WORKSHOP ON
DESIGN OF
SMALL SCALE WATER CONTROL STRUCTURES

VOLUME II

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BANGLADESH WATER DEVELOPMENT BOARD
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PREFACE

This publication includes

a. methodologies and details of computations for the design of a non tidal drainage regulator

b. methodologies and computations for hydrologic and hydraulic design of a non tidal and a tidal regulator and

c. hydrologic and hydraulic design of a flushing regulator

The purpose of these volumes are to train engineers of Bangladesh Water Development Board on design of small scale water control structures.

These publications have been produced with financial assistance from Canadian International Development Agency for the Small Scale Water Control Structures (SSWCS) Project.

The SSWCS Project has a training component which includes following categories of training :

1. Field workshops for Junior level field officers and contractors on construction management and O&M.

2. Dhaka Workshops for mid level engineers on planning and design aspects of small scale projects.

3. Advanced applied programs for mid level engineers at the Asian Institute of Technology, Thailand to provide in-depth training on water resource development.

4. Brief Technical Missions by senior officials to Canada and South East Asian countries to visit similar programs.

These volumes have been prepared to meet the partial requirements of training materials for 2nd and 3rd categories of training.

Any suggestions to improve the quality of contents will be greatly appreciated.

March, 1987

H.R. Khan
Water Resource Advisor.

DESIGN OF SMALL SCALE WATER CONTROL STRUCTURES.

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CHAPTER VI

HYDROLOGIC AND HYDRAULIC DESIGN
OF
BIDIRPUR AND JUBLEE KHAL REGULATOR

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DRAINAGE REGULATOR

DATA REQUIRED FOR HYDROLOGIC DESIGN

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   - area of the drainage basin
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   - map of the basin area with its geographical center
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DATA REQUIRED FOR HYDRAULIC DESIGN

1. Design discharge (from flood routing)

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6. Top width of embankment (from x-sections of the embankment)

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9. Water level data (for frequency analysis)

DESIGN PROCEDURE

A. Develop a Design Storm

1. Point Rainfall

a) Locate basin on Fig.1 and find the 4-month rainfall index.

b) Determine from Fig.2, the ratio to apply to the four-month index to compute the index rainfall for the selected time interval. (Table 1).

c) Determine from Fig.3, the ratio to apply to the index rainfall for the selected time period to determine the rainfall for the selected recurrence interval. (Table 2).

d) Multiply the index rainfall for the selected time period in Table 1 determined by step A.1.b by the ratio in Table 2 determined by step A.1.c. The calculation is in Table 3.

e) Multiply the 4-month index by the daily combined indices in Table 3. This gives the accumulated point rainfall volume in inches for storms of 1,2,3,4 & 5 days duration. Table 4 shows the computation.

2. Equivalent Uniform Depth of Rainfall.

a) Compute the reduction factor for rainfall variability. Computation in Table 5 and Table 6.

b) Multiply the point rainfall volumes by the rainfall reduction factor. Separate accumulative totals into daily increments. Computation in Table 7.

3. Rainfall Time Distribution

a) Arrange the daily increments in an order giving the worst flood condition. For a 3-day storm use the sequence 3,2,1. For a 5-day storm use 3,2,1,4,5. For a 6-day storm use 6,3,2,1,4,5 and so on.

b) Locate the gauging station closest to the basin being investigated. Convert the indicated gaging station rainfall during multiples of the unit duration to rainfall at the basin by using a ratio of the respective 4-month indices.

c) Convert unit duration point rainfall to equivalent uniform depth using the reduction factor. Table 9 shows the computation.

d) Arrange the unit durations for the maximum day into sequence. Use the same sequences given in step 3-a.

e) Compute the volumes and arrange the unit durations for the remaining days into sequence (Table 10).

B. **Determine percentage of paddy and non-paddy land**

Determine land classification as percentages of area in paddy land and non-paddy land. Whenever possible these percentages should be determined by field inspection. This is particularly important for small basins where the proportion of paddy land is disproportionately high. In absence of a survey the percentages on Table 13 can be used as an average.

C. **Determine Rainfall Excess (Runoff Distribution)**

1. Paddy land:

   Separate the rainfall losses from the rainfall time distribution determined in Table 10 by assuming the entire basin area consists of paddy land. (Table 12).

2. Non-paddy land:

   Separate the rainfall losses from the rainfall time distribution determined in Table 10 by assuming the entire basin area consists of non-paddy land. (Table 13).

3. Compute the weighted basin runoff distribution by multiplying the net runoff obtained in steps D-1 and D-2 by the respective land classification percentage as in Table 11. ~~Computation shown in~~ Table 14.

D. <u>Develop a Unit Hydrograph</u>

E. <u>Prepare the Runoff Hydrograph</u>

F. <u>Flood Routing through the sluice</u>

1. Prepare Basin Elevation-Area-Capacity curves (Fig.13)
2. Prepare stage-discharge curves for different flow conditions
3. Prepare 2s/t + D Curve
4. Route the flood through the structure
5. From the flood routing computations plot basin storage water surface elevation vs. time
6. Make assessment of flood damage
7. From the level of flooding and the extent of crop damage, make a judgement whether the capacity of the structure is adequate. If necessary repeat routing procedure using a revised sluice discharge capacity.

A. Develop a Design Storm

1. Point Rainfall

Point rainfall is the quantity of rain falling at a specific point, usually as measured at a rain gauging station. The point rainfall can be defined as the center of a storm, equal to the product of the four-month rainfall index (45") and the combined rainfall indices from Table 3.

a) Determine the four-month index rainfall for the desired location from the isohyetal map (fig.1).

In this case, the rainfall index for four-month period for the project area is 45".

b) Determine from fig.2, the ratio to apply to the four-month index to compute the index rainfall for the selected time interval. A rainfall duration of 5 days is required for our study.

Days	Ratio
1	0.085
2	0.13
3	0.158
4	0.176
5	0.189

Ratio of Index Rainfall
Table - 1

c) Determine from fig.3, the ratio to apply to the index rainfall for the selected time period to determine the rainfall for the selected recurrence interval.

	Ratio	
Days	10 yr.frequency	25 yr.frequency
1	1.51	1.80
2	1.48	1.76
3	1.47	1.727
4	1.465	1.725
5	1.46	1.72

Ratio of Rainfall for 10 yr & 25 yr Recurrence Interval
Table - 2

d) Multiply the index rainfall for the selected time period determined in step B-1-b by the ratio determined in step B-1-c.

Days	Storm Frequencies	
	10 years	25 years
1	0.128	0.153
2	0.192	0.288
3	0.230	0.272
4	0.257	0.303
5	0.276	0.326

Combined Rainfall Index for Selected Frequency
Table-3

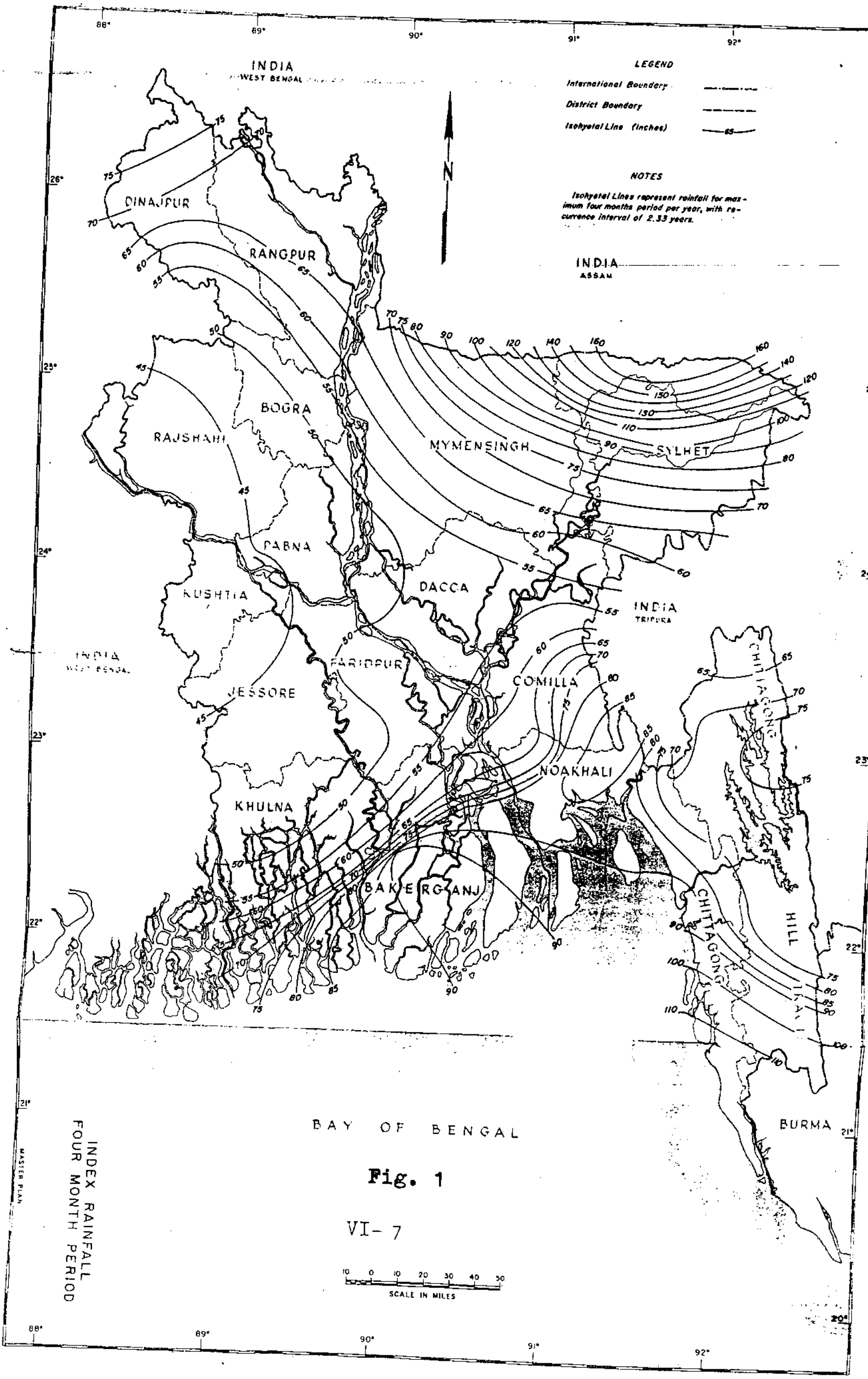

INDIA
WEST BENGAL
LEGEND
International Boundary
District Boundary
Isohyetal Line (Inches)
NOTES
Isohyetal Lines represent rainfall for maximum four months period per year, with recurrence interval of 2.33 years.
N
INDIA
ASSAM
DINAJPUR
RANGPUR
RAJSHAHI
BOGRA
MYMENSINGH
SYLHET
PABNA
DACCA
KUSHTIA
INDIA
TRIPURA
INDIA
WEST BENGAL
FARIDPUR
COMILLA
JESSORE
NOAKHALI
CHITTAGONG
KHULNA
BAKERGANJ
CHITTAGONG
HILL
BAY OF BENGAL
BURMA
SCALE IN MILES
INDEX RAINFALL
FOUR MONTH PERIOD
MASTER PLAN

Fig. 1

VI- 7

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

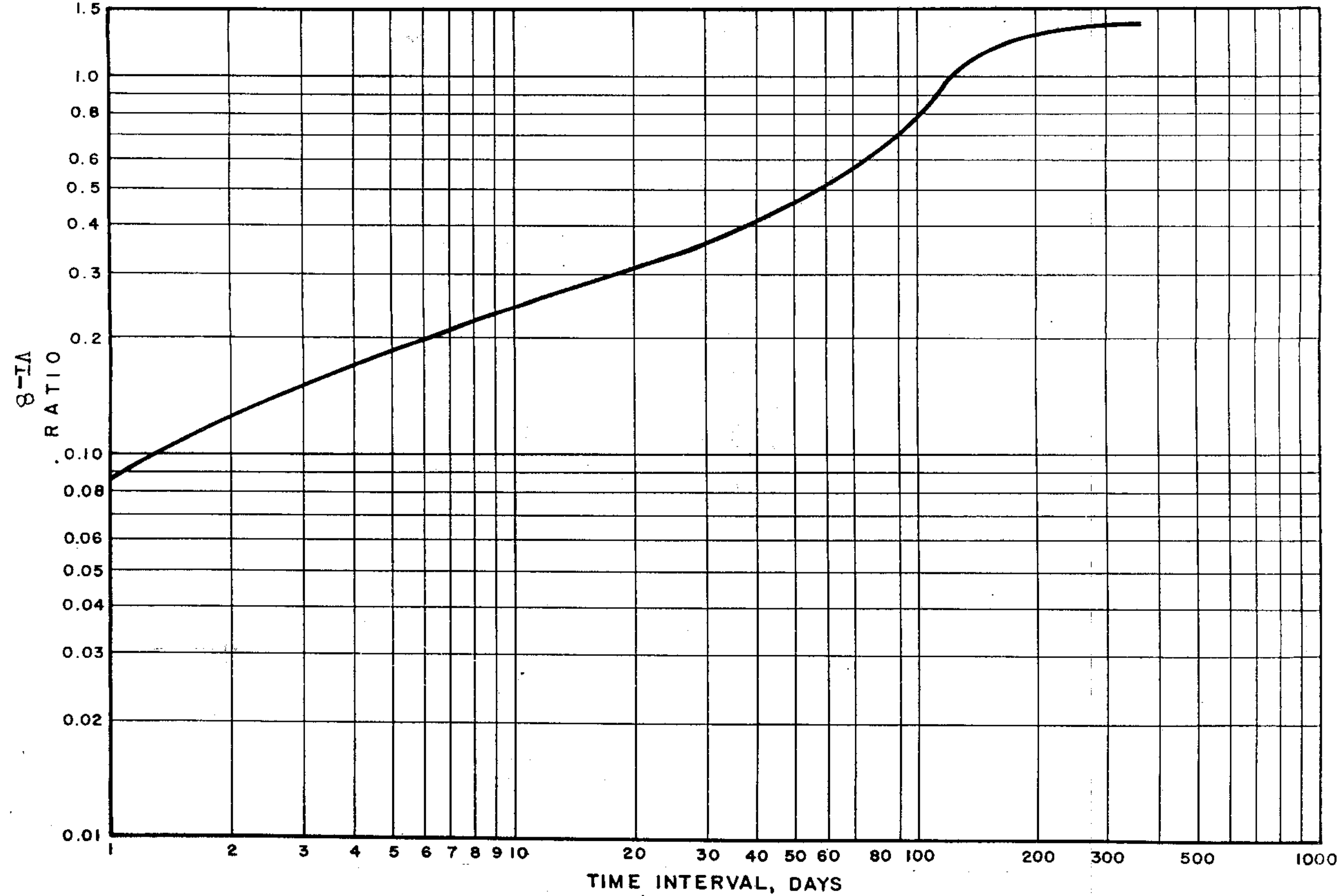
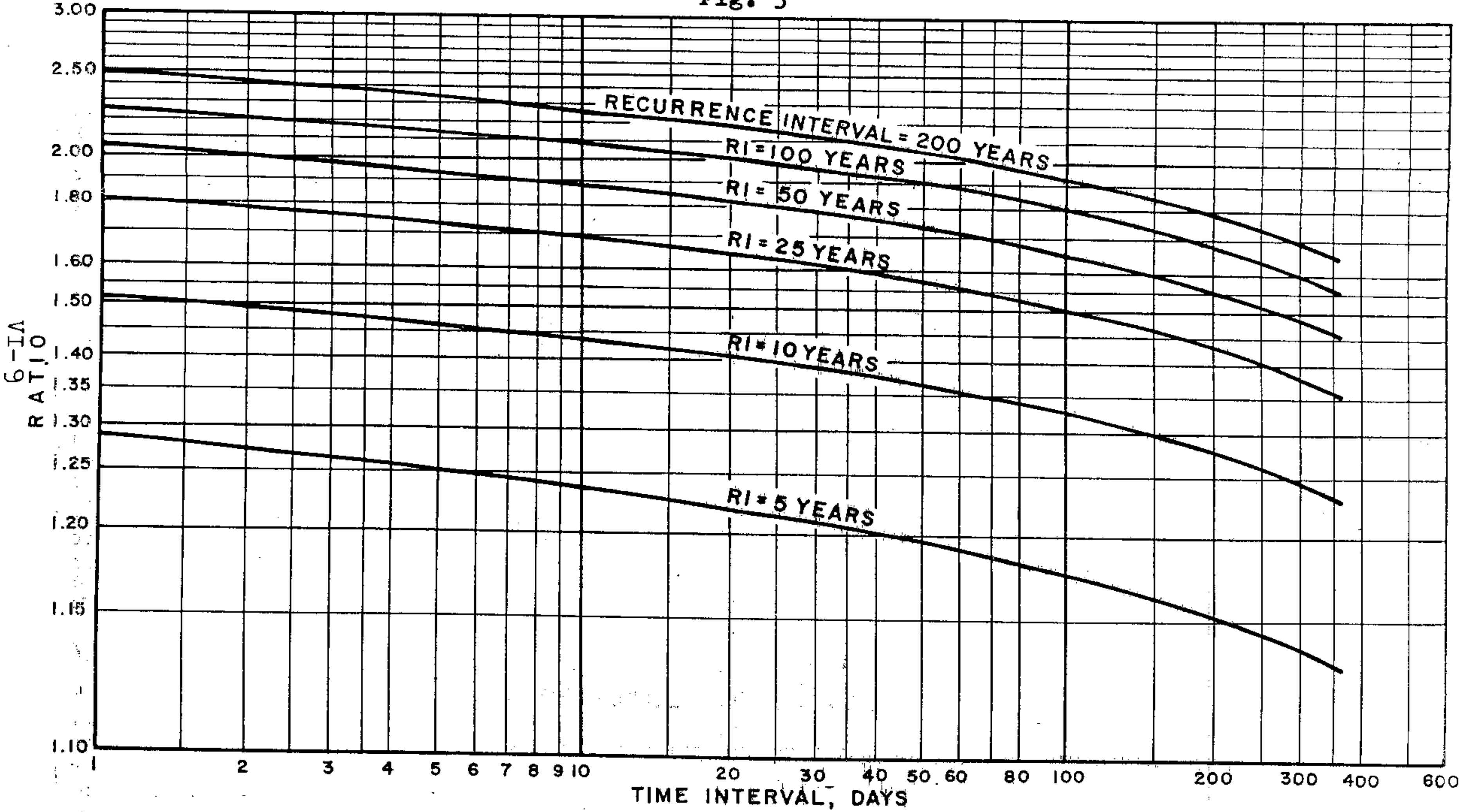


Fig. 2

RATIO OF RAINFALL FOR SELECTED RECURRENCE INTERVALS TO INDEX RAINFALL FOR INDICATED TIME INTERVAL

Fig. 3



e) Rainfall index for four months for the project area = 45"

Accumulated point rainfall for 10 years frequency is as follows:

Days	Accumulated point rainfall for 10 yrs. frequency
1	5.8" (45"x0.128)
2	8.6" (45"x0.192)
3	10.4" (45"x0.23)
4	11.6" (45"x0.257)
5	12.4" (45"x0.276)

Accumulated Point Rainfall
Table-4

If point rainfall were used as the basis for designing a structure the computed volume of water would be unrealistically large. Rainfall intensity decreases as the distance from the storm center increases until at the storm edge it is zero.

Therefore it is necessary to determine an average or equivalent uniform depth of rainfall.

2. Equivalent Uniform Depth of Rainfall

The equivalent uniform depth of rainfall is defined as the depth of water which results from spreading the total volume of basin rainfall uniformly over the total basin area.

The storm center may occur at any point over the basin or may move along any path across the basin. We assume the storm center is stationary over the geographical center of the basin.

The rate of decrease of a storm's intensity away from its center has been studied by the General Consultants the IECO Master Plan states that studies of storm in Bangladesh indicate, that a fairly good relationship exists between depth of rain and distance from storm center.

Isohyetal maps for total rainfall in periods of 3-days, 5-days, 1-month and 4-months of 10-year recurrence interval are shown in figures 4,5,6,7 respectively. Fig.8 shows the relationship plotted as percent of the rainfall of 10-years recurrence interval against distance from the storm center. Table 5 shows the relationship in tabular form for 5-day storm.

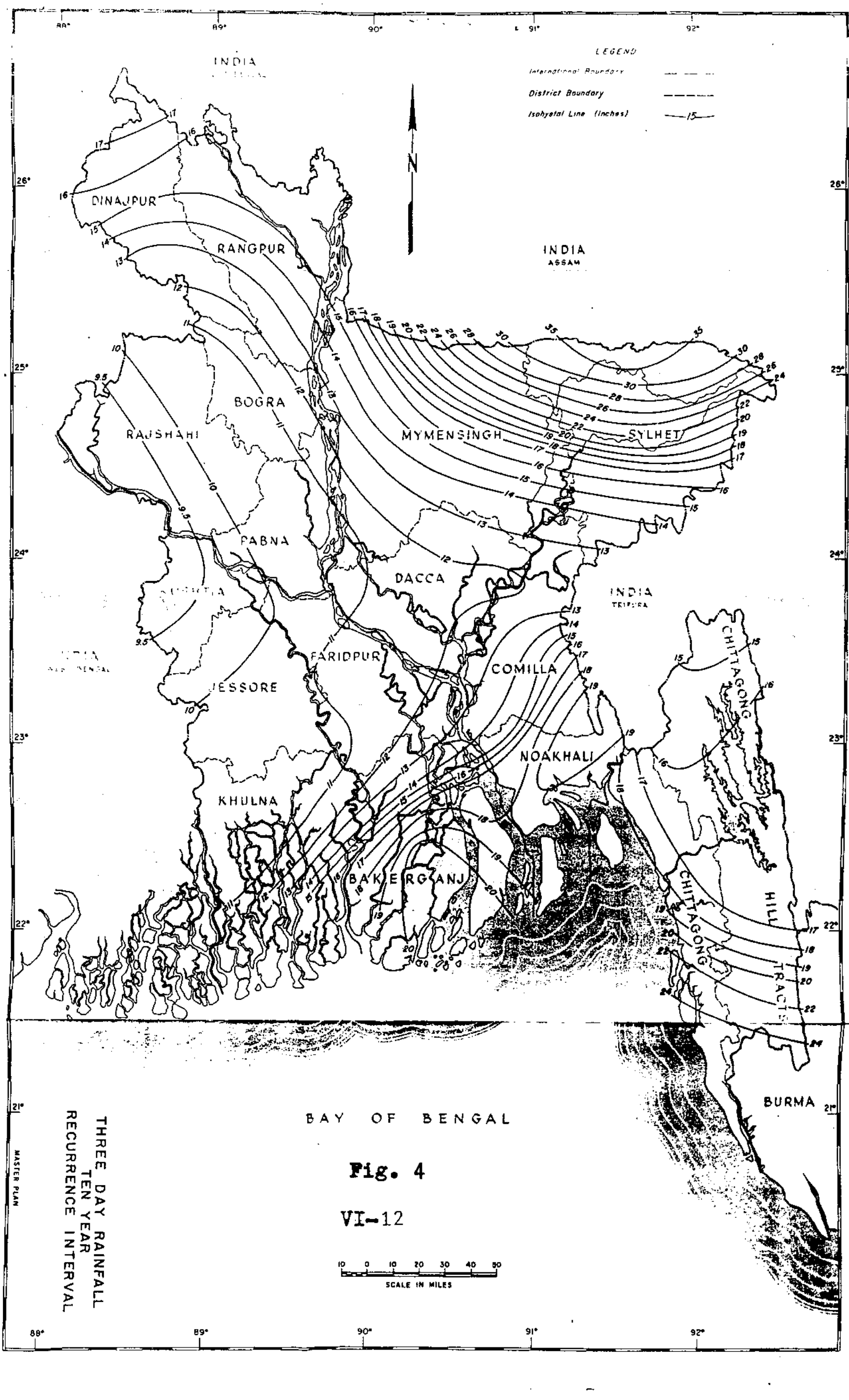

LEGEND
International Boundary
District Boundary
Isohyetal Line (inches)
INDIA
INDIA
ASSAM
INDIA
TRIPURA
DINAJPUR
RANGPUR
BOGRA
RAJSHAHI
MYMENSINGH
SYLHET
PABNA
DACCA
FARIDPUR
COMILLA
JESSORE
KHULNA
NOAKHALI
BAKERGANJ
CHITTAGONG
CHITTAGONG HILL TRACTS
BURMA
BAY OF BENGAL
THREE DAY RAINFALL
TEN YEAR
RECURRENCE INTERVAL
MASTER PLAN
SCALE IN MILES

Fig. 4

VI-12

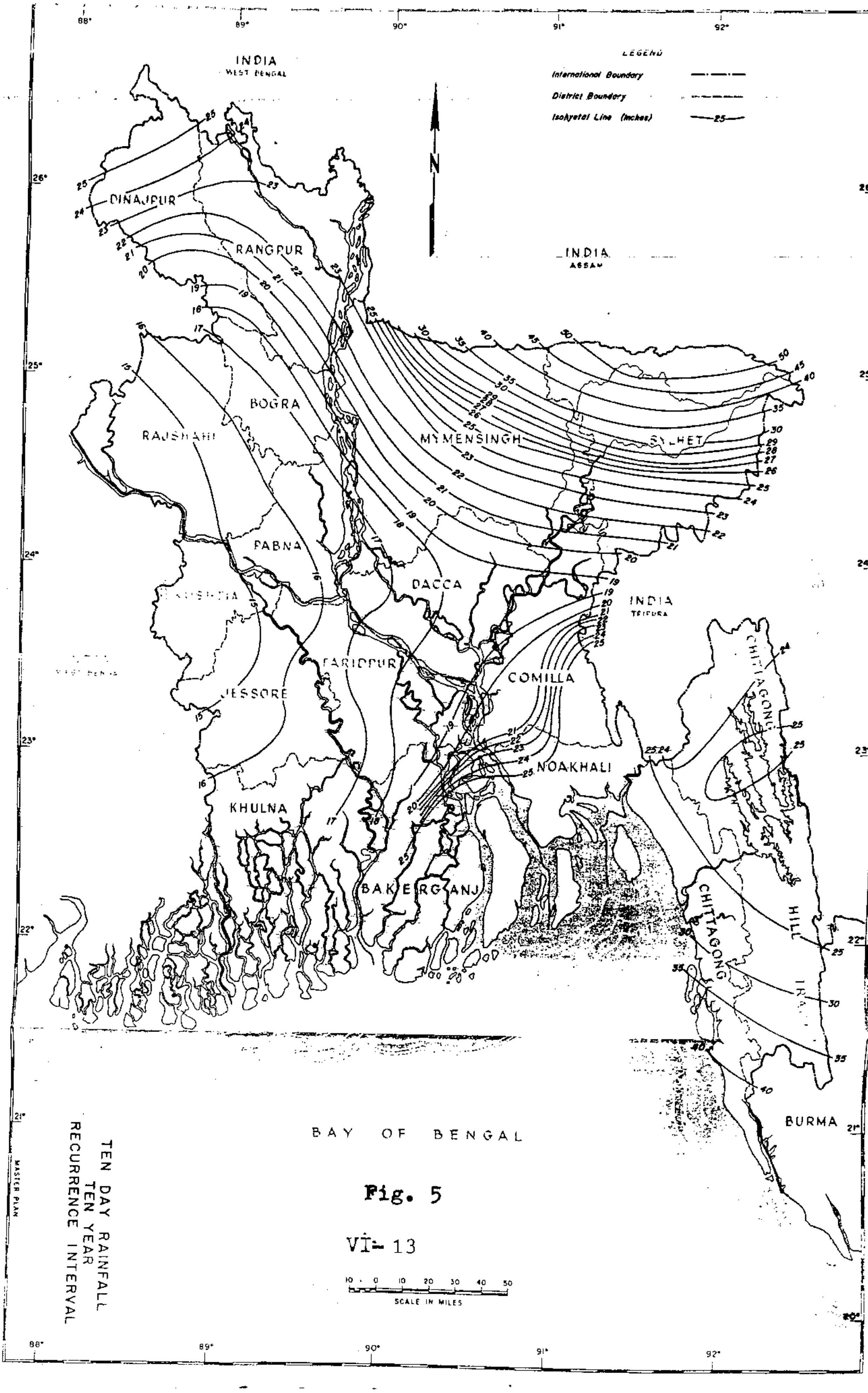
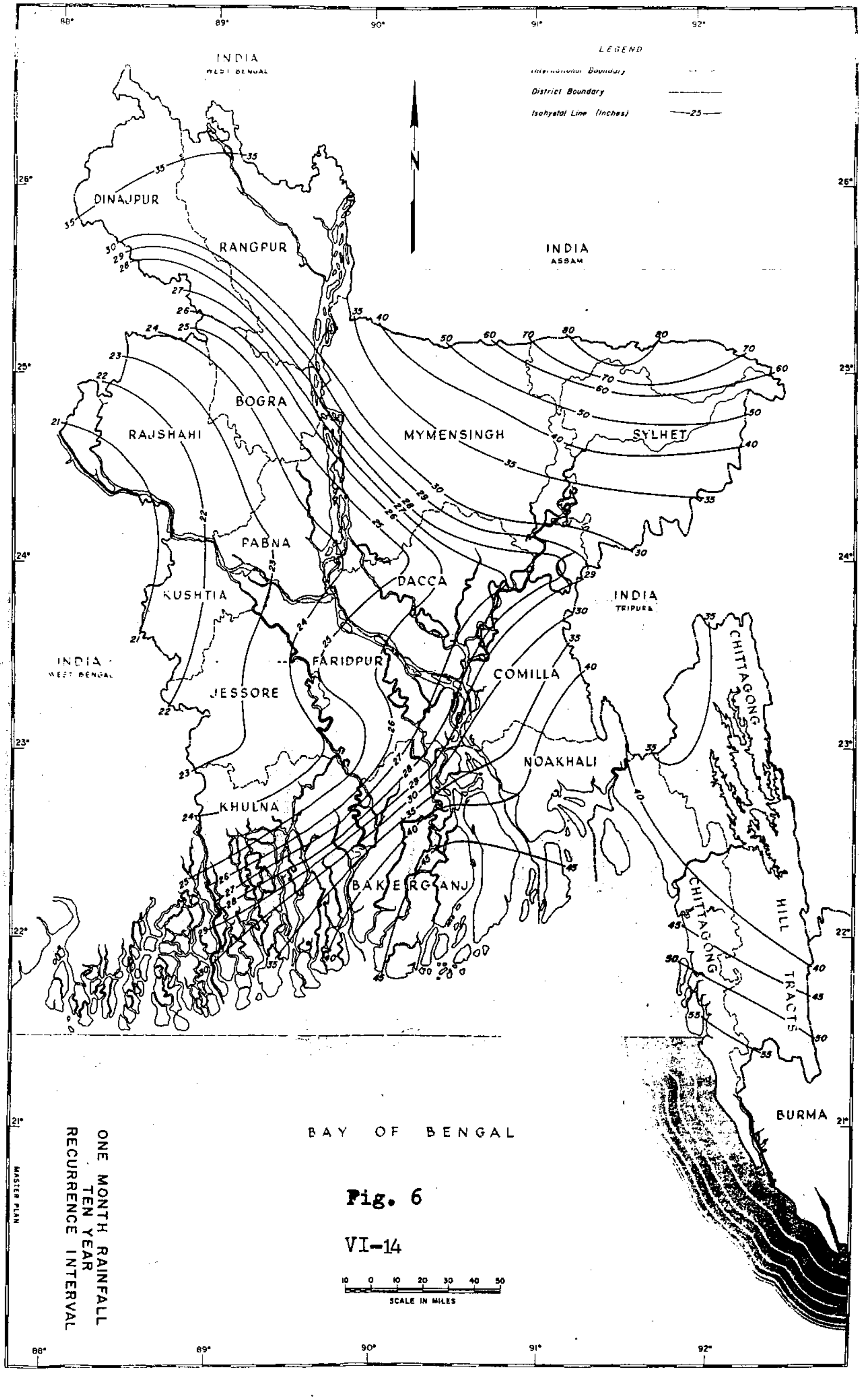

