An average cross section is found by plotting all sections together, (Fig. V-4). A clearer picture can be had by plotting the sections from some common point, in our case the right bank slope. Because the actual drainage channel will be relatively unstable there is not much point in going into great detail in finding the average slope. Usually some simple geometric shape, such as a trapezoid, will be sufficiently accurate. The one feature to which we must pay particular attention is the bed width. On Fig. V-4 we have indicated three sections which might be considered average. We will now decide which one we will use. To do this we will use the following rule of thumb.

> Two-thirds of the maximum runoff hydrograph inflow (Table II-17) should flow within the bankfull level of the channel.

From Fig. V-4 we have estimated bed widths of 72, 85, and 100 feet; side slopes of 3:1; and bankfull depth 7 to 7.5 feet. From Table II-17, 2/3 of the maximum inflow is approximately 2000 CFS.

Using the procedure in King's HANDBOOK OF HYDRAULICS, Table 97, page 7-78.

$$Q = \frac{K^1}{n} b^{8/3} S^{1/2}$$

With

$$Q = 2000 \text{ CFS}$$
$$n = 0.035$$
$$S = .00025$$

$$K' = \frac{4420}{b^{8/3}}$$

b	$b^{8/3}$	K'	D/b	D
72'	90,000	.0492	.1193	8.6'
85'	137,000	.0323	.0943	8.0'
100'	214,000	.0207	.0734	7.34'

UNIFORM FLOW DEPTHS

A bed width of 100 ft. will carry a discharge of 2000 CFS at approximately bankfull level. We will use this value for our design width. The narrower channel sections at stations 1500 and 6500 ft. can be improved and widened if necessary.

5.3.3 COMPUTATION OF K_d FACTOR

We now have the information required to prepare the K_d curve. From pg. V-2,

$$K_d = \frac{1.486}{n} \; a \; r^{2/3}$$

$$\text{with } n = .035$$

$$K_d = 42.5 \; a \; r^{2/3}$$

With 3:1 side slopes, $a = bd + 3d^2$

$$wp = b + 6.32 \; d$$

$$r = \frac{a}{wp}$$

The equation, will be solved for selected values of the flow depth, d, and plotted. Values of d greater than 8 feet will not be used because at this point over-bank flow takes place and the average cross section will no longer be representative.

d	a	wp	r	$r^{2/3}$	K_d
1	103	106	0.97	0.98	4.290
2	212	113	1.88	1.53	13,780
3	327	119	2.74	1.96	27,200
4	448	125	3.58	2.34	44,500
5	575	132	4.35	2.66	65,000
6	708	138	5.13	2.97	89,000
7	847	144	5.88	3.26	117,200
8	992	151	6.56	3.50	147,800

K_d COMPUTATIONS

5.4 COMPUTING WATER SURFACE PROFILES

The computation of water surface profiles consists of starting with a given discharge at a point in the channel where the depth of flow is known, and computing the slopes of successive reaches upstream. The point of known depth will be at some control section in the channel, either an obstruction or a drop where the flow may pass through critical depth. Usually the point where a channel enters a river will constitute a control. With low river stages the channel flow may pass through critical depth as it falls into the river; with high river stages the river itself forms the obstruction that creates a backwater profile.

By assuming an average channel section, as we have with Sarai, the problem is simplified by eliminating the trial and error process required to determine the water levels between two different cross sections. We again wish to point out that we feel this simplification is justified because of the unstable nature of most of the channels we will encounter.

The solution consists of starting with a known water depth, selecting a second depth, either higher or lower depending on the profile, and computing the distance between the occurrence of these depths. The process is continued with progressively higher or lower depths until the accumulated lengths of the reaches equal the distance between the starting point and the structure. Figures V-7 to V-11 illustrate the profile computations.

For backwater curves the starting depth is measured from the river water surface to the average channel bottom (Fig. V-5). For drawdown curves the starting point is the critical depth which can be calculated using the method in King's Handbook, Table 109, pg. 8-75.

$$Q = K_c' \, b^{5/2}$$

$$\text{With } b = 100', \quad b^{5/2} = 100{,}000$$

For Sarai we will compute curves using discharges of 2000, 1500, 1000, and 500 CFS.

Q	K'_c	D_c/b	D_c
2000	.020	.0224	2.24
1500	.015	.0186	1.86
1000	.010	.0139	1.39
500	.005	.0086	0.86

CRITICAL DEPTHS

The drawdown curves are computed on Figures V-7 and 8, and the results plotted on Figure V-12 represent the tailwater curve for rivers stages less than Elev. 60.0.

Similarly tailwater curves are plotted from the results of backwater computations for river elevations 64, 65, and 66. Tailwater elevations for river levels other than these can easily be interpolated from the curves.

The preparation of the tailwater rating curves concludes the required backwater studies.

APPENDIX A

TRIANGULAR UNIT HYDROGRAPHS

A.1 INTRODUCTION

Over the years a considerable effort has been spent by various agencies to determine a method of forecasting peak floods. With only limited basin data available the methods developed had to be of a synthetic nature and usually applied only to the particular region under consideration. In order to transfer a method relating to one region to an entirely different region it is necessary to understand and revise the empirical coefficients associated with the original region. This requires some basic rainfall-runoff data relative to the new region. Unfortunately the relative data often does not exist, yet development works must be planned and built and cannot wait for the collection of data. The engineer must then exercise his judgement. Using the results of studies by others he must develop a method of forecasting the magnitude and duration of a particular flood with reasonable accuracy. This section suggests a procedure for use in East Pakistan until additional data is available and a better method can be developed.

A.2 THE UNITGRAPH SHAPE

When a natural flood wave is defined by plotting discharge Vs. time, the resulting hydrograph forms a curvilinear shape. For small ungauged watersheds with limited hydrologic data available, the preparation of a synthetic unit hydrograph with a similar curvilinear shape is only a refinement of an approximation. Representing the unit hydrograph as triangular, instead of curvilinear, reduces the computations and the error thus introduced results in a slightly more severe condition because the triangular shape distributes a given amount of runoff over a shorter time interval. The triangular unitgraph will be used in this study.

A.3 THE TRIANGULAR UNITGRAPH

Figure A-3 illustrates a curvilinear unitgraph with a triangular unitgraph superimposed. The following are definitions of the various elements:

1. Time of Concentration, T_c:

   The time measured from the cessation of rainfall to the point of contraflexure on the recession limb of a curvilinear hydrograph. This approximates the time of travel for runoff to move from the most remote portion of a basin to the point of interest.

2. Lag Time to Peak, t_p :

The time measured from the center of mass of effective rainfall to the hydrograph peak. This approximates the time of travel from the widest portion of the basin (where runoff volume is greatest) to the point of interest. The U.S. Soil Conservation Service has found that as an average $t_p = 0.6\ T_c$.

3. Time of Rise, T_p :

The time measured from the beginning of effective rainfall to the hydrograph peak.

4. Time of Recession, T_R:

The time measured from the hydrograph peak to the point of zero discharge.

5. Hydrograph Base Length, T_b :

$$T_b = T_p + T_R$$

6. Unit Duration of Effective Rainfall, t_r :

Effective rainfall is the same as rainfall excess discussed in Section 2.6. For unitgraph purposes this time should be about 1/5 t_p .

A.4 TRIANGULAR UNITGRAPH FORMULAE DERIVATION

The unitgraph principle is based on the condition of 1 inch of rainfall excess evenly distributed over a basin area. By definition the total volume of runoff under a curvilinear and triangular unitgraph are equal. Also the peak discharges are equal and the time of rise are equal. The base lengths are not equal.

The area under a unitgraph curve equals the total volume of runoff. For the triangular unitgraph

$$(1)\ \text{Volume} = q_p \times T_b \times 3600 \times \tfrac{1}{2} = \text{cubic-feet}$$

The figure 3600 converts base time from hours to seconds.

Since one inch of effective rainfall equals 3630 cubic-feet of water per acre, the total volume of runoff from a basin is

$$(2)\ \text{Volume} = 3630\ A \doteq \text{Cubic-feet}$$

Where A is the basin area in acres.

Combining (1) & (2)

$$A = \tfrac{1}{2}\ q_p \cdot T_b\ \frac{3600}{3630}$$

or approximately

$$(3) \quad q_p = \frac{2A}{T_b}$$

This equation forms the basis for determining the elements of a triangular unitgraph. To complete the unitgraph the base length time must be found, and the relationship between T_p and T_R known.

Pf the various methods available for developing empirical unitgraphs, it was decided to make use of the Snyder method as it is based on data that can be easily obtained from a topographic map. Snyder's equations for measuring lag and peak discharge are

$$(4) \quad t_p = C_t \, (L\bar{L})^{0.3}$$

$$(5) \quad q_p = \frac{C_p \, A}{t_p}$$

L = Basin length in miles along the principle drainage channel

$\bar{L}$ = Length in miles measured along the principle drainage from the point of interest to a point opposite the basin centroid.

C_t = Coefficient measuring channel slope effect and storage effect

C_p = Coefficient measuring flood wave and storage effect.

Combining equations (3) & (5) and eliminating q_p and A

$$(6) \quad T_b = \frac{2t_p}{C_p}$$

Substituting the U.S. SCS relationship

$$(7) \quad t_p = 0.6 \, T_c$$

In equation (6), we get

$$(8) \quad T_b = \frac{1.2 \, T_c}{C_p}$$

In a later work Snyder developed a relationship for determining time of concentration.

$$(9) \quad T_c = C \left(\frac{10.L \, n}{\sqrt{s}} \right)^{0.6}$$

n = Manning roughness coefficient for the principal channel

C = Coefficient depending on the type of drainage system

s = Weighted channel slope in %

For very flat land the average slope, computed as total fall ÷ total channel length, should closely approximate the weighted slope.

Expressing slope in feet/mile, or,

S = 52.8 s

Equation (9) can be re-written

$$(10) \quad T_c = 13.1\,C \left(\frac{L^2 n^2}{S}\right)^{0.3}$$

The problem now becomes one of selecting values for coefficients C, C_p and C_t relative to East Pakistan.

Snyder found that the value of C in the Washington, D.C. area varies from 1.5 to 2.8 with an overall value of 2.0 evident for natural basin. He selected to use an average value of 1.7, however to be conservative. The units for C are hours/mile, so for basins identical except in slope the flatter basin would have the greater C value giving a longer time of concentration. A limited amount of investigations of basins in East Pakistan suggested a value for C in the neighbourhood of 2.4.

The coefficients C_p and C_t measure slope and storage effects. It was reasoned that possibly a loose relationship existed between them. Assuming two basins with identical shape characteristics, L, $\bar{L}$, and A, but differing in slope and storage, it can be seen from equations (4) & (5) that q_p varies with C_p/C_t. In general it can be expected that flat basins have a higher storage capacity than the steeper basins, and a lower peak discharge. Thus with two similar basins the flatter should have the lower value of C_p, indicating greater storage, and a larger value of C_t, indicating a longer time of concentration.

Three sets of Snyder coefficients were available for greatly different terrain; the flat U.S. Gulf Coast, the Appalachian highlands, and the mountains of Southern California. These points were plotted as C_t vs. C_p and joined by a curve. It was reasoned that this curve might constitute an average relationship between C_p and C_t.

Data were then collected for 93 basins in widely separated regions. Snyder coefficients were computed and plotted on the C_t Vs. C_p curve. The plot of the points tended to follow the original curve.

On the basis of these studies it was felt that the C_p vs. C_t curve could be used to determine the base length of a triangular unitgraph.

A.5 DETERMINING THE TRIANGULAR UNITGRAPH ELEMENTS

The procedure for developing a triangular unitgraph is as follows:

1. Determine the time of concentration:

$$(10) \quad T_c = 31 \left(\frac{L^2 n^2}{S}\right)^{0.3}$$

The included coefficient C is 2.40. A method for determining n is given in Appendix B.

2. Determining lag to peak and time of rise:

$$(7) \quad t_p = 0.6\,T_c$$

$$T_r = t_p + t_r/2$$

The unit rainfall duration is the same unit time interval used for developing the design storm and should be approximately 1/5 t_p.

3. Determine coefficient C_t :

$$(4) \quad C_t = \frac{t_p}{\left(L \bar{L}\right)^{0.3}}$$

4. Find coefficient C_p from the C_p vs. C_t curve:

5. Determine the unitgraph base length:

$$(8) \quad T_b = \frac{1.2\ T_c}{C_p}$$

6. Determine the unitgraph peak discharge:

$$(3) \quad q_p = \frac{2A}{T_b}$$

APPENDIX B

CALCULATION OF RIVER ROUGHNESS COEFFICIENTS

The procedure to be used for determining the roughness coefficients for rivers and drainage channels is the same as was used for the sluice outfall channel in Section V, Article 5.3.1. A breakdown of coefficients to be used for various channel conditions is given in Table V-3, Page V-7.

The following is an example using the Sarai River, the principle drainage course through Sarai Basin, as an illustration.

	Value of n
1. Material involved: Earth	0.020
2. Degree of irregularity: Moderate to severe. (Eroded and sloughed banks; some scouring; sand bars and sand waves present)	0.015
3. Variations of cross-section: Alternating frequently. (Large and small cross-sections alternate frequently; main flow shifts from side to side)	0.010
4. Relative effect of obstructions: negligible. (No fallen logs, stumps, roots, boulders or debris deposits present).	0.000
5. Vegetation: Low. (No bushes or trees present; no cultivation in channel bed; depth of flow at flood is 2 to 3 times height of side vegetation).	0.005
Summation of n values	0.050
6. Degree of meandering: Minor to appreciable. (Ratio of meander length to straight length of channel is about 1.2)	m = 1.10

Estimated Roughness Coefficient:

$$n = 1.10 \times 0.050 = 0.055$$

MANNING ROUGHNESS COEFFICIENTS

As a supplement to the coefficients described in Article 5.3.1, the following are Manning coefficients as described by Sribny in the U.S.S.R.

1.	Natural channels with very favourable conditions (Clean, straight, with no obstructions, earthen bed with a free flow of water.)	.025
2.	Channels of permanent lowland water-courses in favourable conditions of bed and current flow(Mainly large and medium rivers), and, Periodical water-courses(Large and small) where the surfaces and form of the bed are in very good condition.	.033
3.	Comparatively clean channels of constant low land water-courses in ordinary conditions, winding with some irregularities in the direction of flow; or straight but with irregularities in the profile of the bed (Shoals, depressions, boulders in places); and, Earthen channels of intermittant watercourses (Dry wadis) in relatively favourable conditions.	.040
4.	Channels(of large and medium rivers) considerably obstructed, winding and partly overgrown with vegetation, or stony with a turbulent current; Ephemeral (shower or springtime) watercourses bearing considerable amount of alluvial materials during time of flood with a bed covered with large boulders or vegetation (reeds,etc.); or The flood plains of large and medium rivers relatively cultivated, covered with a normal amount of vegetation (grass, shrubs).	.050
5.	The beds of ephemeral watercourses, greatly obstructed and winding. Comparatively overgrown, uneven, little cultivated river flood plains(gullifs), bushes, trees, with backwaters); and Boulder and stone beds of the mountain type with irregular water surface; Rapids in lowland rivers.	.067

6. Rivers and flood plains considerably overgrown (with a weak current) and with large, deep depressions;

Boulder-strewn channels of the mountain type with a turbulent foaming current and a rough water surface with spray flying up from it. .080

7. Flood plains as in the previous category but with highly irregular cross-currents, back-water, etc.;

Channels of the mountain waterfall type with large boulders, a winding bed, marked water-falls and so foaming that the water loses its transparency and looks white. The noise of the stream drowns all other noises and cover-sation difficult. .100

8. Mountain rivers of approximately the same characteristics as in the previous category;

Rivers of the marshy type (bed of reeds, bog, the water almost stagnant in places;

Flood plains with very large dead spaces, and with local depressions, lakes, etc. .133

9. Streams consisting of mud, stones, etc.;

Flood plains without outlet (complete forest, of the Taiga Type.) .200

10. Slopes of catchment areas in their natural state (the factor varies depending on the character of the slope). .250 to 1.000

APPENDIX C

RAINFALL

APPENDIX D

RELATED EMPIRICAL FORMULAE

D.1 THE RATIONAL FORMULA

The lack of records for rainfall intensity, runoff, and stream flow have led to the development of a large number of empirical formulae for estimating flood flow. Each formula is usally based on limited data for a particular region and involves one or more coefficients that must be evaluated by judgement. One of the leading formulae in the class which includes rainfall as a parameter is the rational formula.

$$Q = ciA$$

i = Rainfall intensity in inches/hour
c = A runoff coefficient
A = Basin area in acres
Q = Peak runoff in cfs.

A method for developing the coefficients has been outlined by Bernard. Assuming the area around the Eastern U.S. Gulf Coast to be approximately comparable to East Pakistan, the following coefficients can be developed.

The runoff coefficient:

$$C = C_{Max}\left(\frac{T_p}{100}\right)^x$$

T_p = Recurrence period (Assume 10 years)

$X = 0.18$

$C_{max} = 0.70$

Calculating for C,

$$C = 0.463$$

Rainfall intensity:

$$i = \frac{K\,(T_p)^x}{t^n}$$

$K = 45$

$(T_p)^x = 1.515$

$n = 0.73$

t = Rainfall duration, in minutes, equal to, or greater than the time of basin concentration.

Calculating for i :

$$i = \frac{68.2}{t^{0.73}}$$

Substituting the coefficients into the rational formula,

$$Q = \frac{32\ A}{t^{0.75}}$$

Calculating Q for Sarai Basin:

$$A = 27{,}500 \text{ Acres}$$
$$t = 41 \text{ hours} = 2460 \text{ minutes}$$

$$Q = \frac{32 \times 27{,}500}{(2460)^{0.75}} = 2980 \text{ CFS}$$

This compares with a peak of 3065 CFS from Section II, Table II-17.

D.2 U.S.S.R. Empirical formula

A formula developed for the steppe region in the Soviet Union is

$$Q = a\ S^{4/9}\ (b\ h\ r)^{4/3}$$

a = A channel roughness coefficient

$$a = \frac{0.0040}{n}$$

n = Manning's coefficient

S = Mean channel slope in parts per thousand

b = Mean width of drainage basin.

$$b = \frac{\text{Area (km}^2\text{)}}{\text{Channel length (km)}}$$

h = Total depth of direct runoff, in millimeters, during the time of basin concentration spanning the period of most intense rainfall.

r = Coefficient of regulation for depressions or lakes without outlet.

$$r = \frac{1-f}{1 + 25\ f}$$

f = Ratio of total lake or depression area without outlet to total basin area.

Estimating the peak flood for Sarai Basin:

$$n = 0.055 \quad \text{(Appendix B)}$$

$$a = \frac{0.004}{0.055} = 0.073$$

$$S = \frac{0.56 \text{ ft.}}{5.28} = 0.106 \text{ ft/1000 ft. (Pg. II-27)}$$

$$b = \frac{43 \text{ sq. miles}}{21.5 \text{ miles}} \times 1.61 = 3.22 \text{ km.}$$

To find h the depth of rainfall must be known during the period of greatest intensity spanning the time of basin concentration. Referring to the design storm, Table II-9, this would be the rainfall occurring during the 41 hours prior to the 18th hour of the 3rd day, or 9.06 inches. The total runoff depth can be determined using either Figure E-5 or E-6. Using the SCS method, the curve number is computed(Appendix E) and the net runoff taken off the proper curve as the difference between runoff at the beginning and end of the 41 hour period. (However, since it has already been computed we will use Table II-13.) The net runoff is

$$h = 5.82 \text{ inches} = 148 \text{ mm.}$$

The coefficient of regulation, r, is based on the effect of depression areas that do not release stored water as runoff. From the relationship it can be seen that the effect is about 4 times the actual water surface area in small depressions. This is because the area supplying storage to the depression is greater than the stored water surface area. Storage tanks in East Pakistan usually control little if any of the runoff around its immediate vicinity. Therefore, the effective tank area will be treated as having only ¼ its actual area. In Section E it is estimated that 2% of the total area comprises tanks; using 0.5% as effective,

$$r = \frac{1 - .005}{1 + (25 \times .005)} = 0.885$$

Computing the discharge:

$$\therefore Q = .073(.106)^{4/9}(3.22 \times 148 \times .885)^{4/3}$$

$$Q = 85.5 \text{ cubic-meters/sec.}$$

$$\text{or, } Q = 3020 \text{ cfs}$$

D.3 <u>TIME OF CONCENTRATION</u>

In Appendix A a formula developed by Snyder was given for determining the time of basin concentration.

$$(1) \quad Tc = 13.1\, C\left(\frac{L^2 n^2}{S}\right)^{0.3}$$

Using an estimated value of C = 2.4, and an average n value of 0.050. Equation (1) becomes

$$(2) \quad Tc = 5.2\left(\frac{L^2}{S}\right)^{0.3}$$

The U.S. Soil Conservation Service has develeped a similar relation from studies of a large number of drainage basins.

$$(3) \quad Tc = \left(\frac{11.9\, L^3}{H}\right)^{0.3}$$

The difference in basin elevation, H, from the high point to the outlet can be expressed as slope

$$H = SL$$

Where H is in feet, L in miles and S in Ft/mile. Expressing the relation in the same form as equation (2),

$$Tc = \left(\frac{11.9\ L^2}{S}\right)^{0.385}$$

The SCS formula and Snyder formula are in close agreement for basins with moderate slopes, but with very flat slopes the SCS formula yields shorter times of concentration. Because the Snyder formula takes into account storage, it is suggested that for very flat slopes the SCS exponent be revised to 0.40 bringing the two methods into close agreement. The SCS formula then becomes

$$(4) \quad Tc = 2.7\left(\frac{L^2}{S}\right)^{0.40}$$

Often it is impossible to measure the length of the major drainage path in a basin because of inadequate topography, or because the channel is not well defined. An empirical formula used in the U.S.S.R. is

$$L = 2.4\ A^{0.57}$$

Where A is the basin area in square miles. This should not be used as a substitute when actual measurement is possible.

