5X, F10.0)
      WRITE (6,550) WBRIDG, COSTHU, COSTH
  550 FORMAT (5X, ' HIGHWAY BRIDGES', 8X, F10.0, 1X, ' FT.', 8X, ' 3',
     X11X, F10.2, 5X, F10.0)
      WRITE (6,560) WRAIL, COSTRU, COSTR
  560 FORMAT (5X, ' RAILWAY CROSSINGS', 6X, F10.0, 1X, ' FT.', 8X, ' 2',
     X11X, F10.2, 5X, F10.0, / )
      WRITE (6,570) TCOST
  570 FORMAT (45X, ' TOTAL DIVERSION COST', 12X, ' $', F10.0, / / / )
      GO TO 600
  580 WRITE (6,590) W(1)
  590 FORMAT ( /, ' WIDTH =', F5.0, ' TOO NARROW', / / )
  600 CONTINUE
      CALL EXIT
      END
```

APPENDIX C-2

PORTAGE DIVERSION

TYPICAL OUTPUT

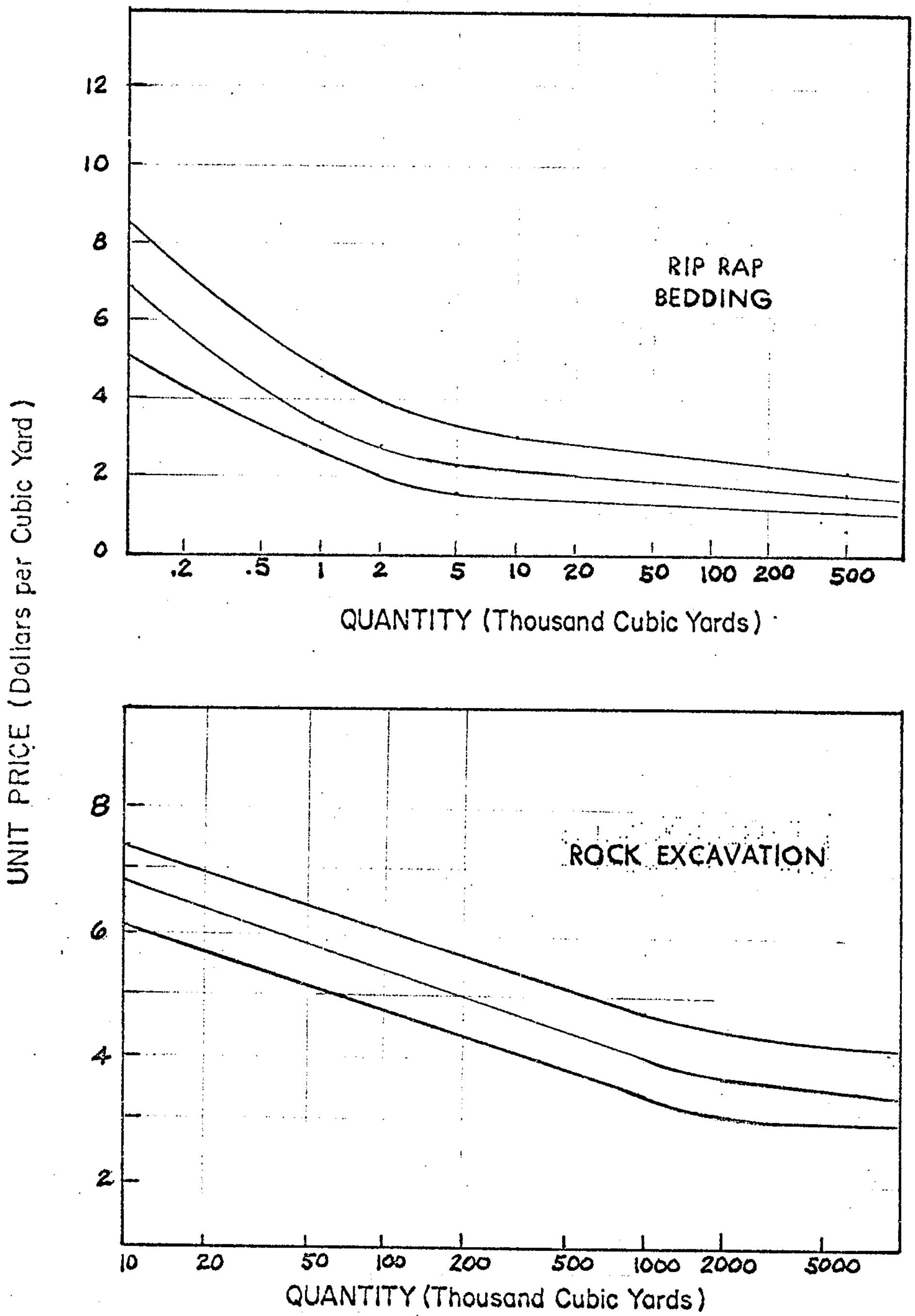
PORTAGE DIVERSION CALCULATIONS WIDTH = 316. SLOPE = 0.000200 QTOT = 30000. CFS

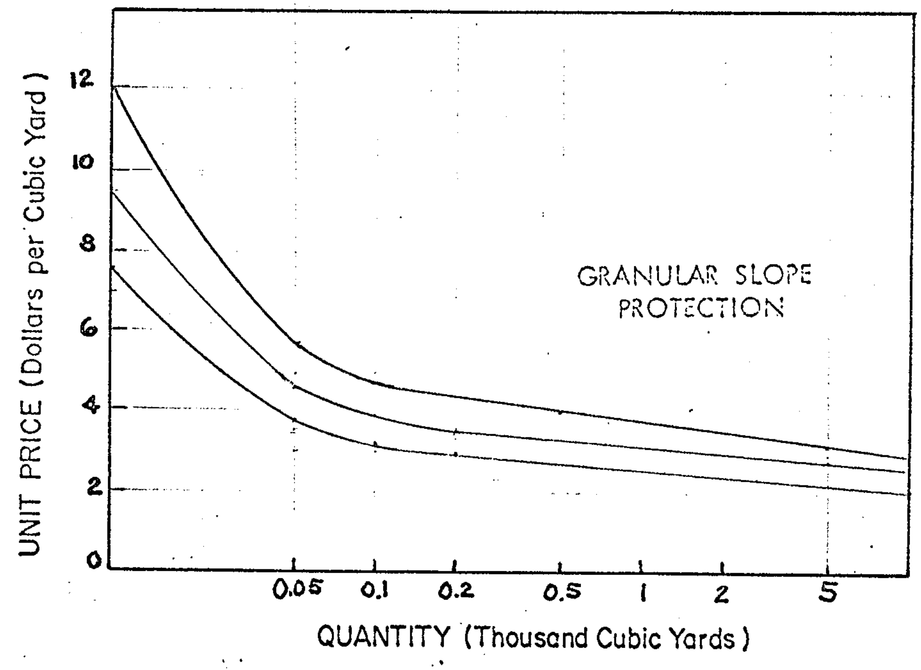
FSL = 728.00 DN = 18.99 VELOCITY = 5.00 FPS

NUMBER	ELINT FT.	ELG FT.	ELC FT.	LINT FT.	L FT.	LTOT FT.	VOLINT CU.YD.	VOL CU.YD.	VOLTOT CU.YD.	DYKE A SQ.FT.	DYKE CU.YD.	TDYKE CU.YD.
1		709.01	709.01									
SLOPE=0.000200			WIDTH=	316 FT.								
3		710.00	709.00		42.	2170.		249.	249.	0.	0.	0.
SLOPE=0.000200			WIDTH=	316. FT.								
4		715.00	708.94		310.	2480.		13462.	13712.	0.	0.	0.
SLOPE=0.000200			WIDTH=	316 FT.								