TEN DAY RAINFALL
TEN YEAR
RECURRENCE INTERVAL
LEGEND
International Boundary
District Boundary
Isohyetal Line (inches)
INDIA
WEST BENGAL
INDIA
ASSAM
INDIA
TRIPURA
DINAJPUR
RANGPUR
BOGRA
RAJSHAHI
MYMENSINGH
SYLHET
PABNA
DACCA
FARIDPUR
COMILLA
JESSORE
NOAKHALI
KHULNA
BAKERGANJ
CHITTAGONG
CHITTAGONG
HILL
BURMA
BAY OF BENGAL
SCALE IN MILES
MASTER PLAN

Fig. 5

VI- 13

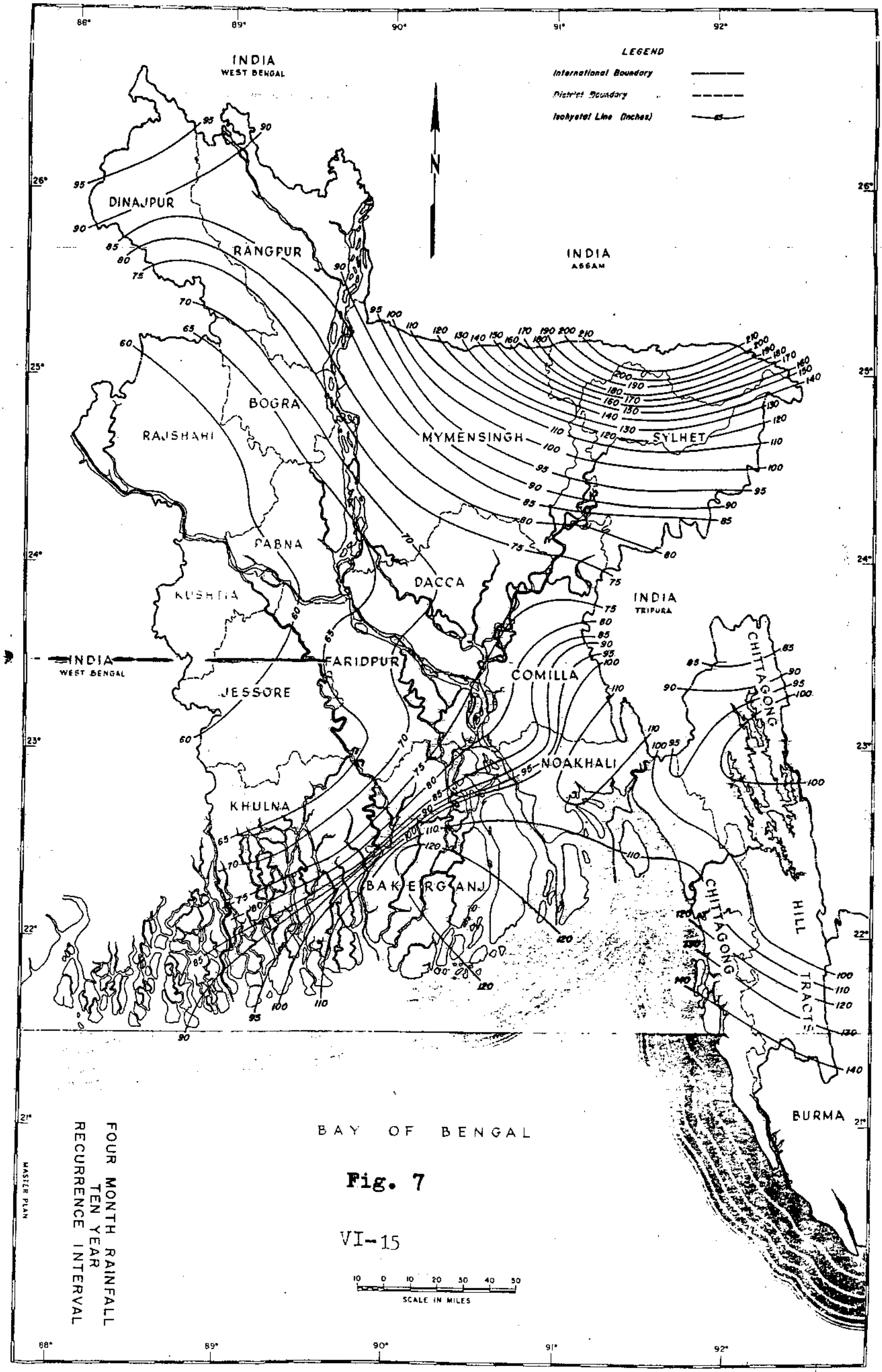


ONE MONTH RAINFALL
TEN YEAR
RECURRENCE INTERVAL

Fig. 6

VI-14

10 0 10 20 30 40 50
SCALE IN MILES



FOUR MONTH RAINFALL
TEN YEAR
RECURRENCE INTERVAL

Fig. 7

RAINFALL VARIABILITY

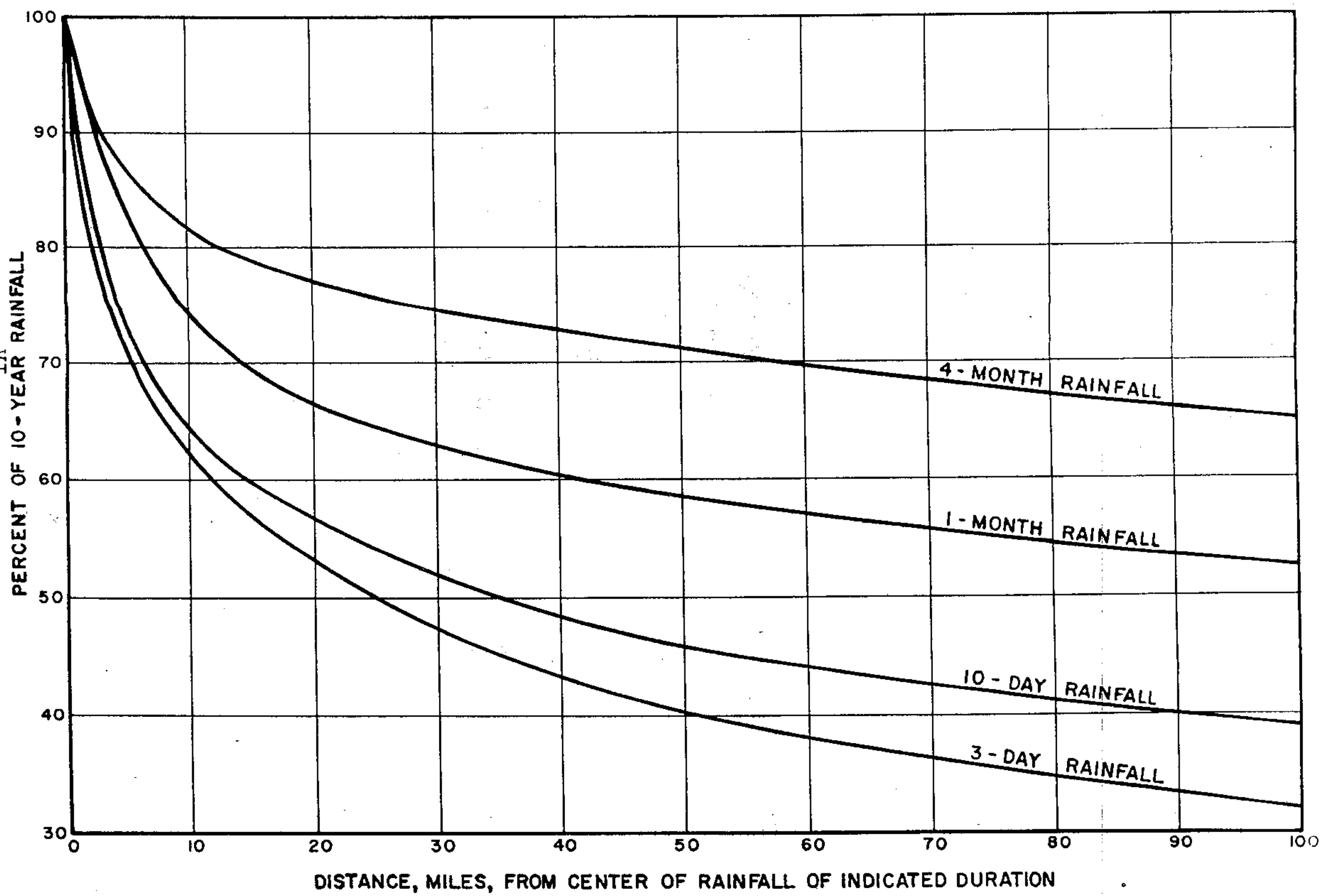


Fig. 8

a) Rainfall Variability

Distance from storm center	5-day Storm Percentage of point rainfall
1/4	100
1	88
2	81.7
3	77.5
4	74.2
5	72.0
6	69.8
7	67.8
8	66.0
9	64.5
10	63.1

Rainfall Variability
Table-5

The equivalent uniform depth of rainfall can be determined by the following way:

Step 1. Locate the geographical center of the basin area. The basin centroid can be found by the following way:

Cut a hardboard identical to the catchment area. Fix three or four points on the hard board. Hang the board with a thread through these fixed points at one end, one by one having a piece of weight on the other end of the thread. Draw straight lines along the thread at each time. The point of intersection of these lines gives the centroid of the basin.

Step 2. Draw concentric circles or isohyetals at even mile intervals around the center. Fig.9 is a map of the basin area with its geographical center located and isohyetals drawn at one mile interval starting one-half mile from the center.

Step 3. Plainmeter the areas between the respective isohyetals within the basin area.

Step 4. Multiply the area by the proper percentage from Table 5 and make a summation of the values.

Average distance from Storm Center	5-day storm percentage of point rainfall	Area [sq.miles]	Area times Percentage
1/4	100	0.79	0.79
1	88	6.21	5.46
2	81.7	12.63	10.32
3	77.5	14.32	11.10
4	74.2	3.05	2.26
		37.00	29.93

Percentage of Point Rainfall
Table - 6

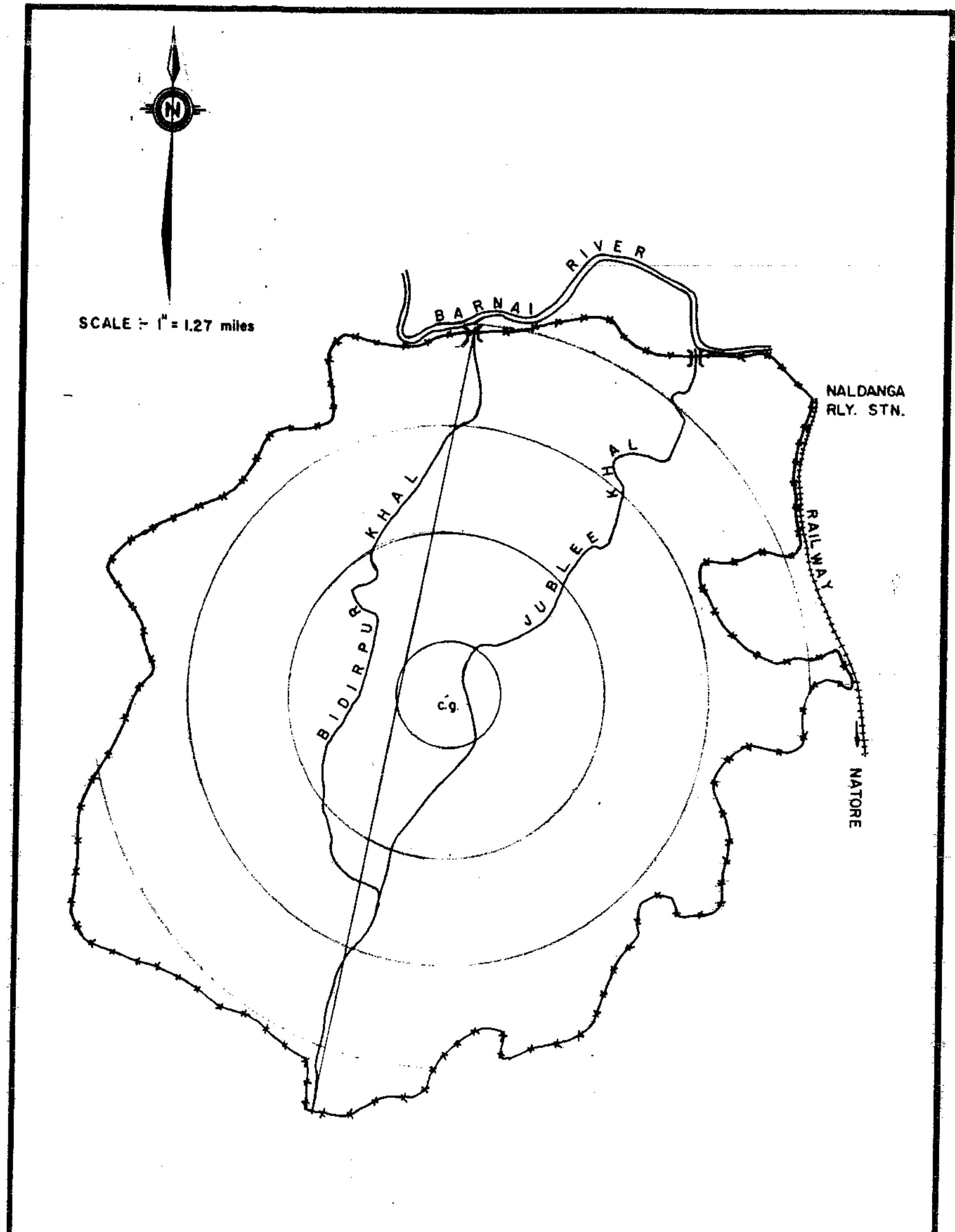

SCALE :- 1" = 1.27 miles
BARNAI RIVER
NALDANGA RLY. STN.
RAILWAY
NATORE
BIDIRPUR KHAL
JUBLEE KHAL
c.g.

Figure - 9

Step 5. Compute the percentage of point rainfall that forms the uniform equivalent depth of rainfall.

$$\frac{29.33}{37.00} \times 100 \approx 81\%$$

b) Convert accumulative point rainfall to daily uniform depth equivalents by multiplying Table-4 values by 81%.

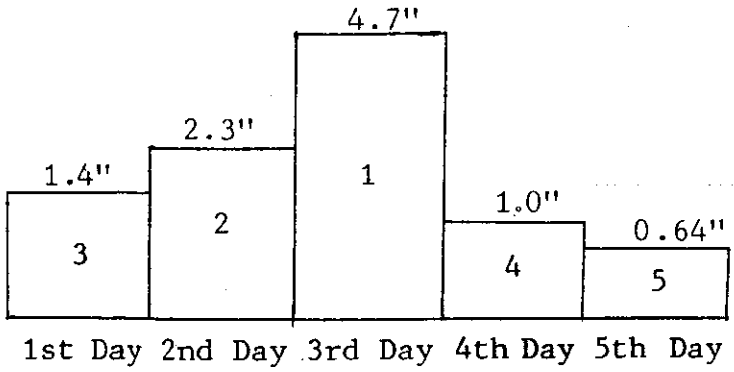
Days	Equivalent Uniform Depth	
	Accumulative total [inch]	Daily Increments [inch]
1	4.7	4.7
2	7.0	2.3
3	8.4	1.4
4	9.4	1.0
5	10.04	0.64

Equivalent Uniform Depth
Table 7

3. <u>Rainfall Sequence of Uniform Equivalent Depth</u>

a) The above supposition says, in effect, a maximum one-day storm forms part of a maximum 2-day storm, which in turn forms part of a maximum 3-day storm etc. And as stated, these daily increments can occur in any order i.e. will arbitarily arrange a sequence of 3,2,1,4,5 in order to satisfy the losses that will take place early in the storm.

A graphical arrangement of this sequence looks like this:



Daily rainfall sequence of Equivalent Uniform Depth

b) Now we want to know the quantity of rain falling during certain hourly intervals. Table 8 has been prepared from Master Plan rainfall data for breaking down rainfall time-distribution into intervals less than 24-hours.

Districts	4-Month Index	Storm Duration in Hours Accumulated rainfall in Inches						
		1	2	3	6	12	18	24
Barisal	65	2.33	2.77	3.11	3.82	5.36	6.85	8.35
Jessore	46	2.37	3.00	3.35	3.80	4.43	5.15	5.90
Cox's Bazar	115	2.33	2.88	3.30	4.40	6.65	10.50	14.10
Chittagong	86	2.43	3.07	3.58	4.56	6.67	9.00	11.00
Dhaka	55	3.10	4.00	5.50	5.25	6.10	6.60	7.06
Bogra	58	3.40	4.23	4.50	5.47	6.30	6.90	7.40
Sylhet	120	2.85	4.15	5.00	6.16	6.20	12.00	15.30

24-hour Point Rainfall Time-Distribution
Table 8

c) The smallest time interval to be used will depend on the length of the main drainage course within the basin. The following intervals are recommended:

1. Less than 2 miles; one hour
2. 2 to 6 miles; 3 hours
3. Over 6 miles; 6 hours

The length of the Bidirpur khal is 8.78 miles (fig.9). So we will use a unit interval of 6-hours. The rainfall data located closest to Rajshahi District is Bogra, with a 24-hour point rainfall of 7.40 inches.

After the adjustment made on the basis of the ratio of the 4-month rainfall indices between Bogra and the project area that is, 45/58 (=0.775). The adjusted maximum accumulative point rainfall of the project area are shown in Table 9.

Stations	Storm Duration Accumulative Rainfall			
	6 hours	12 hours	18 hours	24 hours
Bogra Maximum Accumulative Point Rainfall	5.47" x.775 =	6.30" x0.775 =	6.90" x0.775 =	7.40" x0.775 =
Project Area Maximum Accumulative Point Rainfall	4.24"	4.88"	5.35"	5.74"
Project Area Maximum Incremental Point Rainfall	4.24" x0.81 =	0.64" x0.81 =	0.47" x0.81 =	0.39" x0.81 =
Uniform Equipment Depth	3.43"	0.52"	0.38"	0.32"

24 - hour Rainfall Time Distribution
Table 9

d) As pointed out for rainfall time-distribution with 24-hour increments the increments within a 24-hour period also cannot be predicted. Therefore, we will arbitrarily arrange a sequence of 3,4,1,2.

Thus during the maximum 24-hour rainfall period the 6-hour incremental rainfall is 0.38", 0.32", 3.43" and 0.52". So, the rainfall sequence for the peak 24-hour period is established.

Now, to determine a sequence for the other 24-hour periods we will use two arbitary assumptions to assist in the distribution.

1. In as much as possible, use the same increments and sequence as was used with the maximum 24-hour period.

2. Use the entire time period for distribution.

3) The procedure for calculation of design storm is as follows :

1. Referring to table 10, we have established the sequence for the 3rd day.
2. Moving to the 2nd day, use the same increments for the 1st, 2nd and 4th periods as were used for the 3rd day.
3. For the 3rd period of the 2nd day we will assign the remainder of the total 2nd day rainfall or 1.08 inches.
4. The 1st day is similarly handled with the remainder of 0.18 inches in the 3rd period.
5. The 4th and 5th days are similarly handled and the design storm is complete.

Day	6-hour Increments
1	0.38"
	0.32"
	0.18"
	0.52"
2	0.38"
	0.32"
	1.08"
	0.52"
3	0.38"
	0.32"
	3.43"
	0.52"
4	0.38"
	0.32"
	0.00"
	0.30"
5	0.38"
	0.00"
	0.00"
	0.26"

Design Storm
Table 10

B. Agricultural Land Classification

District	Available for Cultivation	Cultivated as Paddy Land
Dhaka	70	33
Mymensingh	75	40
Faridpur	84	50
Bakerganj	69	57
Chittagong	42	39
Noakhali	66	41
Comilla	79	61
Sylhet	56	35
Rajshahi	74	45
Dinajpur	69	46
Rangpur	66	42
Bogra	84	56
Pabna	81	37
Kushtia	75	26
Jessore	73	35
Khulna	36	25

Percentage of Total Land Area
Table 11

C. Rainfall Excess (Runoff Distribution)

1. Paddy Land Rainfall Excess

We will assume the following paddy land losses for the basin.

1. Initial soil-moisture loss is nil.

   (Paddy land assumed to be saturated)

2. Subsequent soil-moisture loss is assumed to be at the constant infiltration rate of 0.04 inch/hour or 0.25 inch each 6-hour interval.

3. Initial paddy depression storage assumed to be 4 inches.

Days	Hours	Rainfall in Inches	Losses in Inches: Soil Moisture: Initial	Soil Moisture: Subsequent	Depression Storage [inches]	Available Paddy Storage [inches]	Net Runoff [inches]
1	0-6	0.38	0	-0.25	-4.00	-3.87	0
	6-12	0.32		-0.25		-3.80	0
	12-18	0.18		-0.25		-3.87	0
	18-24	0.52		-0.25		-3.60	0
2	0-6	0.38		-0.25		-3.47	0
	6-12	0.32		-0.25		-3.40	0
	12-18	1.08		-0.25		-2.57	0
	18-24	0.52		-0.25		-2.30	0
3	0-6	0.38		-0.25		-2.17	0
	6-12	0.32		-0.25		-2.10	0
	12-18	3.43		-0.25		0	1.08
	18-24	0.52		-0.25		0	0.27
4	0-6	0.38		-0.25		0	0.13
	6-12	0.32		-0.25		0	0.07
	12-18	0.00		-0.25		-0.25	0
	18-24	0.30		-0.25		-0.20	0
5	0-6	0.38		-0.25		-0.07	0
	6-12	0.00		-0.25		-0.32	0
	12-18	0.00		-0.25		-0.57	0
	18-24	0.26		-0.25		-0.56	0

Rainfall Excess
Paddy Land
Table 12

2. Non-paddy Land Rainfall Excess

We will assume the following non-paddy land losses for the basin:

1. Initial soil-moisture loss is 0.50 inch.

2. Subsequent soil-moisture loss is 0.25 inch/6 hour interval.

3. Depression storage at a maximum constant rate of 0.033 inch/hour or 0.20 inch/6-hour interval, provided the rainfall is available, until a total of 1.00 is stored.

Days	Hours	Rainfall in Inches	Losses in Inches			Available Non-Paddy Storage [inches]	Net Runoff [inches]
			Soil Moisture		Depression Storage [inches]		
			Initial	Subsequent			
1	0-6	0.38	-0.50	-	-	-0.12	0
	6-12	0.32		-0.25	0	-0.05	0
	12-18	0.18		-0.25	0	-0.12	0
	18-24	0.52		-0.25	-0.15	0	0
2	0-6	0.38		-0.25	-0.13	0	0
	6-12	0.32		-0.25	-0.07	0	0
	12-18	1.08		-0.25	-0.20	0	0.63
	18-24	0.52		-0.25	-0.20	0	0.07
3	0-6	0.38		-0.25	-0.13	0	0
	6-12	0.32		-0.25	-0.07	0	0
	12-18	3.43		-0.25	-0.05	0	3.13
	18-24	0.52		-0.25	0	0	0.27
4	0-6	0.38		-0.25	0	0	0.13
	6-12	0.32		-0.25	0	0	0.07
	12-18	0.00		-0.25	0	-0.25	0
	18-24	0.30		-0.25	0	-0.20	0
5	0-6	0.38		-0.25	0	-0.07	0
	6-12	0.00		-0.25	0	-0.32	0
	12-18	0.00		-0.25	0	-0.57	0
	18-24	0.26		-0.25	0	-0.56	0

Rainfall Excess
Non-paddy Land
Table 13

3. Rainfall Excess - Weighted Basin Average

The project area is in Rajshahi District. Referring to Table 11 we find that 45% of the total land area is paddy land, leaving 55% as non-paddy land.

Table 14 shows our computation for a weighted average. The net runoff columns are taken from table 12 and table 13 respectively.

The weighted runoff columns are obtained by multiplying paddy land runoff by 45% and non-paddy land runoff by 55%. The basin weighted runoff is the sum of the paddy and non-paddy weighted runoff.

Day	Hours	Paddy Land		Non-Paddy Land		Basin
		Net Runoff	Weighted Runoff	Net Runoff	Weighted Runoff	Weighted Runoff
1	0-6	0	0	0	0	0
	6-12	0	0	0	0	0
	12-18	0	0	0	0	0
	18-24	0	0	0	0	0
2	0-6	0	0	0	0	0
	6-12	0	0	0	0	0
	12-18	0	0	0.63	0.35	0.35
	18-24	0	0	0.07	0.04	0.04
3	0-6	0	0	0	0	0
	6-12	0	0	0	0	0
	12-18	1.08	0.49	3.13	1.72	2.21
	18-24	0.27	0.12	0.27	0.15	0.27
4	0-6	0.13	0.06	0.13	0.07	0.13
	6-12	0.07	0.03	0.07	0.04	0.07
	12-18	0	0	0	0	0
	18-24	0	0	0	0	0
5	0-6	0	0	0	0	0
	6-12	0	0	0	0	0
	12-18	0	0	0	0	0
	18-24	0	0	0	0	0
TOTAL			0.70		2.37	3.07"

Rainfall Excess
Weighted Basin Average
Table 14

Paddy Land = 45%

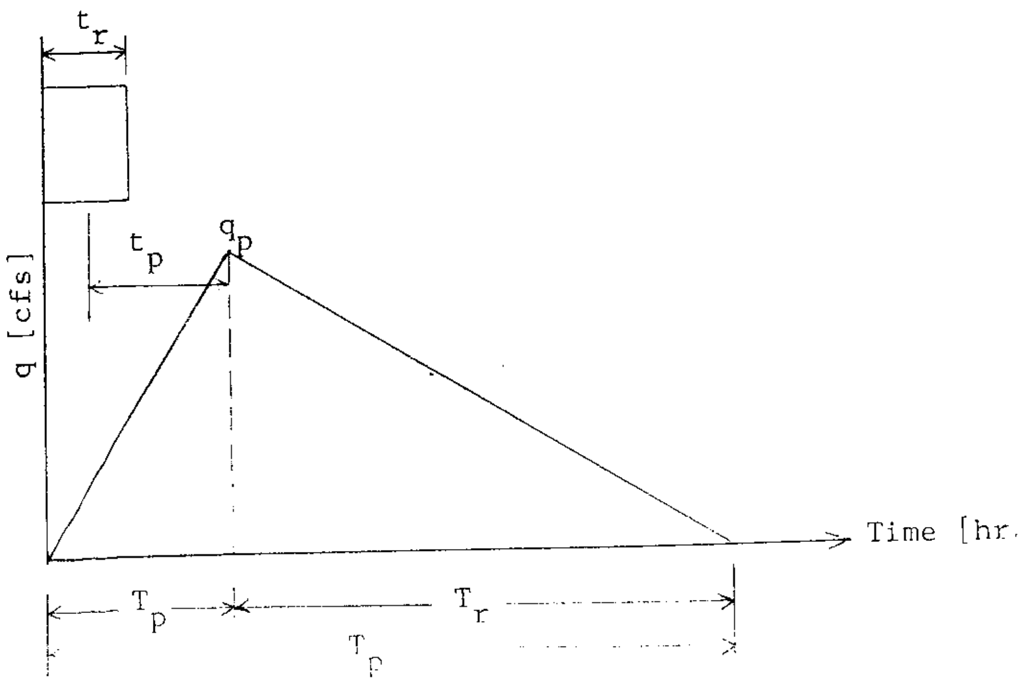
Non-paddy Land = 55%

D. Development of Unit Hydrograph

The hydrograph is a graphical presentation of discharge with respect to time at the drainage basin outlet. The unit hydrograph may be defined as the hydrograph of a unit volume of storm runoff produced by a uniform intensity storm of unit duration. The unit volume corresponds to one inch of rainfall excess spread uniformly over the total basin area.