APPENDIX E

RAINFALL - RUNOFF RELATIONS

The U.S. Soil Conservation Service has conducted rainfall-runoff studies on experimental watersheds to assist the Engineer in estimating the amount of runoff resulting from a given amount of precipitation. The method consists of determining the Hydrologic Soil group classification of the basin, the hydrologic soil-cover complex from land use and treatment, and by use of charts estimating runoff from total volume of rainfall. This method only gives the total amount of runoff and does not reference the runoff to a distribution as was done in Section II, Article 2.5. The SCS method can be used as a check on total runoff obtained by the method in Section II.

<u>HYDROLOGIC SOIL GROUPS</u>

Group A:

Deep sands with little silt or clay: also deep, rapidly permeable loss.

Sandy loam
Loamy sand and sand
Gravelly loam

Group B:

Sandy soils less deep than Group A with above average infiltration after thorough wetting.

Fine sandy loam
Clay loam
Silt loam
Loam

Group C:

Shallow soils and soils containing considerable clay and colloid; below average infiltration after pre-saturation.

Clay Loam
Silt Loam
Silty clay loam

Group D:

Clays of high swelling percent, shallow soils with nearly impermeable sub-horizon near the surface.

Soil textures in East Pakistan range from coarse sand to clay, but silty loams and silty clay loams predominate. Permeability generally ranges from moderate to very slow. Using the above group classification most of these soils will come under groups B and C. An estimate of the soil group can be made by using the Master Plan Supplement B.

ANTECEDENT SOIL CONDITIONS

Condition I:

A condition of watershed soils where the soils are dry, but not to the wilting point, and where satisfactory plowing and cultivation takes place. This condition is not considered applicable as a design condition.

Condition II:

The average case for conditions which have preceded the occurrence of the maximum annual flood on numerous watersheds.

Condition III:

When heavy or light rainfall and low temperatures have occurred during the 5 days previous to the design storm, and the soil is nearly saturated.

ESTIMATING RUNOFF

Figure E-1 is a chart showing various rainfall-runoff curves. Table E-2 relates the curve numbers to land use, soil group, and antecedent condition. Table E-3 is a district breakdown of land use in East Pakistan by percentage of total area. These percentages are approximate because land use changes seasonally and from year to year. Table E-3 should only be used when a suitable survey has not been made. The SCS method does not take into account the additional retention available with paddy land. Therefore, the following example will be used as a check on non-paddy land runoff previously computed in Section II, Table II-11. Using Tables E-2 and E-3, the basin area is broken down into its land use components and the weighted runoff curve number is computed. Table E-4 illustrates the procedure.

In the calculations the paddy area is treated using its actual cover, but the bund retention is not considered. As with the procedure under Section II, paddy land runoff will be considered the same for both conditions II and III. The calcualtions yield a weighted curve number of 66 for Condition II and 75 for Condition III.

On Figure E-1 the rainfall runoff relation using the data from Table II-11 has been plotted. The plots follow curves 68 and 74 for Conditions II and III, respectively, showing close agreement between the two methods.

A weighted basin average including paddy land retention can be estimated by substracting a pro-rated amount of retention from the runoff obtained from the SCS curves. For example, after the 16th hour on the 3rd day of the Sarai storm 9.54 inches of rainfall has occurred. Using the calculated SCS curve number 66 total runoff upto this point is 5.35 inches. The difference between the sum of initial loss and depression storage for non-paddy land and depression storage for paddy land is (4.00" - 1.50") = 2.50". This loss prorated over paddy land is 2.50 x 42% = 1.05". Reducing the net runoff by this amount the weighted basin runoff is

5.35" - 1.05" = 4.30".

This compares with 4.43 inches under Section II.

Another rainfall-runoff relation chart that can be used for comparison has been prepared by Linsley, Kohler & Paulhus (Fig.E-5). This chart relates the various curves to the general conditions of dry, normal, wet, etc. With soil group B Sarai curves follow "normal" for Condition II, and "Wet" for Condition III. It would appear from Figures E-I and E-5 that soil group C should have a different loss rate than Soil Group B. Therefore the following is recommended:

For Soil Group C extract subsequent losses at the rate of 0.03 inches/hr. All other losses remain as stated for Group B.

If the runoff computations for Sarai are repeated using Soil Group C, the Condition II curve would fall along the SCS curve number 74, and Condition III along curve Number 80. Calculating the curves numbers by the SCS method Condition II is 76 and Condition III is 90. Since the SCS procedure does not provide for inclusion of minimum infiltration rate after the soil becomes saturated, the higher curve number is expected for very wet conditions.

APPENDIX F

CROP DAMAGES

The extent of crop damage due to flooding will depend on the age of the plant at the time of flood and the depth of submergence. The cropping pattern of the principle crops and the depth of flooding they can stand has been given in Article 3.2. Table F-1 is the estimated cost of production per acre for the principle crops.

For economic studies it is assumed that if Aus or transplanted Aman is damaged prior to late August, there is sufficient time for the farmer to re-transplant and still raise a crop. The estimated loss to the farmer for seed and work will be Rs. 45 to 50 per acre. After late August it is assumed that total loss of crop occurs. This will be assessed at Rs. 200 per acre for normal conditions and Rs. 300 per acre when the land is part of an extensive irrigation development project.

ESTIMATED AVERAGE PRODUCTION COSTS PER ACRE

AUS RICE

Seed	Rs.	18.12
Plowing		67.50
Raking and weeding		29.50
Harvesting, threshing, storing		44.00
Land rent, 6 months		3.20
Land interest, 6 months		30.00
Total	Rs.	191.32

TRNASPLANTED AMAN

Seed	Rs.	3.75
Plowing		76.50
Transplanting		24.50
Weeding		14.00
Harvesting, threshing, storing		39.75
Land rent, 6 months		3.20
Land interest, 6 months		30.00
Total	Rs.	191.70

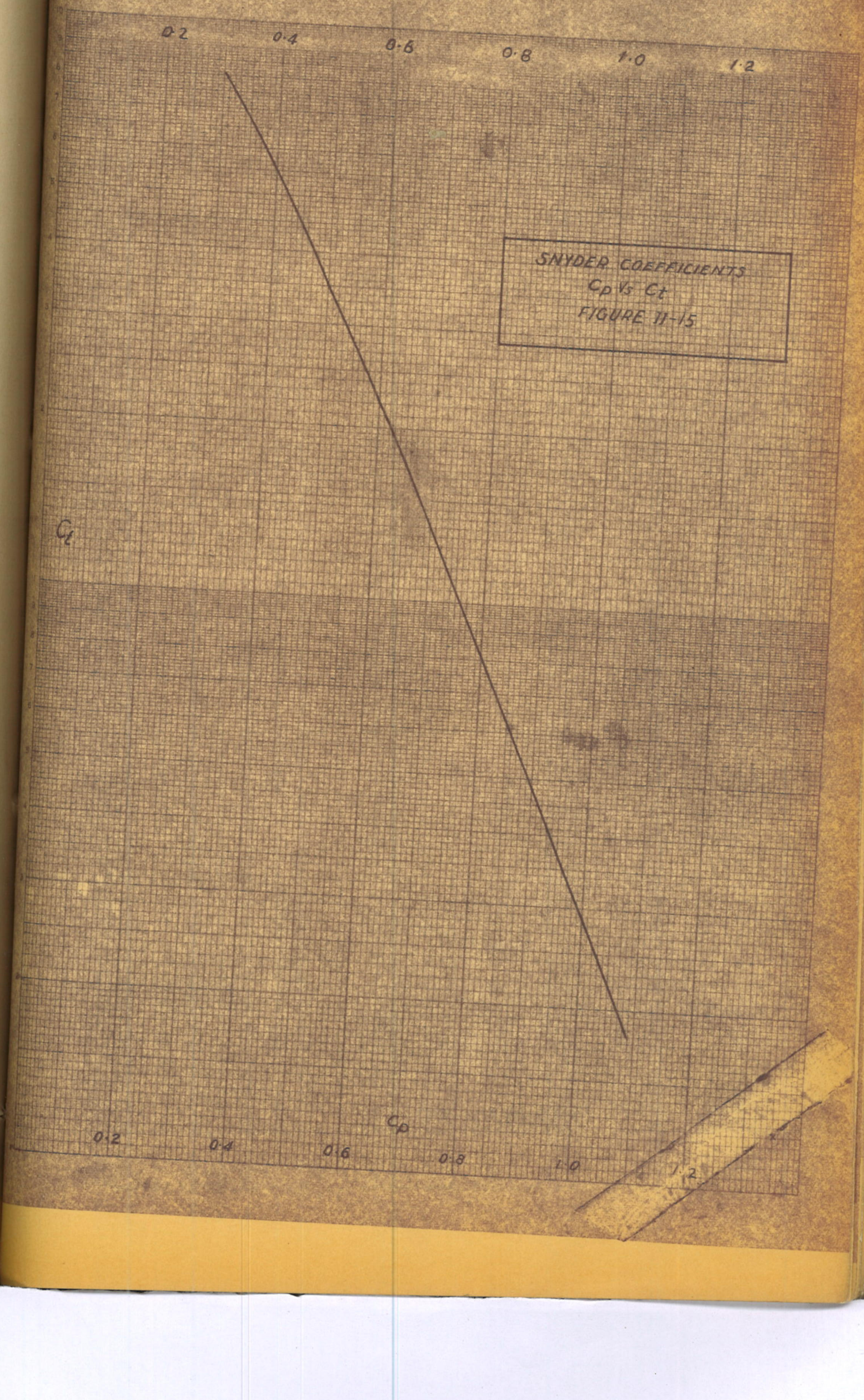
JUTE

Seed		7.80
Mannure		11.00
Plowing		58.73
Weeding		64.01
Separating & Washing		38.86
Harvesting		32.55
Land rent		3.75
Land interest		38.00
Total	Rs.	253.98

SUGARCANE

Seed	Rs.	100.00
Man Labour		300.00
Bullock Labour		62.50
Fertilizer		25.00
Land Rent		3.00
Land interest		60.00
Total	Rs.	550.5