5		720.00	708.84		480.	2960.		52664.	66376.	0.	0.	0.
SLOPE=0.000200			WIDTH=	316. FT.								
6		725.00	708.62		1110.	4070.		203072.	269448.	0.	0.	0.
SLOPE=0.000200			WIDTH=	316 FT.								
7		730.00	708.32		1480.	5550.		390233.	659681.	0.	0.	0.
SLOPE=0.000200			WIDTH=	316. FT.								
8		732.00	708.06		1300.	6850.		422242.	1081923.	0.	0.	0.
SLOPE=0.000200			WIDTH=	316 FT.								
9		730.00	707.65		2070.	8920.		683915.	1765838.	0.	0.	0.
SLOPE=0.000200			WIDTH=	316. FT.								
10		725.00	707.21		2180.	11100.		610748.	2376586.	73.	2958.	2958.
SLOPE=0.000200			WIDTH=	316 FT.								
11		720.00	706.21		5000.	16100.		1064321.	3440907.	355.	39662.	42620.
SLOPE=0.000200			WIDTH=	316. FT.								
12		715.00	705.11		5500.	21600.		849758.	4290665.	815.	119147.	161767.
SLOPE=0.000200			WIDTH=	316 FT.								
13		710.00	703.65		7300.	28900.		749383.	5040048.	1390.	298034.	459802.
SLOPE=0.000200			WIDTH=	316. FT.								
14		705.00	700.88		13850.	42750.		892095.	5932143.	1829.	825716.	1285517.
SLOPE=0.000200			WIDTH=	316. FT.								
15		700.00	695.87		25050.	67800.		1255802.	7187945.	1827.	1696371.	2981888.
SLOPE=0.000200			WIDTH=	316. FT.								
16		695.00	694.33		7700.	75500.		223543.	7411487.	2627.	635172.	3617059.
										2544.	213302.	3830361
FOLLOW GROUND SLOPE												
GROUND SLOPE TOO STEEP												
DROP STRUCTURE REQUIRED												
	693.89			2227.		77727.	8763.		7420249.			
SLOPE=0.000300			WIDTH=	428. FT.								
17		690.00	686.55		7773.	85500.		536441.	7956689.	1034.	305719.	4136080.
SLOPE=0.000300			WIDTH=	748. FT.								
18		685.00	684.72		6100.	91600.		184041.	8140729.	909.	219444.	4355523.
										804.	4200.	4359722.
FOLLOW GROUND SLOPE												
GROUND SLOPE TOO STEEP												
DROP STRUCTURE REQUIRED												
	684.68			132		91732.	290.		8141019.			
SLOPE=0.000300			WIDTH=	748 FT.								
19		680.00	679.09		1968.	93700		163782.	8304600.	817.	52819.	4412540.

COST CALCULATIONS

ITEM	WIDTH		QUANTITY		UNIT COST	COST
DIVERSION EXCAVATION	748.	FT.	8304800.	CU.YD.	0.30	2491439.
LAND ACQUISITION			1229.	ACRES	100.00	122933.
DIVERSION DYKES			4412540.	CU.YD.	0.10	441254.
DROP STRUCTURES	1075.	FT	3		50000.00	537471.
HIGHWAY BRIDGES	1380.	FT.	3		500.00	689919.
RAILWAY CROSSINGS	632	FT	2		300.00	189547.
			TOTAL DIVERSION COST			$ 4472560.