Synthetic Unit Hydrograph (Snyder's Method)

Unit hydrographs can be derived only if sufficient records are available. But synthetic unit hydrographs can be derived for ungauged basins. This requires a relation between the physical geometry of the basin and resulting hydrographs. For ungauged basins with lack of hydrological data available, triangular synthetic hydrograph can be developed as follows :



q_p = peak rate of runoff in cfs

T_p = time of rise from beginning of rainfall excess to peak rate in hours

T_r = time of recession from peak rate to end of hydrograph in hours

T_b = time base of hydrograph in hours

t_r = duration of rainfall excess in hours

t_p = lag time in hours from center of rainfall excess to peak

T_c = time of concentration in hours, travel time of water from hydraulically most distant point to point of interest.

Steps:

1. To calculate basin area from topographic maps:

A = 37 sq.miles

= 23680 acres

2. Length of the principle drainage channel (Fig.9):

L = 8.78 miles

3. To find out the centroid of the basin (Fig.9):

4. Distance from the outlet to a point on the stream nearest to the basin centroid (Fig.9):

L_c = 4.44 miles

5. Average overland slope of the basin measured between contours:

0.85 ft/mile (from topographic map)

6. Straight length of the basin (Fig.9):

7.49 miles

7. Average channel slope :

$$S = \frac{0.85 \times 7.49}{8.78} = 0.72 \text{ ft/mile}$$

8. Channel roughness factor (assumed) :

$n = 0.055$

9. Time of concentration :

$$T_c = 31 \left(\frac{L^2 n^2}{S}\right)^{0.3} = 31\left(\frac{8.78^2 \times 0.055^2}{0.72}\right)^{0.3} = 22.10 \text{ hours}$$

10. Lag time to peak:

$$t_p = 0.6\ T_c = 0.6 \times 22.1 = 13.26 \quad 14 \text{ hours.}$$

11. Time of rise :

$$T_p = t_p + 1/2\ t_r = 14 + (1/2 \times 6) = 17 \text{ hours}$$

where, t_r = time considered for developing the design storm, in this case, 6 hours.

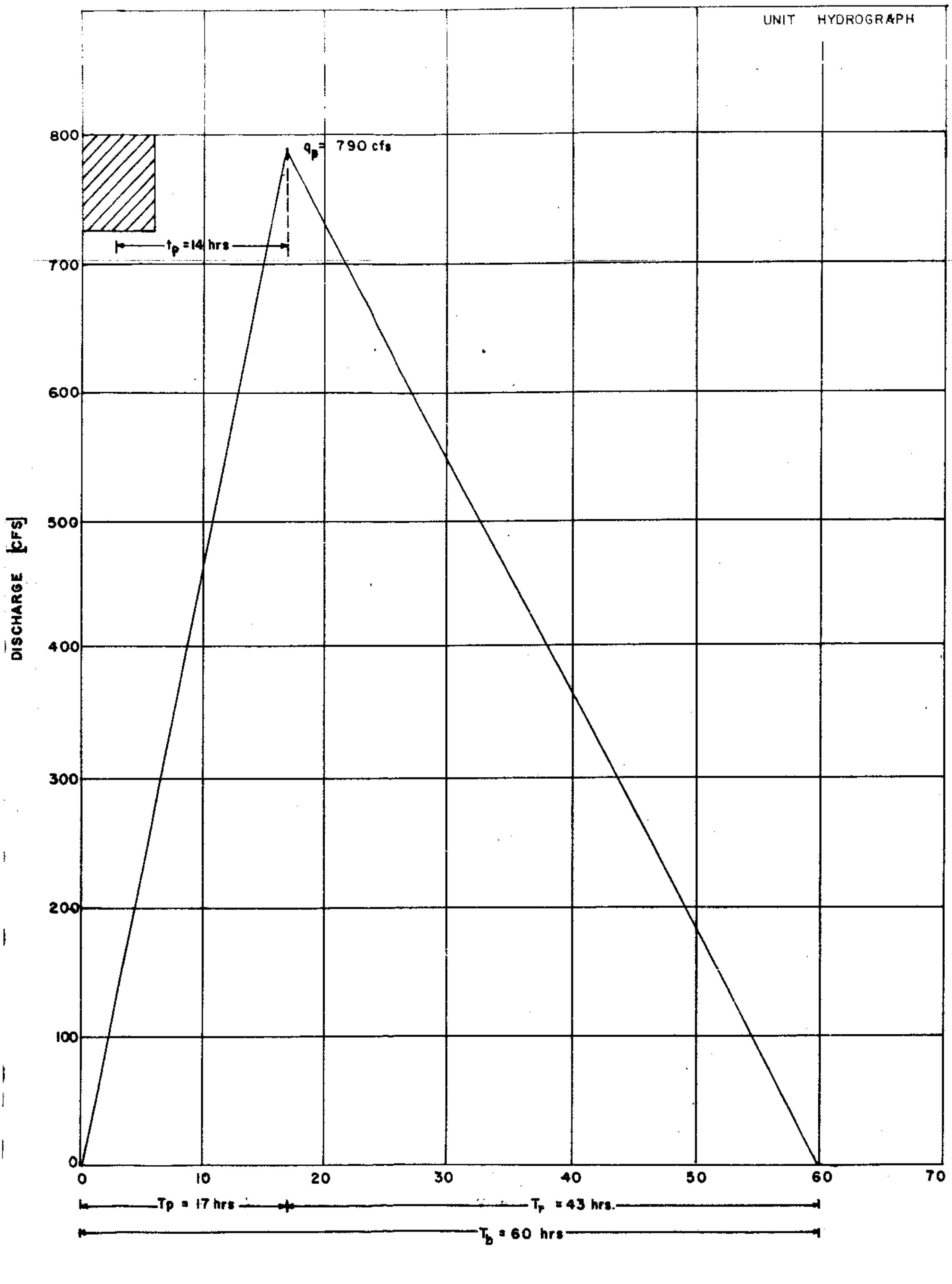

UNIT HYDROGRAPH
q_p = 790 cfs
t_p = 14 hrs
DISCHARGE [CFS]
800
700
600
500
400
300
200
100
0
0
10
20
30
40
50
60
70
Tp = 17 hrs
T_r = 43 hrs.
T_b = 60 hrs

Figure - 11

E. The Runoff Hydrograph

The runoff hydrograph represents the time-dicharge relationship of the total design storm runoff at the basin outlet. The preparation of the storm runoff hydrograph involves treating each 6-hour period as a separate storm, multiplying the unitgraph vertical ordinates by the depth of rainfall excess, relating each hydrograph in time to the other hydrographs and summing up their combined ordinates.

The computations are shown on Table 15. Column 1 are values of the unitgraph vertical ordinates at 6-hour intervals. The intervals are started from the peak rather than from the beginning of the time of rise.

Columns 2 and 3 are 6-hour intervals and days of the storm runoff period respectively.

Columns 4 to 9 are the hydrographs for each 6-hour period of rainfall excess. The values are computed by multiplying the unitgraph values by the amount of rainfall excess at the column head. The zero value of each hydrograph coincides with the first hour of the 6-hour rainfall period.

Column 10 is the summation of values for each 6-hour increment. These values represent the discharge ordinates of the runoff hydrograph and are shown on Fig. 12.

Unitgraph [cfs]	Hours	Day	Rainfall Excess - 6 hour duration						Runoff Hydrograph
			0.35"	0.04"	2.21"	0.27"	0.13"	0.07"	3.07"
1	2	3	4	5	6	7	8	9	10
0	18	1	0						0
225	24		79	0					79
505	6		177	9	0				186
790	12	2	277	20	497	0			794
675	18		236	32	1116	61	0		1445
570	24		200	27	1746	136	29	0	2138
460	6		161	23	1492	213	66	16	1971
350	12	3	123	18	1260	182	103	35	1721
240	18		84	14	1017	154	88	55	1412
130	24		46	10	774	124	74	47	1075
20	6		7	5	530	95	60	40	737
0	12	4	0	1	287	65	46	32	431
	18			0	44	35	31	25	135
	24				0	5	17	17	39
	6	5				0	3	9	12
	12						0	2	2
	18							0	0
	24								
									1217

Table 15

A check on the computations of Runoff Hydrograph in Table 15 can be made using the following formulae:

1. Total volume of water under the hydrograph:

$$V = \frac{It}{12}, \qquad I = \text{from column 10}, \quad t = \text{6-hour time interval}$$

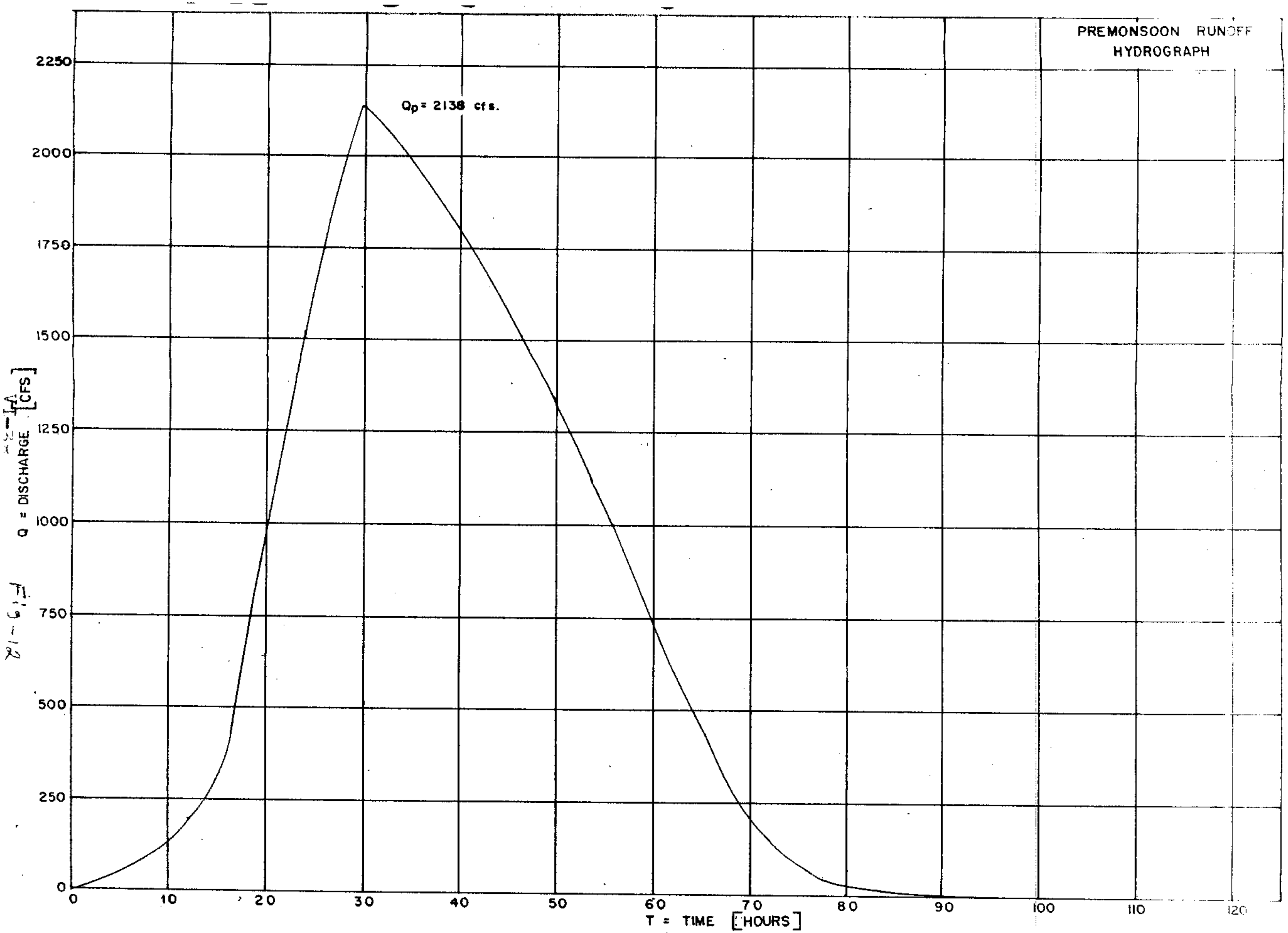
$$V = \frac{12177 \times 6}{12} = 6089 \text{ acre ft.}$$

2. Total rainfall excess, i

$$i = \frac{12V}{A}, \quad A = \text{basin area in acres}$$

$$= \frac{12 \times 6089}{23680} = 3.086"$$

This should equal the summation of weighted rainfall excess in Table 14 or 3.07".



PREMONSOON RUNOFF
HYDROGRAPH
Q_p = 2138 cfs.
2250
2000
1750
1500
1250
1000
750
500
250
0
Q = DISCHARGE [CFS]
0
10
20
30
40
50
60
70
80
90
100
110
120
T = TIME [HOURS]

Fig-12

F.1 Calculation for Elevation Area Capacity

Ground El. [ft.PWD]	Area [acres]	Σ Area [acres]	Volume [acre ft]	Σ Volume [acre ft]
1	2	3	4	5
25	0	0	0	0
			8.51	8.51
26	17.02	17.02		
			68.10	76.61
28	34.05	51.07		
			153.22	229.83
30	51.07	102.14		
			272.38	502.21
32	68.10	170.24		
			425.62	927.83
34	85.13	255.37		
			595.88	1523.71
36	85.13	340.5		
			1011.10	2534.81
38	330.10	670.6		
			3098.30	5633.11
40	1757.10	2427.7		
			8211.90	13845.01
42	3356.50	5784.2		
			16387.60	30232.61
44	4819.20	10603.4		
			26882.20	57114.81
46	5675.40	16278.8		
			37635.60	94750.41
48	5078.00	21356.8		
			44399.90	139150.31
50	1686.30	23043.1		
			46723.10	185873.41
Above 50	636.90	23680 =37 sq miles		

Elevation Area Capacity Table
Table 16

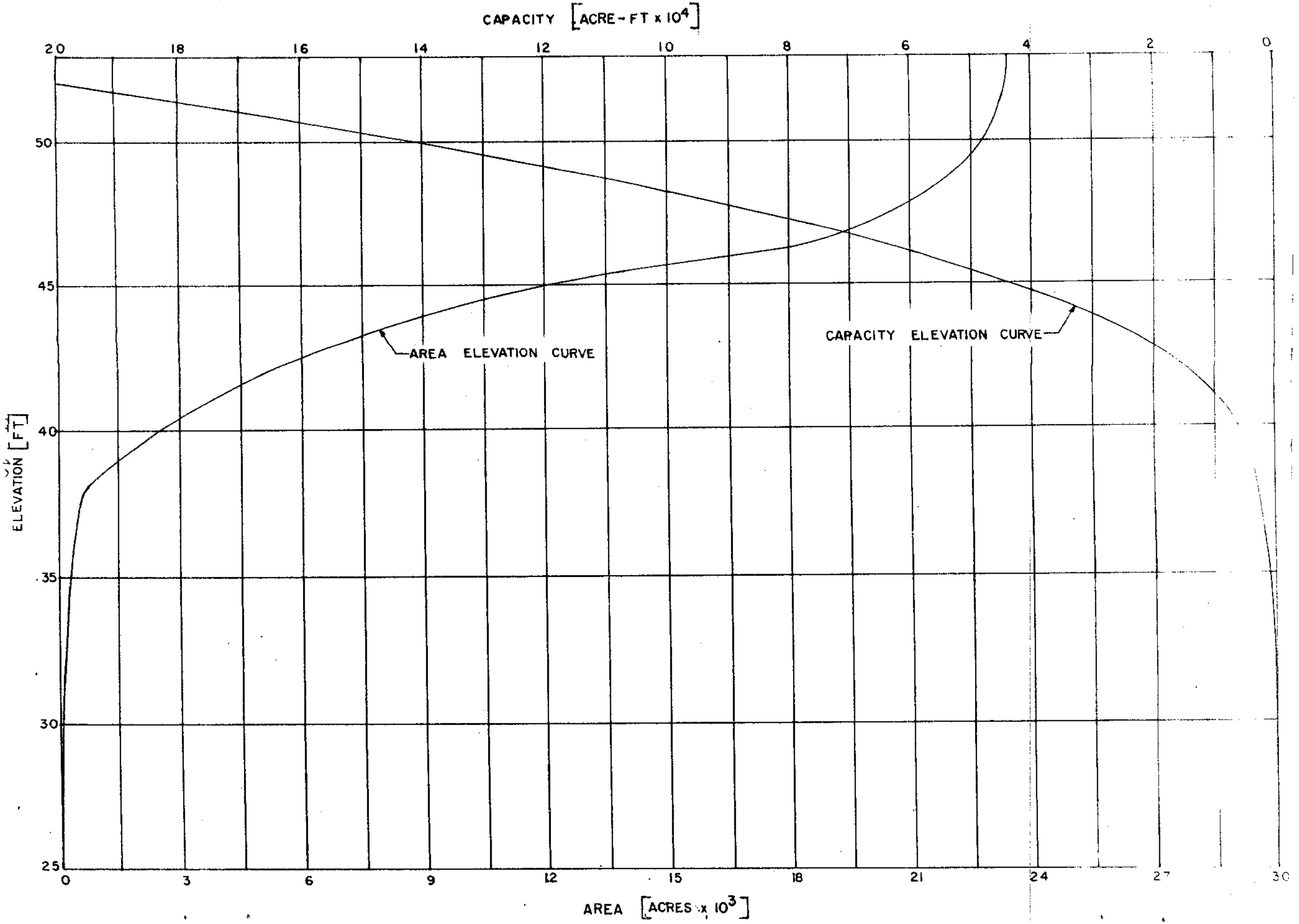

CAPACITY [ACRE-FT x 10^4]
20 18 16 14 12 10 8 6 4 2 0
ELEVATION [FT]
50 45 40 35 30 25
AREA ELEVATION CURVE
CAPACITY ELEVATION CURVE
0 3 6 9 12 15 18 21 24 27 30
AREA [ACRES x 10^3]

Figure - 13

2. Stage-Discharge Curve

During a typical drainage cycle the storage level will at first be low and discharge will begin under flow condition 5. In this condition, it is assumed that there will be a properly designed stilling basin as part of the structure and a critical depth will control at the outlet.

As the storage level rises the entrance to the sluice may become submerged and the dischare will under flow condition 3. After the flood crest passes, the storage level will drop, the entrance will no longer be submerged and discharge will again take place under flow condition 5.

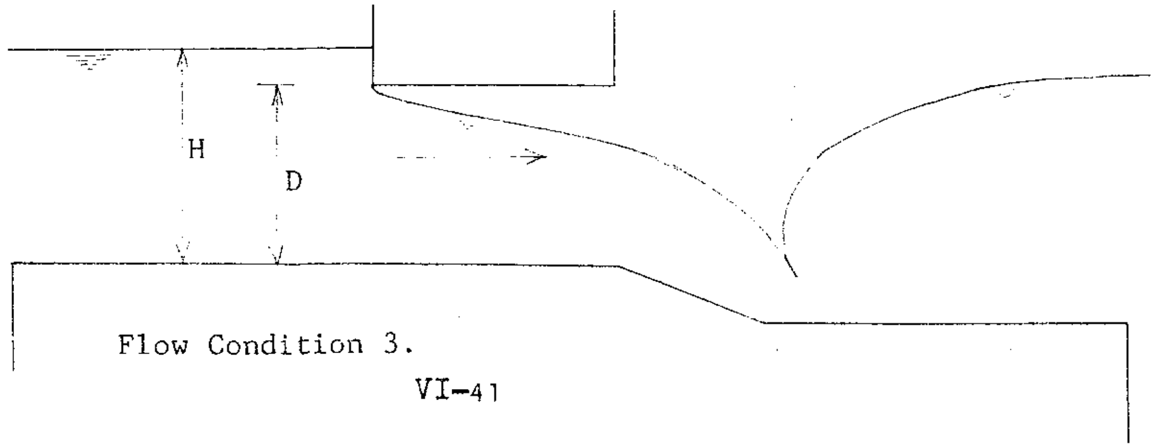
Flow Condition 3

In order for the entrance to be submerged H must be greater than the critical value of 1.2 to 1.5 D. However, in this range the flow is unsteady, so for design purposes 1.5D will be used as limit.

For $H > 1.5\ D$, $q = C_q\ D\sqrt{2gH}$ [Ref: Fluid Mechanics for Hydrualic Engineers-Rouse]

where, q =unit discharge, cfs/unit width

C_q = co-efficient of discharge (taken from Fig.14)



Flow Condition 3.

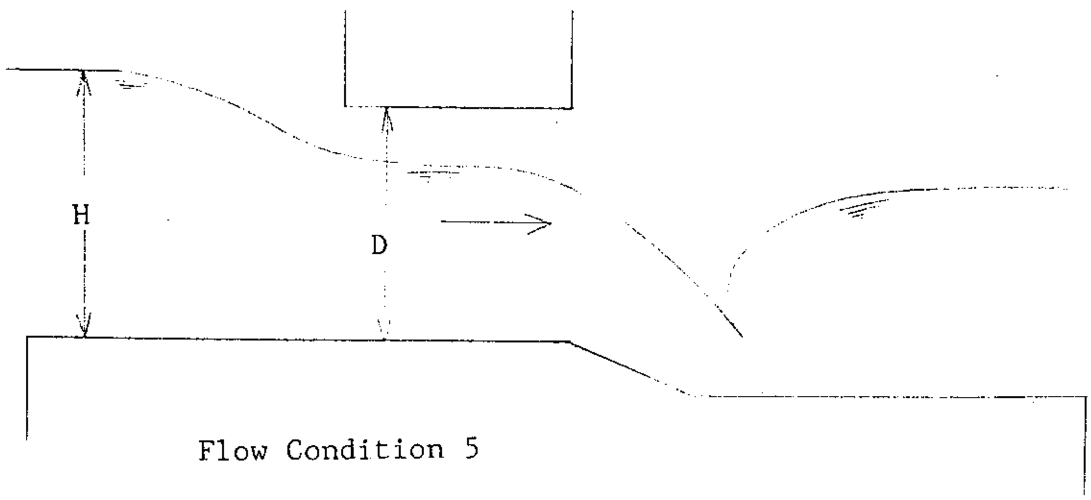
Flow condition 5

This condition will usually occur when a stilling basin with glacis is attached to the sluice, or, when the flow downstream of the vents is able to spread laterally sufficiently to pass through critical depth. In this condition

For $H < 1.5 D$

$q = C H^{3/2}$ [Ref: Hydrologic Design Procedure for Drainage Structures - BWDB]

where C = co-efficient of discharge (Ref. Table 17)



Flow Condition 5

Co-efficient of Discharge
Sluice flowing partially full

Type of entrance	C
Cylinder quadrant	2.86
Simplified straight line	2.80
Straight line transition	2.68
Square ended transition	2.45
Well rounded entrance	2.68
Square entrance	2.45

Co-efficient of Discharge
Table 17

At the beginning, the discharge will follow the flow condition 5. [H ≤ 1.5 D = 9 ft.]. After that the discharge will be under flow condition 3 [H ≥ 1.5 D = 9 ft]. Computation of discharge with different flow conditions for different no. of vents are shown in Table 18.

<u>Discharge with different elevations and ventages</u>

Condition 5

El.	H[ft]	$q=2.68\ H^{3/2}$	Discharge [cfs] Ventages 1	2	3	4	5	6
25	0	0	0	0	0	0	0	0
26	1	2.68	13.4	27	40	54	67	80
28	3	14	70	140	210	280	350	420
30	5	30	150	300	450	600	750	900
32	7	50	250	500	750	1000	1250	1500
34	9	72.4	362	724	1086	1448	1810	2172

Condition 3

El.	H	D/H	Cq (from graph)	$q = C_q D\sqrt{2gH}$ [cfs/unit width]	Discharge [cfs] Ventages 1	2	3	4	5	6
35	10	0.60	0.495	75.37	377	754	1131	1507	1884	2261
36	11	0.54	0.497	79.37	397	794	1191	1587	1984	2381
38	13	0.46	0.506	87.84	439	878	1318	1757	2195	2635
40	15	0.40	0.513	95.67	478	957	1435	1913	2392	2870
42	17	0.35	0.520	103.23	516	1032	1549	2065	2581	3097
44	19	0.31	0.525	110.19	551	1102	1653	2204	2755	330[illegible]
46	21	0.28	0.530	116.94	585	1169	1754	2339	2924	3508

Stage-Discharge Table
Table - 18

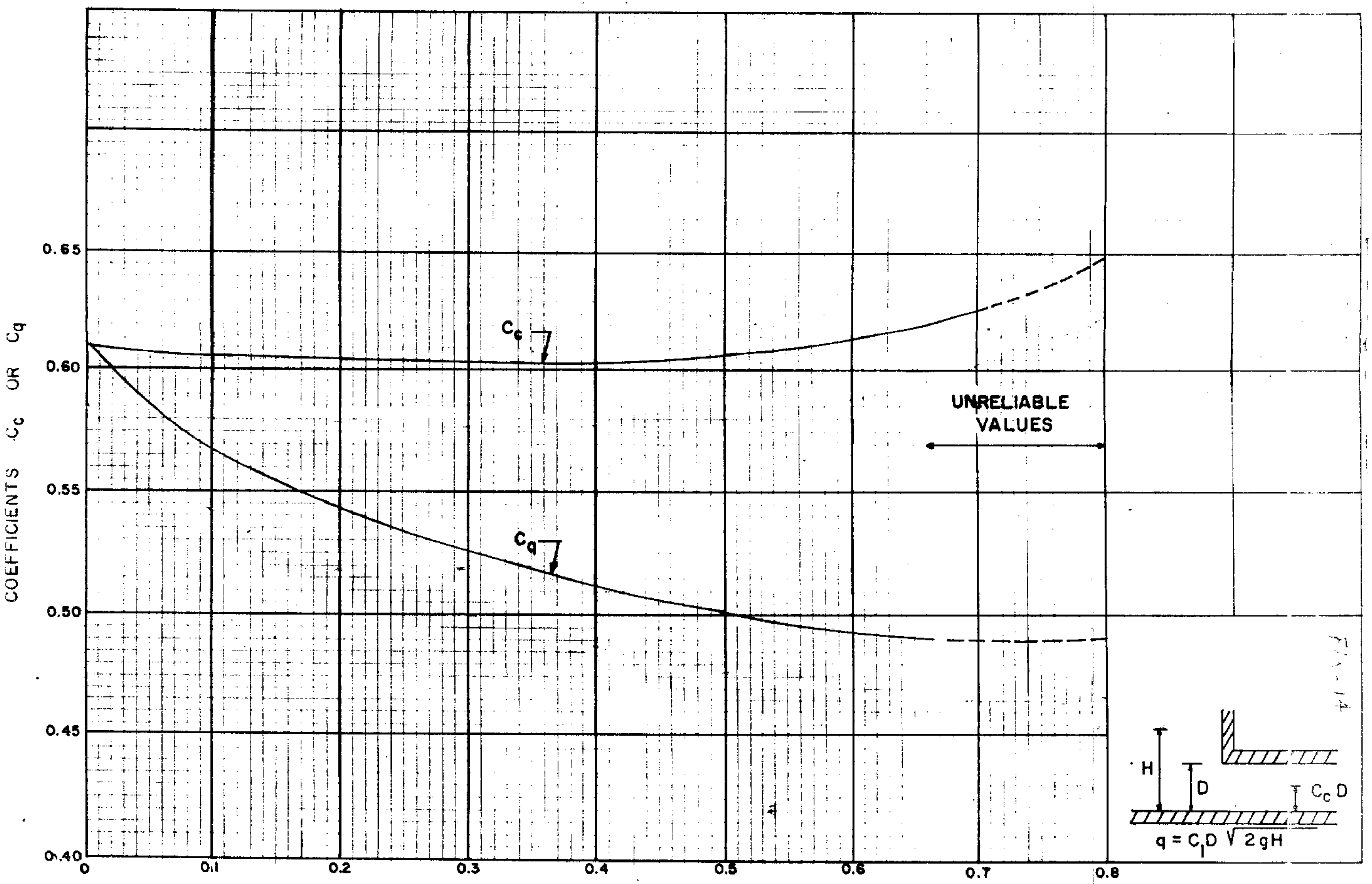
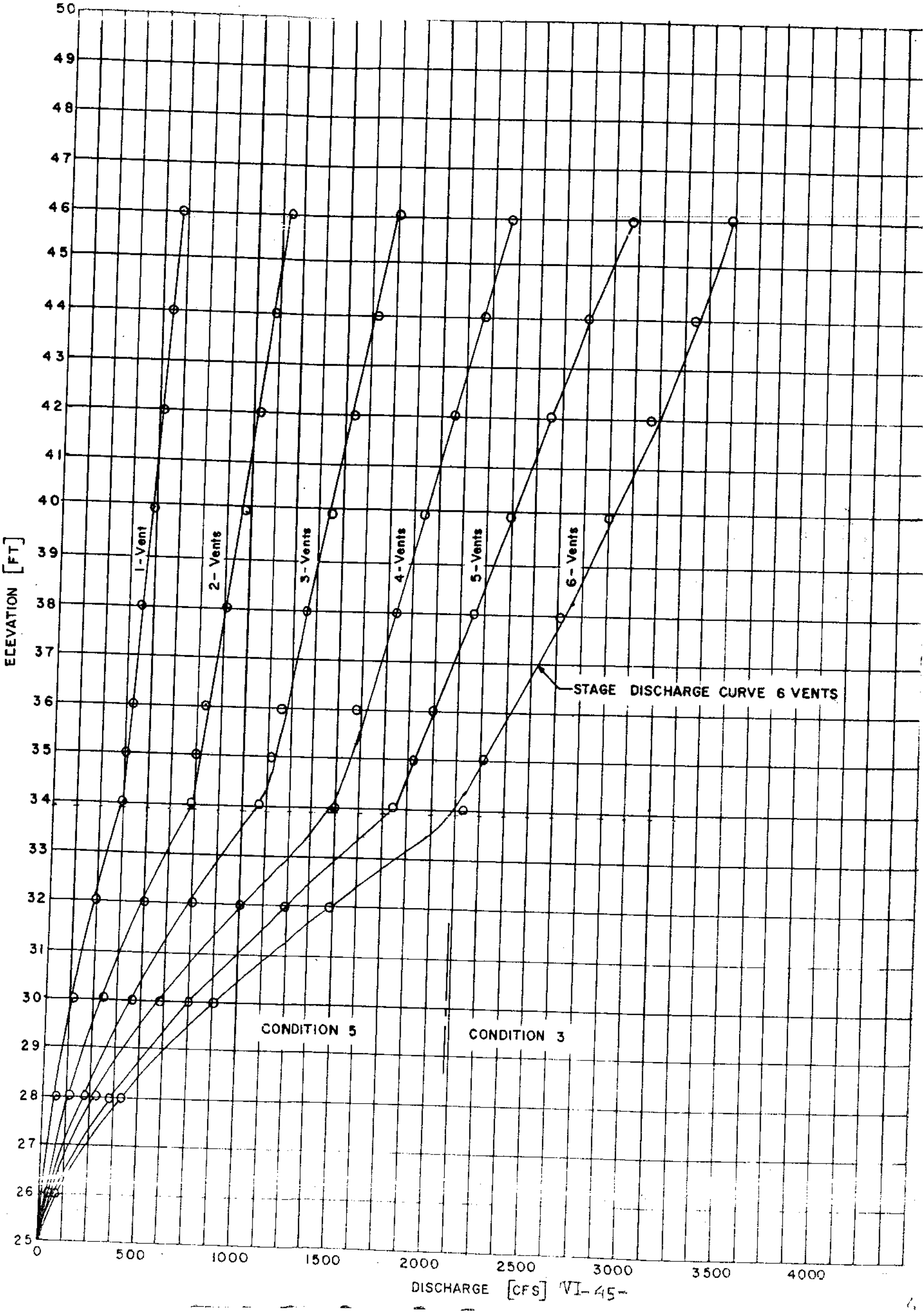