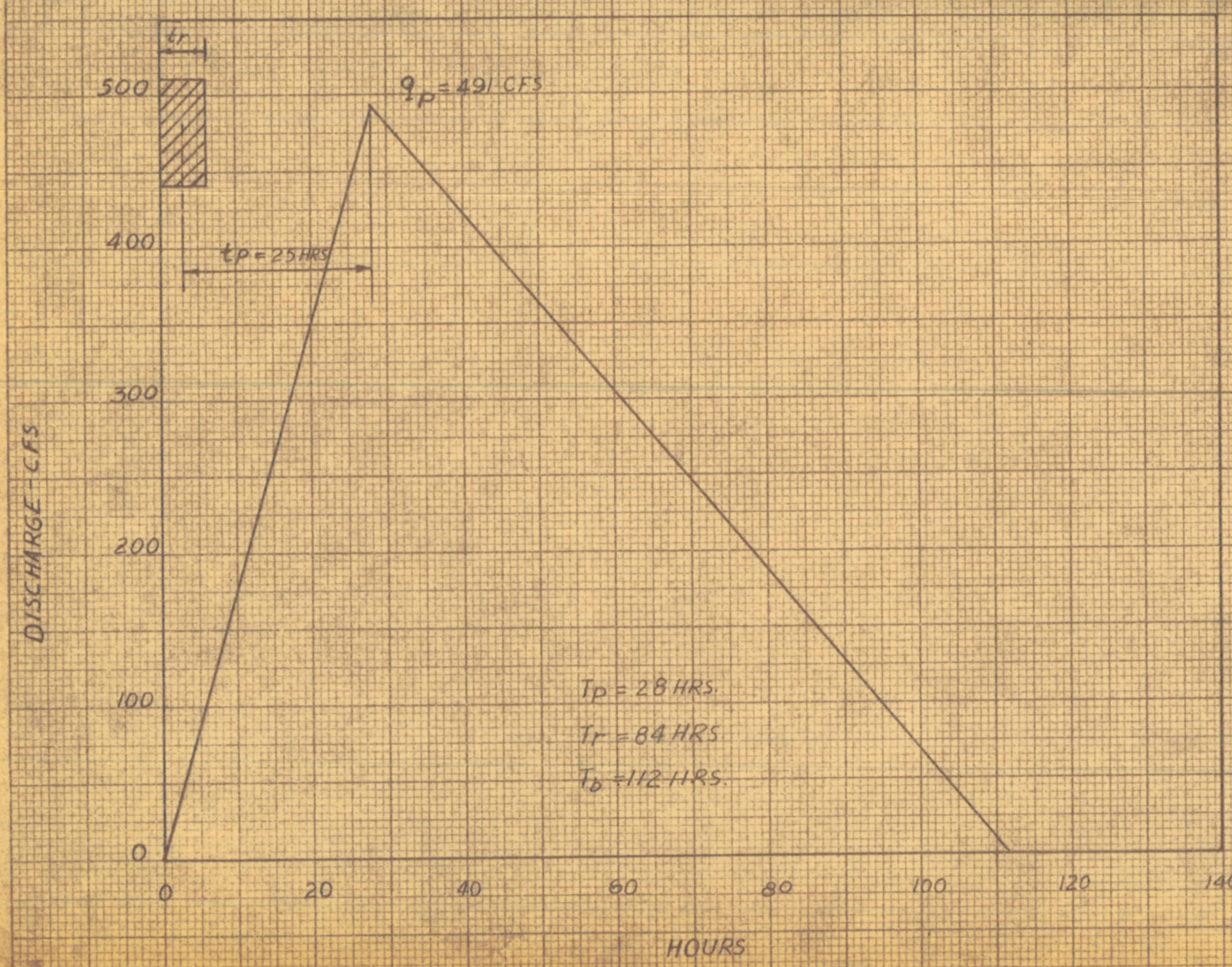
TABLE F-1



SNYDER COEFFICIENTS

C_p Vs C_t

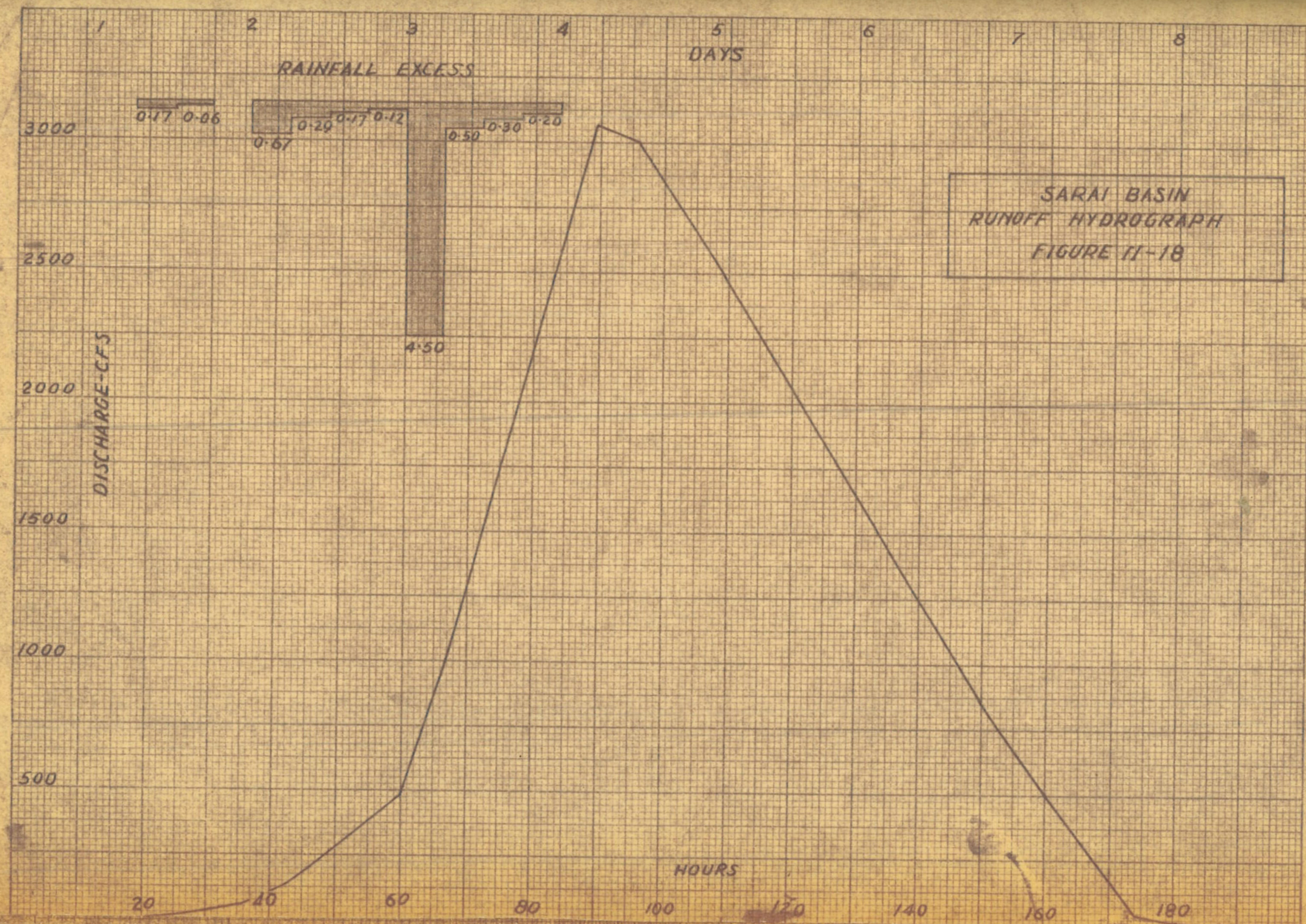
FIGURE 11-15



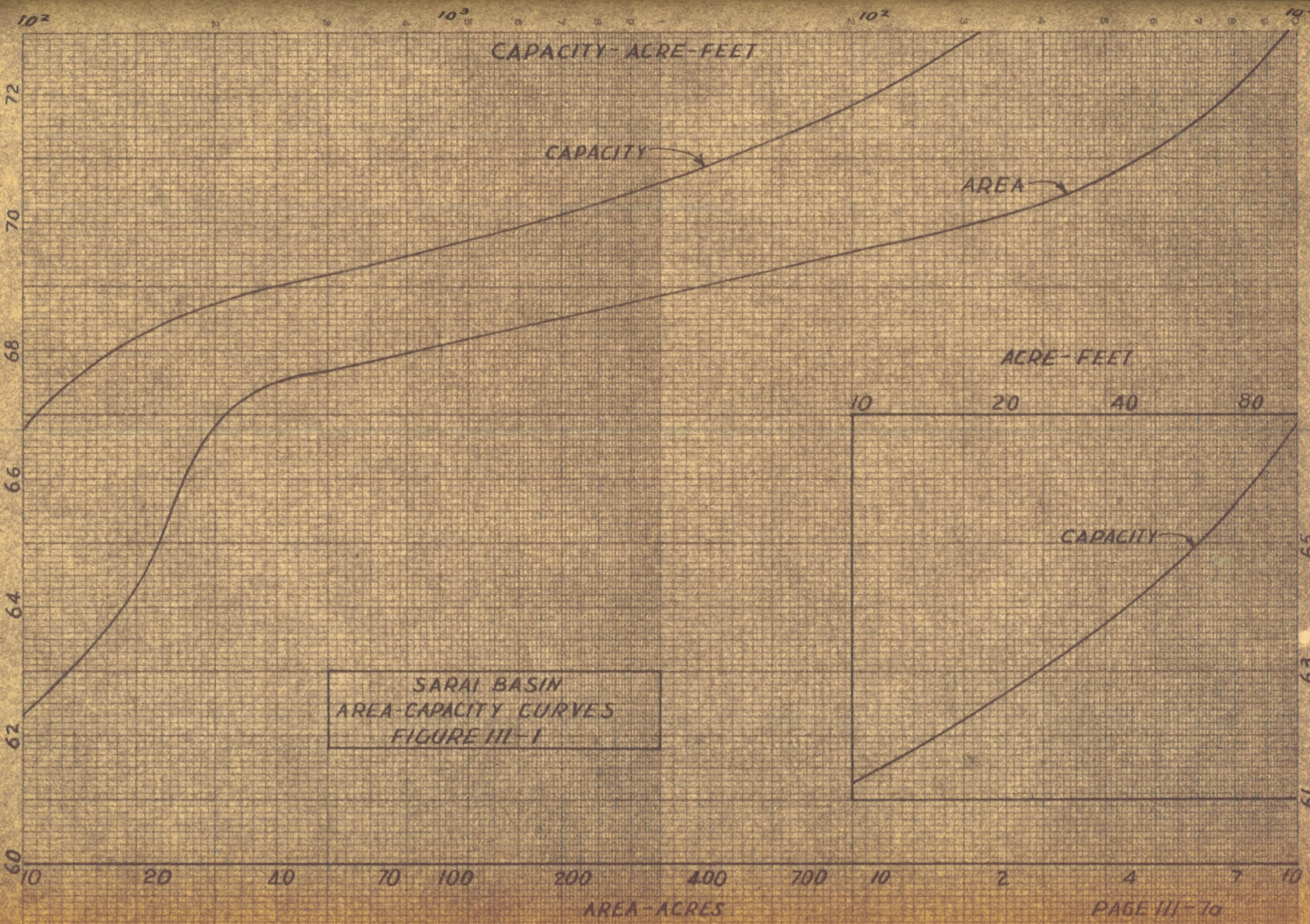
SARAI BASIN
UNIT HYDROGRAPH

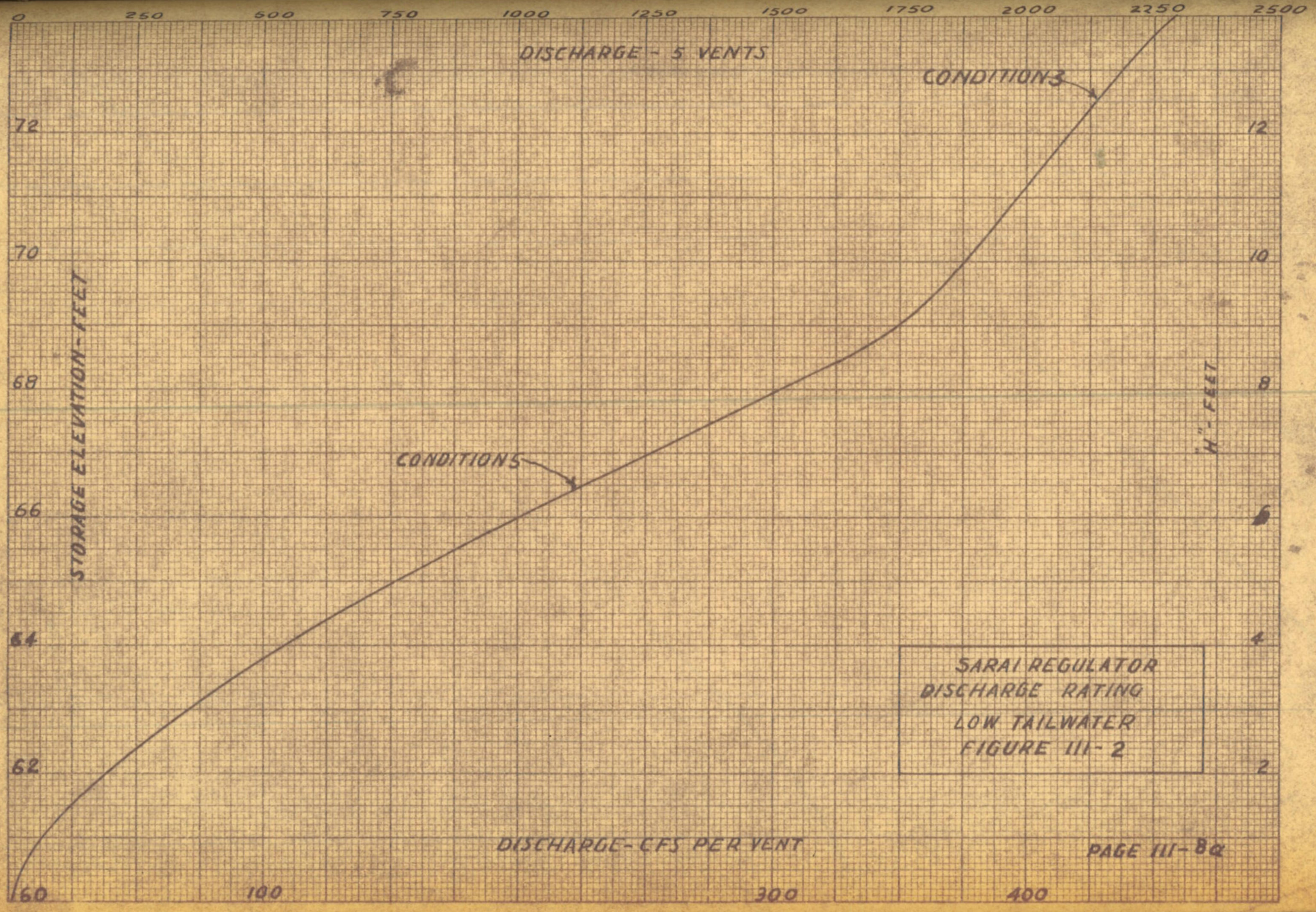
FIGURE II-16

MSC(S.B)2

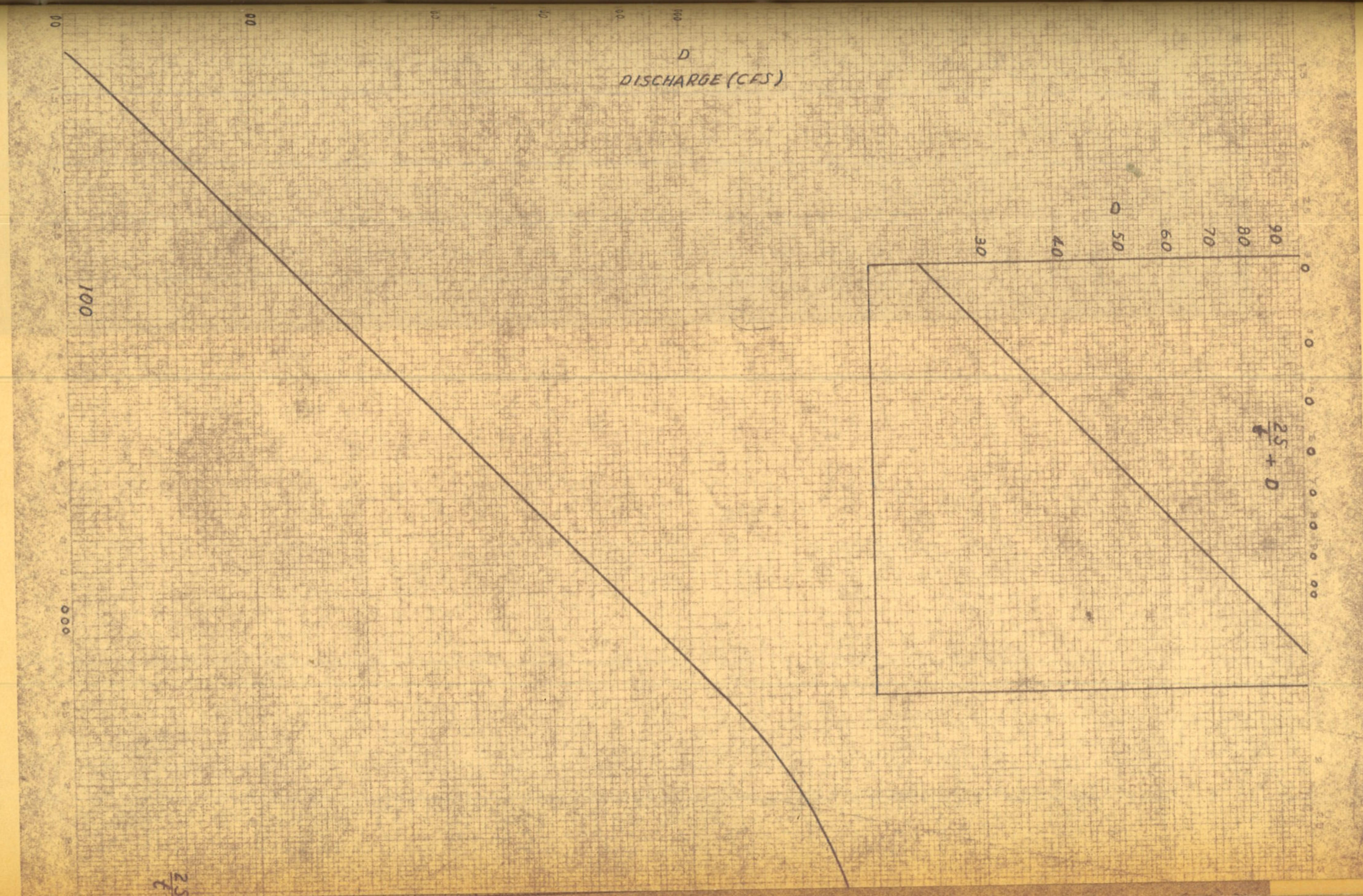


SARAI BASIN
RUNOFF HYDROGRAPH
FIGURE 11-18


CAPACITY - ACRE-FEET
10^2
10^3
10^2
10^5
CAPACITY
AREA
ACRE-FEET
10
20
40
80
CAPACITY
72
70
68
66
64
62
60
65
63
61
10
20
40
70
100
200
400
700
10
2
4
7
10
AREA-ACRES
SARAI BASIN
AREA-CAPACITY CURVES
FIGURE III-1



SARAI REGULATOR
DISCHARGE RATING
LOW TAILWATER
FIGURE III-2


D
DISCHARGE (CFS)
100
30
40
50
60
70
80
90
2S/t + D

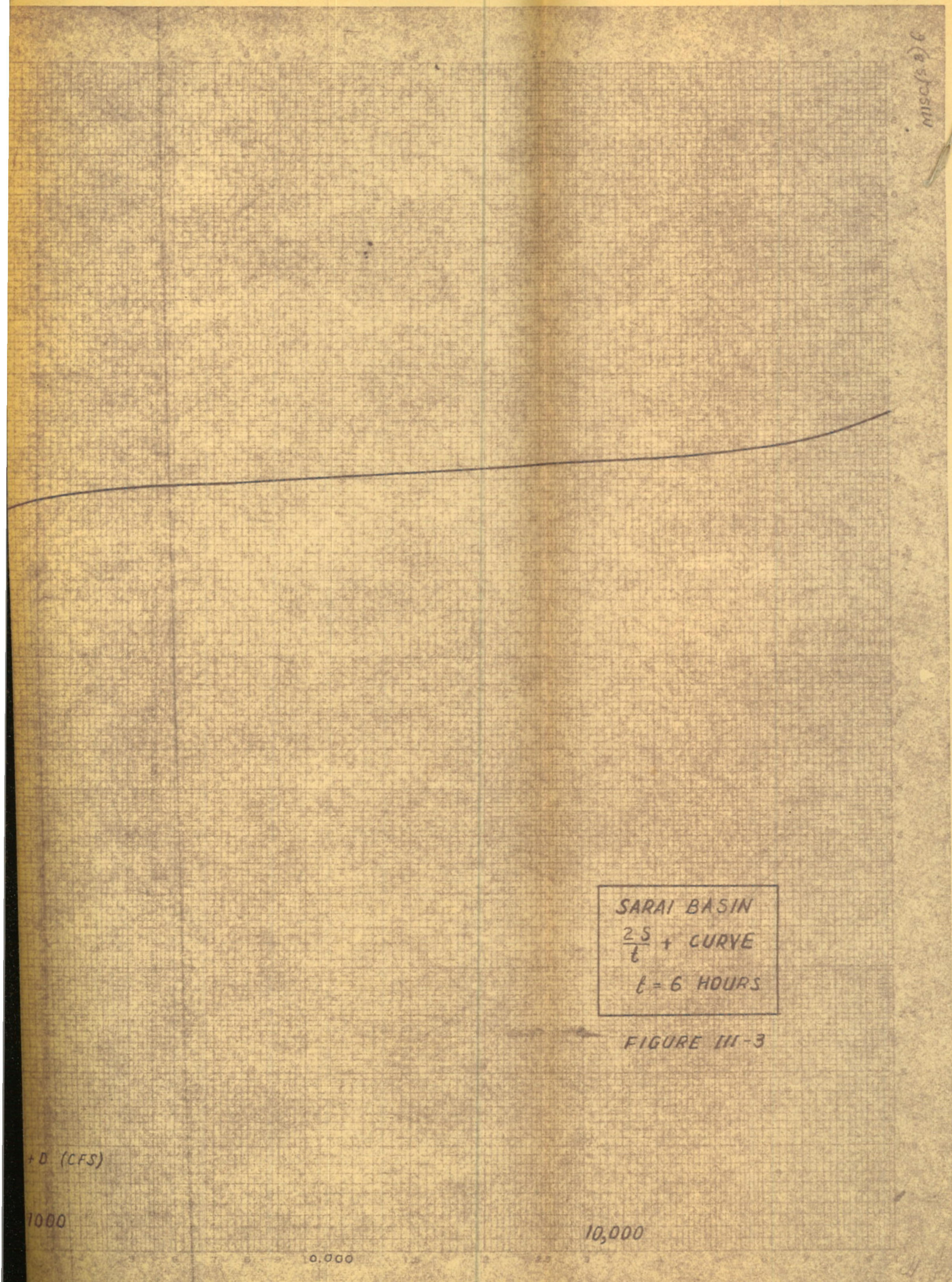
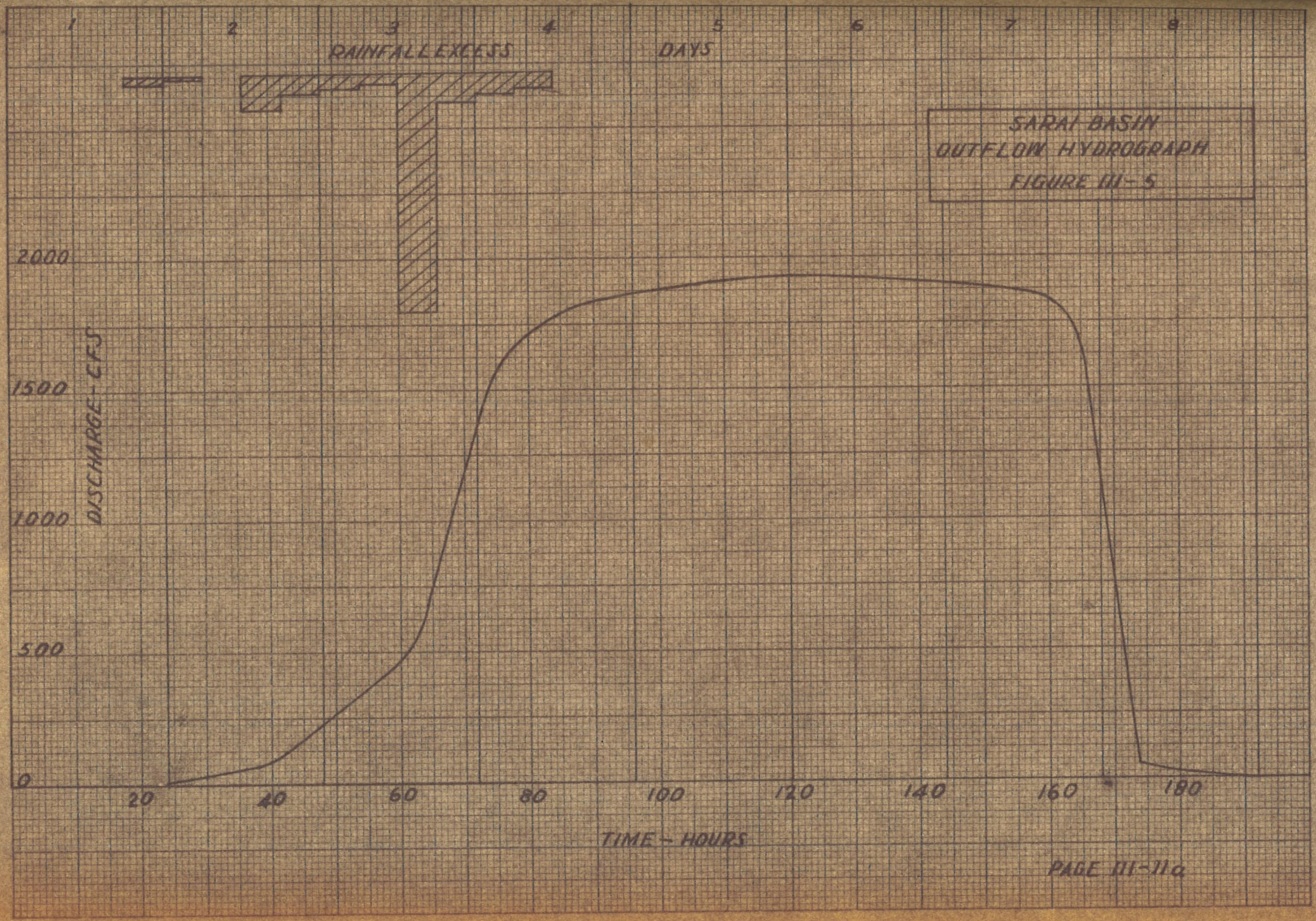
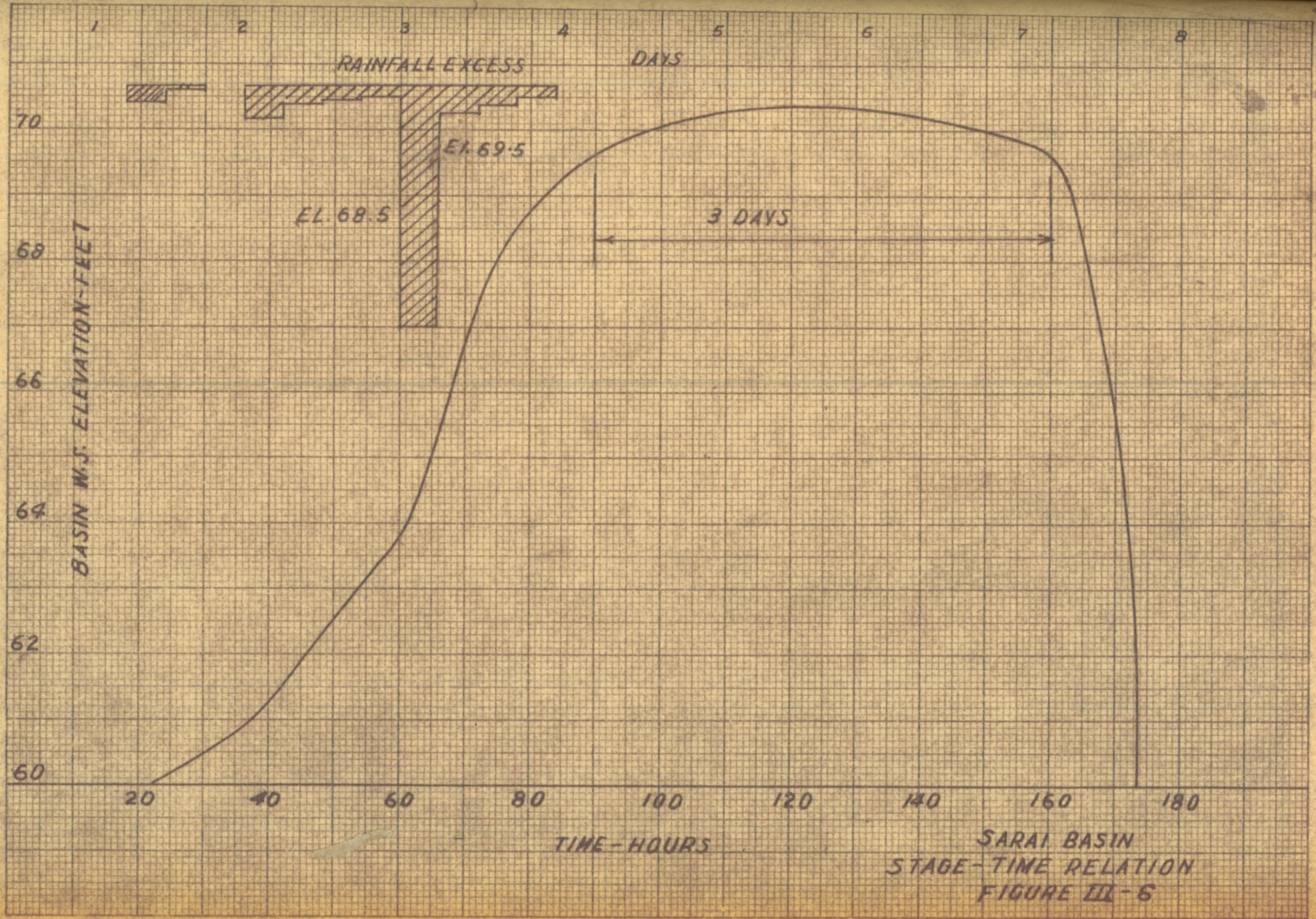


FIGURE III-3


SARAI BASIN
OUTFLOW HYDROGRAPH
FIGURE III-5
RAINFALL EXCESS
DAYS
1
2
3
4
5
6
7
8
DISCHARGE - CFS
2000
1500
1000
500
0
20
40
60
80
100
120
140
160
180
TIME - HOURS