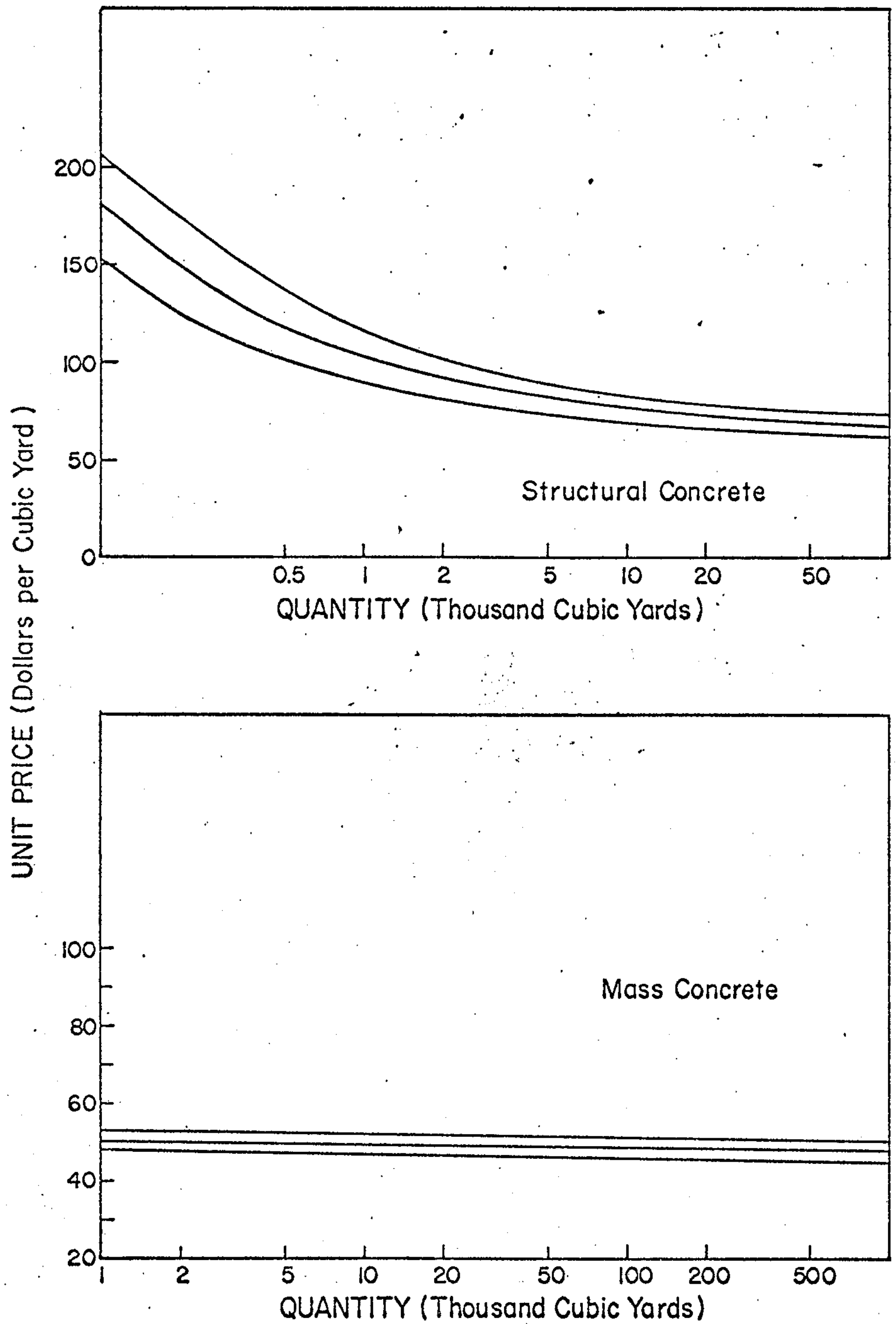


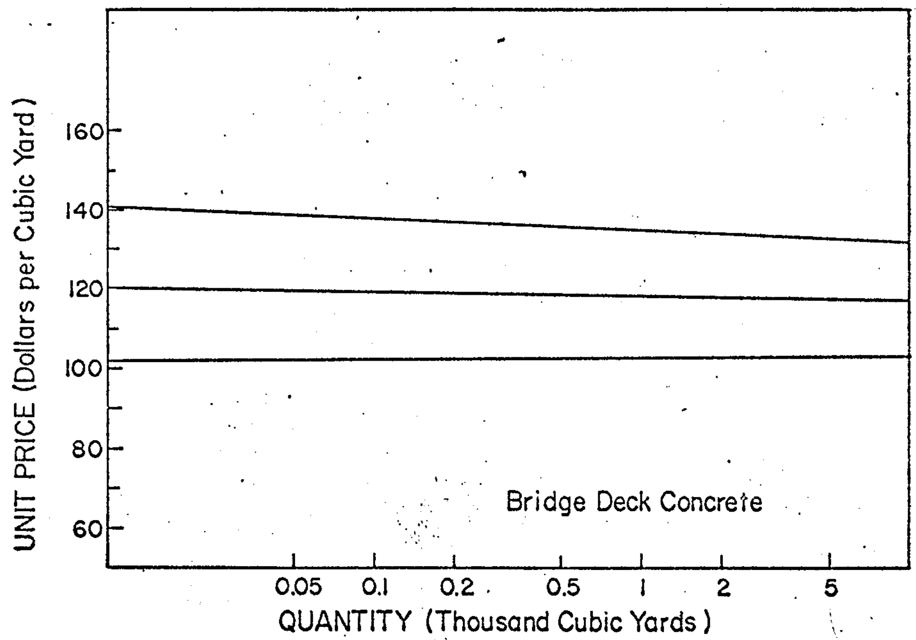


FLOOD CONTROL

APPENDIX D