COEFFICIENTS C_c OR C_q
0.65
0.60
0.55
0.50
0.45
0.40
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
C_c
C_q
UNRELIABLE VALUES
H
D
$C_c D$
$q = C_1 D \sqrt{2gH}$
Fig-14

Figure - 15


ELEVATION [FT]
DISCHARGE [CFS]
1-Vent
2- Vents
3- Vents
4- Vents
5- Vents
6- Vents
STAGE DISCHARGE CURVE 6 VENTS
CONDITION 5
CONDITION 3
50
49
48
47
46
45
44
43
42
41
40
39
38
37
36
35
34
33
32
31
30
29
28
27
26
25
0
500
1000
1500
2000
2500
3000
3500
4000

3. 2s/t ± D Curve

The computations for this curve are shown in Table 19 and Table 20. Column 2 is the storage capacity in acre-feet taken from Fig.13. Column 3 is 2s/t in cfs where S is the storage in cubic feet and t is a selected time interval of 21,600 seconds (6-hours). Column 4 of Table 19 is taken from the stage-discharge curve, Figure 15. Column 5 is the summation of Column 3 and Column 4. Column 4 vs Column 5 is plotted in Figure 15 to get 2S/t +D curve for 1 vent. Similarly 2S/t +D curve for two and three vents are drawn in Figure 16 and that of four, five and six vents are drawn in Fig. 17.

2S/t ±D Vs. D Curve for 1 vent, 2 vents & 3 vents

El.	Storage Volume(S) [acre-ft]	2S/t [cfs]	1 vent		2 vents		3 vents	
			D	2S/t +D	D	2S/t +D	D	2S/t +D
1	2	3	4	5	6	7	8	9
25	0	0	0	0	0	0	0	0
26	9	36	13.4	49.4	27	63	40	76
27	30	121	25	146	75	196	125	246
28	77	311	70	381	140	451	210	521
29	125	504	87	591	200	704	325	829
30	230	928	150	1078	300	1228	450	1378
31	380	1533	175	1708	387	1920	587	2120
32	502	2025	250	2275	500	2525	750	2775
33	720	2904	287	3191	600	3504	900	3804
34	928	3743	362	4105	724	4467	1086	4829
35	1000	4033	378	4411	757	4790	1135	5168
36	1524	6147	398	6545	797	6944	1195	7342
37	2000	8067	425	8492	837	8904	1265	9332
38	2535	10225	440	10665	880	11105	1320	11545
39	4000	16133	450	16583	906	17039	1375	17508
40	5633	22720	480	23200	960	23680	1440	24160
41	9300	37510	500	38010	975	38485	1475	38985
42	13845	55842	515	56357	1030	56872	1545	57387
43	21000	84700	525	85225	1050	85750	1575	86275
44	30233	121940	550	122490	1100	123040	1650	123590
45	43000	173433	562	173995	1125	174558	1675	175108
46	57115	230364	583	230947	1167	231531	1751	232115
47	74700	301290						
48	94750	382158						
49	116000	467867						
50	139150	561238						

2 s/t + D Vs D Table
Table-19

2S/t +D Vs. D Curve for 4 vents, 5 vents & 6 vents

El.	Storage Volume(S) [acre-ft]	2S/t [cfs]	4 vents D	4 vents 2S/t +D	5 vents D	5 vents 2S/t +D	6 vents D	6 vents 2S/t +D
1	2	3	4	5	6	7	8	9
25	0	0	0	0	0	0	0	0
26	9	36	54	90	67	103	80	116
27	30	121	150	271	200	321	238	359
28	77	311	275	586	350	661	418	729
29	125	504	400	904	544	1048	638	1142
30	230	928	587	1515	750	1678	899	1827
31	380	1533	800	2333	1000	2533	1175	2708
32	502	2025	1000	3025	1250	3275	1489	3514
33	720	2904	1250	4154	1525	4429	1813	4717
34	928	3743	1450	5193	1800	5543	2171	5914
35	1000	4033	1575	5608	1892	5925	2268	6301
36	1524	6147	1650	7797	1992	8139	2388	8535
37	2000	8067	1725	9792	2100	10167	2525	10592
38	2535	10225	1760	11985	2200	12425	2638	12863
39	4000	16133	1850	17983	2281	18414	2781	18914
40	5633	22720	1921	24641	2401	25121	2879	25599
41	9300	37510	2000	39510	2475	39985	3013	40523
42	13845	55842	2061	57903	2576	58418	3088	58930
43	21000	84700	2125	86825	2650	87350	3238	87938
44	30233	121940	2200	124140	2750	124690	3310	125250
45	43000	173433	2250	175683	2875	176308	3413	176846
46	57115	230364	2334	232698	2918	233282	3499	233863
47	74700	301290						
48	94750	382158						
49	116000	467867						
50	139150	561238						

Table 20

4. Flood Routing Procedure

The computations are shown in Table 21. The procedure is as follows:

1. The first values in Column 6 and 8 are known or assume.
2. The first value in Column 5 is the value of 2s/t+D taken from Figure 16 &17 corresponding to the known value of D, in this case zero.
3. In subsequent lines Column 5 is Column 4 plus Column 7 from the line above.
4. Column 7 is Column 5 minus twice Column 6 in each case throughout the computations.
5. After the first line, values of D in Column 6 are taken from Figure 16 & 17 corresponding to the value of 2s/t +D on the same line.
6. Column 8 is the basin water surface elevation taken from Figure 15 for the corresponding value of D in Column 6.

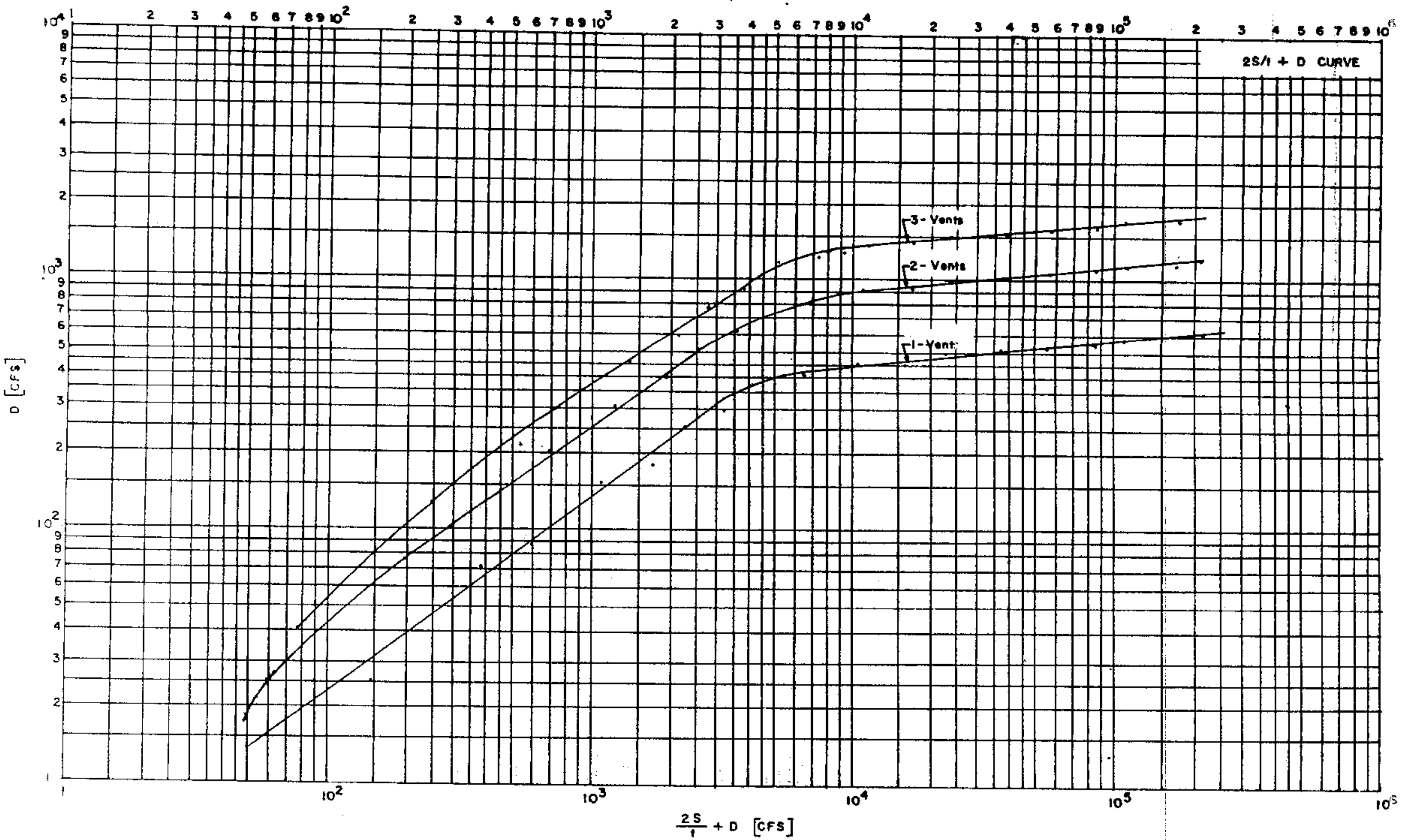


Figure - 16

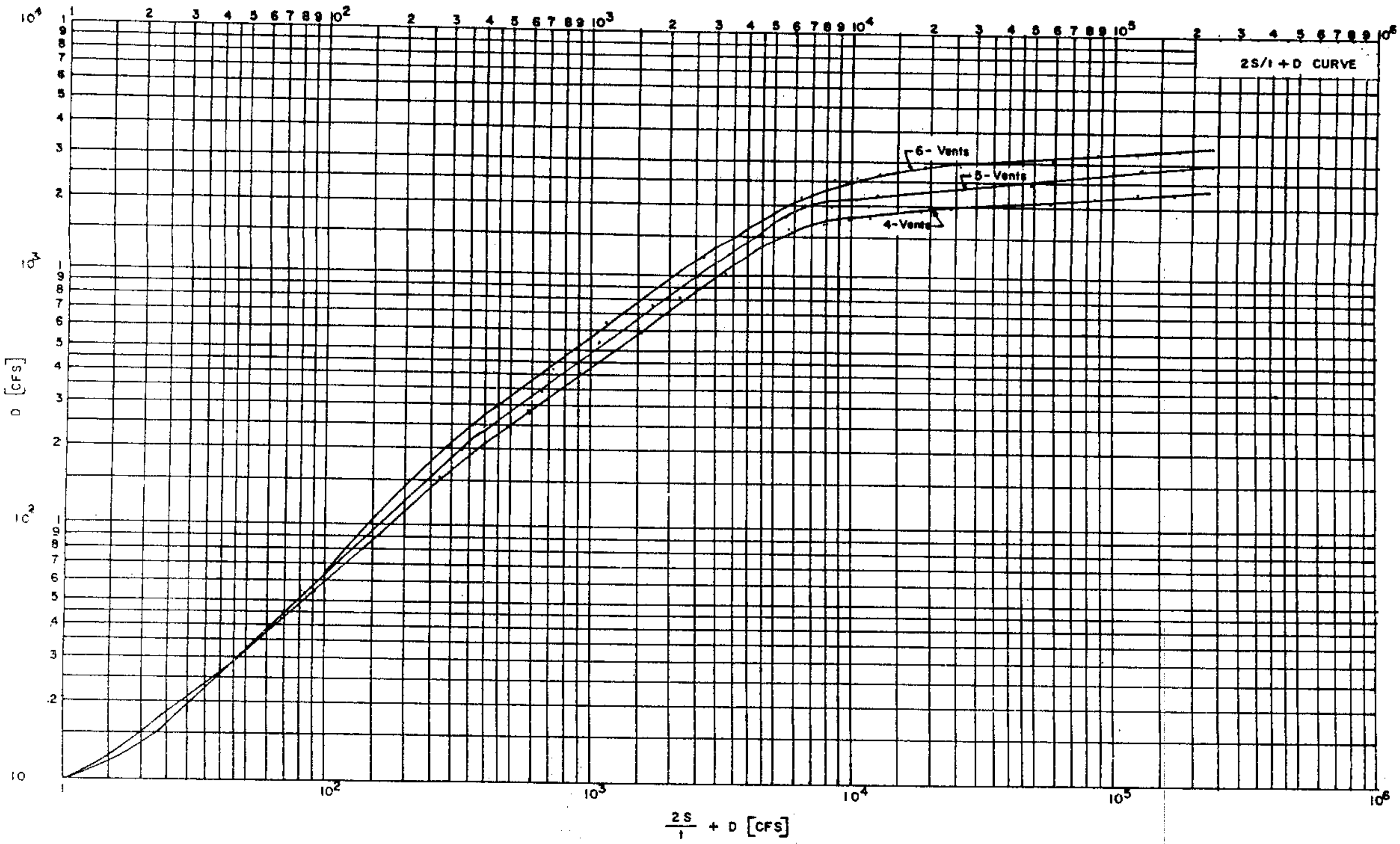


Figure - 17

Flood Routing for 1-vent

Day	Hour	I [cfs]	I 1 + 2	2 s/t + D (4)+(7)	D	2 s/t - D (5)-2x(6)	Basin W.S.El.
1	2	3	4	5	6	7	8
1	18	0	0	0	0	0	25.0
	24	79	79	79	19	41	26.0
	6	186	265	306	52	202	28.0
2	12	794	980	1182	150	882	30.5
	18	1445	2239	3121	315	2491	33.4
	24	2138	3583	6074	400	5274	36.0
	6	1971	4109	9383	430	8523	37.9
3	12	1721	3692	12215	435	11345	38.0
	18	1412	3133	14478	450	13578	38.6
	24	1075	2487	16065	455	15155	39.1
	6	737	1812	16967	460	16047	39.5
4	12	431	1168	17215	462	16291	39.55
	18	135	566	16857	459	15939	39.3
	24	39	174	16113	452	15209	38.7
	6	12	51	15260	450	14360	38.6
5	12	2	14	14374	448	13478	38.5
	18	0	2	13480	445	12590	38.2
	24	0	0	12590	437	11716	38.0
	6	0	0	11716	435	10846	37.9
6	12	0	0	10846	432	9982	37.8
	18	0	0	9982	430	9122	37.8
	24	0	0	9122	428	8266	37.8
	6	0	0	8266	418	7430	37.5
	12	0	0	7430	415	6600	37.0
7	18	0	0	6600	410	5780	36.5
	24	0	0	5780	400	4980	36.0
	6	0	0	4980	385	4210	35.2
8	12	0	0	4210	365	3480	34.3
	18	0	0	3480	335	2810	33.7

Pre-monsoon Flood Routing Table (1 vent 5'x6')
Table 21

To determine the crop damage, a basin water surface elevation vs. time curve is plotted in Figure 17 to 25. This completes the flood routing procedure.

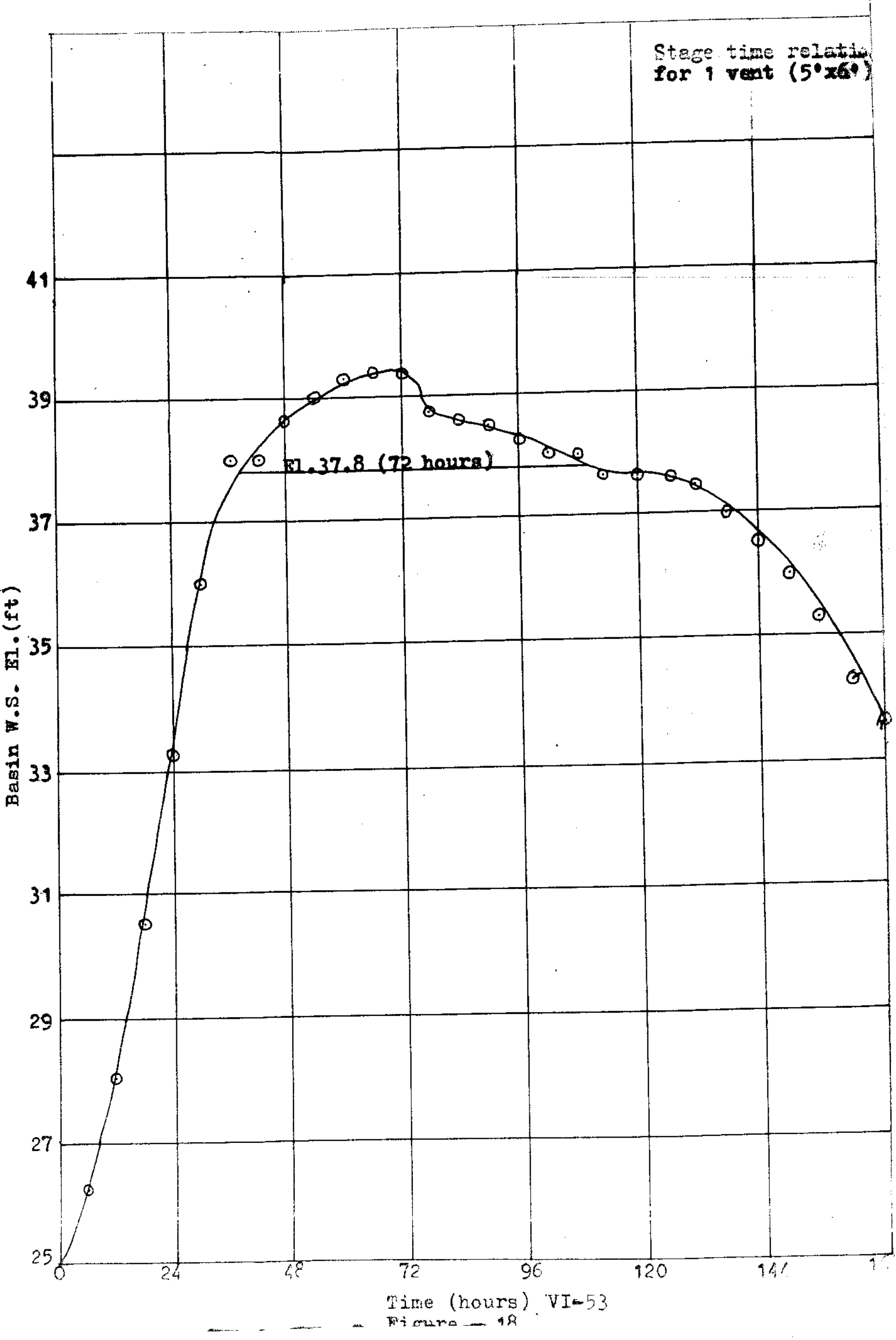

Stage time relati
for 1 vent (5'x6')
El.37.8 (72 hours)
Basin W.S. El.(ft)
41
39
37
35
33
31
29
27
25
0
24
48
72
96
120
144
Time (hours) VI-53

Figure — 18

Flood Routing for 2-vents

Day	Hour	I [cfs]	$I_1 + I_2$	2s/t +D (4) +(7)	D	2s/t -D (5)- 2x(6)	Basin W.S.El.
1	2	3	4	5	6	7	8
1	18	0	0	0	0	0	25.0
	24	79	79	79	34	11	25.7
2	6	186	265	276	96	84	27.4
	12	794	980	1064	260	544	29.7
	18	1445	2239	2783	520	1743	32.3
	24	2138	3583	5326	720	3886	34.0
3	6	1971	4109	7995	830	6335	35.9
	12	1721	3692	10027	870	8287	37.9
	18	1412	3133	11420	880	9660	38.2
	24	1075	2487	11472	880	9712	38.2
4	6	737	1812	11524	880	9764	38.2
	12	431	1168	10932	875	9182	38.0
	18	135	566	9748	860	8028	37.7
	24	39	174	8202	830	6542	36.8
5	6	12	51	6593	780	5023	35.4
	12	2	14	5037	710	3617	33.8
	18	0	2	3619	610	2399	32.9
	24	0	0	2399	470	1459	31.7
6	6	0	0	1459	330	799	30.5
	12	0	0	799	210	379	29.0
	18	0	0	379	130	119	28.0
	24	0	0	119	73	-27	27.0

Pre-monsoon Flood Routing Table (2 vents 5'x6')
Table 22

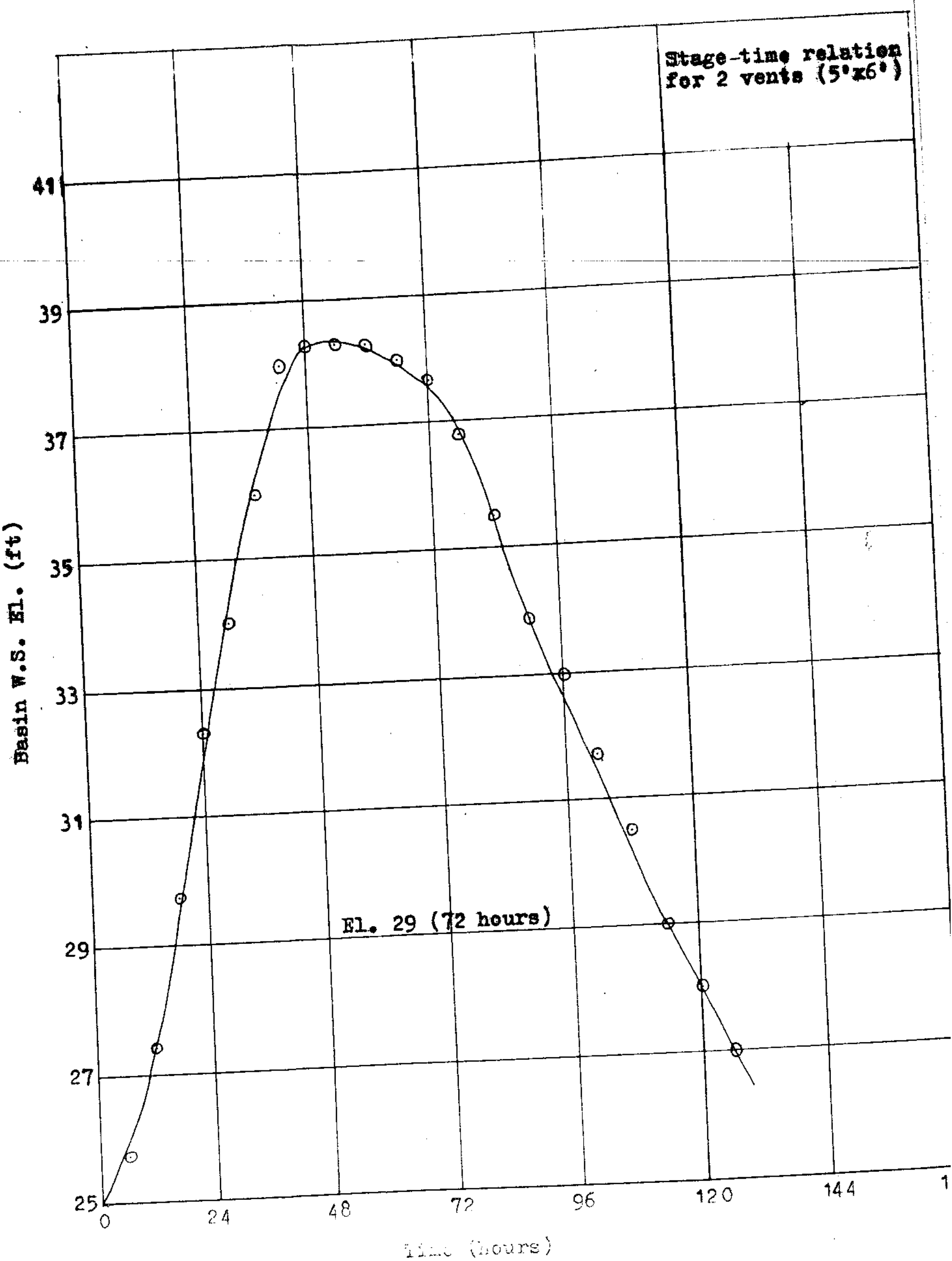

Stage-time relation
for 2 vents (5'x6')
Basin W.S. El. (ft)
41
39
37
35
33
31
29
27
25
El. 29 (72 hours)
0
24
48
72
96
120
144
Time (hours)

Figure - 19

Flood Routing for 4-vents

Day	Hour	I [cfs]	I_1+I_2	2s/t +D (4)+(7)	D	2s/t -D (5)- 2x(6)	Basin W.S.El
1	2	3	4	5	6	7	8
1	18	0	0	0	0	0	25.0
	24	79	79	79	48	-17	25.9
2	6	186	265	248	140	-32	26.9
	12	794	980	948	400	148	28.6
	18	1445	2239	2327	800	727	30.8
	24	2138	3583	4310	1280	1750	33.2
3	6	1971	4109	5859	1500	2859	34.2
	12	1721	3692	6551	1580	3391	34.7
	18	1412	3133	6524	1600	3324	35.4
	24	1075	2487	5811	1500	2811	34.2
4	6	737	1812	4623	1450	1723	33.8
	12	431	1168	2891	940	1011	31.6
	18	135	566	1577	600	377	30.1
	24	39	174	551	270	11	29.2
5	6	12	51	62	38	-14	25.7
	12	2	14	0			
	18	0	2				
	24	0	0				