SARAI BASIN
STAGE-TIME RELATION
FIGURE III-6

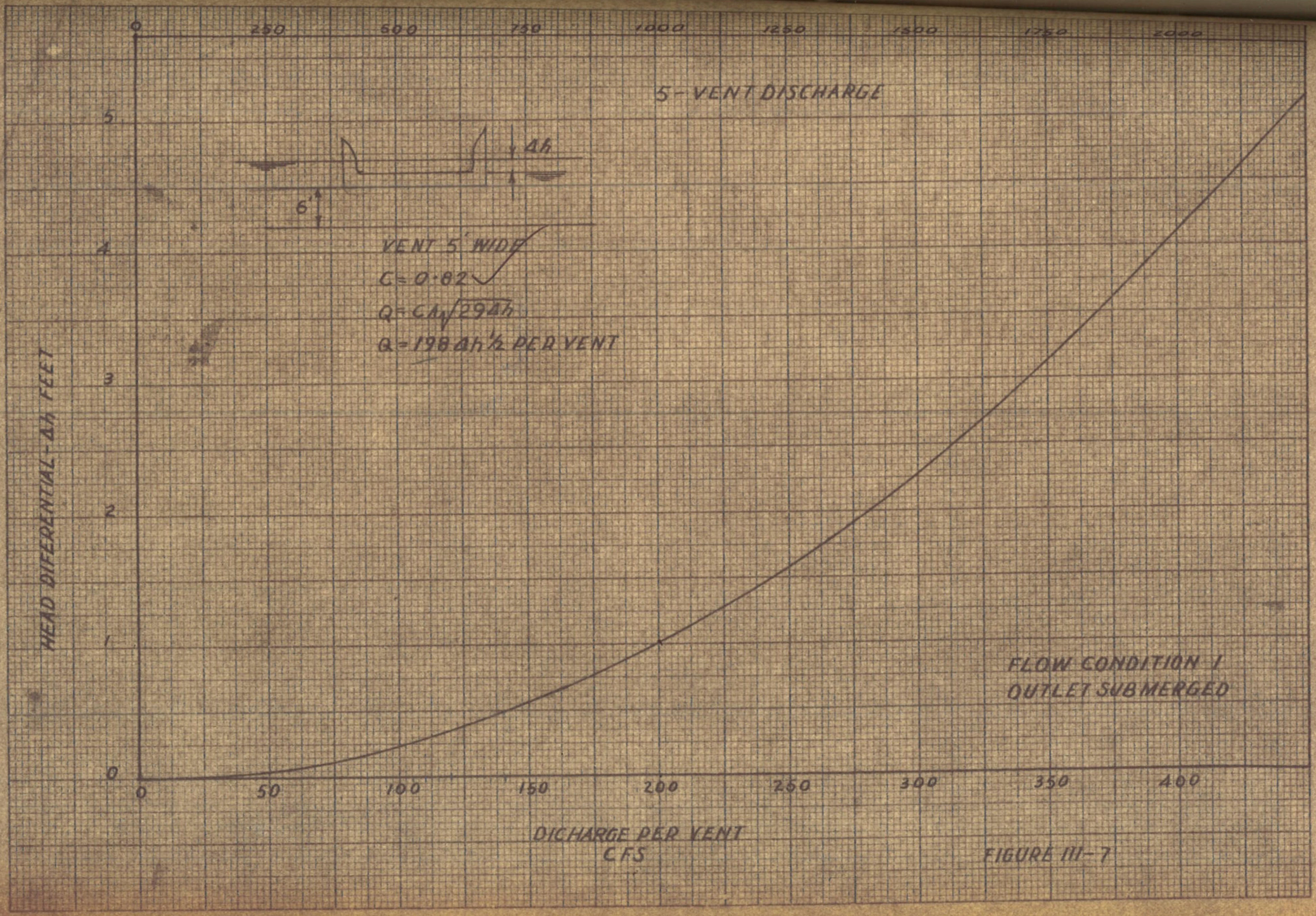


FIGURE III-7

MISC(S.B)9

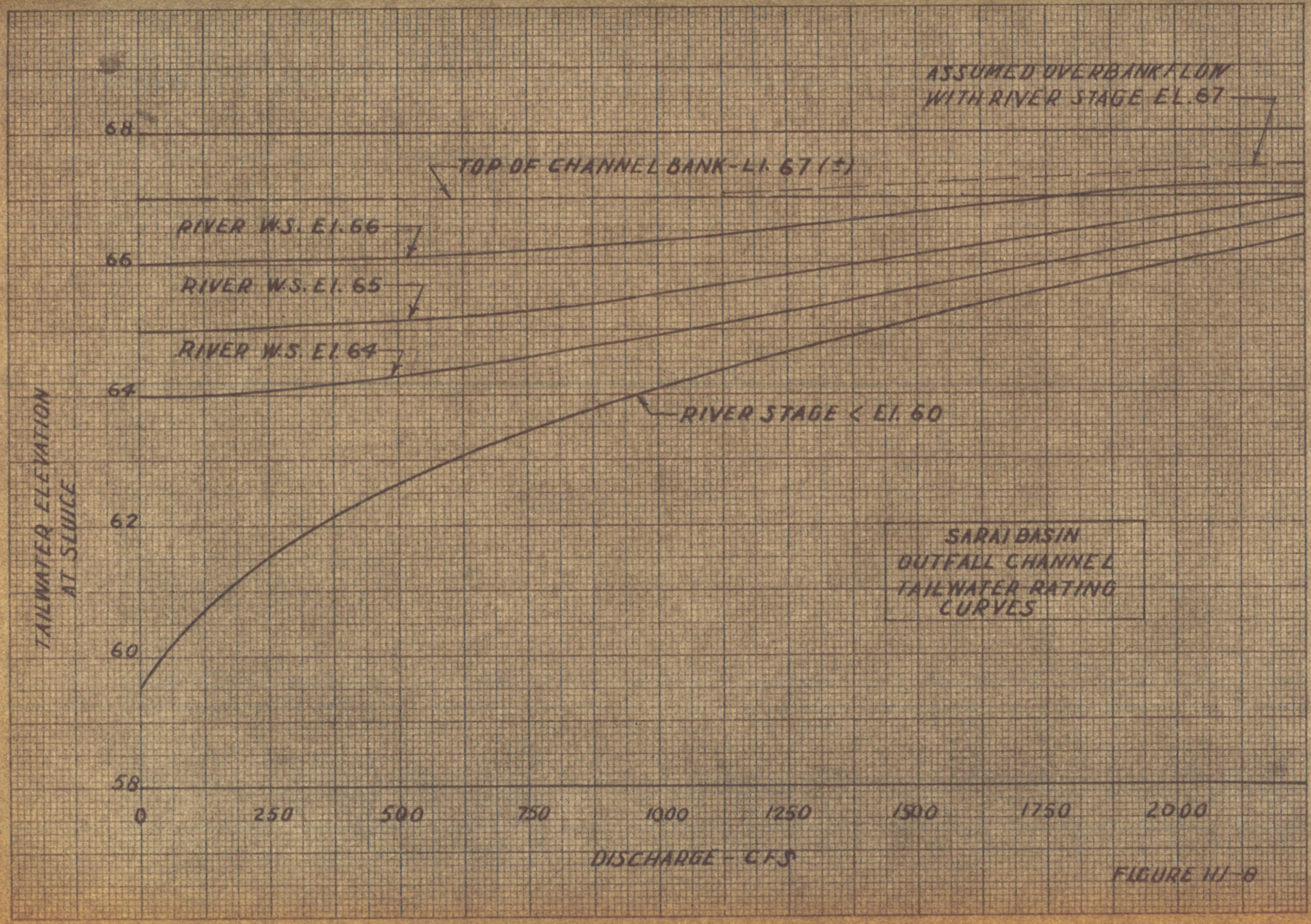

ASSUMED OVERBANK FLOW
WITH RIVER STAGE EL. 67
TOP OF CHANNEL BANK-EL. 67 (±)
RIVER W.S. El. 66
RIVER W.S. El. 65
RIVER W.S. El. 64
RIVER STAGE < El. 60
TAILWATER ELEVATION
AT SLUICE
68
66
64
62
60
58
0
250
500
750
1000
1250
1500
1750
2000
DISCHARGE - CFS
SARAI BASIN
OUTFALL CHANNEL
TAILWATER RATING
CURVES

FIGURE III-8

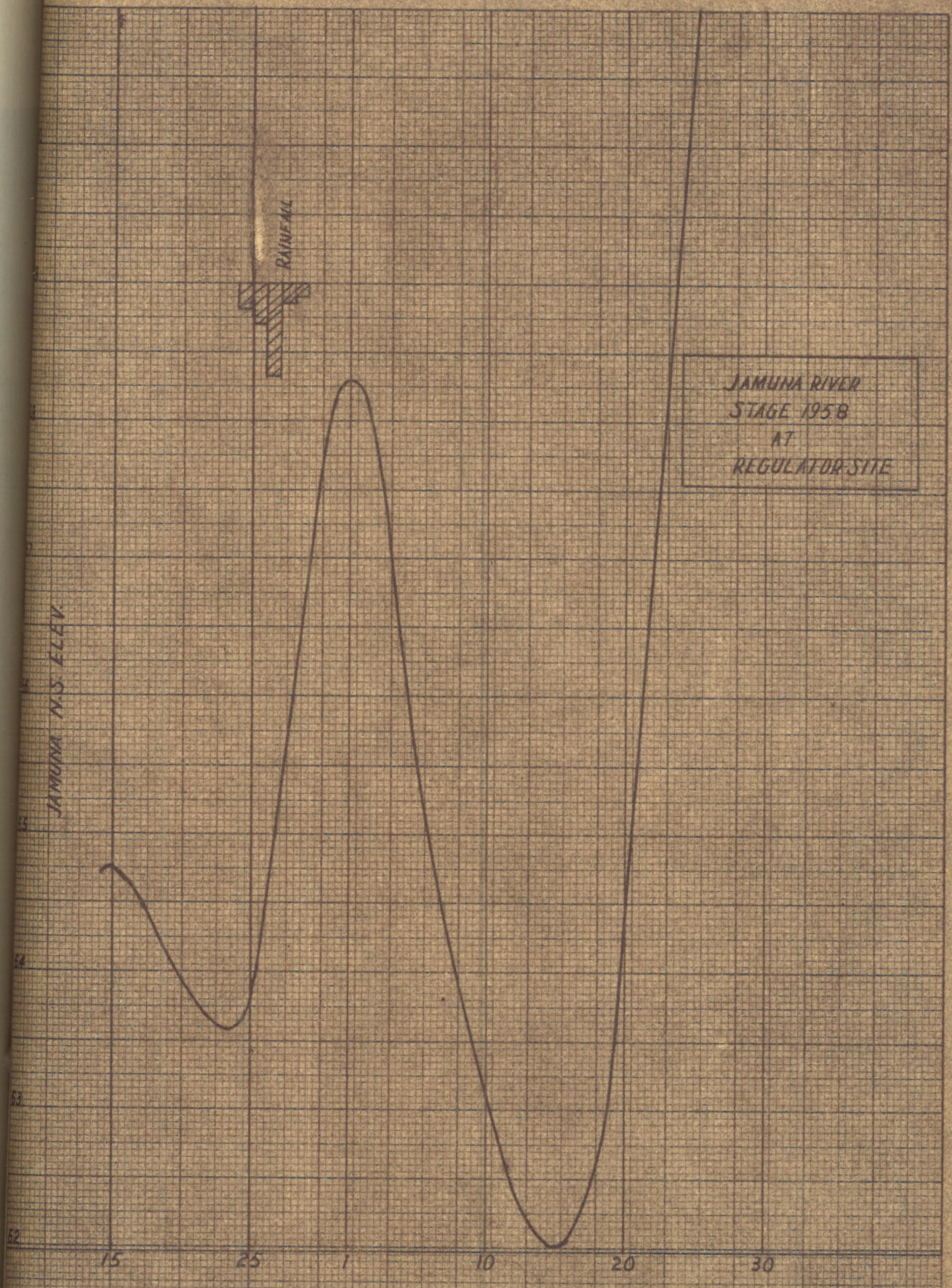
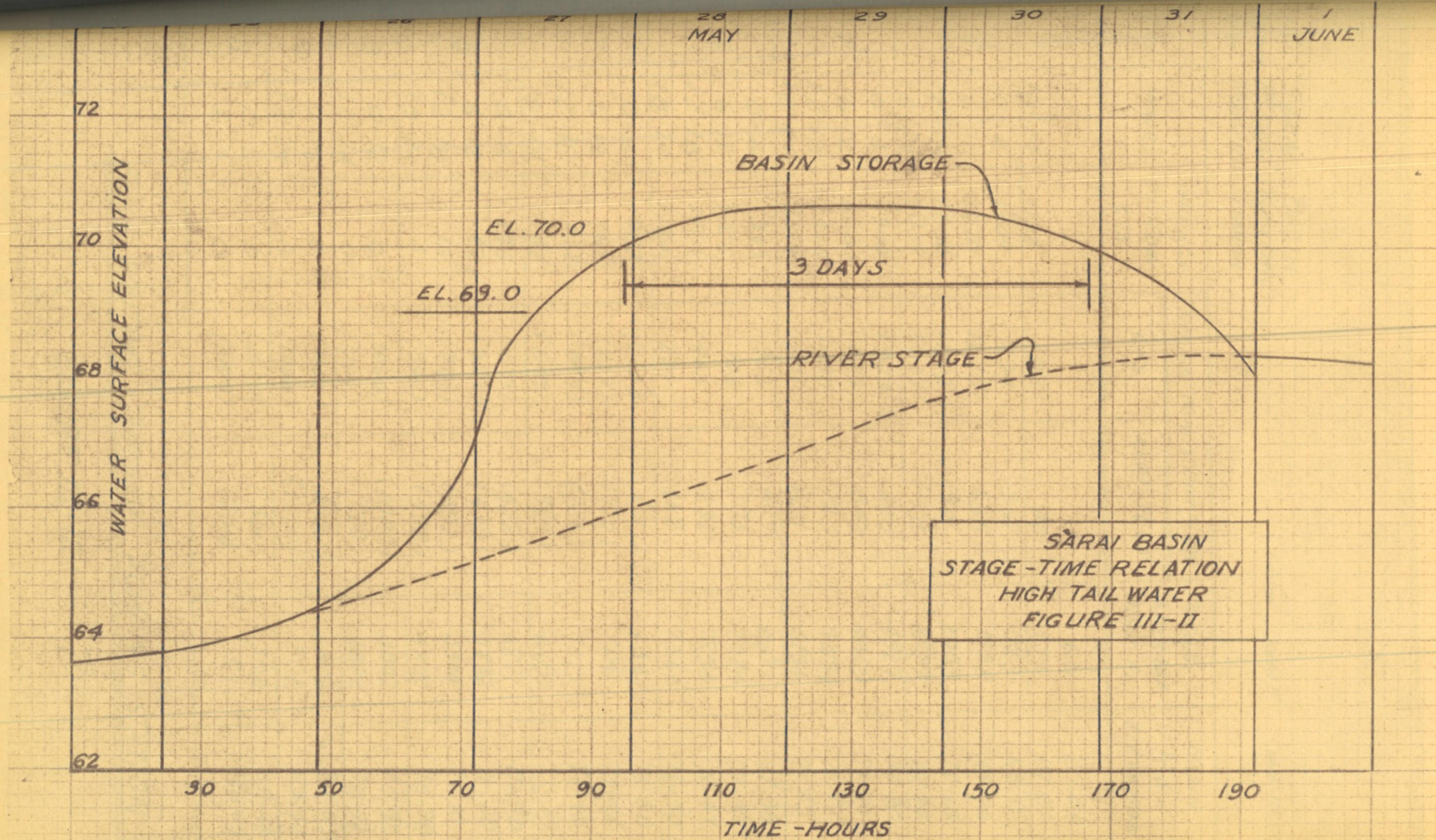

JAMUNA RIVER
STAGE 1958
AT
REGULATOR SITE
RAINFALL
JAMUNA N.S. ELEV.
52
53
54
55
15
25
1
10
20
30
JUNE

FIGURE III-9



SARAI BASIN
STAGE-TIME RELATION
HIGH TAIL WATER
FIGURE III-II

Proj. SARAI BASIN
Feature FLOOD ROUTING Design WME DATE 15.DEC '65
Item PUMPING STATION

DAY	HOURS	AVERAGE INFLOW CFS	INFLOW MINUS OUTFLOW		TOTAL STORAGE ACRE-FT.	BASIN W.S ELEVATION
			CFS	ACRE-FEET		
3					60	65.00
	18-24	1192	192	96	156	67.85
4	0-6	1734	734	367	523	69.15
	6-12	2270	1270	635	1158	69.70
	12-18	2802	1802	901	2059	70.15
	18-24	3037	2037	1018	3077	70.50
5	0-6	2900	1900	950	4027	70.80
	6-12	2683	1683	842	4869	71.00
	12-18	2450	1450	725	5594	71.17
	18-24	2201	1201	600	6194	71.25
6	0-6	1953	953	477	6671	71.38
	6-12	1706	706	353	7024	71.45
	12-18	1463	463	232	7256	71.47
	18-24	1224	224	112	7368	71.50
7	0-6	987	(13)	(7)	7361	71.50
	6-12	761	(239)	(120)	7241	71.46
	12-18	552	(448)	(224)	7017	71.45
	18-24	352	(648)	(324)	6693	71.39
8	0-6	156	(844)	(422)	6271	71.90
	6-12	42	(958)	(479)	5792	71.20
	12-18	16	(984)	(492)	5300	71.10
	18-24	4	(996)	(498)	4802	70.98
9	0-12	0	(1000)	(1000)	3802	70.75
	12-24	0	(1000)	(1000)	2802	70.42
10	0-12	0	(1000)	(1000)	1802	70.03
	12-24	0	(1000)	(1000)	802	69.42
11	0-6	0	(1000)	(500)	302	68.75
	6-9	0	(1000)	(250)	52	64.65

FLOOD ROUTING THROUGH PUMPING STATION

FIGURE III-12

MISC.(S.B)

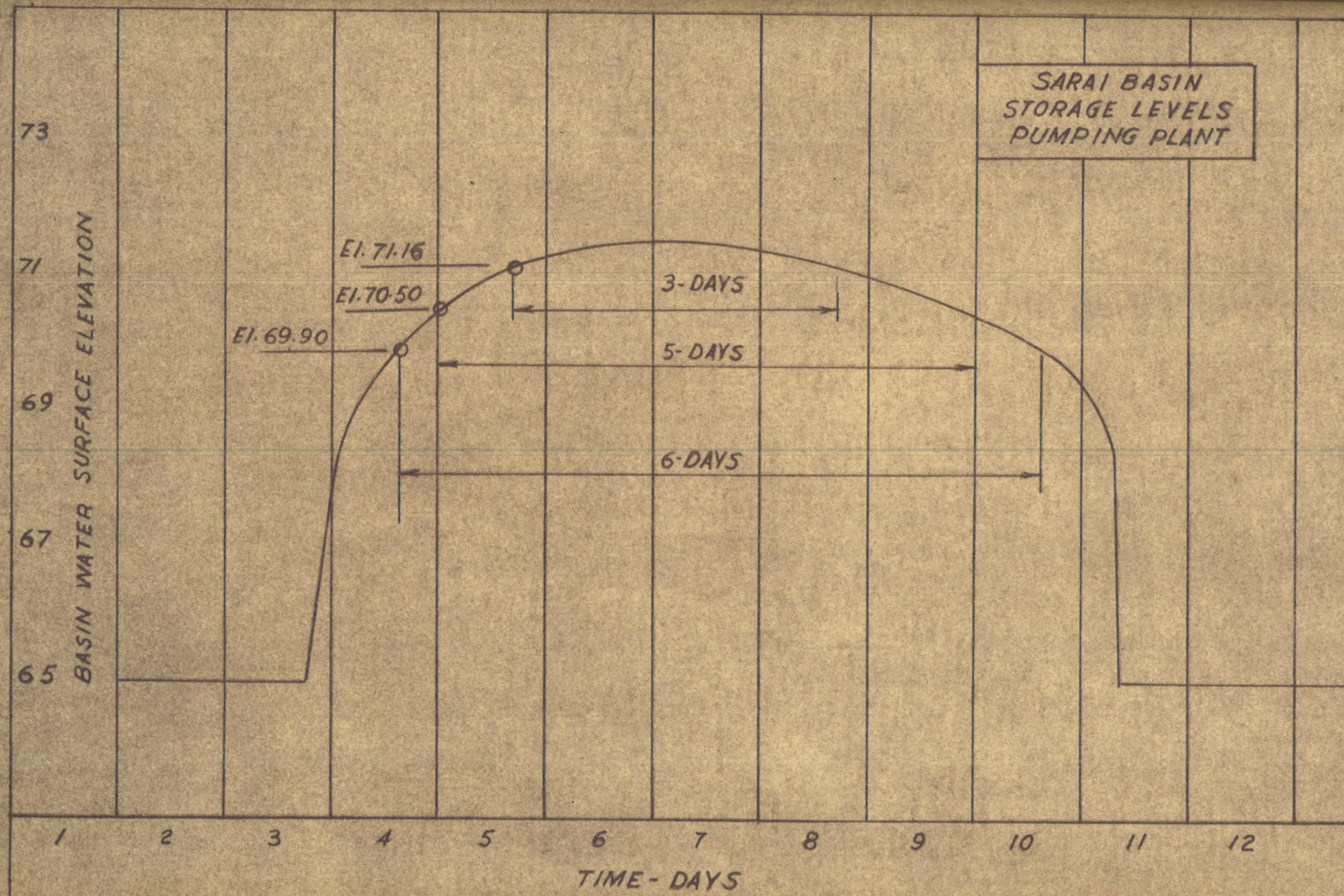
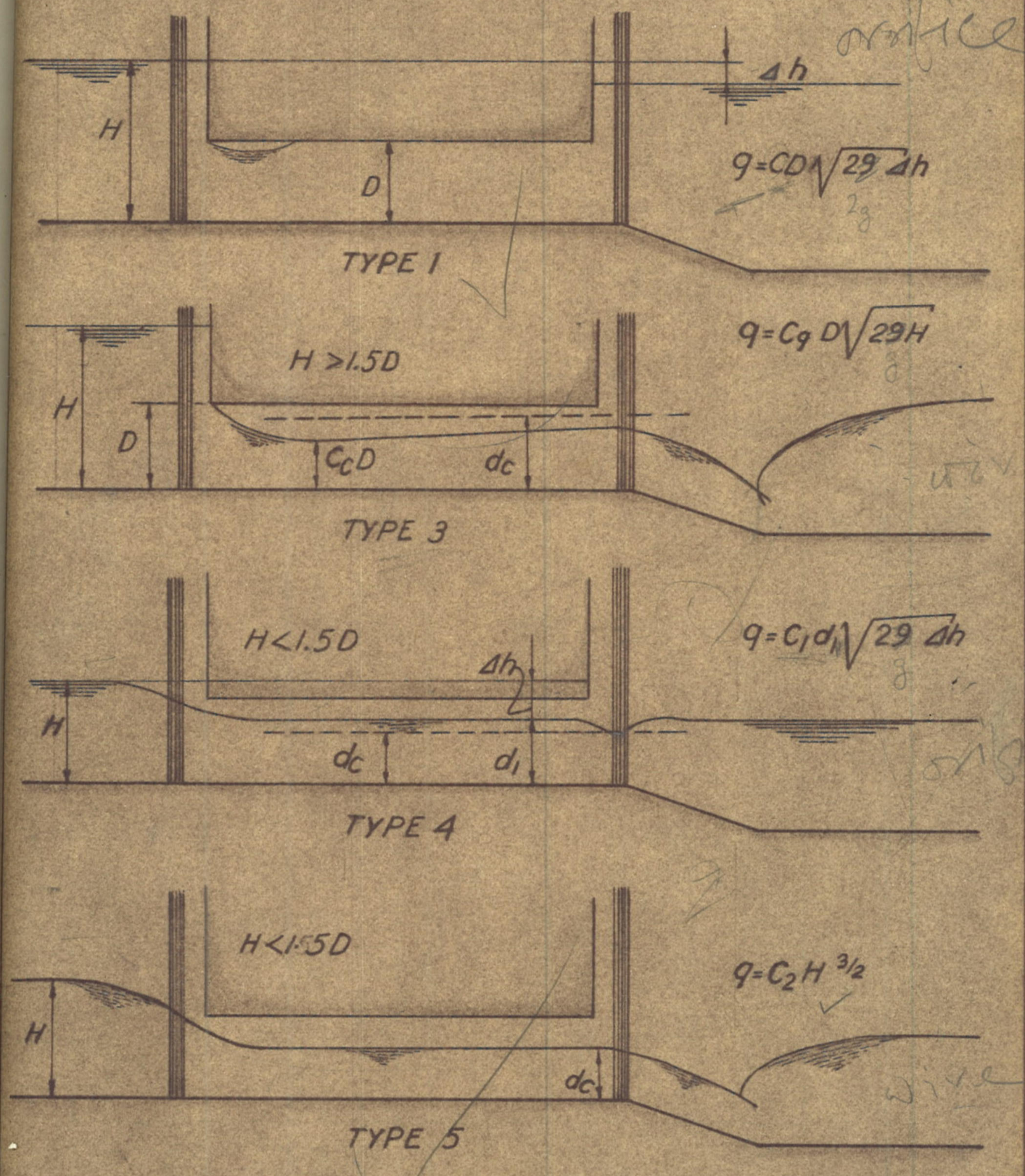