UNIT COST CURVES

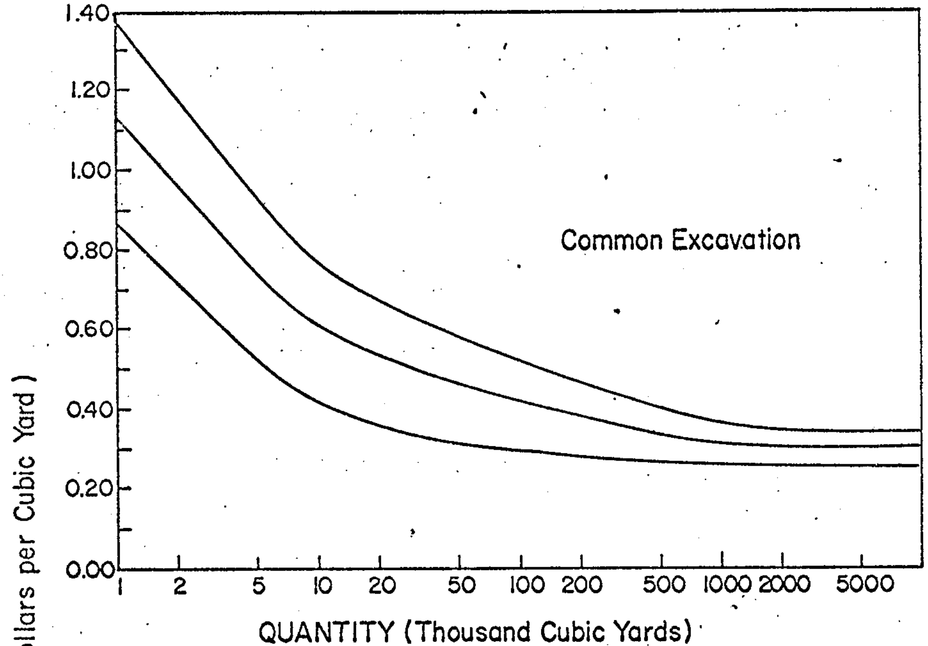


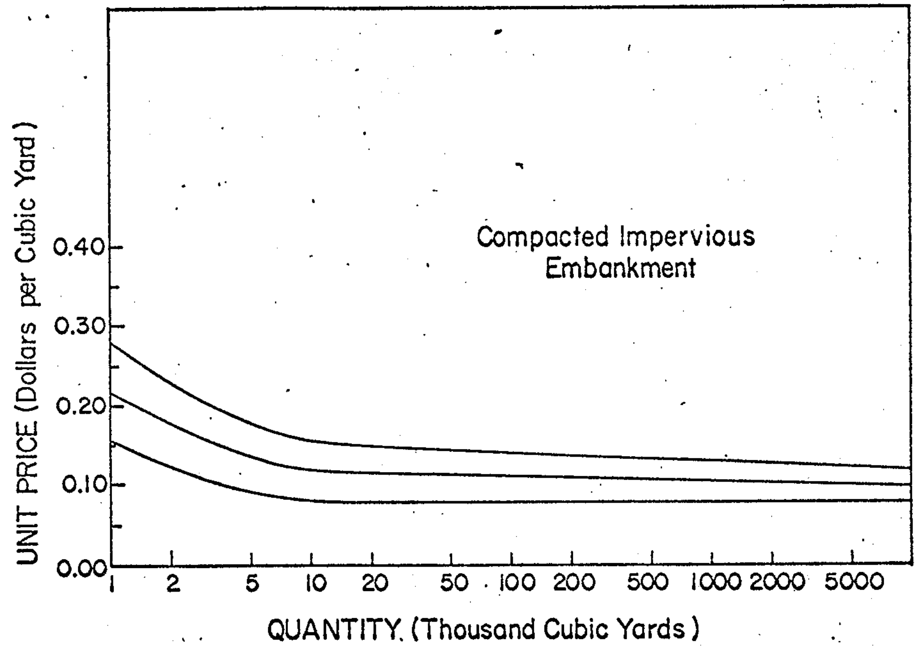


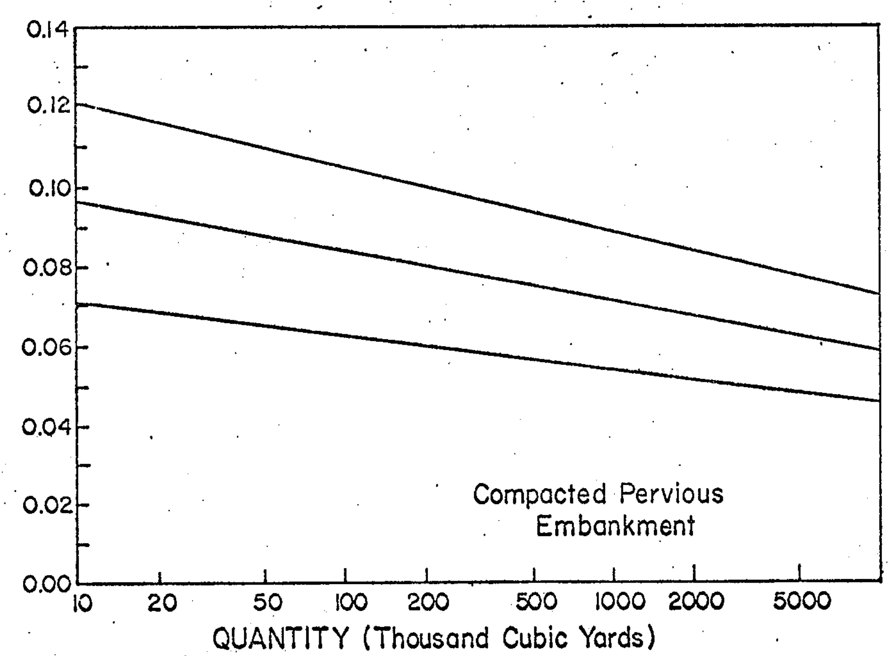