Pre-monsoon Flood Routing Table (4 vents 5'x6')
Table 24

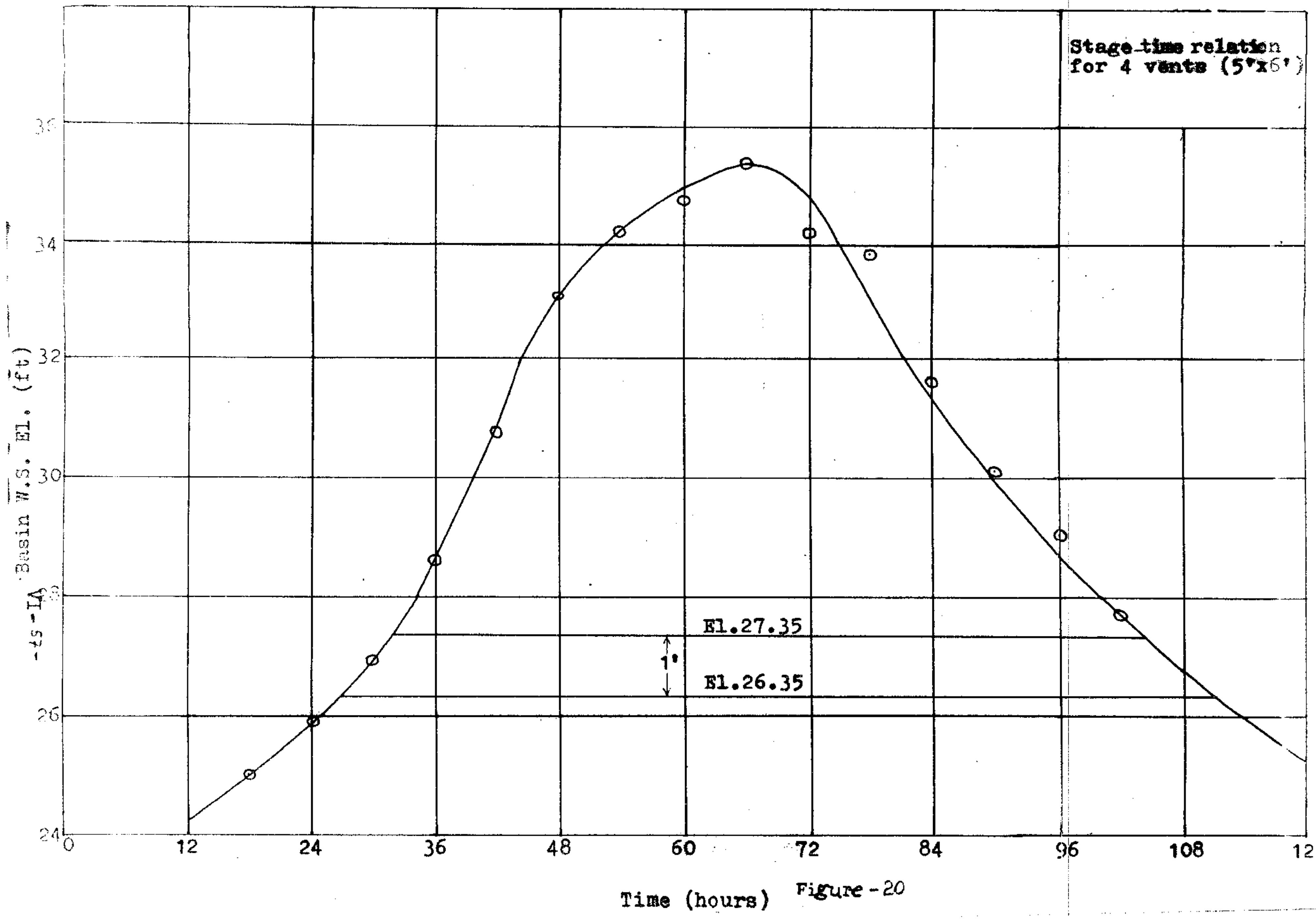

Stage-time relation
for 4 vents (5'x6')
El.27.35
1'
El.26.35
Basin W.S. El. (ft)
24
26
28
30
32
34
36
0
12
24
36
48
60
72
84
96
108
120
Time (hours)

Figure - 20

Flood Routing for 5-vents

Day	Hour	I [cfs]	$I_1 + I_2$	2s/t +D (4) +(7)	D	2s/t -D (5)-2x(6)	Basin W.S.El.
1	2	3	4	5	6	7	8
1	18	0	0	0	0	0	25.0
	24	79	79	79	52	-25	25.5
2	6	186	265	240	155	-70	26.7
	12	794	980	910	470	-30	28.7
	18	1445	2239	2209	910	389	30.6
	24	2138	3583	3972	1425	1122	32.6
3	6	1971	4109	5231	1750	1731	33.8
	12	1721	3692	5423	1800	1823	34.0
	18	1412	3133	4956	1700	1556	35.6
	24	1075	2487	4043	1450	1143	31.9
4	6	737	1812	2955	1150	655	30.9
	12	431	1168	1823	780	263	30.1
	18	135	566	829	430	-31	28.4
	24	39	174	143	92	-41	26.2
5	6	12	51	10			
	12	2	14				
	18	0	2				
	24	0	0				

Pre-monsoon Flood Routing (5 vents 5'x6')
Table 25

- 69-IA -

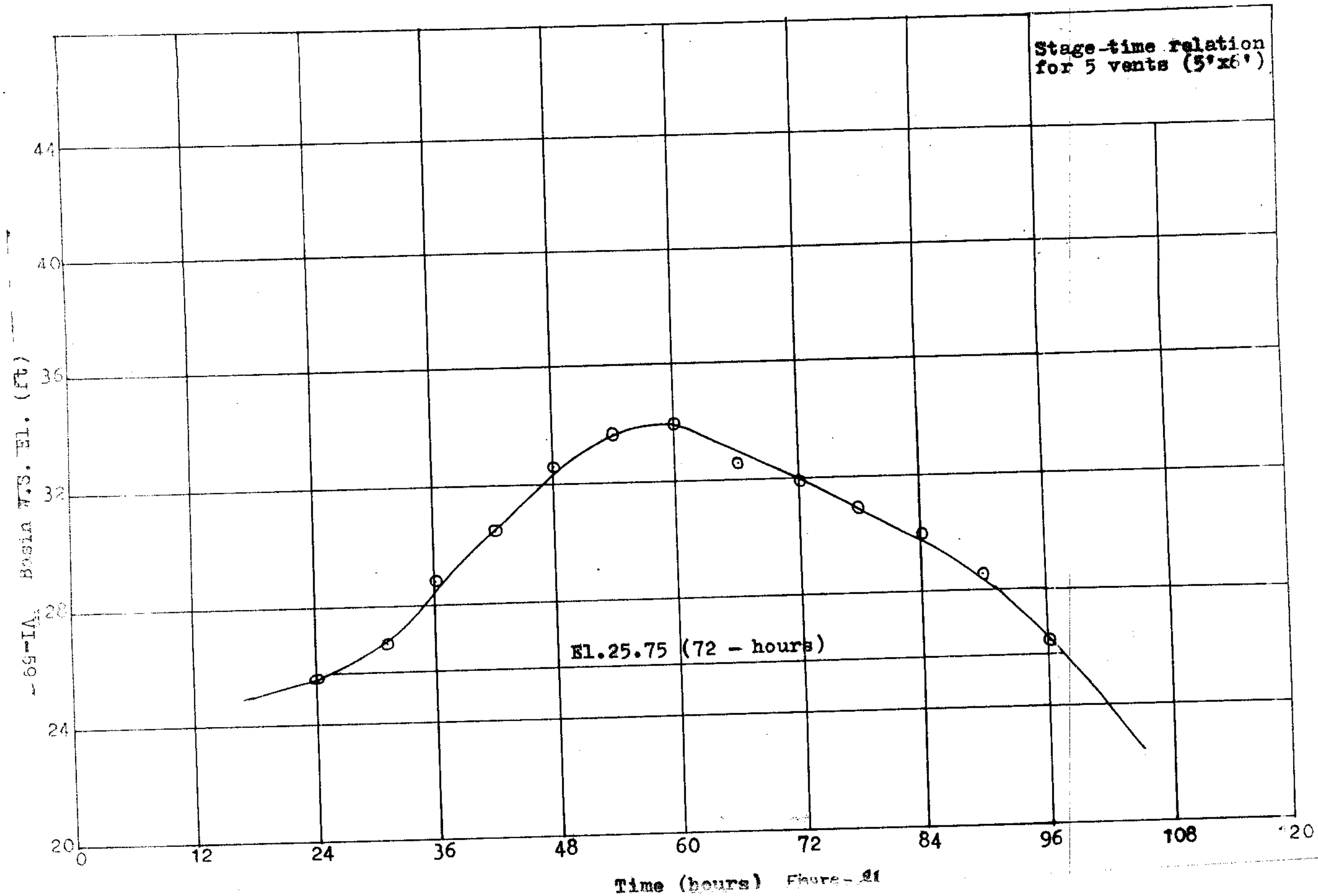
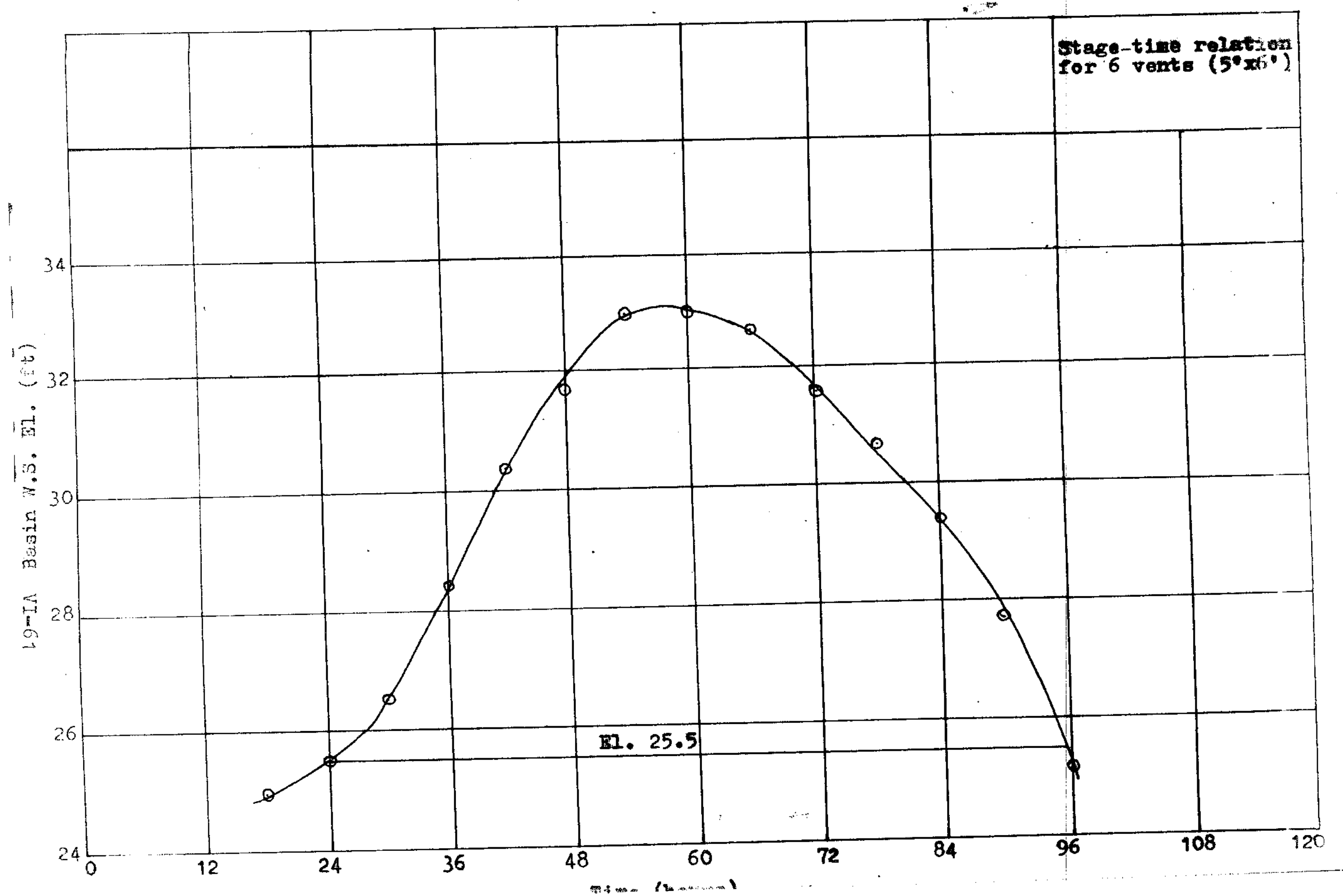


Figure - 21

Flood Routing for 6 vents

Day	Hour	I [cfs]	I_1+I_2	2s/t +D (4) +(7)	D	2s/t -D (5) -2x(6)	Basin W.S.El.
1	2	3	4	5	6	7	8
1	18	0	0	0	0	0	25
	24	79	79	79	47	-15	25.5
2	6	186	265	250	170	-90	26.5
	12	794	980	890	500	-110	28.4
	18	1445	2239	2129	990	-149	30.4
	24	2138	3583	3434	1400	634	31.7
3	6	1971	4109	4743	1800	1143	32.9
	12	1721	3692	4835	1825	1185	33.0
	18	1412	3133	4318	1700	918	32.6
	24	1075	2487	3405	1420	565	31.6
4	6	737	1812	2377	1075	227	30.7
	12	431	1168	1395	715	35	29.4
	18	135	566	601	370	-139	27.7
	24	39	174	35	18	-1	25.1
5	6	12	51				
	12	2	14				
	18	0	2				
	24						

Pre-monsoon Flood Routing Table (6 vents 5'x6')
Table 26



Stage-time relation
for 6 vents (5'x6')
19-IA Basin W.S. El. (ft)
34
32
30
28
26
24
El. 25.5
0
12
24
36
48
60
72
84
96
108
120

Post Monsoon Routing (5 vents 5'x6')

Computation of Runoff Volume

Month	Monthly Rainfall (inch)	Monthly Evapotranspiration (inch)	Initial soil Moisture loss (inch)	Depression Storage (inch)	Runoff (inch)	Runoff Volume (Acre ft)
July	12.06	4.2	0.5	1.0	6.36	12550
August	13.04	4.1	-	-	8.94	17642
Sept	8.08	3.8	-	-	4.28	8446
Oct	4.93	3.6	-	-	1.33	2625

Rainfall runoff volume (non paddy land)
Table 27

Month	Monthly Rainfall (inch)	Monthly Evapotranspiration (inch)	Initial soil Moisture loss (inch)	Depression Storage (inch)	Runoff (inch)	Runoff Volume (Acre ft)
July	12.06	5.6	-	4.0	2.46	4855
August	13.04	5.3	-	-	7.74	15273
Sept	8.08	5.1	-	-	2.98	5881
Oct	4.93	5.2	-	-	-	-

Rainfall runoff volume (paddy land)
Table 28

Computation of runoff volume

Month	Non paddy land (55%)		Paddy land (45%)		Basin Weighted Runoff (Acre-ft)
	Net Runoff	Weighted Runoff	Net Runoff	Weighted Runoff	
July	12550	5648	4855	2670	8318
August	17642	7939	15273	8400	16339
Sept	8446	3801	5881	3235	7036
Oct	2625	1181	-	-	1181

Rainfall runoff volume (Weighted Basin Average)
Table 29

Date	River W/L (in ft pwd)	Runoff Volume (Acre-ft)	Gate position	Inside	
				Storage (Acre-ft)	W.L. (in ft pwd)
13th July	39.00	7182	Gate closed	7182	39.00
31st July	43.20	4830		12012	41.55
31st August	44.0	16339		28351	43.77
5th Sept	43.90	1173	Gate opened	29524	43.90

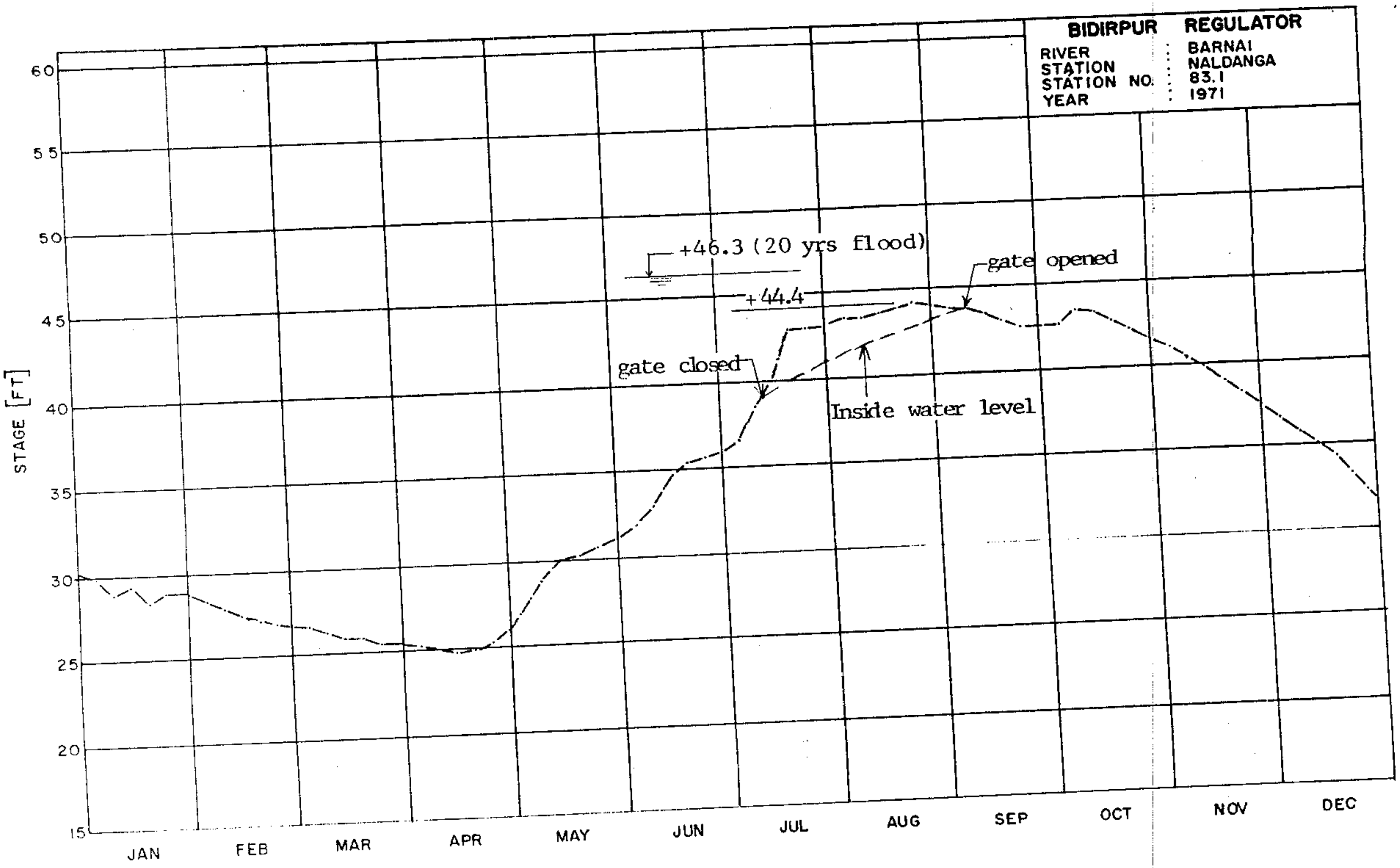
Computation of Inside storage & water level
Table 30

Post Monsoon Flood Routing

5 vents 5'x6'

Date	C/s Water level W1	R/s Water level W2	(W1 - W2) (ft)	Total discharge Q (Acre ft)	Residual storage (Acre ft)	Corresponding C/s Water level
Sept 5	43.9	43.9	0	0	29524	43.9
6	43.9	43.74	0.16	783.07	28740.93	43.82
7	43.82	43.58	0.24	999.06	27781.87	43.70
8	43.70	43.42	0.28	1037.58	26744.29	43.57
9	43.57	43.26	0.31	1089.99	25654.30	43.44
10	43.44	43.10	0.34	1141.52	24512.78	43.30
11	43.30	42.94	0.36	1174.61	23338.17	43.16
12	43.16	42.78	0.38	1206.80	22131.37	43.01
13	43.01	42.62	0.39	1222.57	20908.80	42.86
14	42.86	42.46	0.40	1238.15	19670.65	42.71
15	42.71	42.30	0.41	1253.53	18417.12	42.56
16	42.56	42.14	0.42	1268.73	17148.39	42.40
17	42.40	41.98	0.42	1268.73	15879.66	42.25
18	42.25	41.82	0.43	1283.74	14595.92	42.09
19	42.09	41.66	0.43	1283.74	13312.18	41.87
20	41.87	41.50	0.37			

Table 31


BIDIRPUR REGULATOR
RIVER : BARNAI
STATION : NALDANGA
STATION NO. : 83.1
YEAR : 1971
STAGE [FT]
+46.3 (20 yrs flood)
+44.4
gate closed
gate opened
Inside water level
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC
60
55
50
45
40
35
30
25
20
15

DISCUSSION & CONCLUSION

Jublee Khal & Bidirpur are two defined channels draining a common catchment area of about 37 sq.mile to Barnai river. Bidirpur khal takeoff from the Jublee khal at about ~~6.5 mile from its~~ offtake & flows almost parallel with the Jublee khal near its offtake. The topography of the catchment area is such that it is rather impossible to identify the individual catchment area for Jublee khal & Bidirpur khal. Attempt has therefore been made to complete the flood routing procedure assuming a single channel for the whole catchment area & then distributing the number of vents between the two khals according to their existing carrying capacity. From the results of the premonsoon routing the extent of crop damage or the percentage of total area that will remain submerged for more than 72 hours for different ventages are presented in Table below.

Number of vents	% of total area submerged more than 72 hours
1 vent 5'x6'	2.7%
2 vents 5'x6	0.33
4 vents 5'x6'	0.2%
6 vents 5'x6'	0.04%

From the above Table it may be observed that even with 1 vent of size 5'x6' there is no appreciable damage. But to satisfy other hydraulic conditions like tolerable velocity a minimum of 5 vents of size 5'x6' is required.

From the study of the existing x-sections of Jublee Khal and Bidirpur Khal it is observed that 75% of the runoff volume of the catchment area drains through Jublee Khal and rest 25% through Bidirpur Khal. By proportioning the number of vents according to the carrying capacities of these khals a regulator of 3.75 vents and another 1.25 vents of size 5'x6' are required respectively at the outfall of Jublee Khal and Bidirpur Khal. For convenient a 4 vent regulator is selected at the outfall of Jublee Khal a 2 vent at the outfall of Bidirpur Khal. For hydraulic design the design discharge is also proportioned according to the number of vents selected for the khals.

HYDRAULIC DESIGN

HYDRAULIC DESIGN OF JUBLEE KHAL

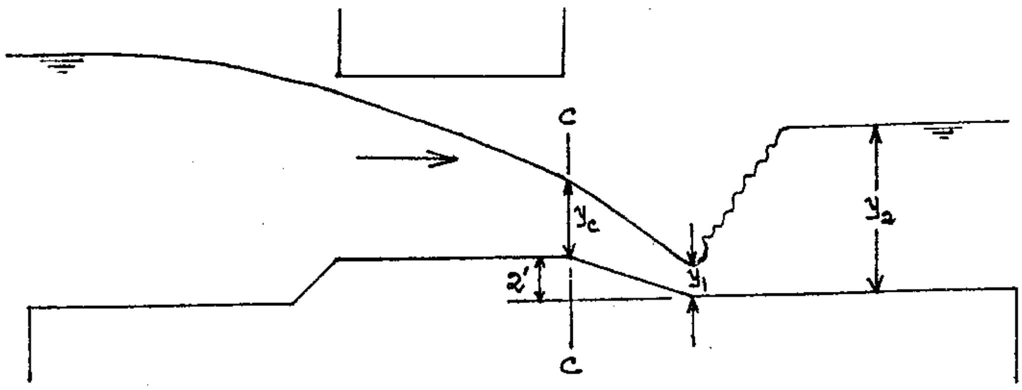
Discharge, Q = 1200 cfs.

No. of vents = 4

Size of vent = 5'x 6'

Unit discharge, $q = \dfrac{1200}{4\times 5} = 60$ cfs/ft

Critical depth, $Y_c = (q^2/g)^{1/3} = (60^2/32.2)^{1/3} = 4.82$ ft.



Applying Bernoullis Equation between the critical section c-c and sec. 1-1.

$$Y_c + \frac{V_c^2}{2g} + Z = Y_1 + \frac{V_1^2}{2g}$$

$b_1 = (4\times 5') + (3\times 1.25') + 2\times 6\times \tan 10^o = 25.86$ ft.

$q_1 = 1200/25.86 = 46.40$ cfs/ft

At the critical state $\dfrac{V_c^2}{2g} = \dfrac{Y_c}{2}$ and $V_1 = q/y_1$

Assuming Z = 2 ft.

So eq. (1) becomes,

$$Y_c + \frac{Y_c}{2} + Z = Y1 + \frac{q_1^2}{2gY_1^2}$$

$$\text{or, } 4.82 + \frac{4.82}{2} + 2 = Y_1 + \frac{46.40^2}{2\times32.2\times y_1^2}$$

$$\text{or, } 9.23 = y_1 + \frac{33.43}{y_1^2}$$

By trial and error, we get $y_1 = 2.18$ ft.

Since $Y_1 < Y_c$, the flow is supercritical.

$V_1 = q/y_1 = 46.40/2.18 = 21.28$ fps

$F_1 = V_1/(gy_1)^{0.5} = 21.28/(32.2 \times 2.18)^{0.5} = 2.50$ OK

$$\frac{Y_2}{Y_1} = \frac{1}{2}\ [(1+8F_1^2)^{0.5} - 1]$$ [Ref: Open Channel Hydraulics - Ven Te Chow]

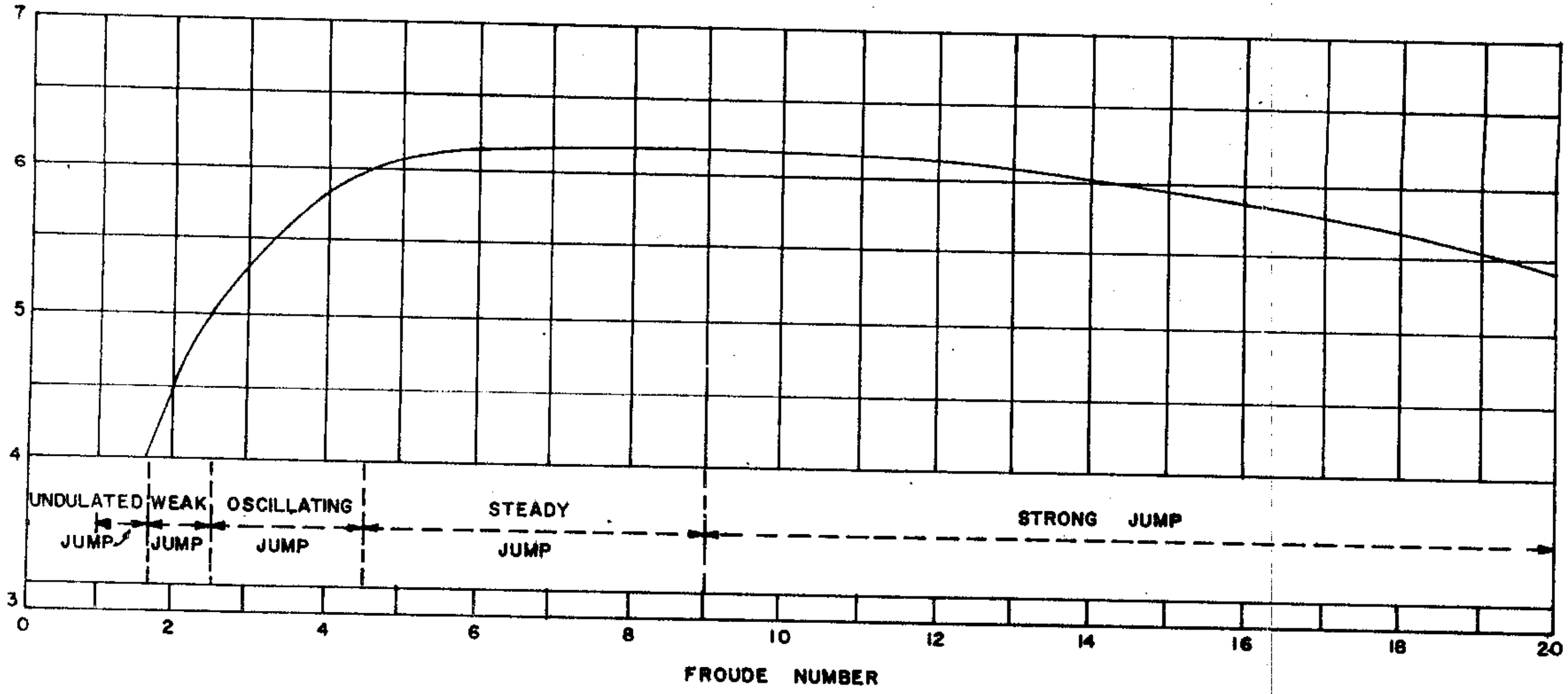
$$= \frac{1}{2}\ [(1+8\times2.5^2)^{0.5} - 1]$$

$$= 3.07$$

$Y_2 = 3.07\ Y_1 = 3.07 \times 2.18 = 6.7$ ft

From curve L/Y_2 vs. F (Fig.23),

$L/Y_2 = 5.0$



LENGTH OF JUMP

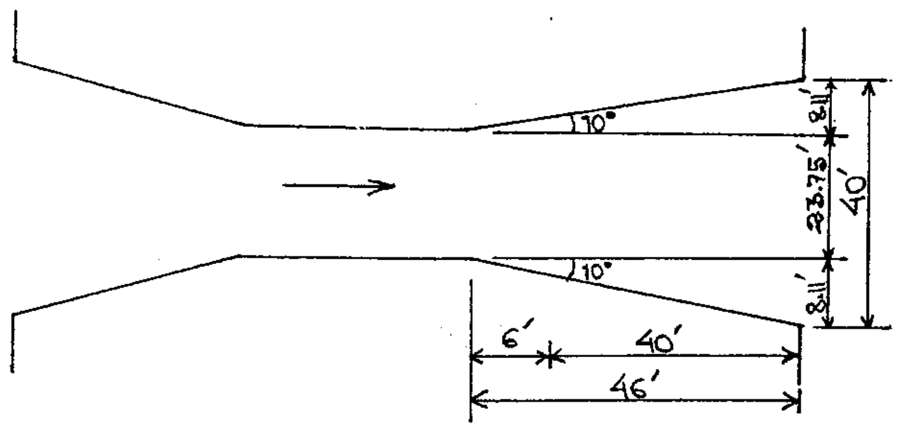
Figure - 23

$$L = 5.0\ Y_2 = 5.0 \times 6.7 = 33.5 \text{ ft.}$$

Make it 40 ft.