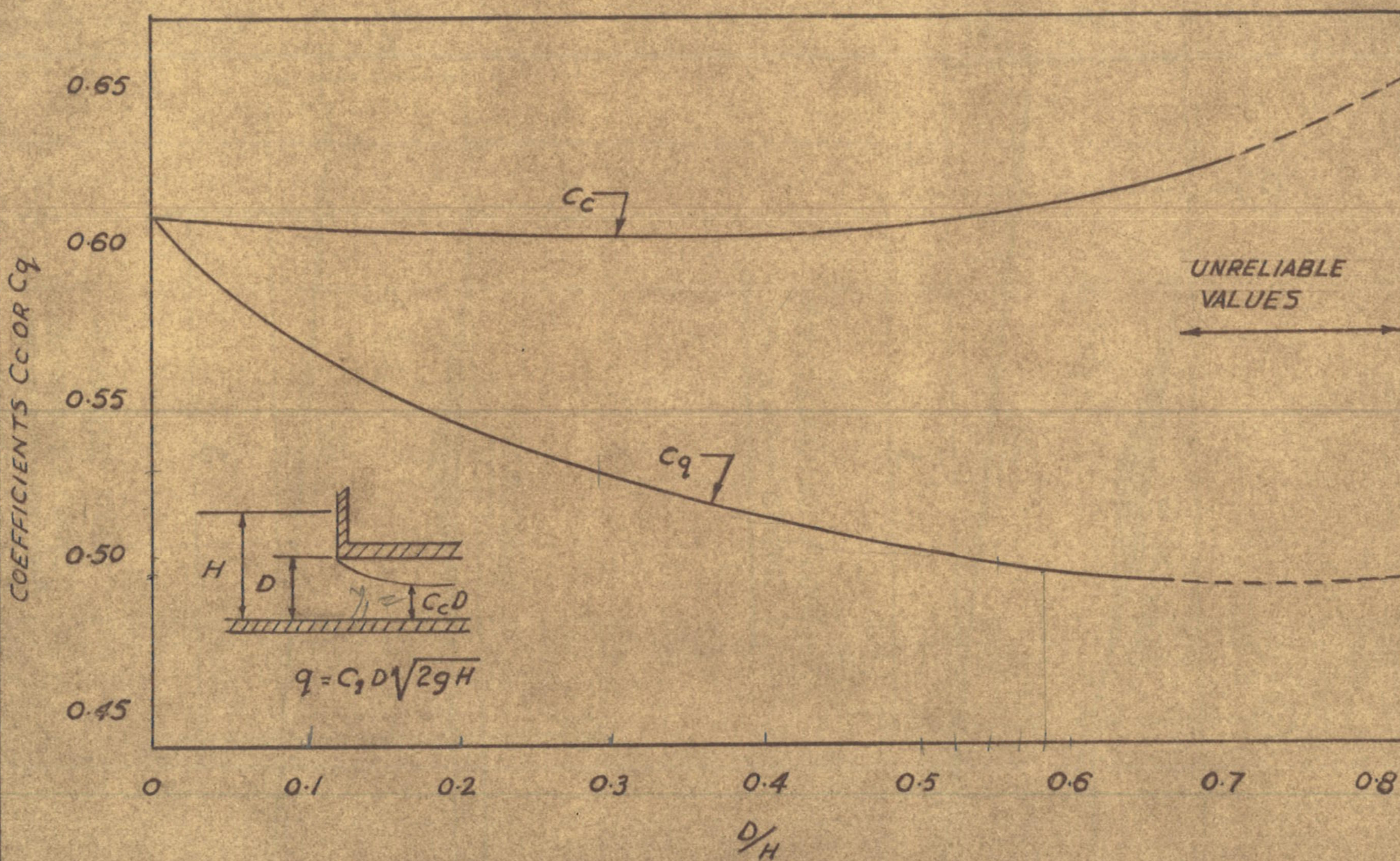
FIGURE III-13

Q = CIA
q = Q/7.75



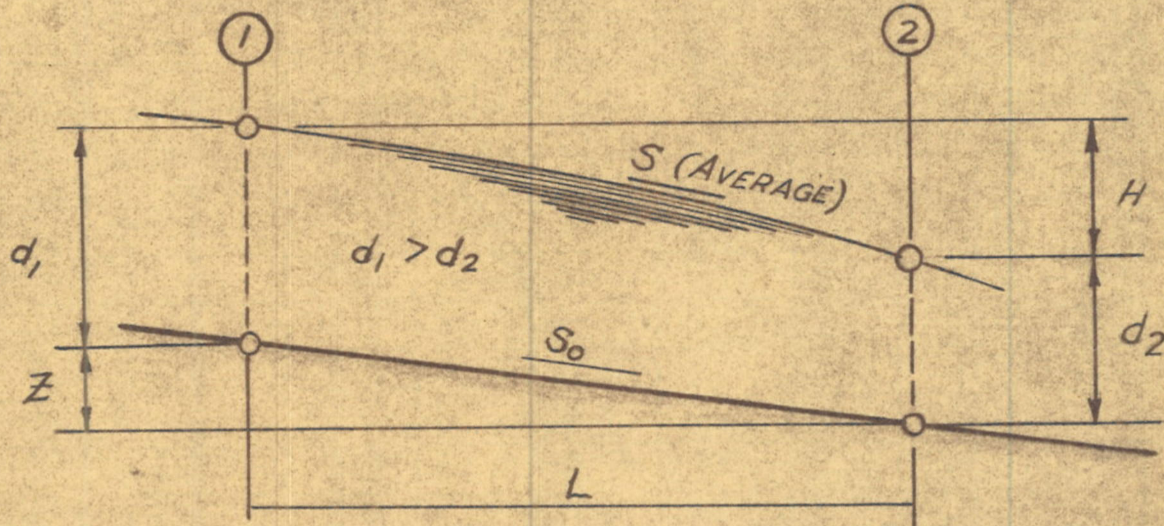
CONDITIONS OF FLOW THROUGH SLUICES

~~FIGURE IV~~
Page IV-2a

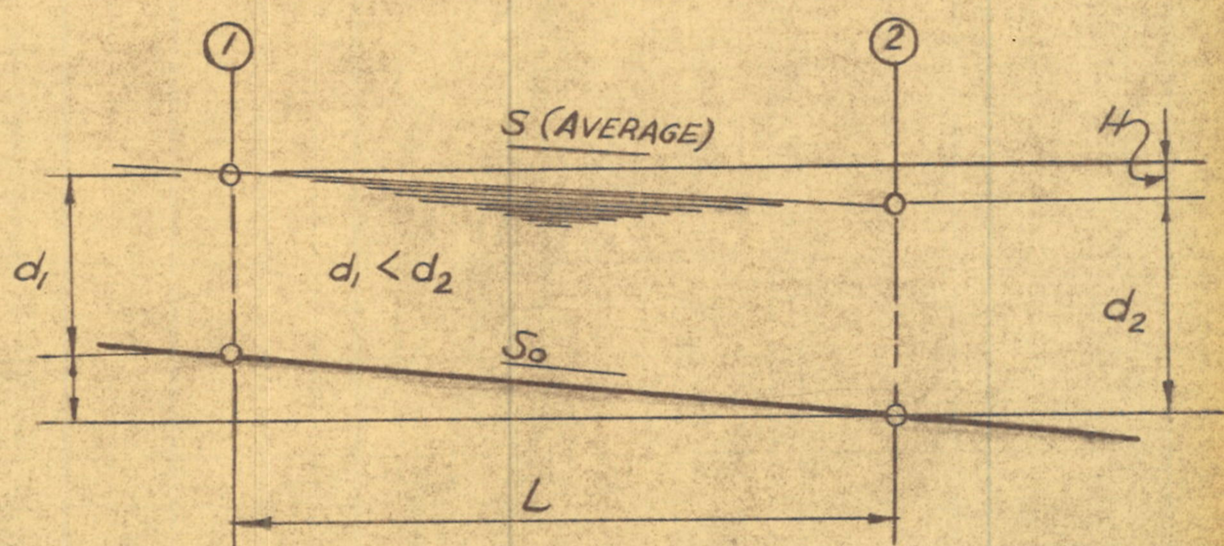


FLOW CONDITION TYPE-3
COEFFICIENTS

FIGURE IV-5



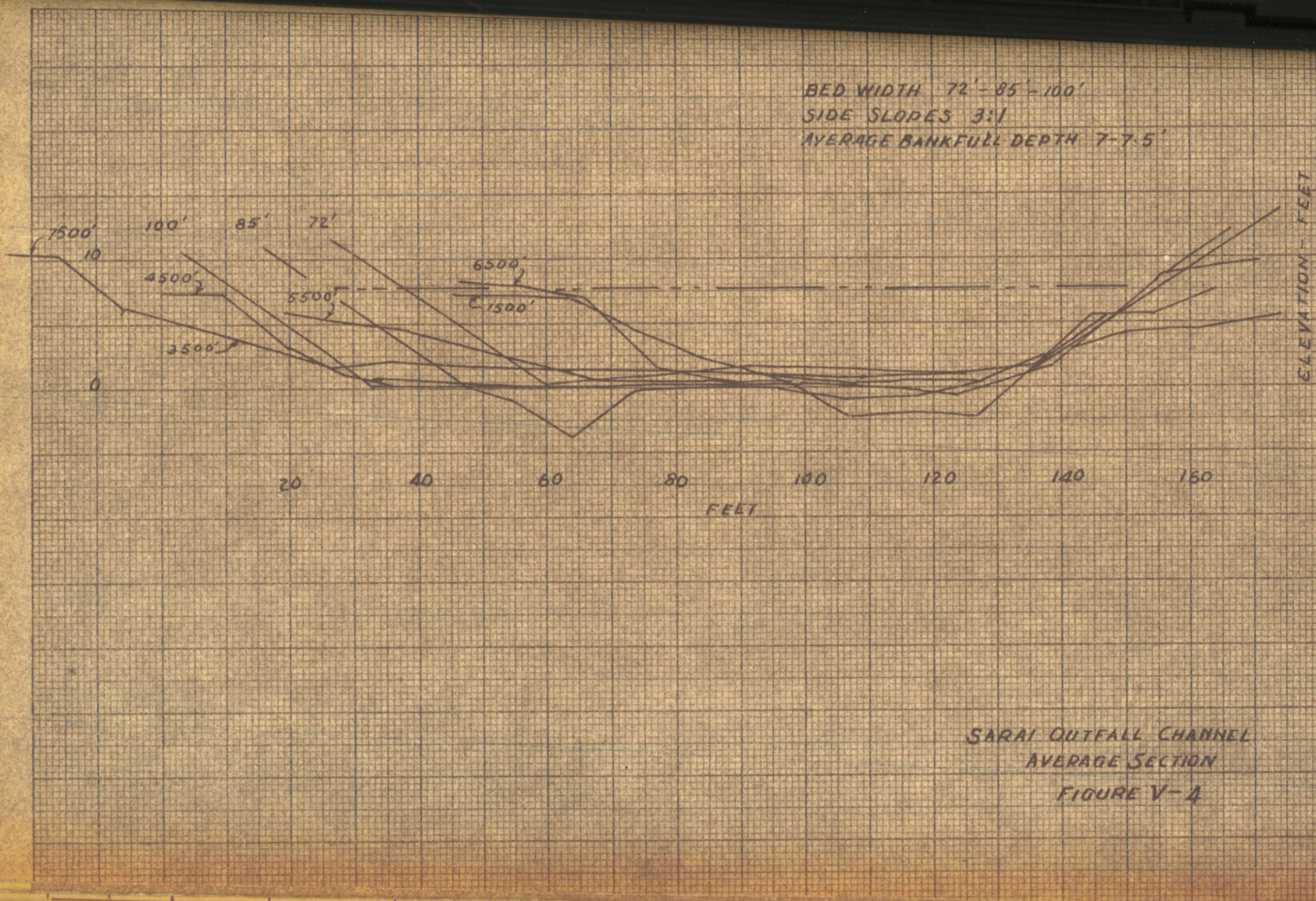
ACCELERATING FLOW
DRAWDOWN CURVE



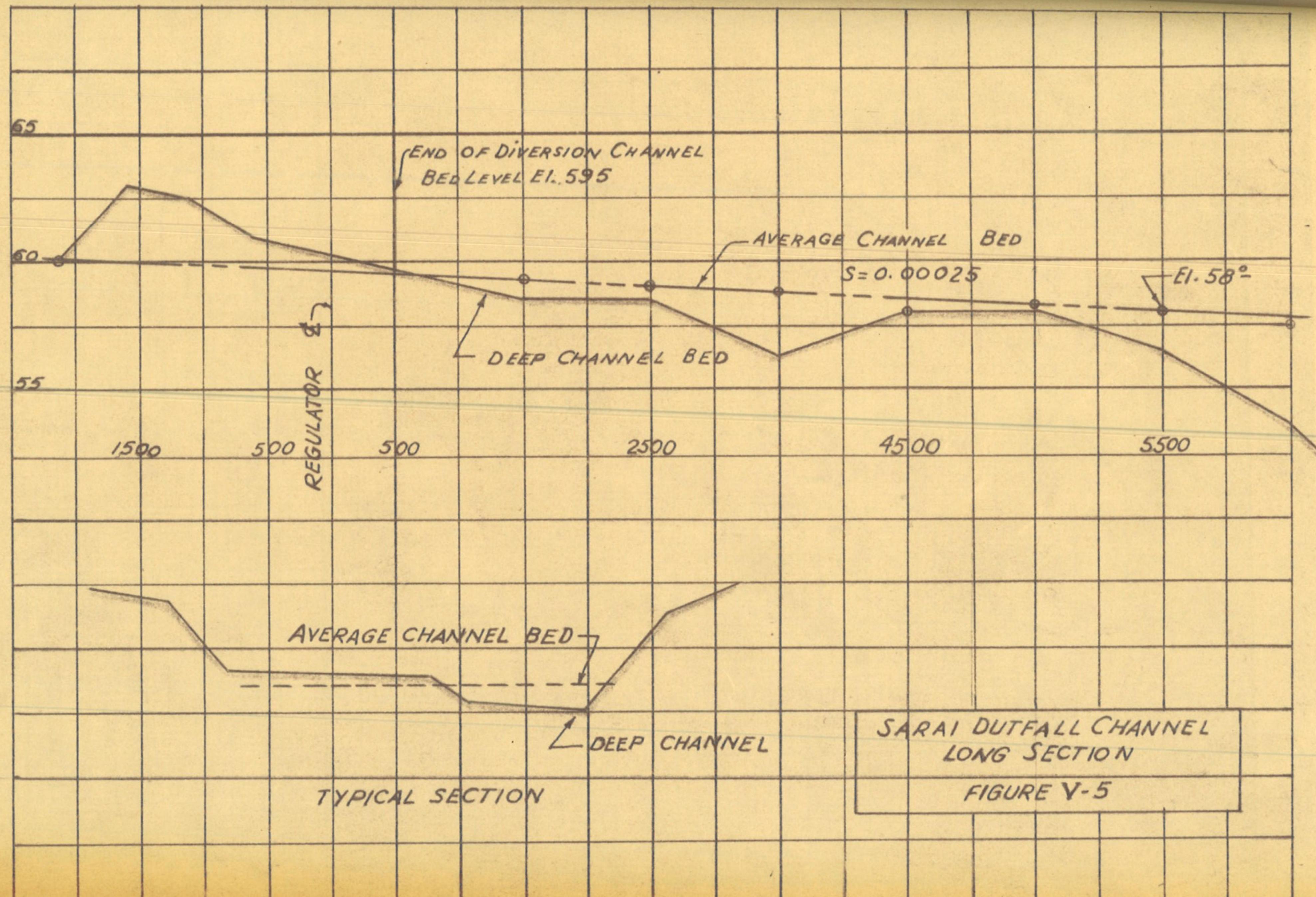
RETARDED FLOW
BACKWATER CURVE

FIGURE V-1

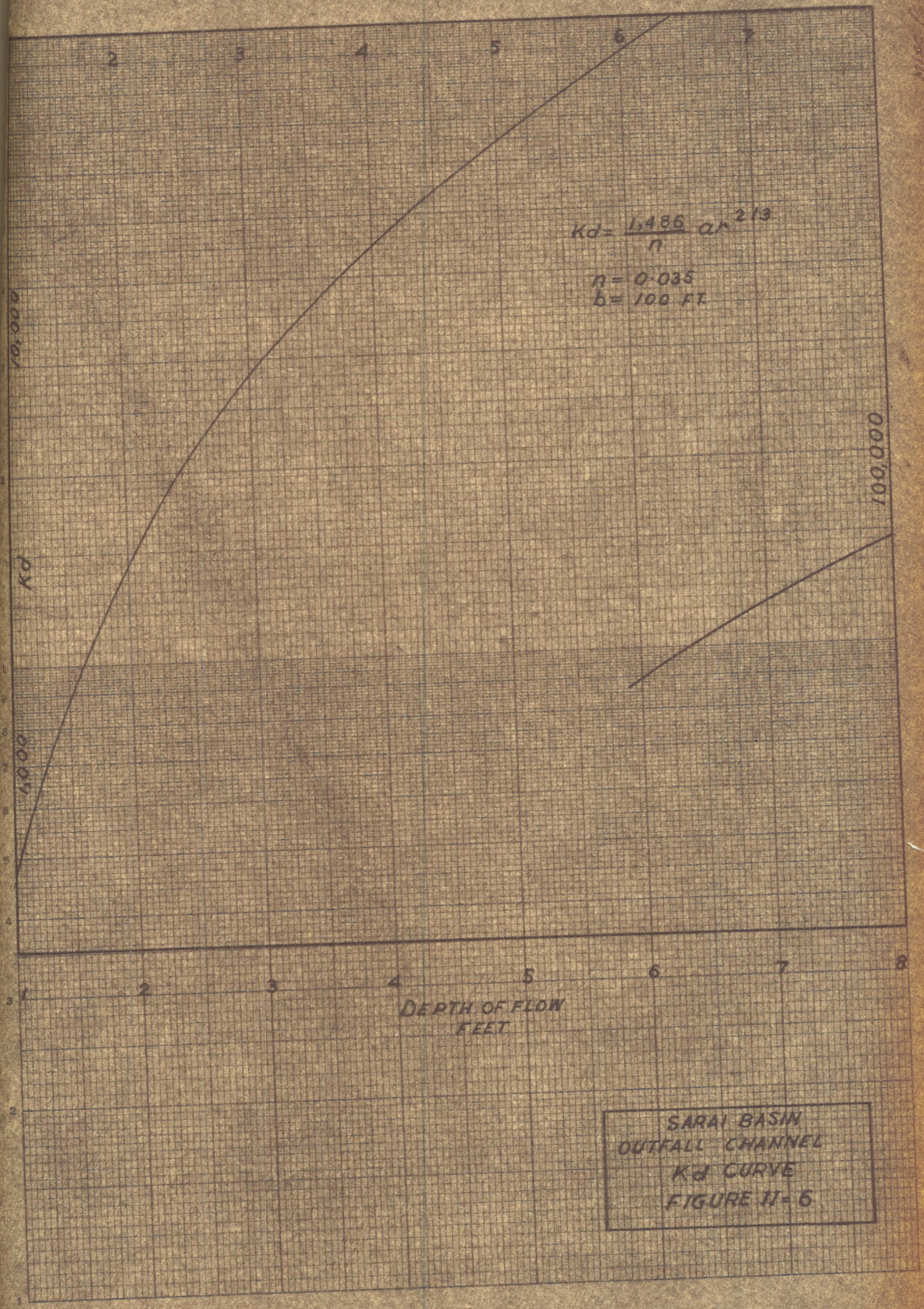
misc. CS



SARAI OUTFALL CHANNEL
AVERAGE SECTION
FIGURE V-4



SARAI OUTFALL CHANNEL
LONG SECTION
FIGURE V-5



SARAI BASIN
OUTFALL CHANNEL
Kd CURVE
FIGURE II-6

d	Δd	K_d	K_d AVERAGE	Q/K_d	$S=(Q/K_d)^2$	$S-S_o$	$L=\Delta d$	ΣL	REMARKS
2.24		16,500						0	Q=2000
	·26		18,100	.110	.012	.01175	22		
2.50		19,700						22	RIVER W.S. BELOW 60.00 LENGTH OF CHANNEL 6000' So = 0·00025
	·50		23,450	.0855	.0073	.00705	71		
3.00		27,200						93	
	·50		31,350	.0638	.00406	.00381	131		
3.50		35,500						224	
	·50		40,000	.050	.0025	.00225	222		
4.00		44,500						446	
	·50		49,750	.0401	.00161	.00136	368		
4.50		55,000						816	
	·50		60,000	.0333	.00111	.00086	582		
5.00		65,000						1396	
	·50		71,000	.0282	.000794	.000544	920		
5.50		77,000						2316	
	·50		83,000	.0241	.00058	.00033	1514		
6.00		89,000						3830	
	·50		95,500	.0210	.00044	.00019	2680		
6.50		102,000						6460	W.S. El. 66.00
2.00		13,500						0	Q = 500 CFS
	·50		16,600	.030	.0009	.00065	770		
2.50		19,700						770	RIVER W.S. 62.00
	·50		23,450	.0213	.000455	.000205	2440		
3.00		27,200						3210	
	·15		28,350	·0176	·00031	000068	2500		
3.15		29,500						5710	W.S. El. 62·65