FLOOD CONTROL

APPENDIX D

UNIT COST CURVES



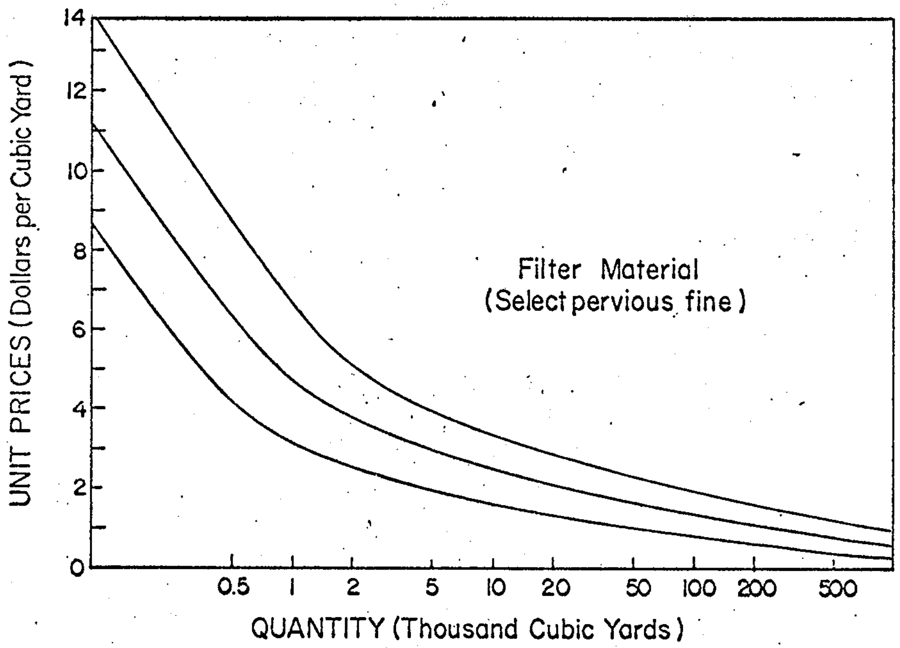