Scour Depth

Assuming : Angle of flaring of d/s wing wall = 10^o
Slope of d/s glacis = 1:3



Width of flow at the end of river side

$$b_2 = (5'\text{x}4) + (1.25'\text{x}3) + 2\text{x}46'\text{x}\tan 10^o = 40 \text{ ft.}$$

$$\text{Discharge at exit, } q_2 = \frac{1200}{40} = 30 \text{ cfs/ft}$$

Depth of scour is determined by Lacey's regime scour depth

$$R = 0.91\ (q_2^2/f)^{1/3}$$

where, silt factor, $f = 1.76\ (\text{dmm})^{0.5}$

From grain size distribution, dmm = 0.029

i.e. $f = 1.76\ (0.029)^{0.5} = 0.30$

$$R = 0.91\ (q_2^2/f)^{1/3} = 0.91\ (30^2/0.3)^{1/3} = 13.12 \text{ ft.}$$

Scour depth at upstream = 1.25 R = 1.25 x 13.12 = 16.4 ft

Scour depth at downstream = 1.50 R = 1.50 x 13.12 = 19.7 ft

[Ref : Irrigation Engineering and Hydraulic Structures
-Santosh Kumar Garg]

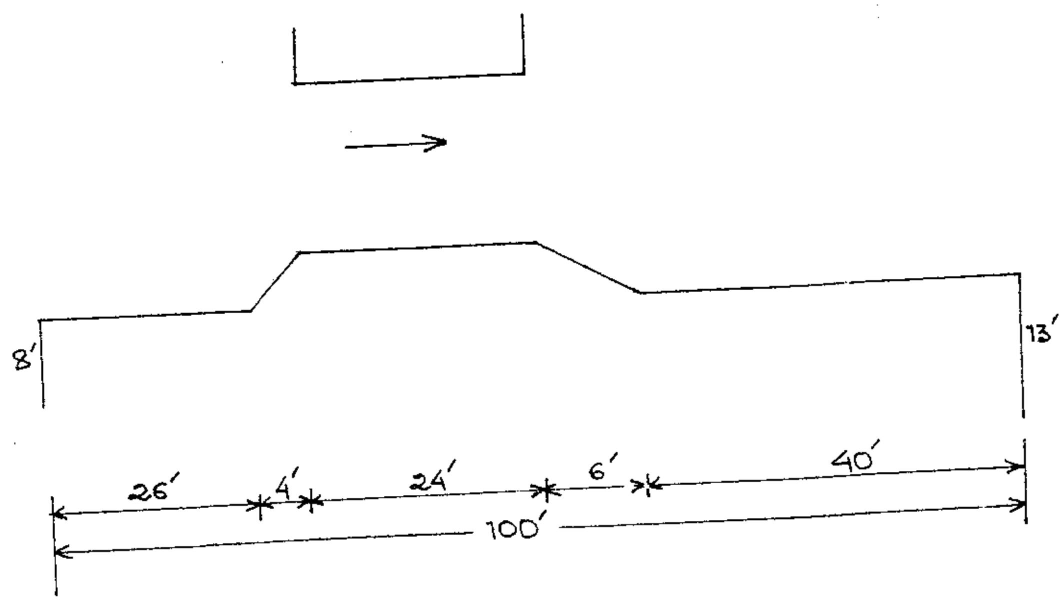
Depth of upstream cutoff = 16.4 - 8 8 ft.
Depth of downstream cutoff = 19.7 - 6.7 = 13 ft.

Floor length

Length of u/s glacis = 4 ft. [with 2 ft drop and 1:2 slope]
Lenght of barrel = 4+1+14+1+4 = 24 ft.
Length of d/s glacis = 6 ft [with 2 ft. drop and 1:3 slope]
Length of d/s apron = 40 ft.
Length of u/s apron = 40 x 2/3 = 26 ft.

Total floor, length, b = 26+4+24+6+40 = 100 ft.

Now we will check this floor length whether it is adquate with respect to exit gradient.



CHECK FOR EXIT GRADIENT

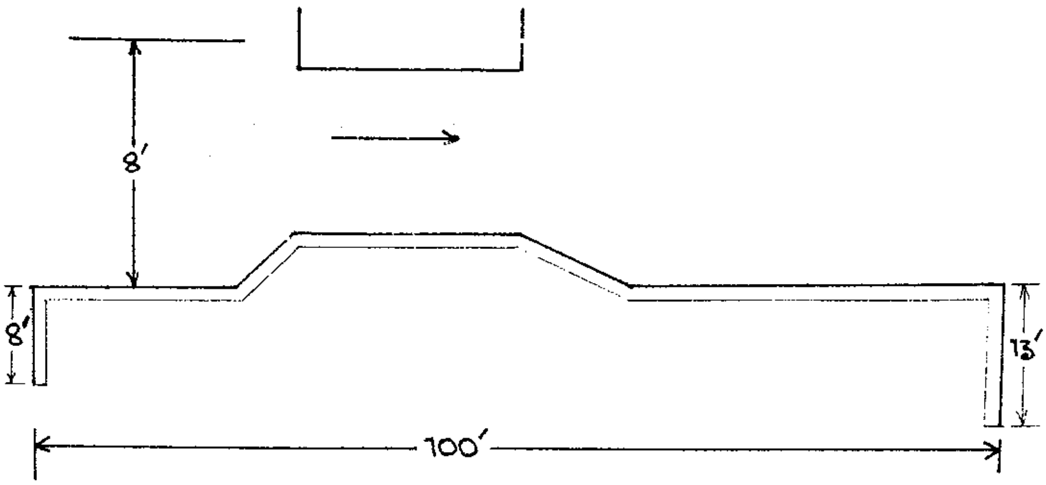
$$G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{L}}$$

where, H = maximum difference of water level between country side and river side

d = depth of cutoff wall

$$L = \frac{1+(1+a^2)^{0.5}}{2} \quad \text{where } a = \frac{b}{d}$$

b = total apron length



H = 8' u/s cutoff depth, d_1 = 8'

b = 100' d/s cutoff depth, d_2 = 13'

The exit gradient at the downstream end,

$$G_E = \frac{H}{d_2} \times \frac{1}{\pi\sqrt{L}}$$

$$a = \frac{b}{d_2} = \frac{100}{13} = 7.69$$

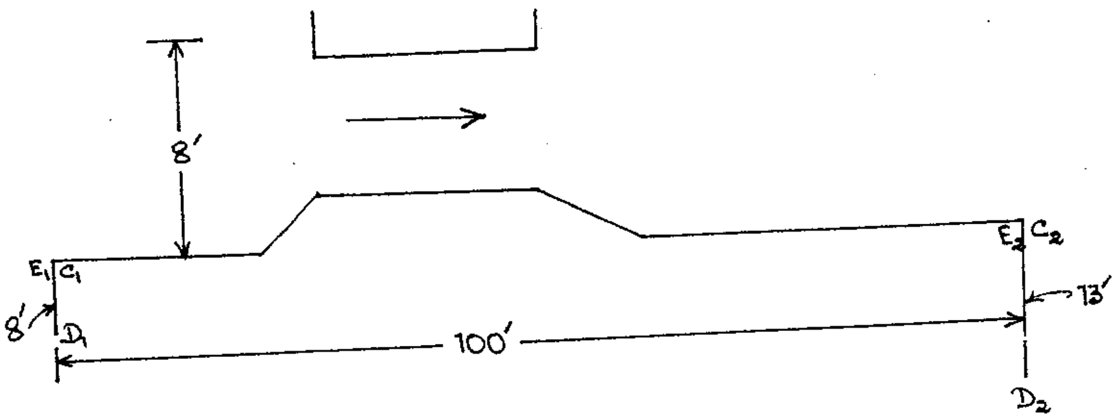
$$L = \frac{1 + (1 + 7.69^2)^{1/2}}{2} = 4.38$$

$$G_E = \frac{8}{13} \times \frac{1}{\pi \sqrt{4.38}}$$

$$= 0.093 < 0.143 \quad [1/7 = 0.143]$$

Hence O.K.

<u>Uplift Pressure Calculation</u>



<u>Upstream cutoff</u>

b = 100'

d = 8'

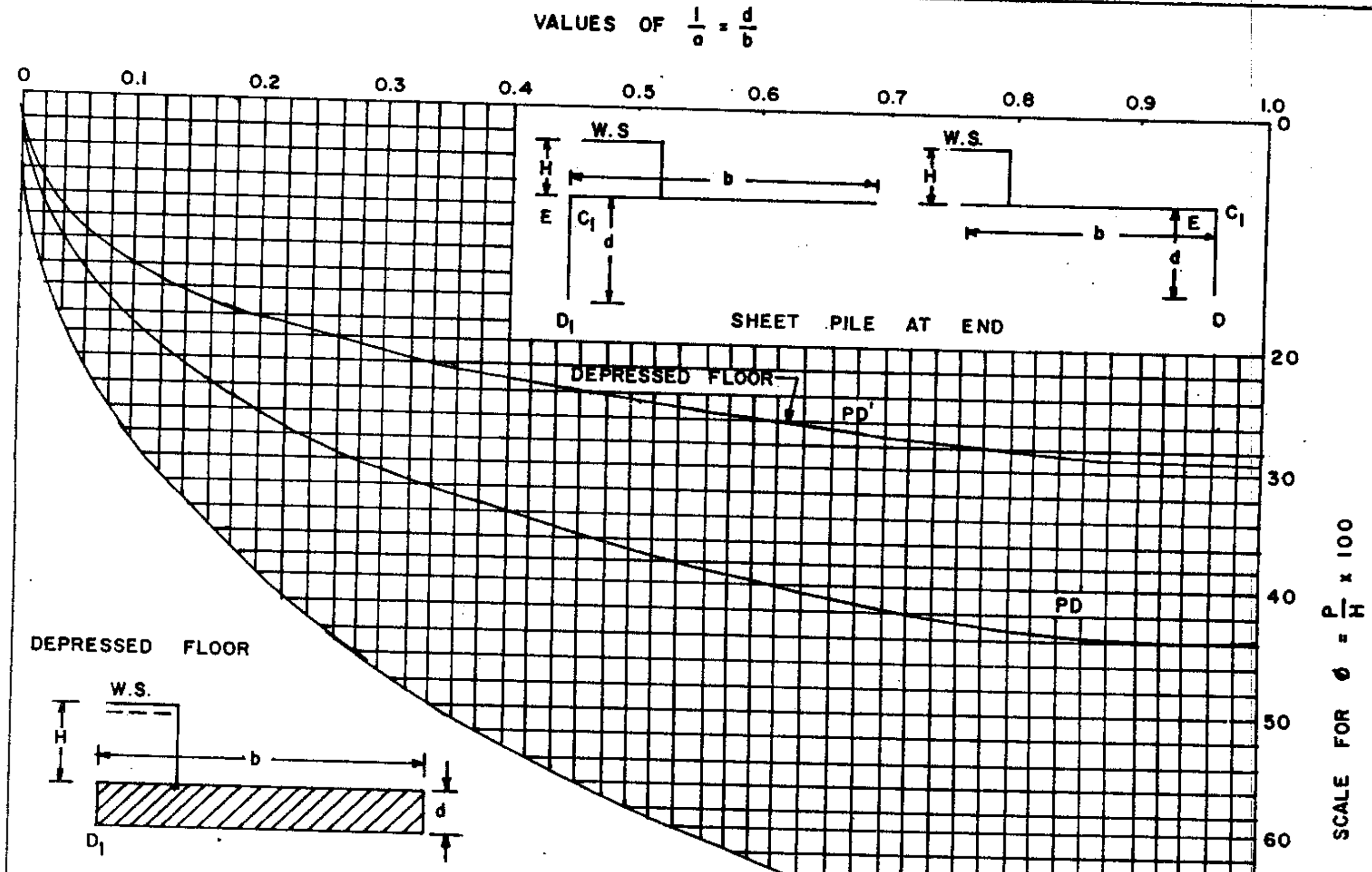
1/a = d/b = 8/100 = 0.08

From fig. 24

PE_1 = 100%

PE = 26% $\quad$ PC_1 = 100-PE = 100-26 = 74%

PD = 17% $\quad$ PD_1 = 100-PD = 100-17 = 83%



$$PE = \frac{1}{3.1416} \cos^{-1}\left(\frac{L-2}{L}\right)$$

$$PD = \frac{1}{3.1416} \cos^{-1}\left(\frac{L-1}{L}\right)$$

$$PC_1 = 100 - PE$$

$$PD_1 = 100 - PD$$

$$PD' = 100 - PD' \text{ (DEPRESSED FLOOR)}$$

$$PD' = PD - \frac{2}{3}(PE - PD) + \frac{3}{a^2} \text{ (DEPRESSED FLOOR)}$$

$$L = 1 + \frac{\sqrt{1+a^2}}{2}$$

Figure – 24

Figure - 24

Corrections for PC_1

a) Effect of downstream cutoff on upstream cutoff:

$$\text{Correction} = 19\left[\frac{D}{b'}\right]^{1/2}\left[\frac{d+D}{b}\right]$$

Where, D = depth of downstream pile
d = depth of upstream pile
b'= distance between two piles
b = total floor length

Assumed 2 ft. thickness throughout the floor length.

D = 13 - 2 = 11 ft.
d = 8 - 2 = 6 ft.
b' = 100 - 2 = 98 ft.
b = 100 ft.

$$\text{Correction} = 19\left[\frac{11}{98}\right]^{1/2}\left[\frac{6+11}{100}\right] = 1.08\ \%\ (+ve)$$

b) Correction for depth :

$$\text{Correction} = \frac{PD_1 - PC_1}{d} \times \text{thickness}$$

$$= \frac{83 - 74}{8} \times 2$$

$$= 2.25\ \%\ (+ve)$$

$$PC_1\ (\text{corrected}) = 74\ \% + 1.08\ \% + 2.25\ \%$$
$$= 77.33\ \% \approx 77\ \%$$

<u>Downstream cutoff</u>

b = 100'
d = 13'

1/a = d/b = 13/100 = 0.13

From fig. 24

$PC_2 = 0\%$

$PE_2 = 33\%$

$PD_2 = 22\%$

Corrections for PE_2
----

a) Effect of upstream cutoff on downstream cutoff:

$$\text{Correction} = 19 \left[\frac{D}{b'}\right]^{1/2} \left[\frac{d+D}{b}\right]$$

where, $D = 8 - 2 = 6$ ft.
$d = 13 - 2 = 11$ ft.
$b' = 100 - 2 = 98$ ft.
$b = 100$ ft.

$$\text{Correction} = 19 \left[\frac{6}{98}\right]^{1/2} \left[\frac{11+6}{100}\right] = 0.80\ \%\ (-ve)$$

b) Correction for depth:

$$\text{Correction} = \frac{PE_2 - PD_2}{d} \times \text{thickness}$$

$$= \frac{33 - 22}{13} \times 2$$

$$= 1.69\ \%\ (-ve)$$

PE_2 (corrected) $= 33\ \% - 0.80\ \% - 1.69\ \% = 30.51\ \% \approx 31\ \%$

Upstream pile

$PE_1 = 100\ \%$

$PD_1 = 83\ \%$

$PC_1 = 77\ \%$

Downstream pile

$PE_2 = 31\ \%$

$PD_2 = 22\ \%$

$PC_2 = 0\ \%$

WATER LEVEL FREQUENCY CURVE

RIVER : BARNAI

STATION : NALDAGA

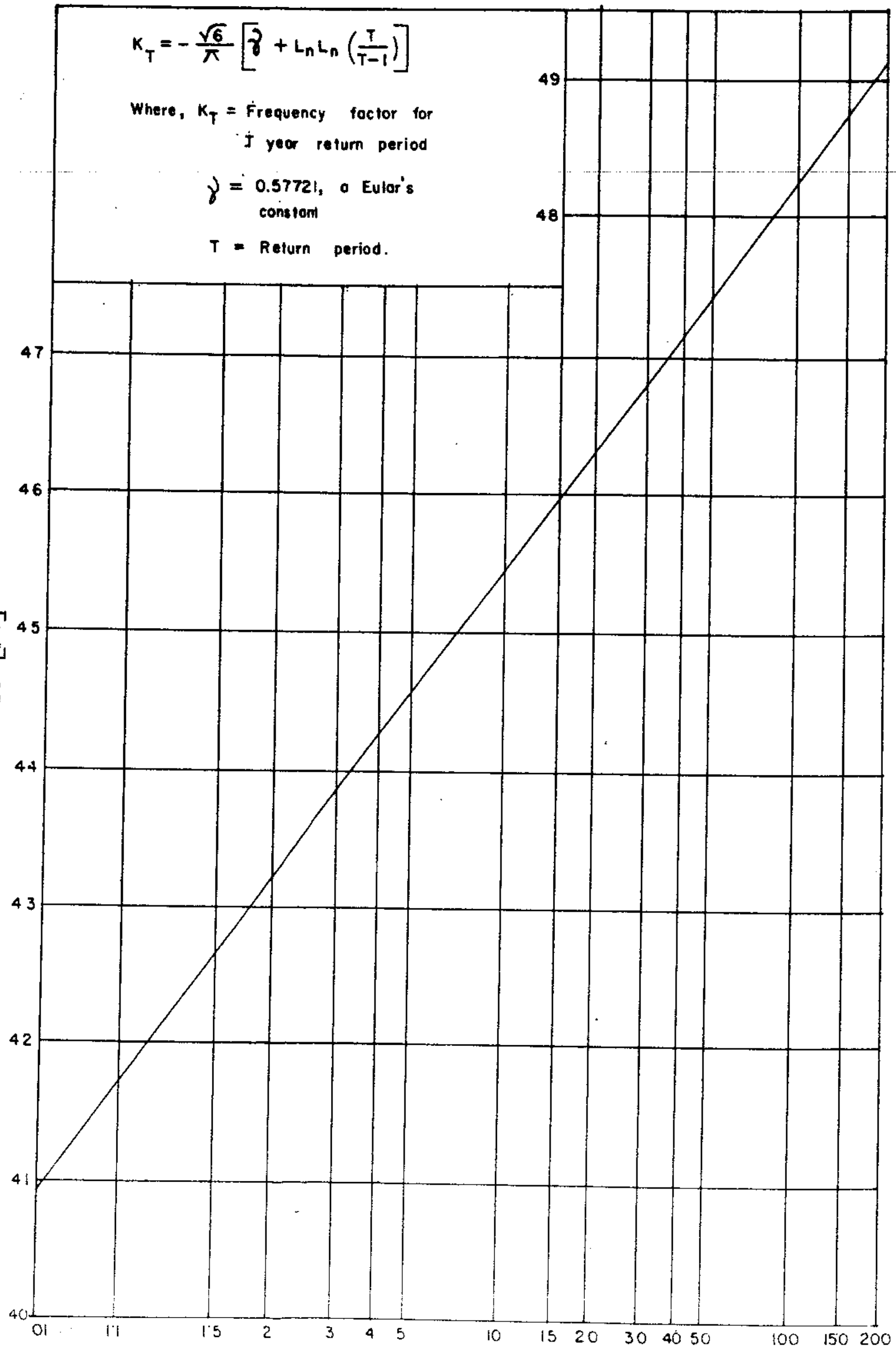


Figure 25

VI-72

20 years flood (Fig.25) = + 46.30

C/s water level (July 31) = + 41.55

Head for designing country side floor thickness

= 46.30 - 41.55 = 4.75 ft. 5 ft.

$H = 5'$ u/s cutoff depth, $d_1 = 13'$

$b = 100'$ d/s cutoff depth, $d_2 = 8'$

The exit gradient at the downstream end,

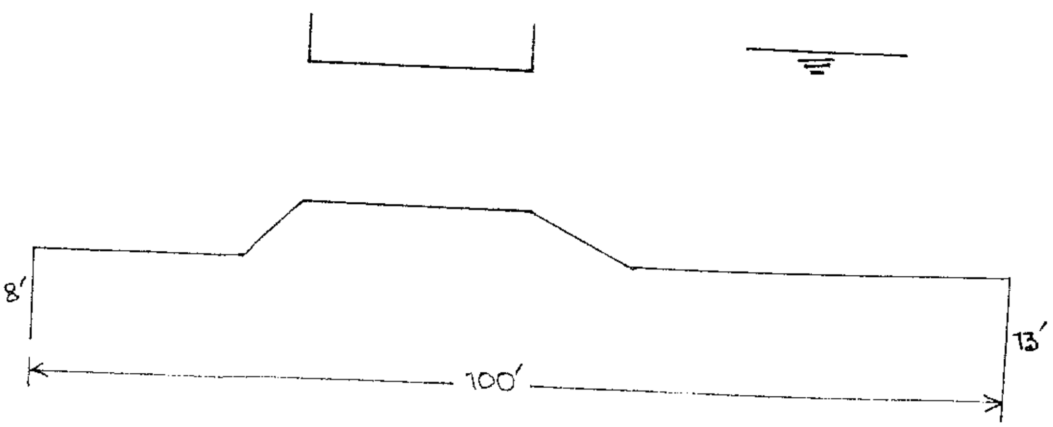
$$G_E = \frac{H}{d_2} \times \frac{1}{\pi \sqrt{L}}$$

$$a = \frac{b}{d_2} = \frac{100}{8} = 12.50$$

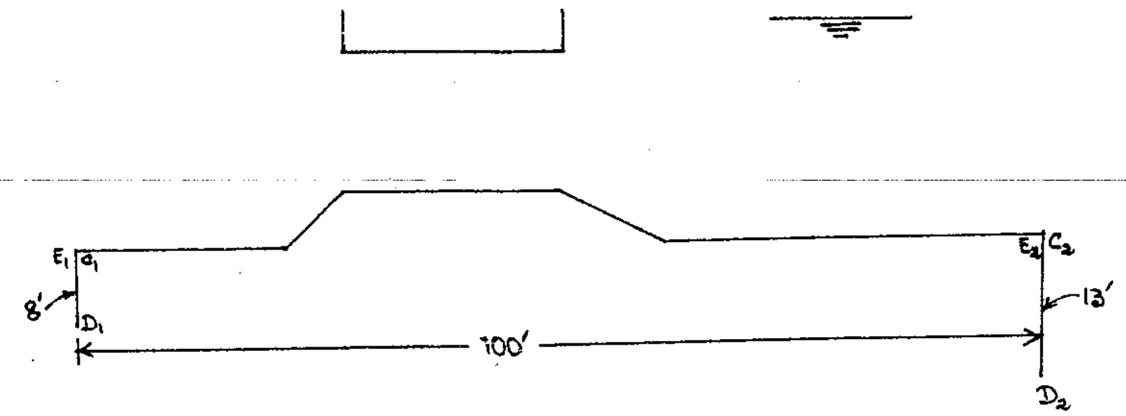
$$L = \frac{1 + (1 + 12.50^2)^{1/2}}{2} = 6.77$$

$$G_E = \frac{5}{8} \times \frac{1}{\pi \sqrt{6.77}}$$

$$= 0.076 < 0.143 \quad [1/7 = 0.143]$$



Uplift Pressure Calculation



Upstream cutoff

b = 100'

d = 13'

1/a = d/b = 13/100 = 0.13

From fig. 24

PE_1 = 100%

PE = 33% PC_1 = 100-PE = 100-33 = 67%

PD = 22% PD_1 = 100-PD = 100-22 = 78%

Corrections for PC_1

a) Effect of downstream cutoff on upstream cutoff:

$$\text{Correction} = 19\left[\frac{D}{b'}\right]^{1/2}\left[\frac{d+D}{b}\right]$$

Where, D = depth of downstream pile
d = depth of upstream pile
b'= distance between two piles
b = total floor length

Assumed 2 ft. thickness throughout the floor length.

$$D = 8 - 2 = 6 \text{ ft.}$$
$$d = 13 - 2 = 11 \text{ ft.}$$
$$b' = 100 - 2 = 98 \text{ ft.}$$
$$b = 100 \text{ ft.}$$

$$\text{Correction} = 19 \left[\frac{6}{98}\right]^{-1/2} \left[\frac{11+6}{100}\right] = 0.80\ \% \quad (+ve)$$

b) Correction for depth :

$$\text{Correction} = \frac{PD_1 - PC_1}{d} \times \text{thickness}$$

$$= \frac{78 - 67}{13} \times 2$$

$$= 1.69\ \%\ (+ve)$$

$$PC_1 \text{ (corrected)} = 67\ \% + 0.80\ \% + 1.69\ \%$$
$$= 69.5\ \%$$
$$\approx 70\%$$

<u>Downstream cutoff</u>

$b = 100'$
$d = 8'$

$1/a = d/b = 8/100 = 0.08$

From fig. 24

$PC_2 = 0\%$

$PE_2 = 26\%$

$PD_2 = 17\%$

Corrections for PE_2

a) Effect of upstream cutoff on downstream cutoff:

$$\text{Correction} = 19 \left[\frac{D}{b'}\right]^{1/2} \left[\frac{d+D}{b}\right]$$

where, $D = 13 - 2 = 11$ ft.
$d = 8 - 2 = 6$ ft.
$b' = 100 - 2 = 98$ ft.
$b = 100$ ft.

$$\text{Correction} = 19 \left[\frac{11}{98}\right]^{1/2} \left[\frac{6+11}{100}\right] = 1.08\ \%\ (-ve)$$

b) Correction for depth:

$$\text{Correction} = \frac{PE_2 - PD_2}{d} \times \text{thickness}$$

$$= \frac{26 - 17}{8} \times 2$$

$$= 2.25\ \%\ (-ve)$$

$$PE_2\ (\text{corrected}) = 26 - 1.08 - 2.25 = 22.67\ \% \approx 23\ \%$$

Upstream pile

$PE_1 = 100$ %

$PD_1 = 78$ %

$PC_1 = 70$ %

Downstream pile

$PE_2 = 23$ %

$PD_2 = 17$ %

$PC_2 = 0$ %

$$\text{Specific weight of concrete, SWC} = \frac{UWC - UWW}{UWW}$$

where, UWC = unit weight of concrete
UWW = unit weight of water
SWc = specific weight of concrete

$$SWc = \frac{150-62.4}{62.4} = 1.40$$

<u>Thickness of the floor slab</u>

Point a $\quad t = \dfrac{5 \times 0.23}{1.40} = 0.82'$

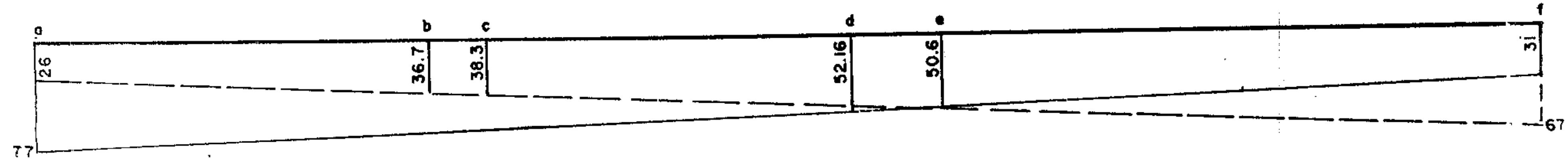
" b $\quad t = \dfrac{5 \times 0.367}{1.40} = 1.31'$

" c $\quad t = \dfrac{5 \times 0.383}{1.40} = 1.37'$

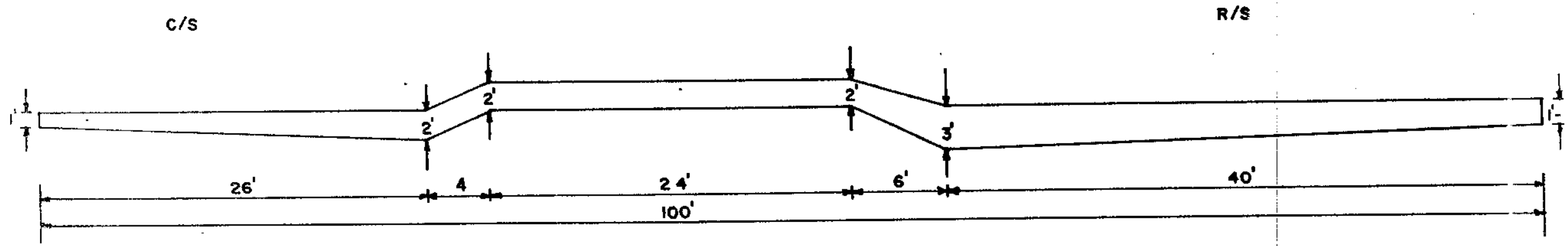
" d $\quad t = \dfrac{8 \times 0.5216}{1.40} = 2.98'$

" e $\quad t = \dfrac{8 \times 0.506}{1.40} = 2.89'$

" f $\quad t = \dfrac{8 \times 0.31}{1.40} = 1.77'$



PERCENTAGE OF PRESSURES



THICKNESS OF THE FLOOR SLAB

REFERENCES

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2. Flood routing through drainage structures, by M.F.A. Siddique & W.M. Emerson.

3. Estimate Flood Peaks for small drainage basins, by M.F.A. Siddiqui & W.M. Emerson.

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5. Design Report of Jublee Khal Regulator (4 vents), by A.N.M. Wahedul Huq.