SARAL BASIN
OUTFALL CHANNEL PROFILES
DRAWDOWN - RIVER El. 60

FIG V-7

Proj: SARAI BASIN
Feature OUTFALL CHANNEL PROFILES
Item DRAWDOWN - RIVER EL. 60

d	Δd	Kd	Kd Average	Q/Kd	$S=(Q/Kd)^2$	$S-S_o$	$L=\frac{\Delta d}{S-S_o}$	Σ L	REMARKS
1.86		15,000						0	Q = 1500 CFS.
	0.64		17,350	.0865	.00747	.00722	89		
2.50		19,700						89	River W.S. El. 60 ±
	0.50		23,450	.064	.0041	.00385	130		LENGTH OF CHANNEL 6000'
3.00		27,200						219	
	1.00		35,850	.042	.00176	.00151	663		S_o = 0.00025
4.00		44,500						882	
	1.00		54,750	.0274	.00075	.00050	2000		
5.00		65,000						2882	
	0.60		72,000	.0208	.000433	.000183	3280		
5.60		79,000						6162	W.S. El. 65.10
2.00		13,500						0	Q = 1000 CFS
	1.00		20,350	.0492	.00242	.00217	460		
3.00		27,200						460	River W.S. El. 60 ±
	1.00		35,850	.028	.000785	.000535	1870		
4.00		44,500						2330	
	.50		49,750	.0201	.000405	.000156	3220		
4.50		55,000						5550	
	.05		55,300	.0181	.000327	.000077	650		
4.55		55,600						6200	W.S. El. 64.05

Sheet FIG. V-8

Project SARAI BASIN
Feature OUTFALL CHANNEL PROFILES
Item BACKWATER - RIVER EL. 64

d	Ad	Kd	Kd. Average	Q/Kd	$S=(Q/Kd)^2$	So - S	$L=\frac{Ad}{So-S}$	ΣL	REMARKS
6.00		89,000						0	Q = 500 CFS.
	.50		83,000	.00603	.000036	.000214	2340		RIVER W.S. El. 64 LENGTH OF CHANNEL 6000' So = 0.00025
5.50		77.000						2340	
	.50		71,250	.00702	.000049	.000201	2490		
5.00		65.500						4830	
	.23		63,000	.00795	.000063	.000187	1230		
4.77		60,500						6060	W.S. E.l. 64.27 ←
6.00		82,000						0	Q = 1000 CFS.
	.50		83,000	.01205	.000145	.000105	4750		
5.50		77,000						4750	
	.10		75,750	.01320	.000174	.000076	1310		
5.40		74,500						6060	W.S.E.l. 64.90 ←
6.00		89,000						0	Q = 1500 CFS.
	.10		90,250	.0166	.000276	.000026	3850		
6.10		91,500						3850	
	.03		92,000	.0163	.000265	.000015	2000		
6.13		92,500						5850	W.S.E.l. 65.63 ←

SARAI BASIN
OUTFALL CHANNEL PROFILES
BACKWATER - RIVER El. 65

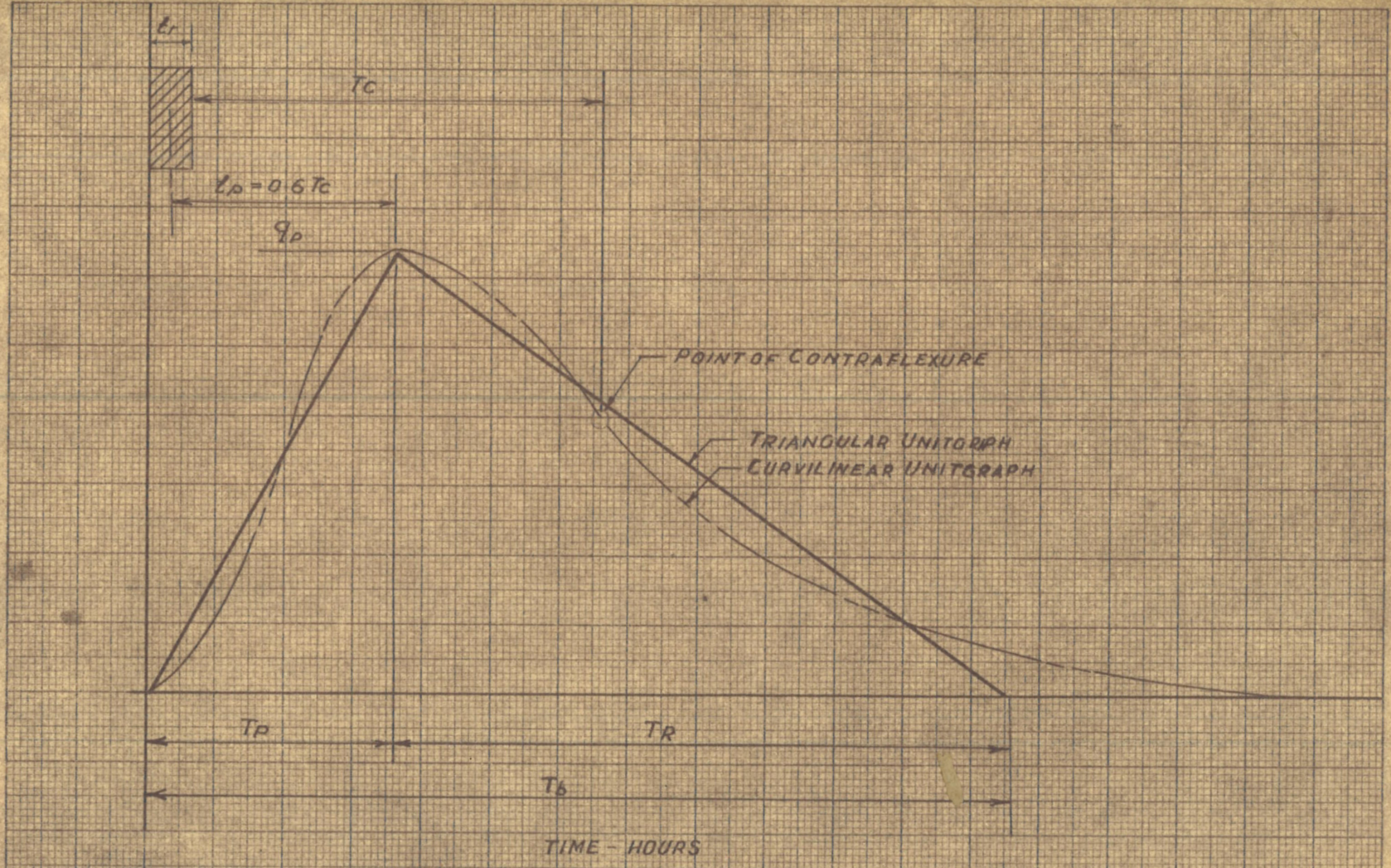
FIG. V-10

d	Δd	K_d	K_d AVERAGE	Q/K_d	$S=(Q/K_d)^2$	S_o-S	$L=\frac{\Delta d}{S_o-S}$	Σ L	REMARKS
7.00		116,000						0	Q=500 CFS
	.50		108,500	.0046	.0000212	.0002288	2180		RIVER W.S. El. 65.00
6.50		101,000						2180	LENGTH OF CHANNEL 6000'
	.50		95,000	.00526	.0000277	.0002223	2240		
6.00		89,000						4420	S_o = 0.00025
	.35		84,500	.00591	.000035	.000215	1630		
5.65		80,000						6050	W.S. El. 65.15 ←
7.00		116,000						0	Q = 1000 CFS
	.50		108,500	.0092	.000085	.000165	3030		
6.50		101,000						3030	
	.42		95,750	.01042	.000109	.000141	2980		
6.08		90,500						6010	W.S. El. 65.58 ←
7.00		116,000						0	Q = 1500 CFS
	.25		112,500	.0133	.000178	.000072	3480		
6.75		109,000						3480	
	.13		107,000	.0140	.000196	.000054	2410		
6.62		105,000						5890	W.S. El. 66.12 ←
7.00		116,000						0	Q = 2000 CFS
	.10		117,500	.0170	.00029	.00004	2500		
7.10		119,000						2500	
	.10		120,000	.0167	.000278	.000028	3570		
7.20		121,000						6070	W.S. El. 66.70 ←

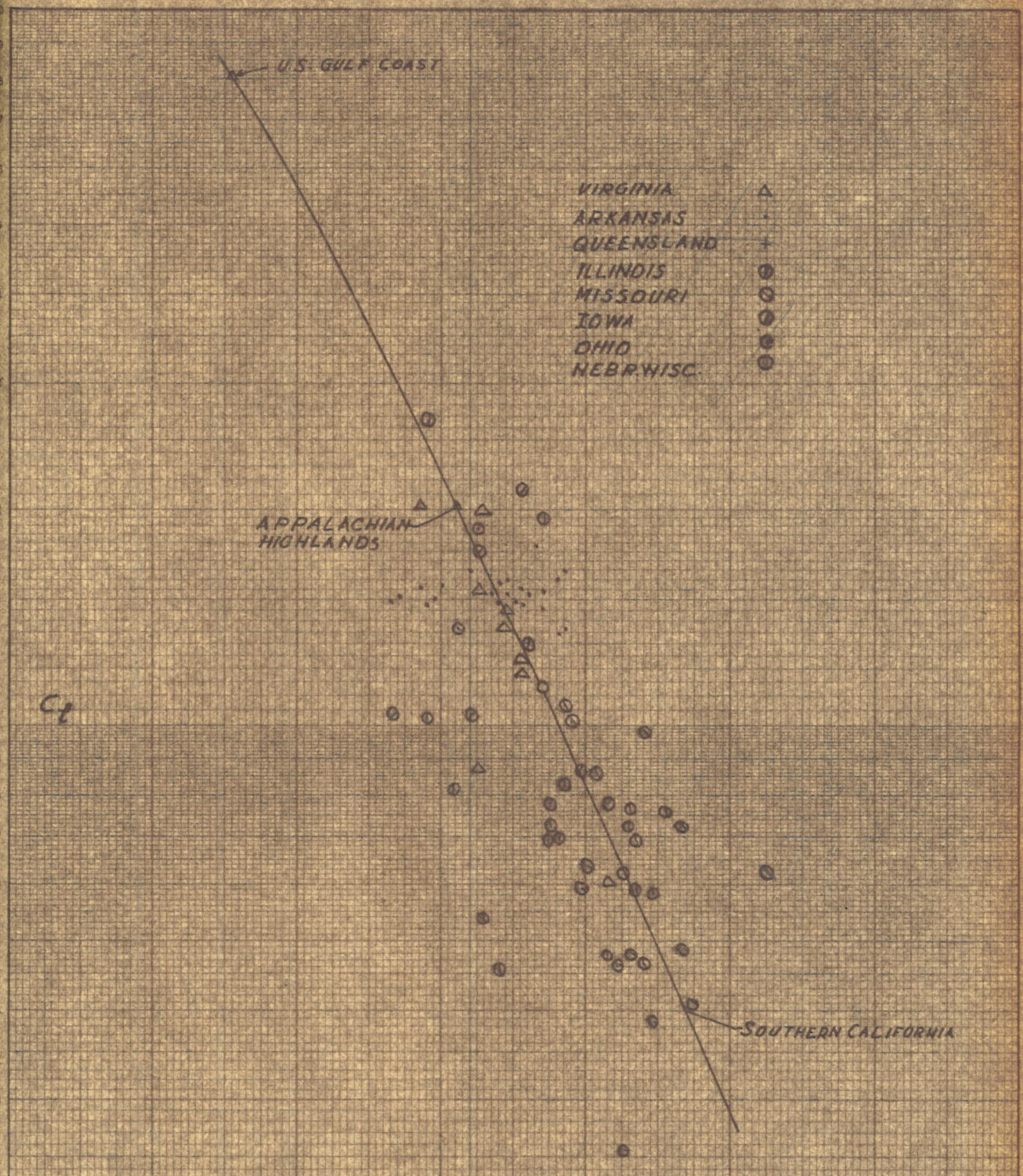
SARAI BASIN
OUTFALL CHANNEL PROFILES
BACKWATER - RIVER El. 66

d	Δd	K_d	K_d AVERAGE	Q/K_d	$S = (Q/K_d)$	$S_o - S$	$L = \frac{\Delta d}{S_o - S}$	ΣL	REMARKS
8.00		147,000						0	Q = 500 CFS
	.50		138,500	.00361	.000013	.000237	2110		RIVER W.S. El. 66.00
7.50		130,000						2110	LENGTH OF CHANNEL
	.50		123,000	.00406	.0000166	.000233	2140		6000'
7.00		116,000						4250	S_o = 0.00025
	.40		110,500	.00452	.0000205	.000229	1750		
6.60		105,000						6000	W.S. El. 66.10 ←
8.00		147,000						0	Q = 1000 CFS
	.50		138,500	.00722	.000052	.000198	2520		
7.50		130,000						2520	
	.50		123,000	.00812	.000066	.000184	2720		
7.00		116,000						5240	
	.13		113,500	.0088	.000078	.000172	755		
6.87		111,000						5995	W.S. El. 66.37 ←
8.00		147,000						0	Q = 1500 CFS
	.50		138,500	.0108	.000117	.000133	3760		
7.50		130,000						3760	
	.25		126,500	.01185	.000140	.000110	2270		
7.25		123,000						6030	W.S. El. 66.75 ←
8.00		147,000						0	Q = 2000 CFS
	.25		142,500	.01402	.000197	.000053	4720		
7.75		138,000						4720	
	.05		137,500	.01455	.000212	.000038	1310		
7.70		137,000						6030	W.S. El. 67.20 ←

FIG. V-11.



UNITGRAPH ELEMENTS
FIGURE A-3



SNYDER COEFFICIENTS

FIGURE A-1

UNIT HYDROGRAPH
SNYDER COEFFICIENTS
FOR SELECTED BASINS

ARKANSAS

$$T_c = 13.1\,C n^{.6}\left(\frac{L^2}{S}\right)^{.3}$$

Assumed:
- $C = 2.0$
- "n" as noted
- $t_p = 0.6\,T_c$

$$t'_p = \frac{21}{22} t_p + \frac{t_R}{4}$$

t_R = 2 Hours

A Sq. MI.	q_p CSM	L	$\bar{L}$	$(L\bar{L})^{.3}$	S FT/MI	$\left(\frac{L^2}{S}\right)^{.3}$	n	T_c	t_p	t'_p	C_t	C_p
1238	15.6	182	95	17	3	13.6	.040	51.6	31.0	30.1	1.82	.73
987	20.4	72	49	11.6	5	8.1		30.8	18.5	18.2	1.60	.58
2038	16.5	146	83	16.8	5	12.2		46.4	27.8	27.0	1.65	.70
1162	23.3	107	49	13	6	9.6		36.5	21.9	21.4	1.69	.78
1091	21.6	118	58	14.2	6	10.2		38.8	23.3	22.7	1.64	.77
1667	20.4	107	62	14.1	6	9.6		36.5	21.9	21.4	1.55	.68
1123	18.5	117	65	14.6	6	10.2		38.8	23.3	22.7	1.59	.66
1020	18.2	84	54	12.5	6	8.3		31.5	18.9	18.6	1.51	.53
225	35	53	27	8.7	7	6.0	K = 3.80	22.8	13.7	13.6	1.58	.74
476	27	73	33	10.3	7	7.3		27.8	16.7	16.5	1.62	.70
262	31	44	23	8.0	7	5.4		20.5	12.3	12.3	1.54	.60
1612	18	96	50	12.7	7	8.6		32.7	19.6	19.2	1.54	.54
398	33.3	58	28	9.2	8	6.1		23.2	13.9	13.8	1.51	.72
1272	30.2	74	30	10.1	8	7.1		27.0	16.2	16.0	1.60	.76
247	28.2	33	16	6.6	8	4.4	N.G. →	17.8	10.7	10.7	1.62	.47
561	33.2	62	29	9.8	9	6.0	.045	24.3	14.6	14.4	1.49	.75
825	26	84	39	11.3	9	7.4	K =	30.0	18.0	17.7	1.59	.72
517	31.2	62	32	9.7	9	6.2	4.05	25.1	15.1	14.9	1.56	.73
1141	20.6	92	46	12.2	9	7.8		33.7	20.2	19.8	1.66	.64
957	27	68	45	11.1	9	6.5	.050	28.1	16.9	16.6	1.52	.70
793	20.5	84	46	11.9	9	7.4		32.0	19.2	18.8	1.61	.60
211	40.5	36	19	7.1	11	4.2		18.2	10.9	10.9	1.54	.69
361	35	49	24	8.3	13	4.8	.33	20.8	12.5	12.4	1.51	.68
74	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

UNIT HYDROGRAPH
SNYDER COEFFICIENTS
FOR SELECTED BASINS

	A Sq. MI.	q_p CSM	L MILES	$\bar{L}$	$(L\bar{L})^{.3}$	t_p	t_r	t'_p
QUEENSLAND AUSTR	290	98	25	ASSUMED: $\bar{L} = .5L$	5·6	4·8	1	4·83
	208	85	39		7·3	5·4	1	5·40
	180	83	41·7		7·6	5·4	1	5·40
	87	160	14·3		3·98	3·0	1	3·11
	233	125	27		5·85	3·6	1	3·69
	260	123	29		6·1	4·2	1	4·26
	660	68	53		8·7	7·2	1	7·12
ILLINGIS	10·1	103	4·9	2·74	2·18	3·88	1	3·95
	22·5	113	10·1	5·4	3·32	3·50	1	3·59
	32·2	49	15·6	8·42	4·33	3·30	2	8·44
	39·6	101	11·7	6·16	3·42	4·50	1	4·55
	45·0	28	23·3	11·4	5·38	12·60	2	12·60
	62·5	79	19·9	12·2	5·20	6·00	1	5·98
	71·4	30	16·8	9·74	4·60	12·45	2	12·38
	83·3	69	19·6	9·89	4·85	5·05	2	5·83
	101	57	23·3	11·60	5·36	5·65	2	5·90
	37·0	46	18·8	8·43	4·49	9·80	2	9·85
	72·7	130	16·8	9·14	4·53	3·93	1	4·00
	60·9	154	17·3	7·86	4·37	3·08	1	3·15

REF. ASCE SEPT '62 VOL. 88 HYS
MAY '63 VOL. 89 HYS

NOTE :-

THE ILLINOIS DATA ARE FROM BASINS THAT DO NO[illegible]
GOOD RESULTS. FOR ANY UNITGRAPH METHOD FO[illegible]
THESE BASINS. IT WAS FOUND THAT BETTER RESUL[illegible]
WERE DETAINED USING THE RELATION.

$$t_p = 0.5\,t \quad \text{INSTEAD OF} \quad t.\ 2.80\ t_p^{0.81}$$

"t" IS THE TIME FROM CENTER OF MASS OF RA[illegible]
TO THE CENTER OF GRAVITY OF THE UNITGRAPTH.