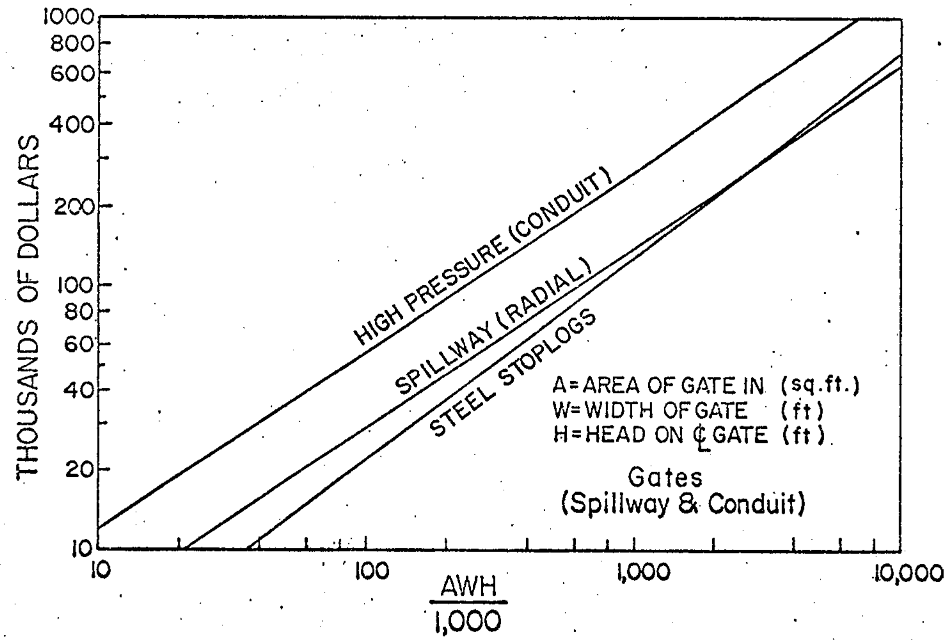


FLOOD CONTROL

APPENDIX D

UNIT COST CURVES





FLOOD CONTROL

APPENDIX D

UNIT COST CURVES