CHAPTER VII

HYDRAULIC DESIGN OF DRAINAGE SLUICE (TIDAL) AT BAMNADANGA IN KHULNA

Design of tidal drainage sluice

Introduction

A drainage sluice is a structure through which excess water from an area within finite boundaries are to drain off. The number of standard 5'x6' vents is based on drainage requirements determined by catchment area, the assumed rainfall, and the tide lockage. In addition to catchment area consideration must be given to inflow from outside the area through spill channels. To design a tidal drainage sluice one must be familiar to the following terms.

Tide lockage

The tide lockage is determined by the range of tide and the elevation to which it is proposed to drain the country side of the embankment. This elevation has been taken usually one foot higher than the average ground level on the assumption that crops can withstand this amount of flooding for short periods. Tide lockage occurs from this elevation upto the elevation of high tide as the water outside the dykes is then higher than the water on the inside. No drainage takes place until the tide falls bellow the elevation of the water on the country side behind the dykes.

Discharge through sluice

The control for the discharge through the sluice is at the inlet when the tailwater elevation is lower than one half of the sum of the critical depth plus the height of the vent. This relationship is designated by the formula $\frac{Dc + D}{2}$. This means the discharge is constant and independent of the tailwater fluctuations until the tailwater raises above this elevation which is designated by the height $\frac{Dc + D}{2}$ above the horizontal flow line of the vent. As the tailwater continues to rise the tailwater assumes control of the discharge. The discharge then decreases rapidly and becomes zero as the tide level becomes at a height of average country side ground level plus one foot as assumed.

The amount of discharge possible for each culvert is to be determined through the use of the curves based on experiments and published by the U.S. Bureau of Public Roads. The relationship between tide lockage and the amount of drainage possible with respect to ground level and vent height during a tide cycle is shown by Fig. 1. For simplicity the tide curve is shown as a straight line (Fig. 2) from low tide to high tide and back down to low tide, over a tide cycle period of twelve hours. This makes the tide curve a triangle with the apex at the high tide, the ordinate being elevation and the abscissa a twelve hour period. Also plotted on the ordinate scale is the drainage discharge in cubic feet per second. The discharge is constant

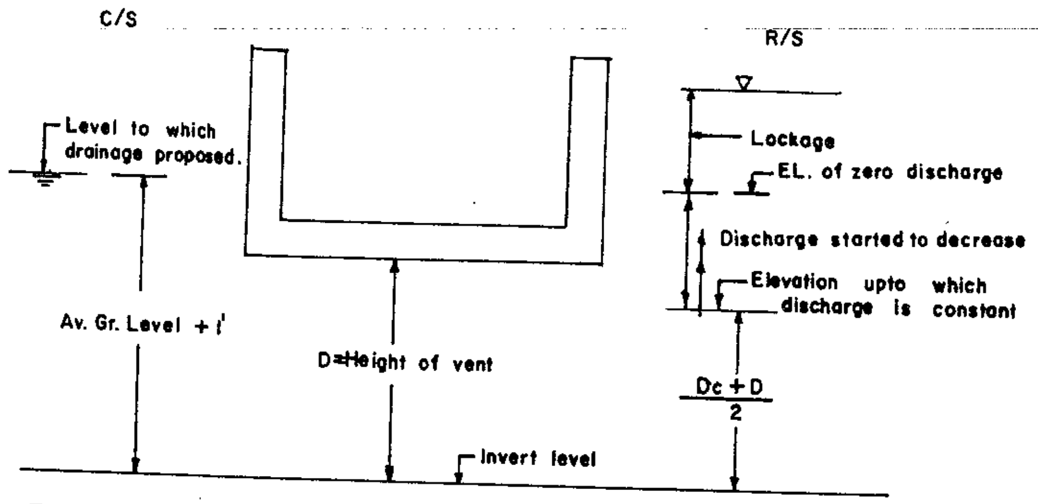
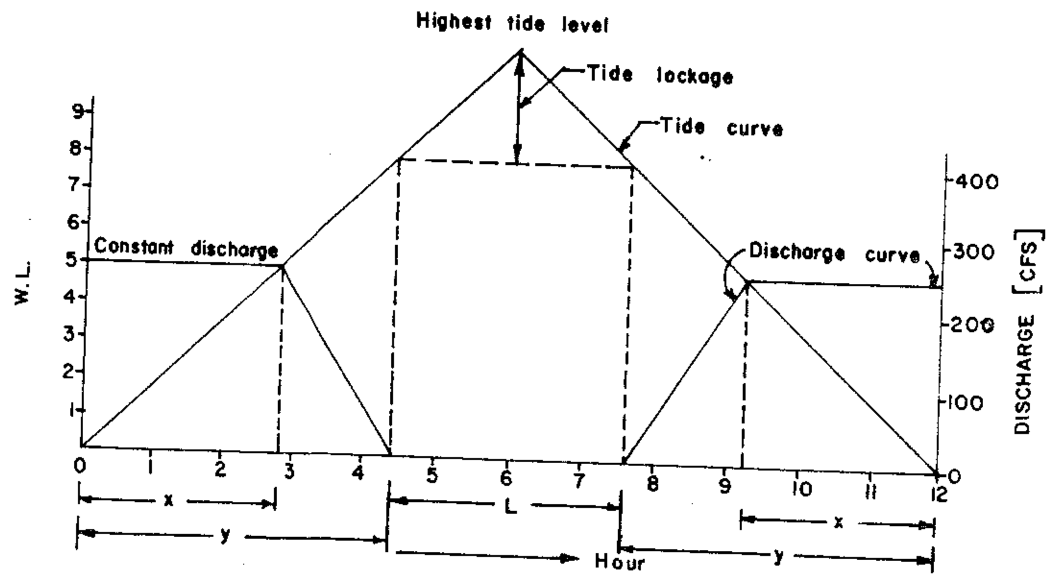


FIG. 1 Elevation of zero discharge, constant discharge and lockage



2x = Time through which discharge is constant.

2y = Total time through which discharge takes place.

L = Lockage period.

FIG. 2. TYPICAL TIME DISCHARGE CURVE

during the time the tide rises from its low point to elevation $\frac{D + Dc}{2}$. The length of this time is x. As the tide continues to rise the discharge is controlled by the tail water, the discharge diminishes very rapidly and becomes zero at y hours from the point of low tide, while the corresponding tide water level is equal to average ground level plus one feet. Above this level upto highest tide level is the lockage period. The drainage curve is duplicated in reverse order on the falling tide.

The area under the curve represents the total volume of drainage flow. Tide lockage is L hours for each tide cycle or approximately 2L hours/day.

Data required to design a tidal drainage sluice

1. A brief description of the project and the purpose it is to serve.

2. A topographic map of the entire drainage basin area.

3. Rainfall record within the basin.

4. Estimate of percentage of paddy & non paddy lands.

5. Tide records for atleast one year.

6. Profile and cross-section of the main drainage channel for atleast one mile upstream of the structure.

PROCEDURE

1) The geological location and size of basin is to be located from the topographic maps.

2) The geographical centre (centroid) of the basin area is to be located. The basin centroid can be found by vertically suspending a cardboard cutout of the basin shape successively from three or more points and finding the intersection of plumb lines from each point.

3) The location of the basin is to be find out on the isohyetal map of 4-month rainfall index of Bangladesh presented by IECO the general Consultant of BWDB, to get 4 month rainfall index of the area.

4) The 4-month rainfall index is to be multiplied by daily combined indices from Table 1 of frequency 10 yrs or 25 years. This gives the accumulated point rainfall in inches for storms of 1,2,3,4 & 5 days.

Table 1.

Storm Frequency for 10 yrs and 25 yrs.

DAYS	STORM FREQUENCIES	
	10-year	25-year
1	0.128	0.153
2	0.192	0.228
3	0.230	0.272
4	0.257	0.303
5	0.276	0.320

5) Equivalent uniform depth of rainfall

The equivalent uniform depth of rainfall is defined as the depth of water which results from spreading the total volume of basin rainfall uniformly over the total basin area. The IECO Master Plan states that a fairly good relationship exists between depth of rain and distance from the storm centre. Table 2 shows the relationship in tabular form. Basin centroid is assuemed as storm centre.

Table 2.

Rainfall Intensities with distance from storm centre.

Distance from storm Centre (mile)	5-day storm percentage of point rainfall
1/4	100
1	88.0
2	81.7
3	77.5
4	74.2
5	72.0
6	69.8

The procedure for determining the uniform equivalent depth of rainfall is as follows:

a) The geographical centre of the basin is to be located.

b) Concentric circles or isohyetals at even miles intervals around the centre is to be drawn.

c) The areas between the respective isohyetals within the basin area is planimetered.

d) The area is to be multiplied by the proper percentage from table 2.

e) A summation of the values from step (d) is to be made and divide by the total area.

f) The result is a percentage that when multiplied with point rainfall gives the equivalent uniform depth of rainfall.

6) The accumulative total of equivalent uniform depth of rainfall is to be separated into daily increments. The daily increments is then arranged in an order (arbitrary) giving the worst of flood condition. A unit time interval (3 hrs or 6 hrs) is to be chosen for breaking down the daily increments.

The smallest time interval to be used will depend on the length of main drainage course within the basin. The following intervals are recommended:

1. Less than 2 miles; one hour
2. 2 to 6 miles ; 3 hours
3. Over 6 miles ; 6 hours

7) Determination of rainfall losses:

To determine rainfall losses, the land is to be classified as percentage of area in Paddy land & Non-paddy land. The losses are assumed as follows:

a) Initial soil moisture loss:

Paddy land : No initial loss under all conditions. It is assumed the soil is saturated.

Non paddy land: 0.50 inch

b) Subsequent Soil-Moisture loss:

Paddy land and Non paddy land: under all conditions a constant rate of infiltration of 0.04"/hr or 1.00"/day is to be assumed.

c) Depression storage:

Paddy land: under all conditions the first $\frac{1}{2}$" of rain falling on paddy land is assumed to go into storage.

Non paddy land: A maximum constant storage rate of 0.033 inch/hour provided the rainfall is available. until a total of 1.00 inch is stored to be assumed.

8) Determination of rainfall excess (Runoff distribution)

Paddy land: The losses of rainfall computed by step 7 is to be separated from rainfall time distribution of step (6) by assuming the entire basin area consist of paddy land.

Non paddy land: The same process is to be repeated assuming the entire basin area consists of Non-paddy land.

9) Weighted Basin Average:

The net runoff of paddy and non paddy land of step (8) is to be multiplied by respective land classification percentage to get weighted runoff. The combined weighted runoff of paddy and non-paddy land will give the basin weighted runoff.

10) Computation of number of vents:

Once the basin weighted runoff is known, the total volume of water to be discharged through the sluice can be computed as follows:

Since 1" of rainfall excess over an area of 1 sq mile would generate 27 cfs, total discharge for the basin would be

Q = 27 x basin weighted runoff x area in square miles.

From tide-discharge curve volume of water to be discharged by each vent can be computed. If this discharge is q/vent. Then the no. of ventage required will be Q/q.

DESIGN OF BAMONDANGA SLUICE

HYDROLOGIC DESIGN

DESIGN DATA AVAILABLE

1.	Basin Area in sq. mile	: 5 sq. miles
2.	Purpose	: Drainage
3.	Average Ground Level	: + 3.50
4.	Highest water level (R/s)	: + 10.50
5.	Lowest water level (R/s)	: - 6.50
6.	Monsoon lowest water level (R/s)	: - 5.40
7.	Crest level of Embankment	: + 13.00
8.	Top width of Embankment	: 14'-0"
9.	C/s slope of Embankment	: 1:2
10.	R/s slope of Embankment	: 1:3

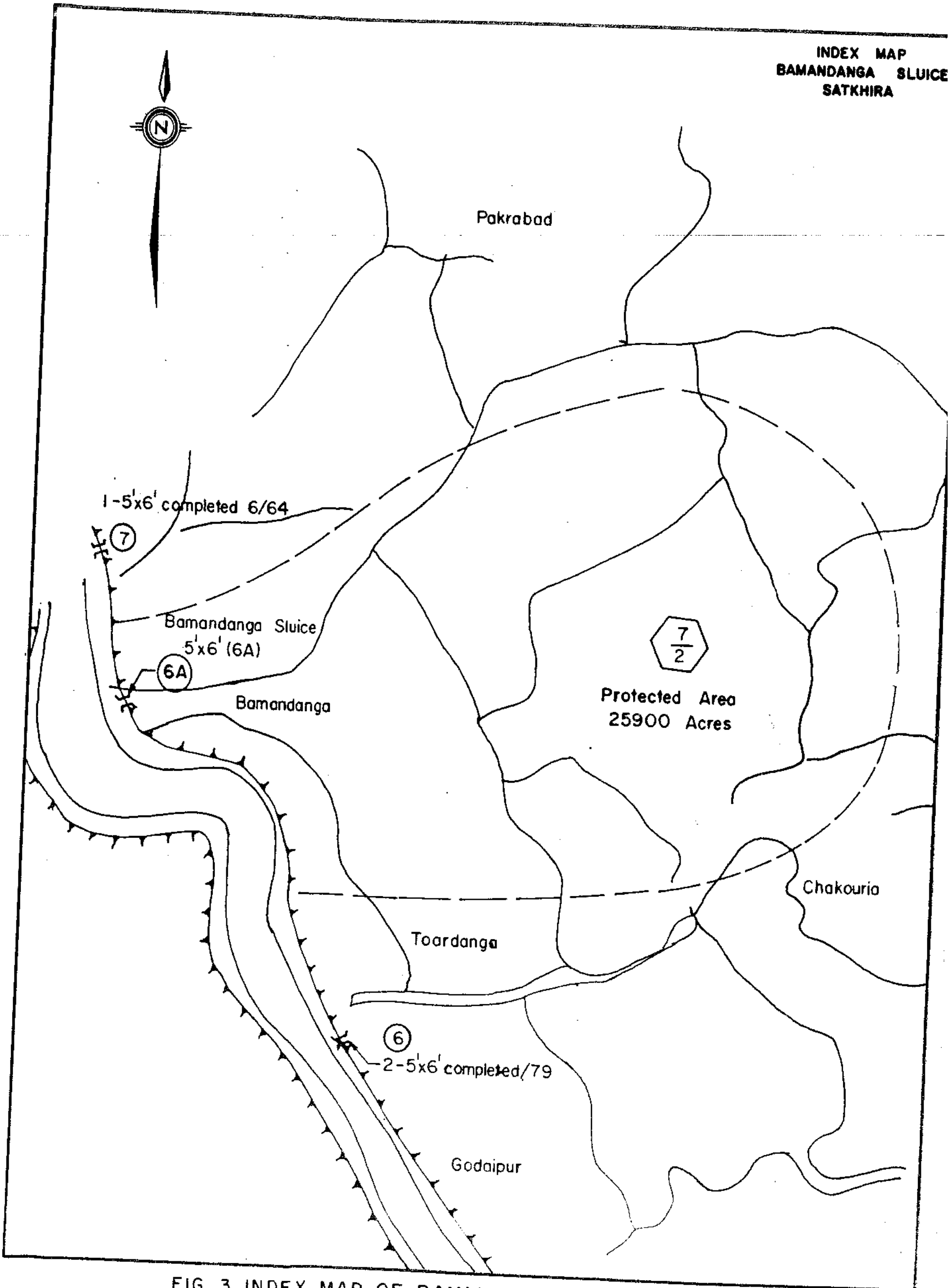


FIG. 3 INDEX MAP OF BAMANDANGA SLUICE

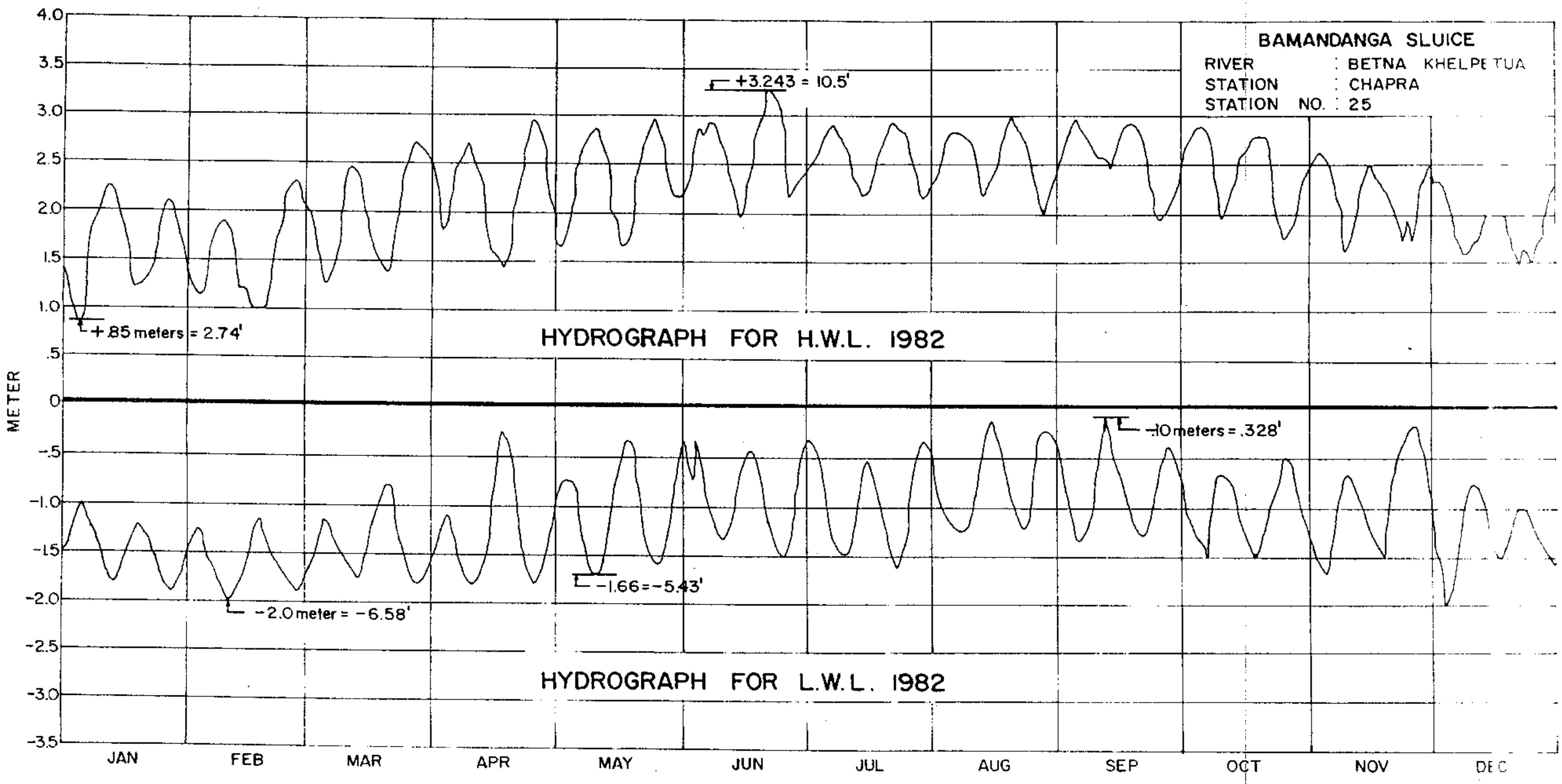
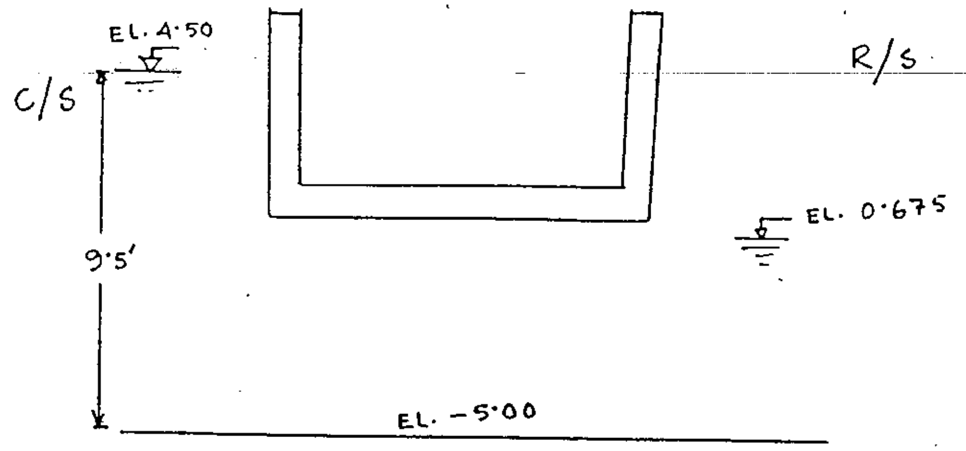


FIG. 4 HYDROGRAPH OF BETNA KHELPETUA RIVER AT CHAPRA



The highest W.L. on the C/s is assumed to be at EL.4.50 (Av.Ground level 3.50+1' = 4.50)

Considering the lowest tide level the invert level is fixed at EL. - 5.00.

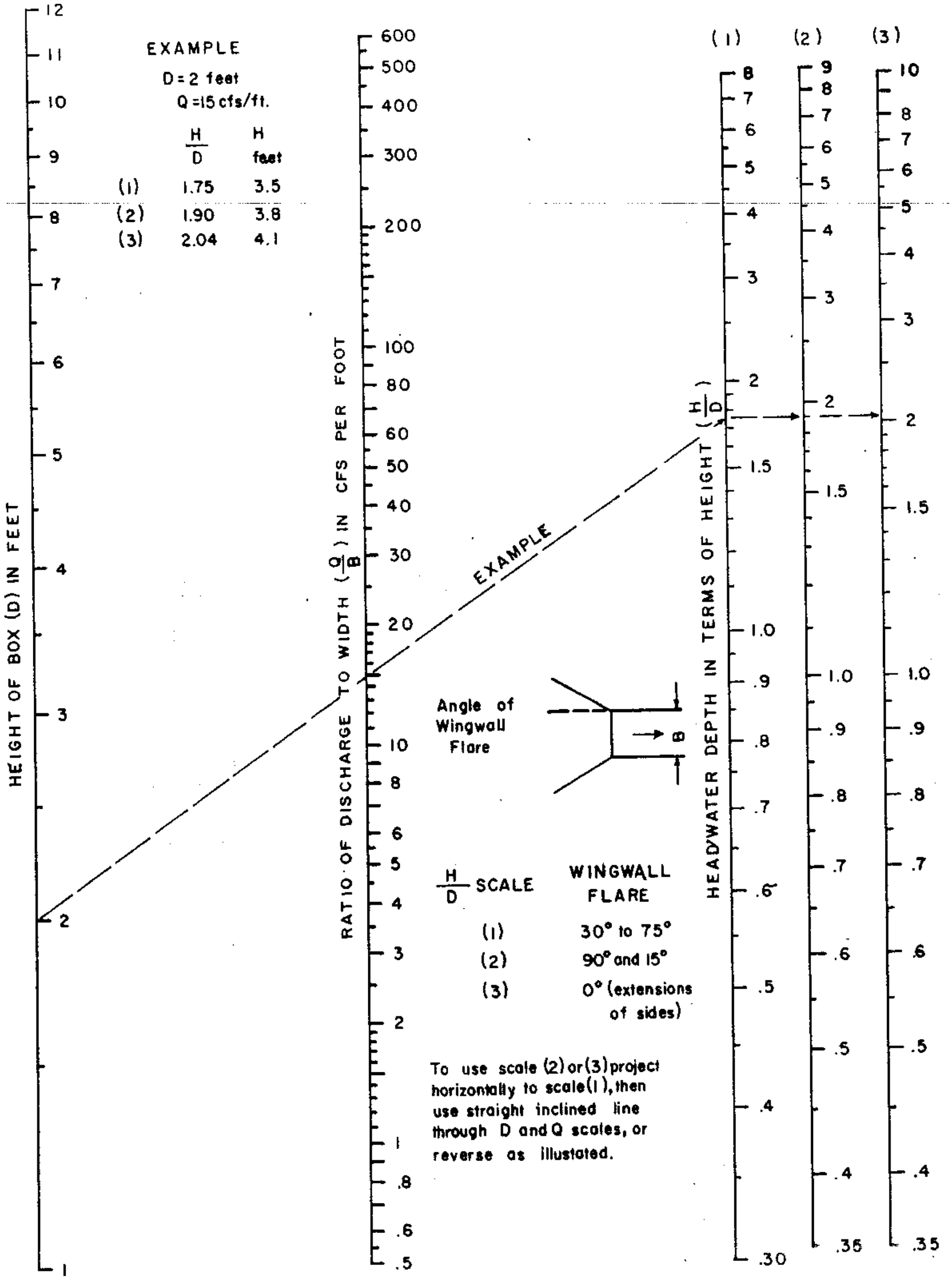
$$HW = 5.00 + 4.50 = 9.50'$$

From curve of U.S. Bureau of public roads for box culvert with entrance control (Fig. 5):

$$\frac{HW}{D} = \frac{9.50}{6} = 1.58$$

From Fig. 5 $\frac{Q}{B} = 70$ cfs/ft width

or $Q = 70 \times B = 70 \times 5 = 350$ cfs/vent



Headwater depth for box culverts with entrance control. (U.S. Bureau of Public Roads)

FIG. 5 DESIGN MONOGRAPH - FLOW THROUGH CULVERTS

Critical depth may be computed from the formula

$$Dc = 0.315 \left(\frac{Q}{B}\right)^{2/3} \quad \text{or} \quad Dc = \left(\frac{q^2}{g}\right)^{1/3}$$

$$= 0.315 \left(\frac{350}{5}\right)^{2/3} \qquad = \left(\frac{70^2}{32.2}\right)^{1/3}$$

$$= 5.35 \text{ ft.} \qquad = 5.34 \text{ ft.}$$

$$= 5.35'$$

The control remains at the entrance until the following elevation reached by the tailwater.

$$\frac{Dc + D}{2} = \frac{5.35 + 6}{2} = 5.675'$$ above from horizontal flow line of the structure, which is at elevation of + 0.675.

The discharge at all stages of tailwater from -5.00 to + 0.675 remains constant at 350 cfs/vent.

The discharge will be controlled by tail water from Elevation +0.675 to elevation + 4.5, the discharge decreases in a straight line from 350 cfs to zero.

The total volume under the discharge curve from one tide cycle and average for 24 hour period is computed from the curve as follows:

Time of constant discharge by Similar Triangle LMN & LBC

$$\frac{6}{15.50} = \frac{x}{5.675}$$

$$\text{or} \quad x = \frac{6 \times 5.675}{15.50} = 2.196 \text{ hour}$$

Time drainage reaches zero from change of tide to tide lockage, from similar triangles LMN & LQR

$$\frac{6}{15.5} = \frac{Y}{9.5}$$

$$\text{or} \quad Y = \frac{9.5 \times 6}{15.5} = 3.67 \text{ hrs.}$$

Total volume under drainage curve for one tide cycle

$$V = 2(350 \text{x} 2.196) + \frac{2(3.67 - 2.196) \times 350}{2} \times 3600$$

= 7391160 cft for 12 hours

= 169.67 = 170 Acre ft. 43560 cft = 1 Acre ft.

= 340 Acre ft. for 24 hours 1 cfs = 2 Acre ft.

= 170 cfs/vent for 24 hours

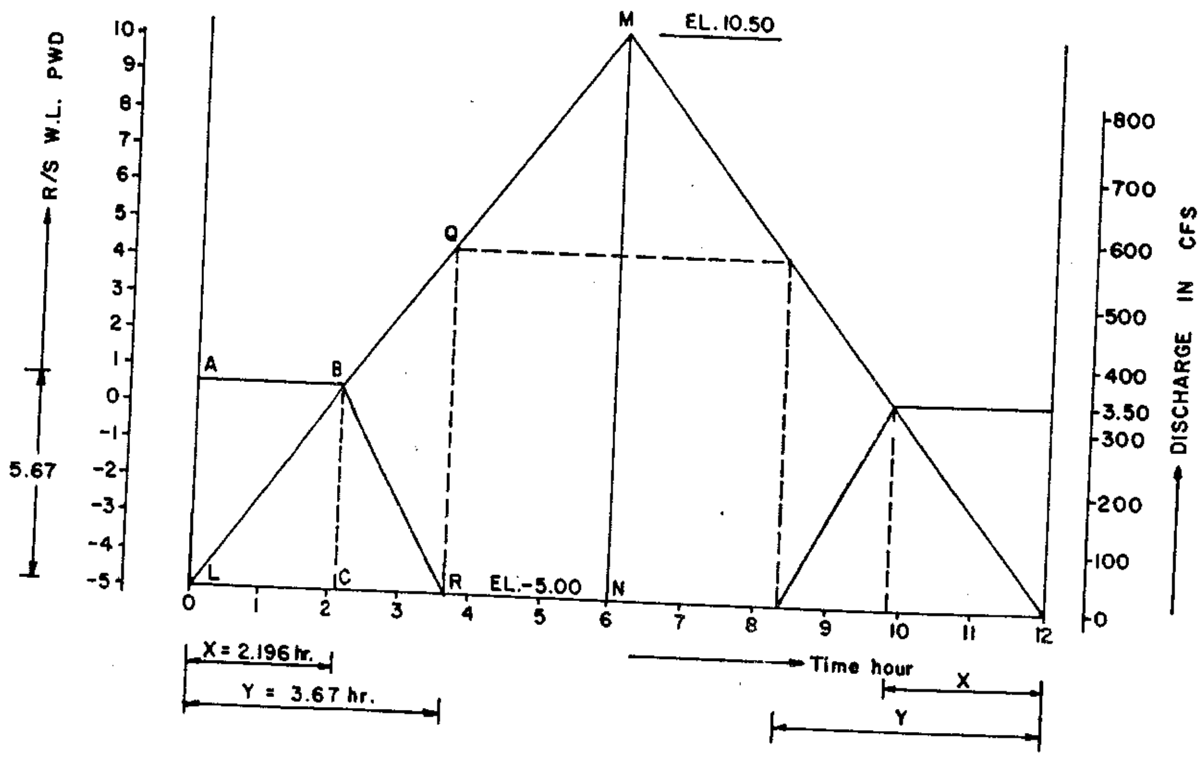


FIG. 6. 12 HOUR TIME DISCHARGE CURVE

Computation of Design runoff

From the isohyetal map the 4 (Fig. 7) month rainfall index for the project area is 50".