-VIRGINIA-

UNIT HYDROGRAPH
SNYDER COEFFICIENTS
FOR SELECTED BASINS

A Sq Mi.	q_p CSM	L MILES	$(L\bar{L})^{.3}$	S FT/Mi	$\left(\frac{L^2}{S}\right)^{.3}$	n	C	T_c	t_p	t'_p	C_t	C_p
546	20.3	40	7.45	5.6	5.45	.06	1.9	25	15.0	20.55	2.01	.655
23.6	81	11	3.42	15.3	1.86	.05	1.7	6.8	4.1	5.6	1.20	.71
338	33.4	33	6.6	11.4	3.92	.06	1.7	16.2	9.7	13.3	1.47	.69
1600	23.6	78	11.1	10.6	6.72	.05	1.6	23.2	13.9	19.1	1.25	.71
247	50	35	6.9	42	2.75	.05	1.7	10.1	6.06	8.32	0.88	.65
9650	9.7	212	20	4.5	16.0	.05	1.7	52	31.2	42.8	1.56	.65
72.8	31.6	15	4.1	28.5	1.86	.07	2.8	14	8.4	11.5	2.05	.57
4.11	426	2.75	1.47	52.8	0.558	.035	1.5	1.5	0.9	1.24	0.61	.83
21.3	88.2	7	2.61	19	1.33	.05	2.1	6.05	3.62	4.97	1.39	.685

FORM SNYDER: $T_c = C\left(\frac{10Ln}{\sqrt{S}}\right)^{.6}$ $S' = S/52.8$

REF. ASCE JOURNAL
OCT, 58 VOL 84 HY5
OCT, 59 VOL 85 HY 10

$$T_c = 13.1\, C\, n^{.6}\left(\frac{L^2}{S}\right)^{.3}$$

ASSUMED: $\bar{L} = .5L$
$t_p = 0.6\,T_c$

GIVEN: $t_r = T_c$

$$t'_p = \frac{21}{22}\,t_p + \frac{T_c}{4}$$

TABLE A-
9-III-66

UNIT HYDROGRAPH
SNYDER COEFFICIENTS
FOR SELECTED BASINS

	A	q_p csm	L	$(L\bar{L})^{.3}$	t_p
IOWA	0·91	1280	1·80	1·16	·47
	7·56	630	5·69	2·31	·90
	0·69	1515	0·85	·72	·35
	0·35	1755	0·75	·68	·32
	3·00	419	3·50	1·72	1·32
	24·57	154	9·50	3·13	2·56
	0·89	1805	1·78	1·15	·30
	32·64	261	12·50	3·70	2·17
NEBR.	20·00	298	11·59	3·13	1·87
	0·75	620	1·96	1·22	·73
OHIO	0·55	1120	0·82	·72	·53
	0·46	945	1·17	·89	·53
	0·46	1220	1·31	·95	·45
	1·44	550	1·56	1·06	·92
	2·37	565	2·41	1·38	·75
	4·02	372	3·25	1·66	1·30
	7·15	308	5·11	2·20	1·73
WISC.	0·52	1480	0·99	·81	·40
	0·27	1320	0·46	·51	·32
ILL	0·45	1055	0·54	·56	·47
MISSOURI	13·70	395	5·95	2·37	1·09
	8·36	495	2·45	1·39	·97
	21·30	330	6·00	2·38	1·67
	20·00	286	7·80	2·78	2·02
	0·62	1340	0·98	·80	·50
	2·72	348	2·50	1·41	1·45
	0·23	441	–	·64	·89
	24·10	370	9·40	3·12	1·45
	6·27	543	3·10	1·60	1·00
	12·20	450	3·70	1·78	1·13
	4·90	485	3·00	1·57	1·20
	2·87	416	2·45	1·39	1·36
	3·86	307	2·70	1·48	1·60
	4·72	292	1·98	·82	1·62

APPENDIX - C

LOCATION	DATE	HOURS									DAYS		
		1	2	3	4	5	6	12	18	24	2	3	4
BOGRA	24.6.57	2.40	3.91										
	23.7.58	1.00	2.00	2.54	2.80	3.77	4.40	6.90	8.11	9.05	9.33	11.09	
	14.8.59	1.62	2.20	2.62	2.96	3.28	3.66	5.41	5.88	6.83	11.27	12.79	13.93
MASTER PLAN	10 YR.	3.40	4.23	4.70	-	-	5.47	6.30	6.90	7.40			
DACCA	4.8.58	1.53	2.15	3.54	3.97	4.15	4.19	4.42	4.87	6.90			
	20.5.60	2.62	2.63	2.66	-	2.67	-	2.95	4.51	4.66	7.98	9.97	
MASTER PLAN	10 YR.	3.10	4.00	4.50	-	-	5.25	6.10	6.60	7.06			
CHITTAGONG	29.10.59	1.78	2.12	3.16	3.76	4.16	4.50	5.96	6.86	7.50			
MASTER PLAN	10 YR.	2.43	3.07	3.58	-	-	4.56	6.67	9.00	11.00			
COX'S BAZAR	11.7.59	2.20	3.04	3.80	4.00	4.57	4.60	6.66	7.24	9.50	13.71	15.83	
	10.6.60	1.77	3.31	3.36	3.42	3.44	3.50	3.57	5.65	6.00	7.83	9.81	11.80
MASTER PLAN	10 YR	2.23	2.88	3.30	-	-	4.40	6.65	10.50	14.10			
JESSORE	3.9.59	.66	1.04	1.24	1.72	2.18	2.68	4.14	4.22	6.16			
	10.7.60	1.31	2.41	2.72	3.01	3.19	3.25	3.36	3.68	5.61			
MASTER PLAN	10 YR.	2.37	3.00	3.35	-	-	3.80	4.43	5.15	5.90			
SYLHET	13.5.58	2.26	4.07	4.25									
	19.6.58	2.02	3.12	4.22	5.09	5.46	5.57	5.60	7.84	12.28			
	3.9.58	2.86	3.10	3.15									
MASTER PLAN	10 YR.	2.85	4.15	5.00	-	-	6.16	8.20	12.00	15.30			

STORM RAINFALL DISTRIBUTION
EAST PAKISTAN

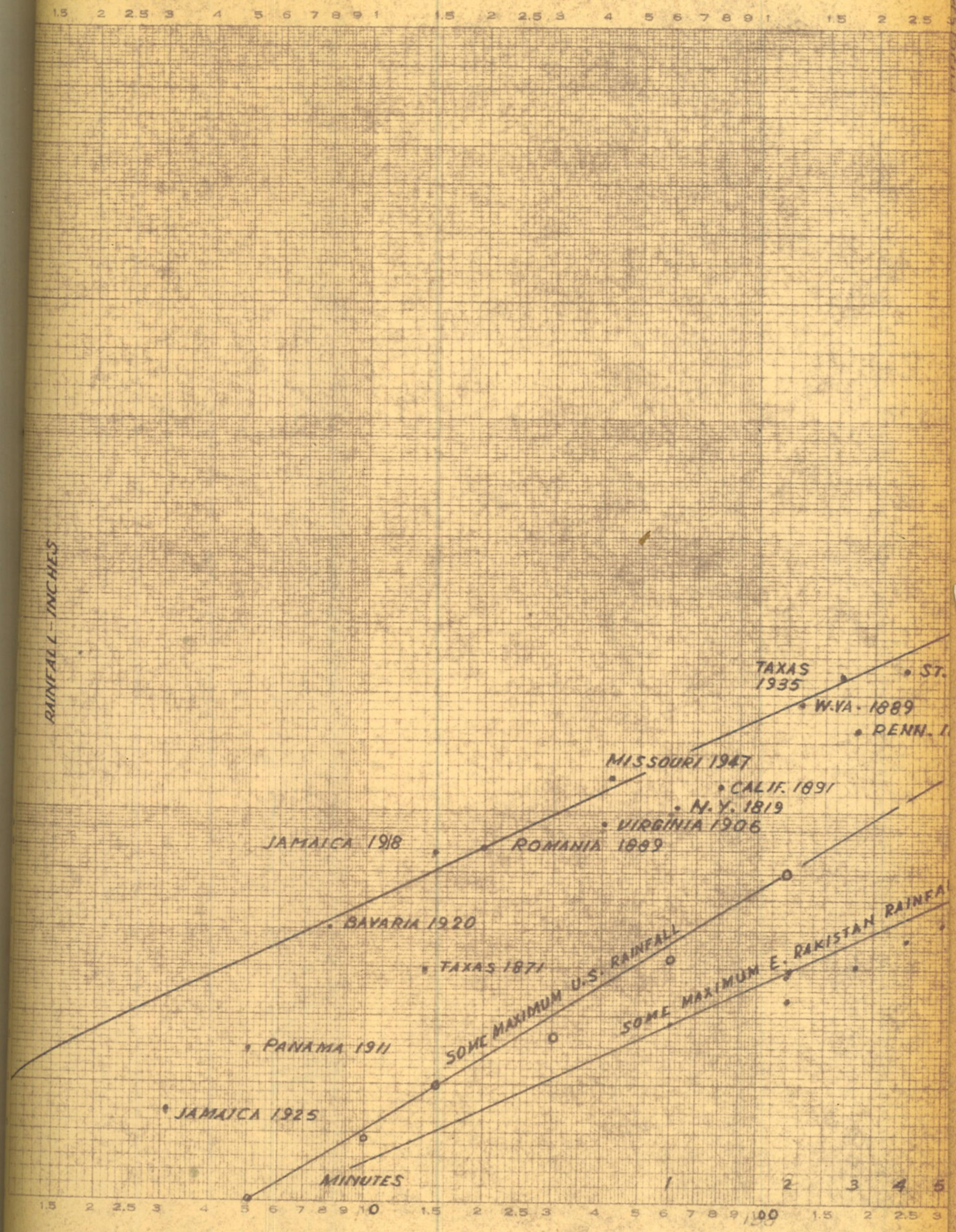
TABLE C-1

SOME MAXIMUM RECORDED U.S.
POINT RAINFALL
U.S. GULF COAST

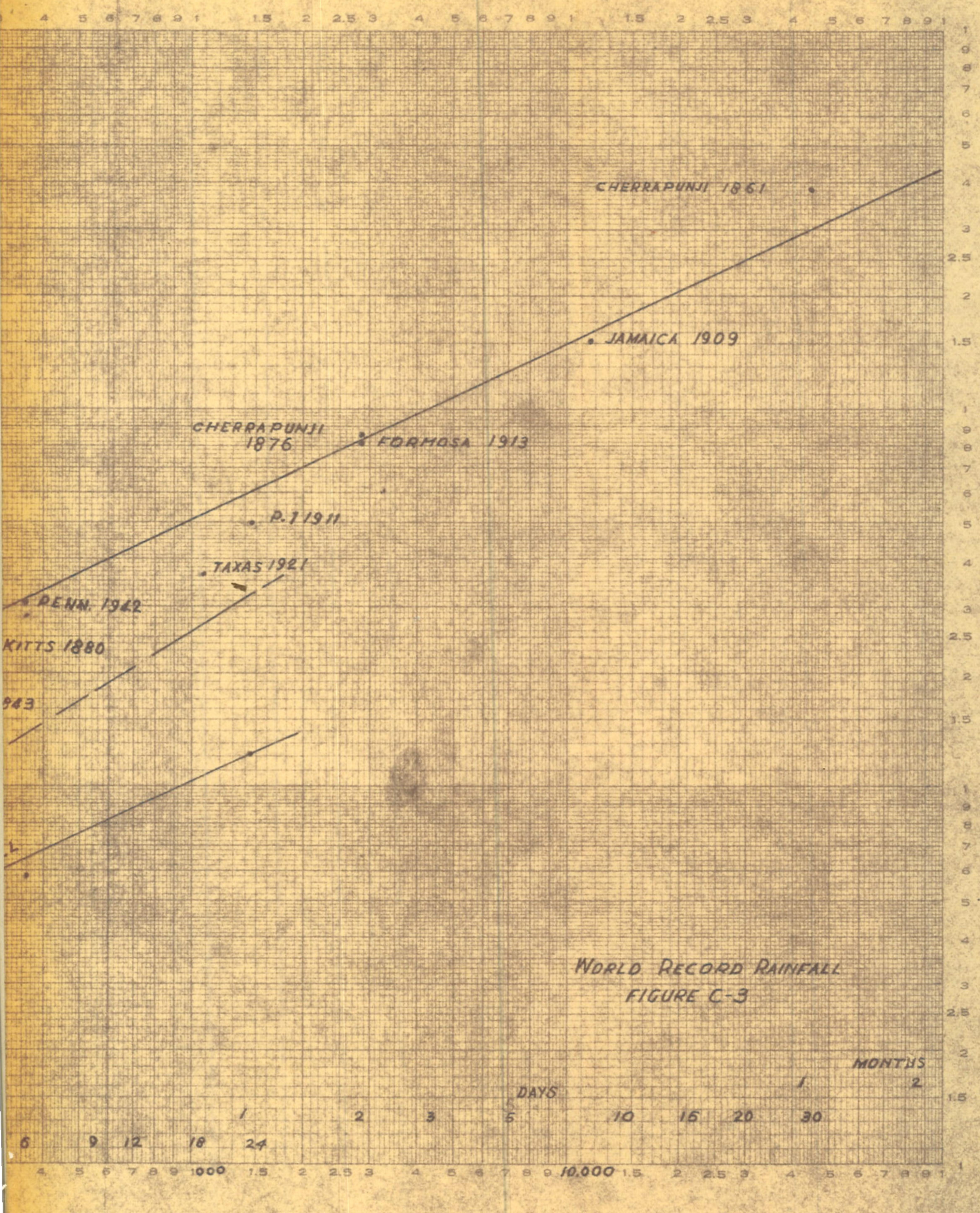
LOCATION	HOURS 2	6	12	24
TEXAS				
GALVESTON	7·58	11·79	12·75	14·35
HOUSTON	6·05	8·67	9·92	10·83
LOUISIANA				
NEW ORLEANS	5·71	7·95	12·76	14·01
SHREVEPORT	5·19	7·74	8·52	12·44
MISSISSIPPI				
VICKSBERG	4·17	7·10	7·55	7·99
ALABAMA				
MOBILE	4·47	8·22	12·98	12·98
FLORIDA				
TAMPA	4·87	6·81	9·01	10·41
PENSACOLA	6·14	10·57	15·64	17·07
MIAMI	6·11	10·64	13·86	15·10
JACKSOH VILLE	4·19	6·75	8·65	9·86
GEORGIA				
SAVANNAN	4·89	5·41	6·75	11·44
ATLANTA	4·32	4·63	5·40	7·36
VIRGINIA				
NORFOLK	4·80	4·81	4·82	6·84
RICHMOND	6·33	7·24	7·24	7·26
SOUTH CAROLINA				
CHARLESTON	6·64	8·62	9·03	10·57
COLUMBIA	3·84	4·10	4·89	5·50

TABLE C-2

MISC. PS. B

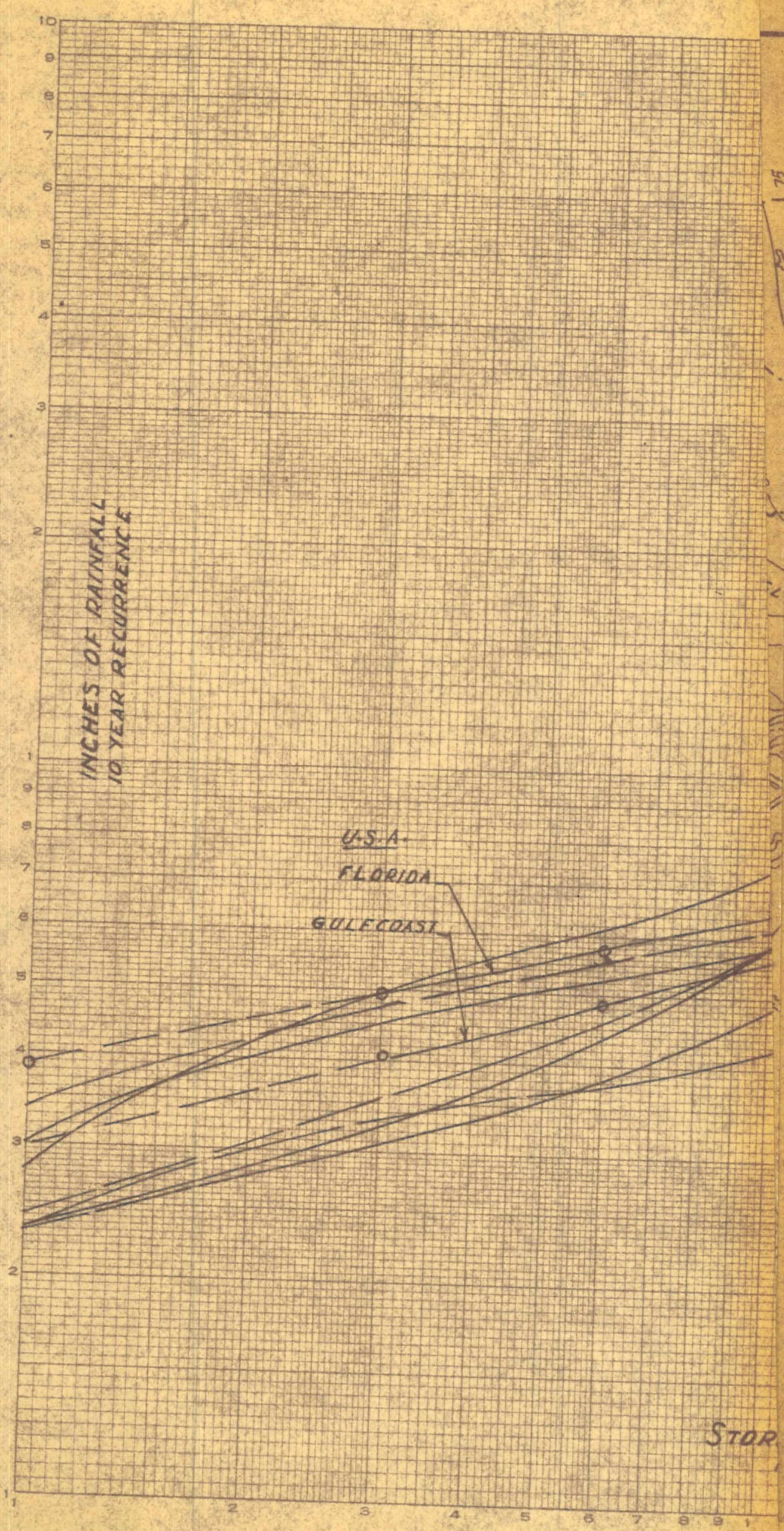


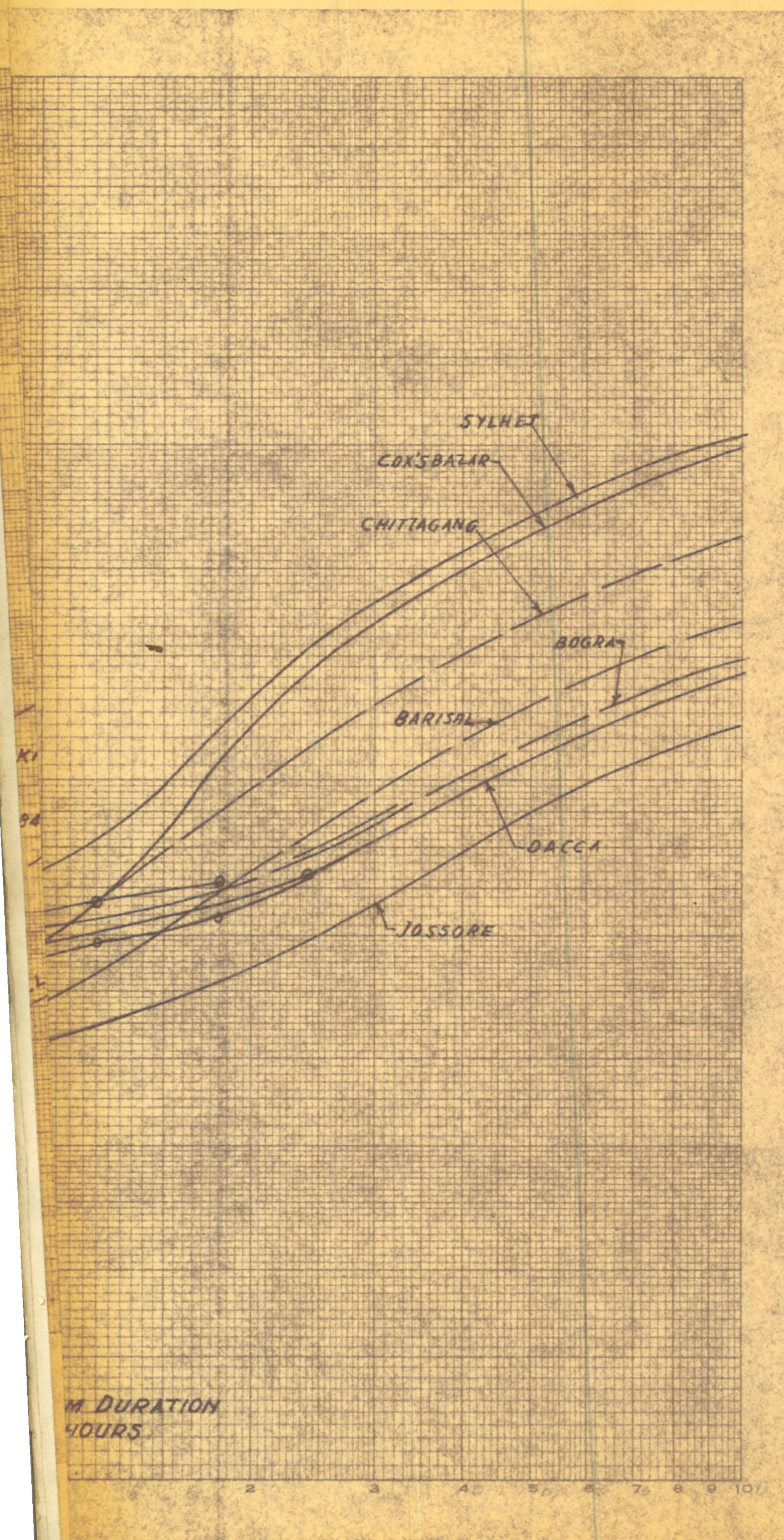
RAINFALL-INCHES
MINUTES
TAXAS 1935
ST.
W.VA. 1889
PENN.
MISSOURI 1947
CALIF. 1891
N.Y. 1819
VIRGINIA 1906
JAMAICA 1918
ROMANIA 1889
BAVARIA 1920
TAXAS 1871
SOME MAXIMUM U.S. RAINFALL
SOME MAXIMUM E. PAKISTAN RAINFA
PANAMA 1911
JAMAICA 1925



WORLD RECORD RAINFALL
FIGURE C-3

MISC (S.B) 24


INCHES OF RAINFALL
10 YEAR RECURRENCE
U.S.A.
FLORIDA
GULF COAST
STOR



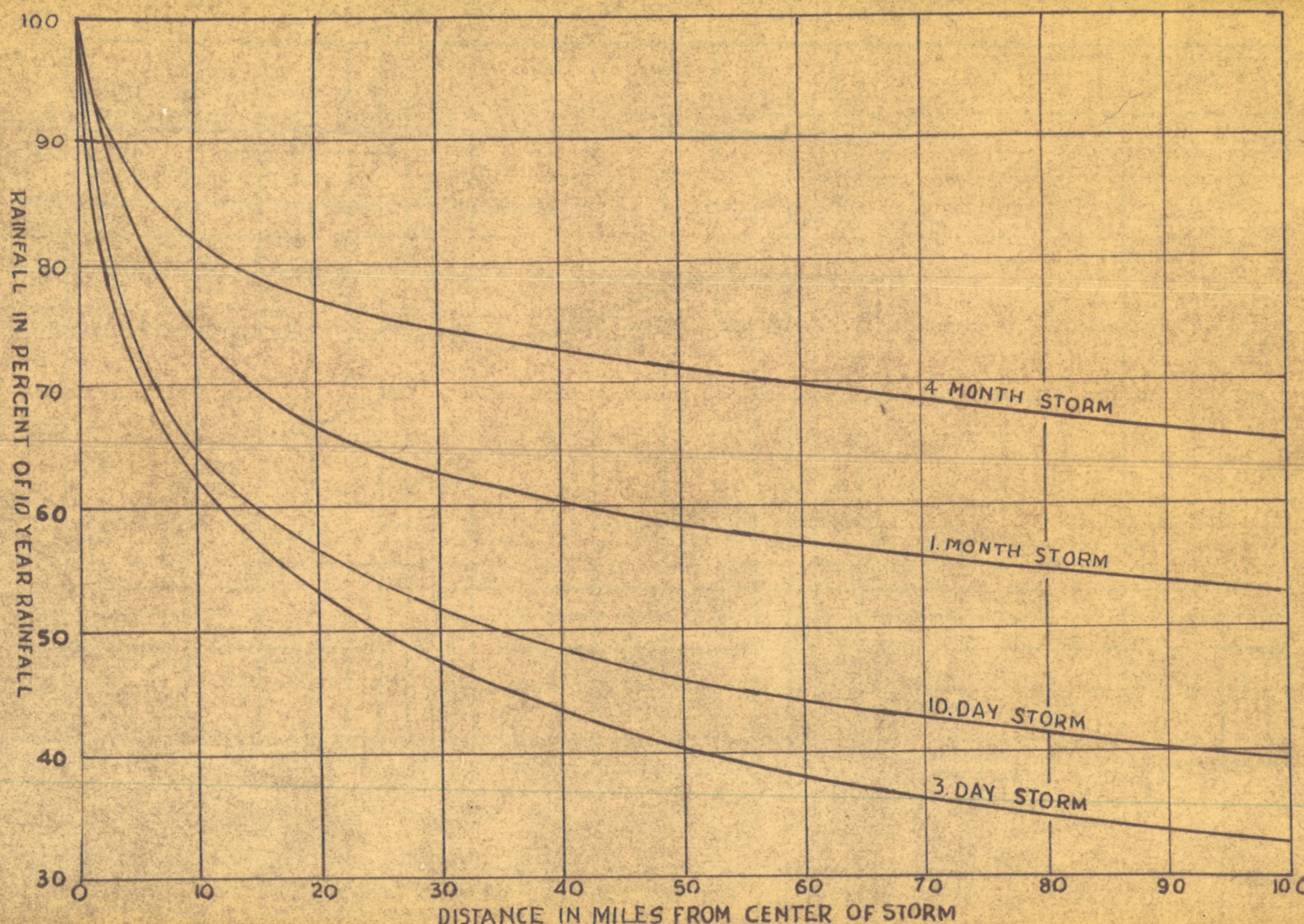
STORM RAINFALL
10 YEAR RECURRENCE
FIGURE C-4


INDIA
N
DINAJPUR
RANGPUR
BOGRA
NASIRABAD
RAJSHAHI
QUAIDABAD
SYLHET
TANGAIL
PABNA
DACCA
KUSHTIA
FARIDPUR
COMILLA
INDIA
TRIPURA
INDIA
INDIA
28°
25°
24°
25°
24°
INDIA
BURMA
EAST
PAKISTAN
THAILAND
ARABIAN SEA
BAY OF BENGAL
CEYLON
MALAYA
15°
0°
60°
75°
90°
VICINITY MAP

45-50

MISC-(S.B) 36.

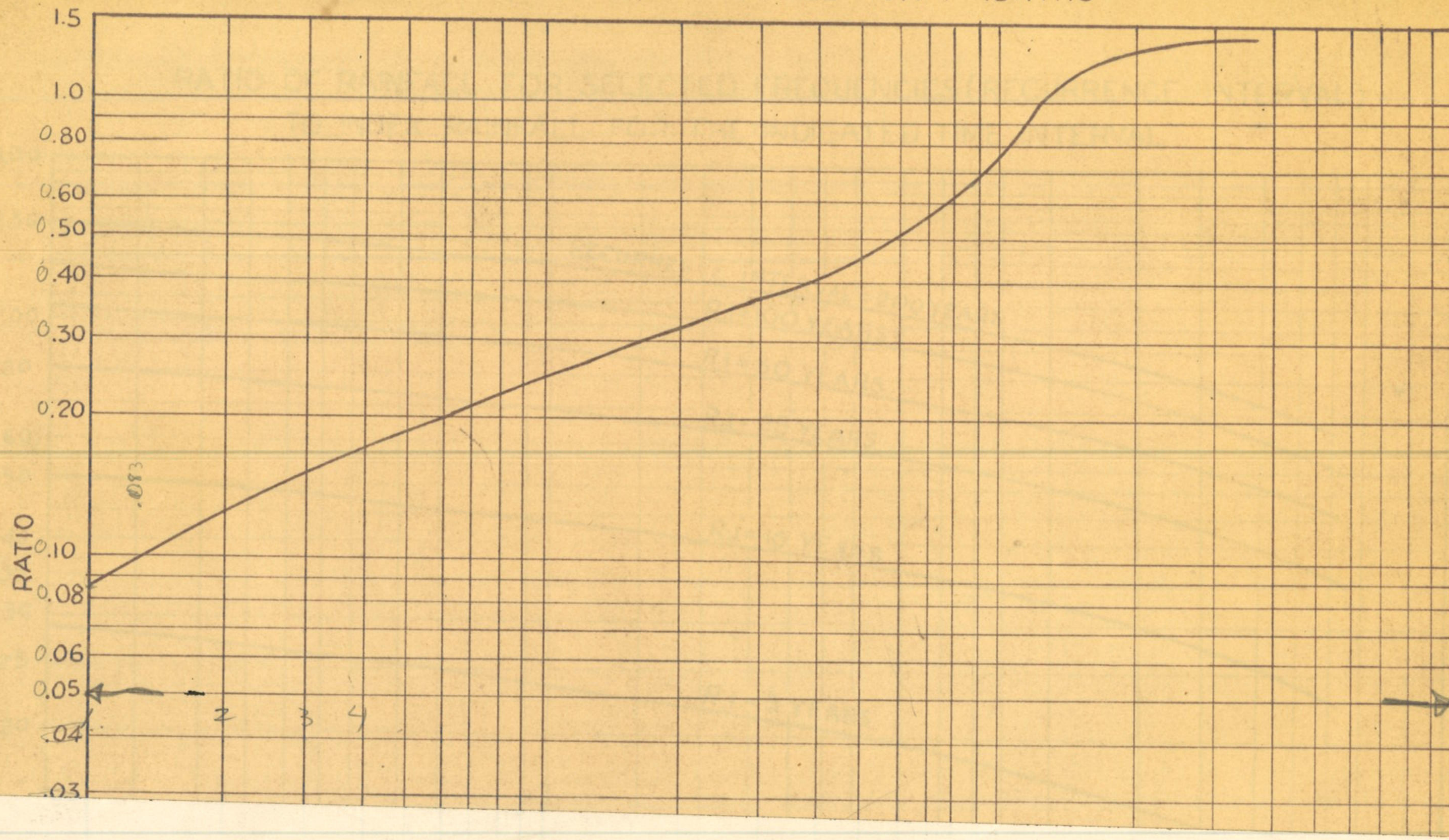
REDUCTION IN RAINFALL FROM STORM CENTER



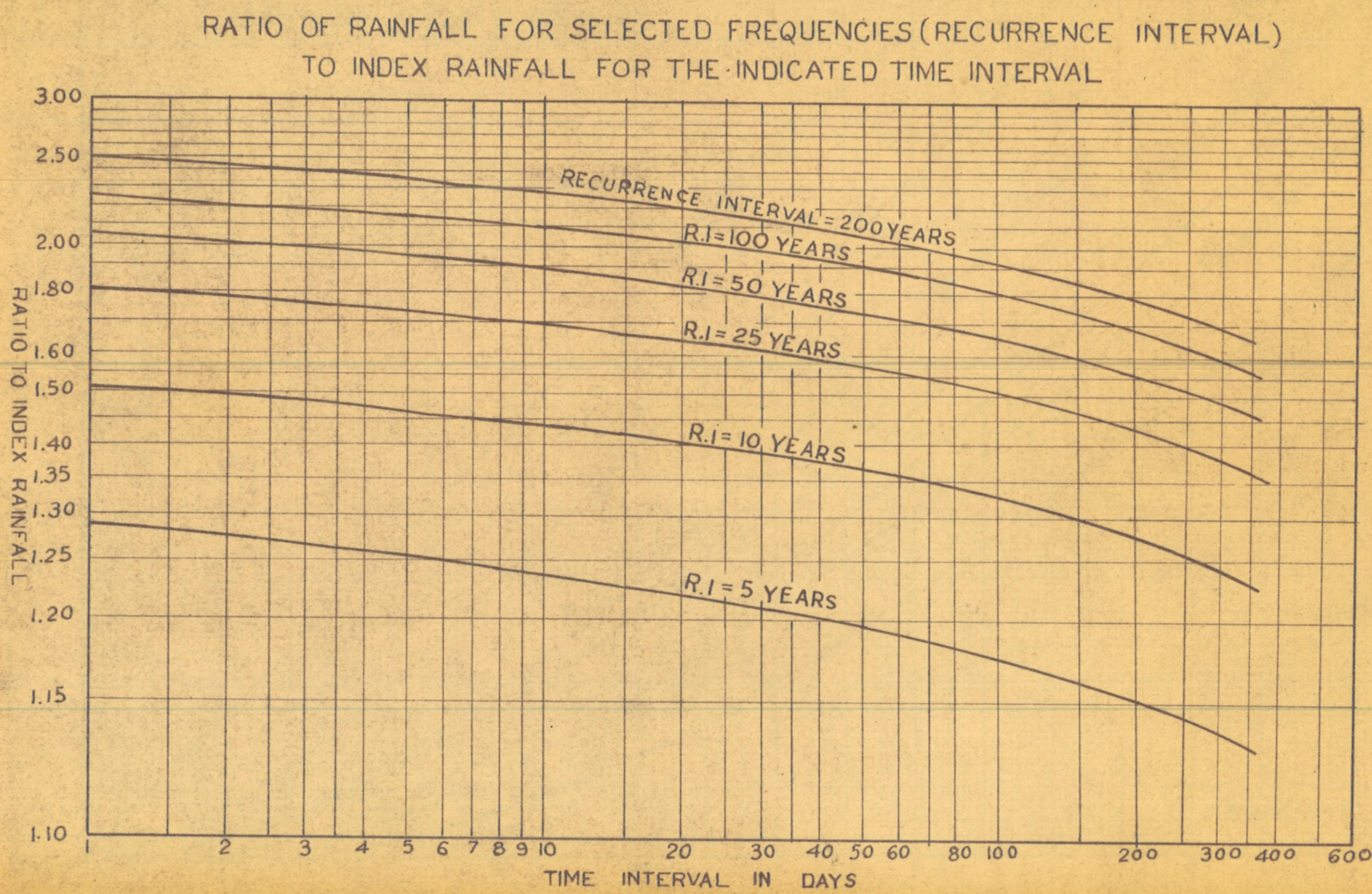
DR. NO. AW-04-051

misc (S.B.) 37

RATIO OF INDEX RAINFALL FOR A GIVEN TIME INTERVAL TO INDEX RAINFALL FOR 4 MONTHS



DR.NO.AW-04-049



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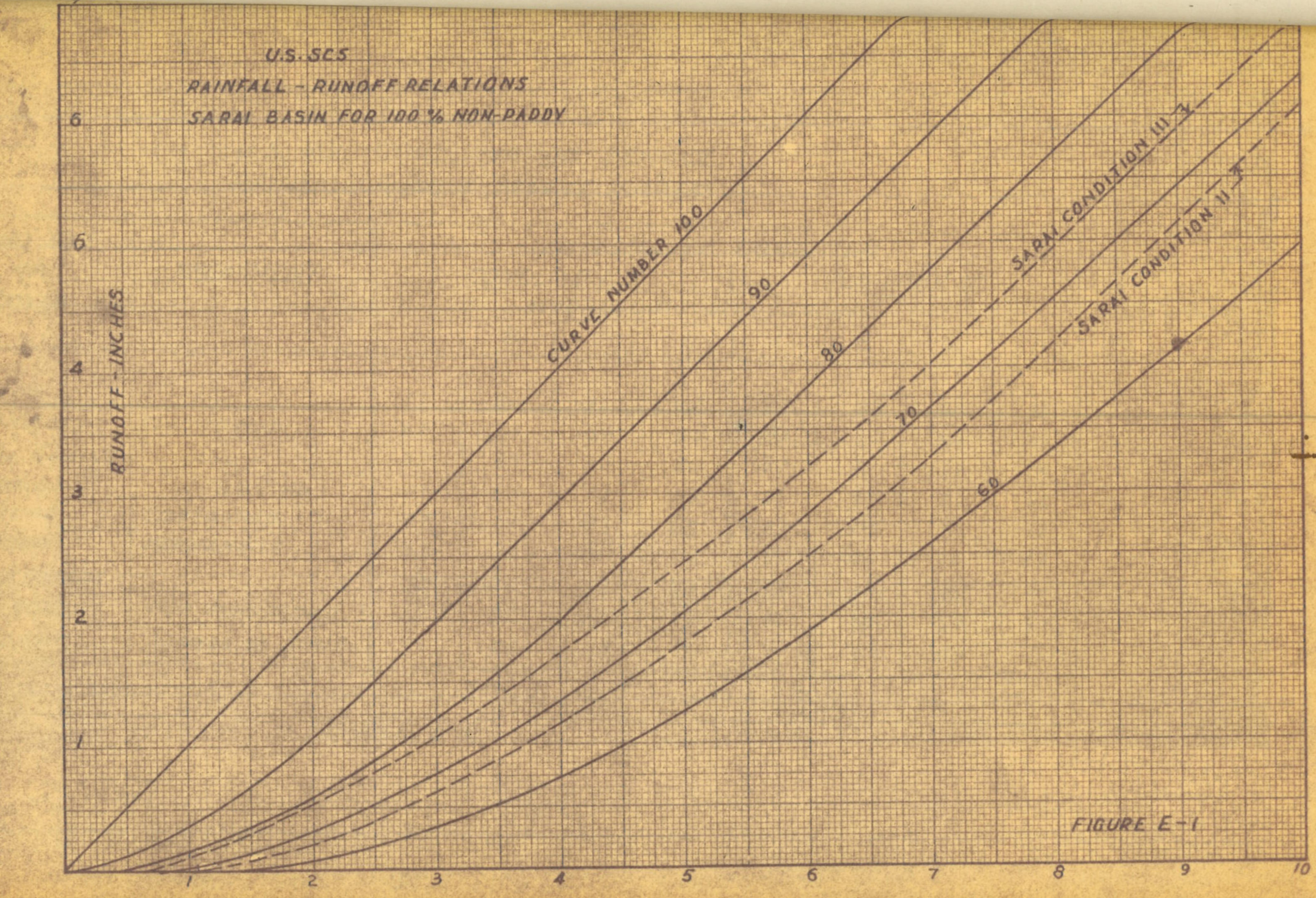

FIGURE E-1

MISC. (S.B.) 40

U.S. SOIL CONSERVATION SERVICE
RAINFALL-RUNOFF SERVICE NUMBERS

LAND USE OR COVER	TREATMENT	ANTECEDENT CONDITION							
		CONDITION II				CONDITION III			
		HYDROLOGIC SOIL GROUPS							
		A	B	C	D	A	B	C	D
FALLOW		77	86	91	94	92	97	98	99
ROW CROPS	CONTOURED AND TERRACED	66	74	80	82	84	90	94	95
SMALL GRAINS		57	68	77	80	77	85	92	94
LEGUMES (3)		41	62	73	78	61	81	89	92
PASTURE	CONTOURED	47	67	81	88	67	85	94	98
MEADOW	PERMANENT	30	58	71	78	50	77	88	92
HOMESTEADS	-	59	74	82	86	78	90	95	97
DIRT ROADS	NOT MAINTAINED	62	72	77	79	81	89	92	93
WOOD LOT	BAMBOO OR BRUSH	-	46	65	-	-	66	83	-

TABLE E-2

DISTRICT	TOTAL AREA ACRES (1000)	PERCENT OF TOTAL AREA						
					PADDY	NON-PADDY		
		FOREST	NOT AVAILABLE FOR CULTIVATION	OTHER LAND	SUMMER RICE	JUTE	SUGAR CANE	OTHER
DACCA	1,835	4	22	29	33	9	2	1
MYMENSINGH	4,073	4	18	26	40	10	1	1
FARIDPUR	1,657	-	16	22	50	9	1	2
BAKERGANJ	2,730	1	17	22	57	1	1	1
CHITTAGONG	1,723	38	16	6	39	-	-	1
NOAKHALI	1,277	-	15	40	41	2	-	2
COMILLA	1,699	-	17	13	61	7	-	2
SYLHET	3,055	7	39	17	35	1	-	1
RAJSHAHI	2,324	-	26	23	45	3	2	1
DINAJPUR	1,639	1	27	18	46	4	3	1
RANGPUR	2,386	-	32	14	42	3	2	2
BOGRA	981	-	11	24	56	5	2	2
PABNA	1,210	-	15	40	37	6	1	1
KUSHTIA	882	-	24	43	26	3	3	1
JESSORE	1,675	-	27	32	35	4	1	1
KHULNA	2,992	50	17	5	25	1	-	2

LAND USE CLASSIFICATION

TABLE - 3

LAND USE	COVER	PERCENT OF AREA	CONDITION II CURVE NUMBER		CONDITION III CURVE NUMBER	
			E-2	WEIGHTED	E-2	WEIGHTED
PADDY LAND	SMALL GRAINS	42	68	28·6	68	28·6
NON PADDY	ROW CROPS	12	74	8·9	90	10·6
OTHER LAND		14				
PASTURE	HEAVILY GRAZED	10	67	6·7	84	8·4
FALLOW		4	86	3·5	97	3·9
NOT AVAILABLE FOR CULTIVATION		32				
WOODLOT	BAMBOO OR BRUSH	10	46	4·6	66	6·6
HOMESTEAD		10	74	7·4	90	9·0
ROAD		1	72	0·7	89	0·9
WATER WAYS		1	100	1·0	100	1·0
TANKS		2	0	0	0	0
MEADOW	PERMANENT	8	58	4·6	77	6·7
TOTAL WEIGHTED BASIN				66·0		75·3

APPENDIX E

RAINFALL-RUNOFF CURVE NUMBER
WEIGHTED BASIN
PADDY LAND RETENTION NOT INCLUDED

TABLE E-4

BRUNING 40-8403
10x10 TO 1/2 INCH
MADE IN U.S.A.

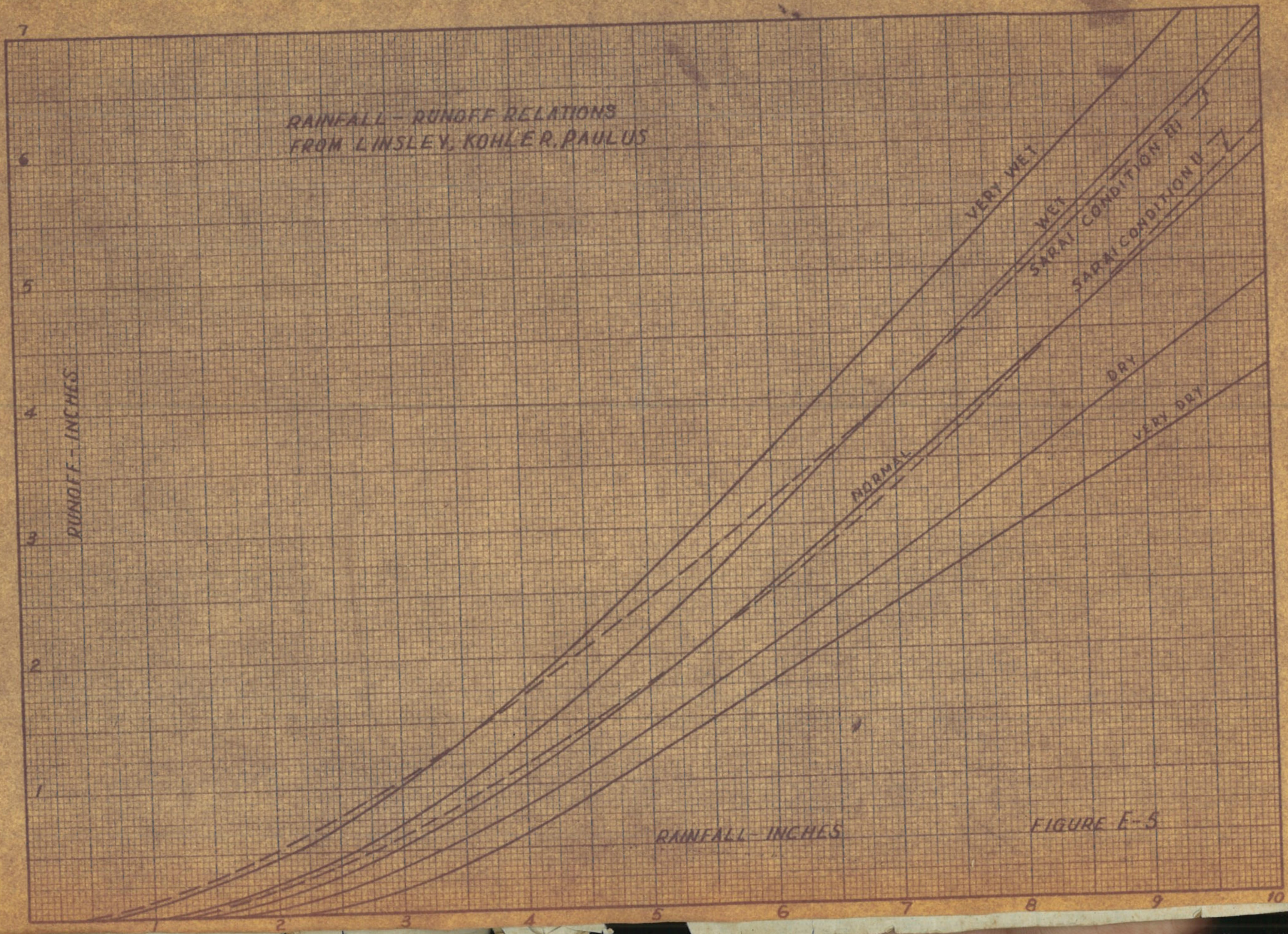

FIGURE E-5