Rainfall indices of storm frequencies of 10 years and 25 years are shown in Table 1. The rainfall duration are taken for 5 days.

Point Rainfall

Point rainfall is the quantity of rain falling at a specific point, usually measured at a rain gaging station. We will define point rainfall as the product of the four month rainfall index and the combined rainfall indices. These rainfall volumes for the period from one to five days form the basis for our design storm.

Table 3. Storm percentage of point rainfall.

Days	Accumulative point rainfall (inch) = 4 month rainfall index rainfall indices
1	6.4"
2	9.6"
3	11.5"
4	12.9"
5	13.8"

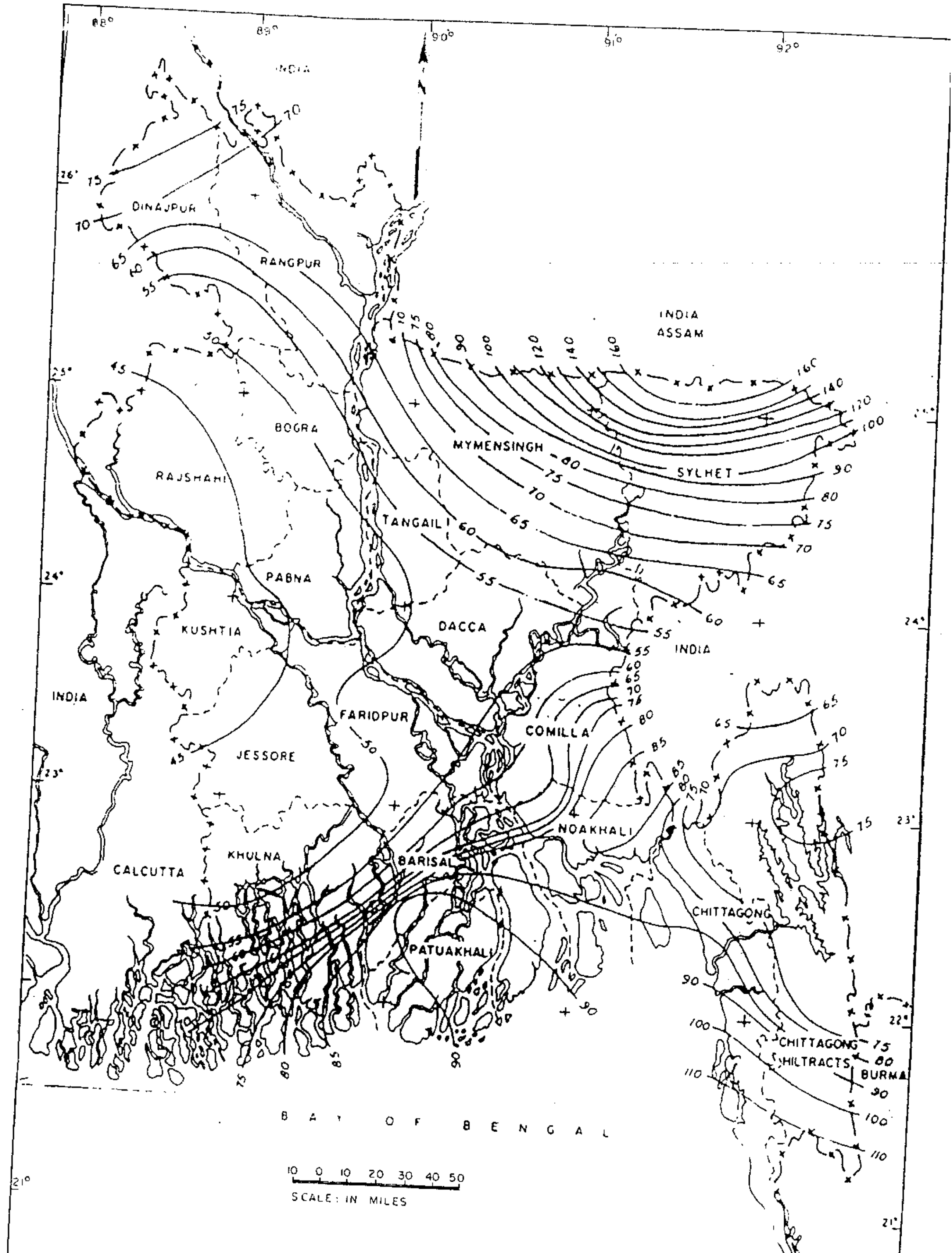


FIG. 7 4 MONTHS RAINFALL INDEX

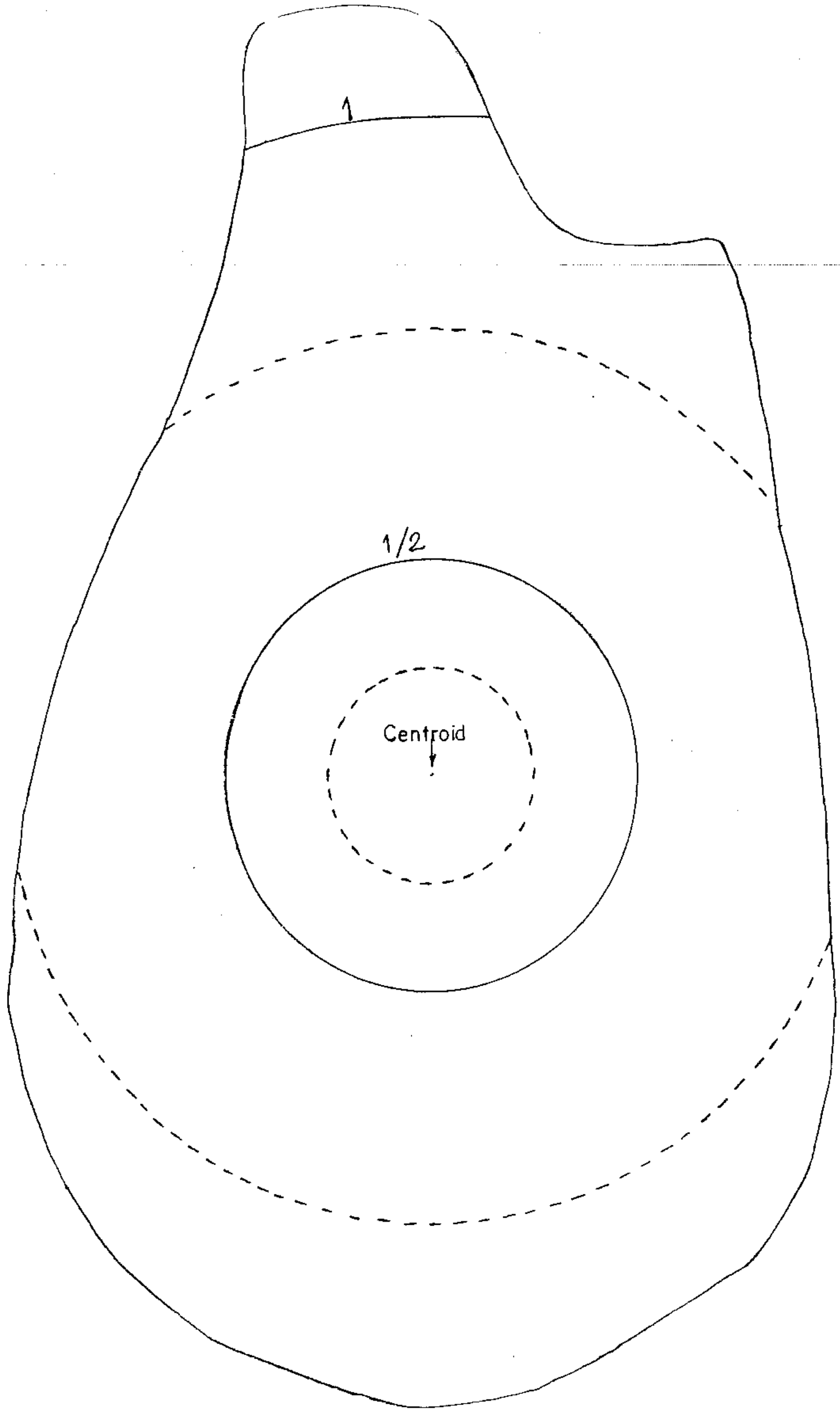


FIG. 8 Basin area showing centroid and isohyetals at even mile intervals

Area from isohyetals

Table 4. Area times storm percentage.

Average Distance From Storm Centre	Area Sq. miles	Area Times Percentage = Area x storm percentage
1/4	0.183	0.183
1	4.20	3.70
2	0.62	0.50
	5.00	4.39

The percentage of point rainfall that form the uniform equivalent depth of rainfall is

$$\frac{4.39}{5.00} \times 100 = 87.8\%$$

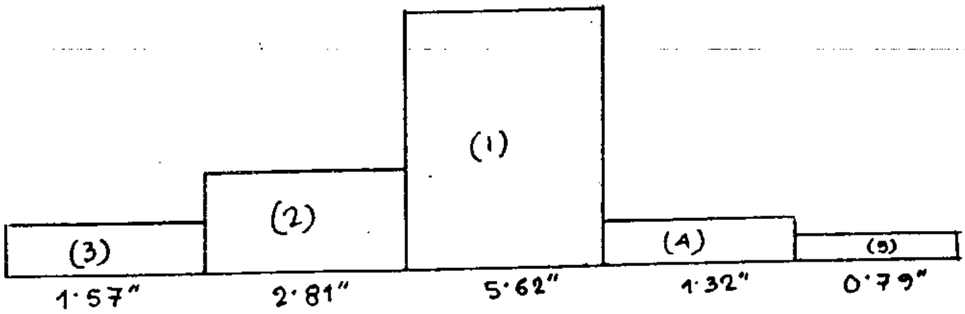
Equivalent uniform depth

The equivalent uniform depth is obtained by multiplying the point rainfall by 87.8%.

Table 5. Equivalent uniform depth.

Days	Equivalent uniform depth Accumulative Total	Daily increments
1	5.62	5.62
2	8.43	2.81
3	10.00	1.57
4	11.32	1.32
5	12.11	0.79

The daily increments of rainfall can occur in any order, we will arbitarily arrange a sequence of 3,2,1,4,5. A graphical arrangments of this sequence looks like this:



24 hour rainfall time distribution

We shall use the unit storm interval of 6 hours. The rainfall time distribution of Satkhira is not available. The available rainfall time distribution closest to project area is Jessore having 4 month rainfall index 46". So an adjustment is the ratio of 50/46 is to be made to get the rainfall time distribution for the project area.

Table 6. Rainfall time distribution.

	Hours			
	6	12	18	24
Jessore maximum accumulative point rainfall	3.80	4.43	5.15	5.90
Project area maximum accumulative point rainfall	4.13	4.82	5.60	6.41
Project area maximum incremental point rainfall	4.13	0.69	0.78	0.81
Project area uniform equivalent depth	3.63	0.60	0.69	0.71

The increments of rainfall within 24 hour period can not be predicted. We will arrange arbitarily a sequence of 3,4,1,2. Thus during the maximum 24 hour rainfall period the 6 hour incremental rainfall is 0.69", 0.71", 3.62" & 0.60".

Design Storm

Table 7. 5-day design storm with 6-hour interval.

Day	6 hour increment
1	0.69 0.71 0.00 0.17
2	0.69 0.71 0.81 0.60
3	0.69 0.71 3.63 0.59
4	0.69 0.00 0.00 0.63
5	0.69 0.00 0.00 0.10

Rainfall Excess - Paddy Land

Table 8. rainfall excess for paddy land.

Days	Hour	Rainfall inches	Losses in inches			Available paddy storage	Net runoff
			Soil moisture		Depression Storage		
			Initial	Subsequent			
1	0-6	0.69	0	-0.25	-4.00	-3.56	0
	6-12	0.71		-0.25		-3.10	0
	12-18	0.00		-0.25		-3.35	0
	18-24	0.17		-0.25		-3.43	0
2	0-6	0.69		-0.25		-2.99	0
	6-12	0.71		-0.25		-2.53	0
	12-18	0.81		-0.25		-1.97	0
	18-24	0.60		-0.25		-1.62	0
3	0-6	0.69		-0.25		-1.18	0
	6-12	0.71		-0.25		-0.72	0
	12-18	3.63		-0.25		0	2.66
	18-24	0.59		-0.25		0	0.34
4	0-6	0.69		-0.25		0	0.44
	6-12	0.00		-0.25		-0.25	0
	12-18	0.00		-0.25		-0.50	0
	18-24	0.63		-0.25		-0.12	0
5	0-6	0.69		-0.25		0	0.32
	6-12	0.00		-0.25		-0.25	0
	12-18	0.00		-0.25		-0.50	0
	18-24	0.10		-0.25		-0.65	0

Rainfall Excess - Non paddy Land

Table 9. Rainfall excess for non-paddy land.

Days	Hour	Rainfall inches	Losses in inches			Available non paddy storage	Net runoff
			Soil moisture		Depression Storage		
			Initial	Subsequent			
1	0-6	0.69	-0.50	-	-0.19	0	0
	6-12	0.71		-0.25	-0.20	0	0.26
	12-18	0.00		-0.25	0	-0.25	0
	18-24	0.17		-0.25		-0.33	0
2	0-6	0.69		-0.25	-0.11	0	0
	6-12	0.71		-0.25	-0.20	0	0.26
	12-18	0.81		-0.25	-0.20	0	0.36
	18-24	0.60		-0.25	-0.10/1"	0	0.25
3	0-6	0.69		-0.25	0	0	0.44
	6-12	0.71		-0.25	0	0	0.46
	12-18	3.63		-0.25	0	0	3.38
	18-24	0.59		-0.25	0	0	0.34
4	0-6	0.69		-0.25	0	0	0.44
	6-12	0.00		-0.25	0	-0.25	0
	12-18	0.00		-0.25	0	-0.50	0
	18-24	0.63		-0.25	0	-0.12	0
5	0-6	0.69		-0.25	0	0	0.32
	6-12	0.00		-0.25	0	-0.25	0
	12-18	0.00		-0.25	0	-0.50	0
	18-24	0.10		-0.25	0	-0.65	0

Rainfall Excess - Weighted Basin Runoff

Assuming paddy land 50% & non paddy land 50%
(From Land classification)

Table 10. Rainfall excess - weighted basin runoff.

Days	Hour	Paddy Land		Non Paddy Land		Basin Weighted
		Net runoff	Weighted runoff	Net runoff	Weighted runoff	
1	0-6	0	0	0	0	0
	6-12	0	0	0.26	0.13	0.13
	12-18	0	0	0	0	0
	18-24	0	0	0	0	0
2	0-6	0	0	0	0	0
	6-12	0	0	0.26	0.13	0.13
	12-18	0	0	0.36	0.18	0.18
	18-24	0	0	0.25	0.125	0.125
3	0-6	0	0	0.44	0.22	0.22
	6-12	0	0	0.46	0.23	0.23
	12-18	2.66	1.33	3.38	1.69	3.02
	18-24	0.34	0.17	0.34	0.17	0.34
4	0-6	0.44	0.22	0.44	0.22	0.44
	6-12	0.00	0	0	0	0
	12-18	0.00	0	0	0	0
	18-24	0.00	0	0	0	0
5	0-6	0.32	0.16	0.32	0.16	0.32
	6-12	0	0	0	0	0
	12-18	0	0	0	0	0
	18-24	0	0	0	0	0
						5.13"

Total runoff = 5.13"

No. of vents required

Now from hydraulic computation, the basin weighted runoff is 5.13" for 5 days storm

So the runoff per day = 5.13/5 = 1.026" per day.

So the total discharge = 27 x 1.026 x 5

= 138.51 cfs.

Discharge/vent = 170 cfs. (Ref: page)

$$\text{No. of ventage required} = \frac{138.51}{170}$$

= 0.814 vents.

= 1 vent.

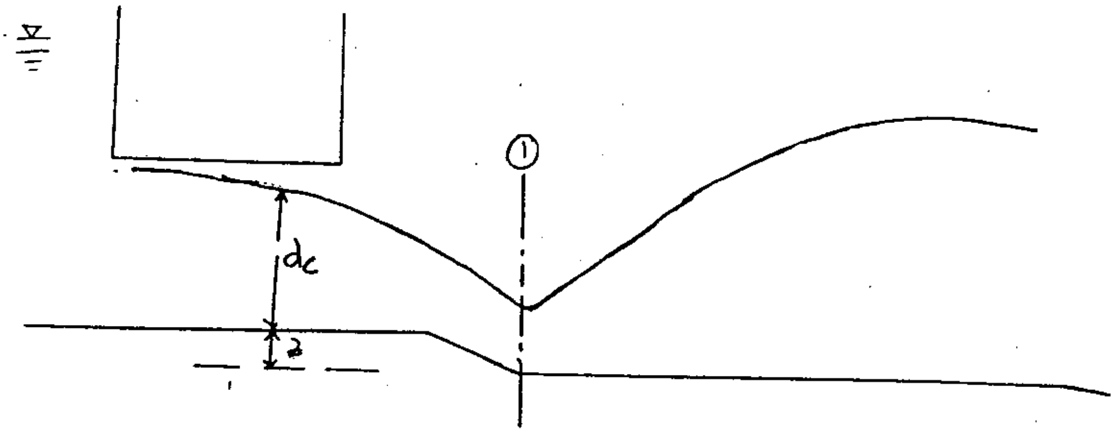
Hydraulic Design

Floor length by hydraulic jump

Q = 350 cfs. (From hydraulic analysis)

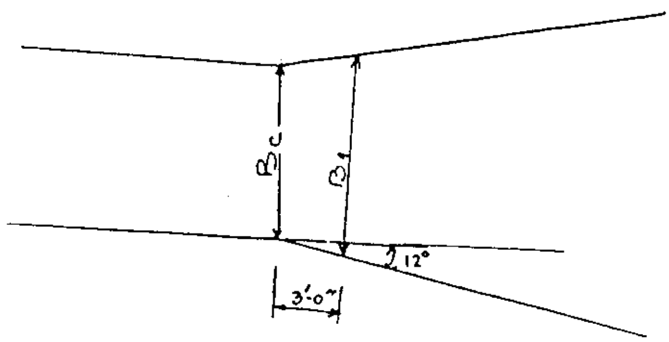
(This discharge is constant for 2.19 hours)

$$\text{discharge/ft width} = \frac{350}{5} = 70 \text{ cfs.}$$



$$d_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{70^2}{32.2}\right)^{1/3} = 5.33 \text{ ft.}$$

$$V_c = \frac{q}{dc} = \frac{70}{5.33} = 13.13 \text{ ft/sec.}$$



B_c = 5' - 0"

B_1 = $5 + 2 \times 3 \tan 12^{o}$ = 6.27'

Applying Bernoulli's Equation at sections (1) and (c)

$$d_c + \frac{Vc^2}{2g} + Z_c = d + \frac{V_1^2}{2g}$$

[At the critical state flow velocity head is equal to half the hydraulic depth.]

$$\text{or} \quad 1.5\, d_c + 1 = d_1 + \frac{V_1^2}{2g} \qquad \text{- - - - - - - - - - - -} \quad (1)$$

At point (1) the discharge/ft width q_1 is given by

$$q_1 = \frac{70 \times 5}{6.27} = 55.82 \text{ cfs/ft.}$$

From equation (1)

$$1.5 \times 5.33 + 1 = d_1 + \frac{55.82^2}{d_1^2 \times 2g} \qquad [\, V = \frac{V_1}{d_1} \,]$$

$$\text{or} \quad 7.99 + 1 = d_1 + \frac{48.38}{d_1^2}$$

$$\text{or} \quad 8.99 = d_1 + \frac{43.38}{d_1^2}$$

By trial $d_1 = 2.80'$

$$\text{Now} \quad V_1 = \frac{q_1}{d_1} = \frac{55.82}{2.80} = 19.93 \text{ ft/sec.}$$

$$\text{Froude No.} \quad F = \frac{V_1}{gd_1} = \frac{19.93}{32.2\text{x}2.80} = 2.09$$

Length of Jump

We know $\frac{d_2}{d_1} = \frac{1}{2}[(1 + 8F^2)^{1/2} - 1]$

or $\frac{d_2}{d_1} = \frac{1}{2}[(1 + 8 \times 2.09^2)^{1/2} - 1]$

or $\frac{d_2}{d_1} = 2.49$

or $d_2 = 2.49\, d_1 = 2.49 \times 2.80$

or $d_2 = 6.94$ ft.

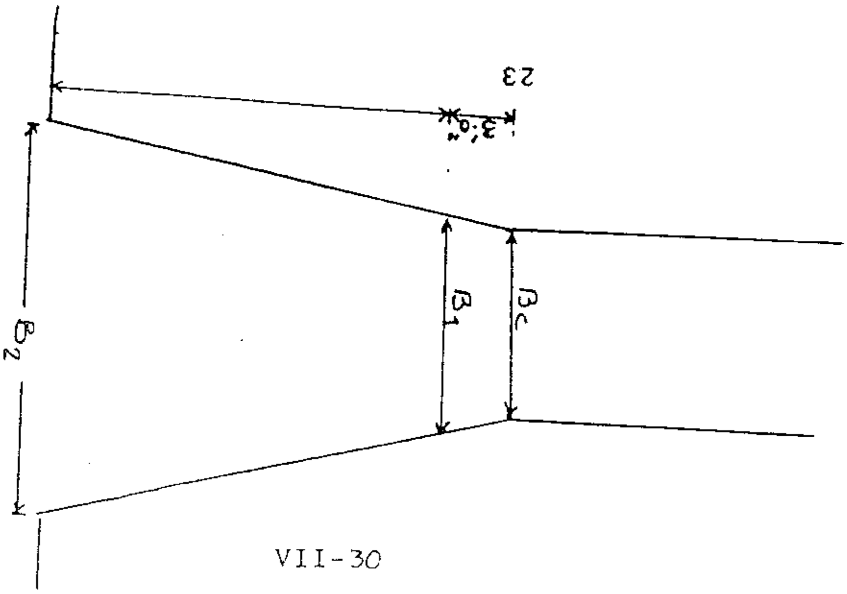
Length of Jump $= 6.9\,(d_2 - d_1)$

$= 6.9\,(6.94 - 2.80)$

$= 28.56$

$= 29\text{-}0"$

We may provide 29'+ 3' = 31'- 0" floor length considering hydraulic jump.



$$B_2 = B_c + 2 \times 31 \tan 12^o$$

$$= 5 + 13.17$$

$$= 18.17$$

Provide 20 - 0"

discharge/ft width at end

$$q_2 = \frac{350}{20} = 17.5 \text{ cfs/ft.}$$

<u>Scour depth with respect to hydraulic jump</u>

$$R = 0.91 \left(\frac{q_2^2}{f} \right)^{1/3}$$

f = silt factor
= 1.76 dm

where dm = Average diameter of particle in mm.

$$= 0.91 \left(\frac{17.5^2}{0.40} \right)^{1/3}$$

$$= 8.32'$$

<u>Design scour depth</u>

U/s = 1.25xR = 1.25x8.32 = 10.40 ft.

D/s = 1.5 xR = 1.5 x8.32 = 12.48 ft.

<u>Scour level</u>

U/s = U/s W.L. - 10.40

= 3.50 - 10.40

= -6,90

(Considering C/s water level at average ground level)

It means the cut off depth required = -5-(-7.93)

= 1.90' = 2'-0"

D/s scour level = D/s W.L. - 12.48

= 0.675 - 12.48

= -11.80

Depth of cutoff required = -6- (-11.80)

= 5.8' = 6'-0"

Floor length and cutoff by exit gradient

In calculating the floor length and depth of cutoff by exit gradient the water level on the R/s will be considered to be at highest tide level while the water level on the C/s to be at retention level to get the maximum head difference.

Now H.W.L. (R/S) : 10.50

Retention level (C/s) : 4.50

Head difference H = 10.50 - 4.50

= 6'- 0"

Considering the safe exit gradient 1/7. Taking maximum head difference 6'-0". Assuming depth of cutoff 6'-0" at C/s.

$$G_E = \frac{H}{d} \times \frac{1}{}$$

$$\text{or} = \left(\frac{H}{G_E \cdot x\ d.x}\right)^2$$

$$\text{or} \quad = \left(\frac{6 \times 7}{6 \times}\right)^2$$

$$= \quad 4.96$$

$$\text{again} \quad = \quad \frac{1 + \quad 1+}{2}$$

$$\text{or} \quad 1 + \quad 1 + \quad ^2 \quad = \quad 2$$

$$\text{or} \quad 1 + \quad ^2 \quad = (2 \quad - 1 \ )^2$$

$$\text{or} \quad = \quad (2 \quad - \quad 1)^2 \ - 1$$

$$= \ (2 \times 4.96 - 1)^2 \ - 1$$

$$= \quad 8.88$$

$$\text{Now} \quad = \ \frac{b}{d}$$

$$\text{or } 8.88 = \ \frac{b}{6}$$

or b = 53'-0"

It is seen that if total floor length is 53'- 0" the depth of cutoff 6' will satisfy the existence of the structure from exit gradient consideration.

The total length required from hydraulic jump consideration.

Total floor length = U/s floor length + barrel + glacis + D/s floor.

For barrel and gate fixing = 24'-0"

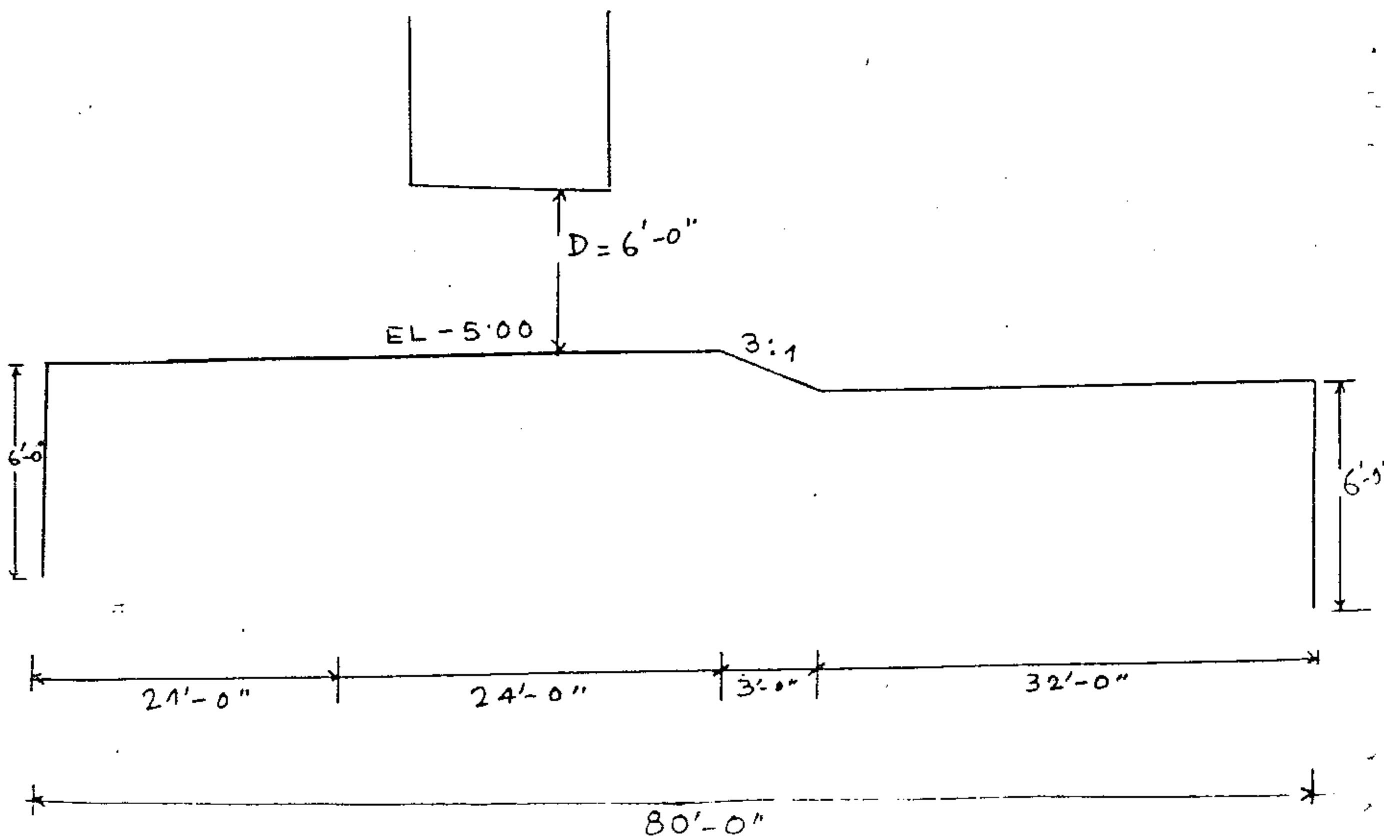
glacis = 3' - 0"

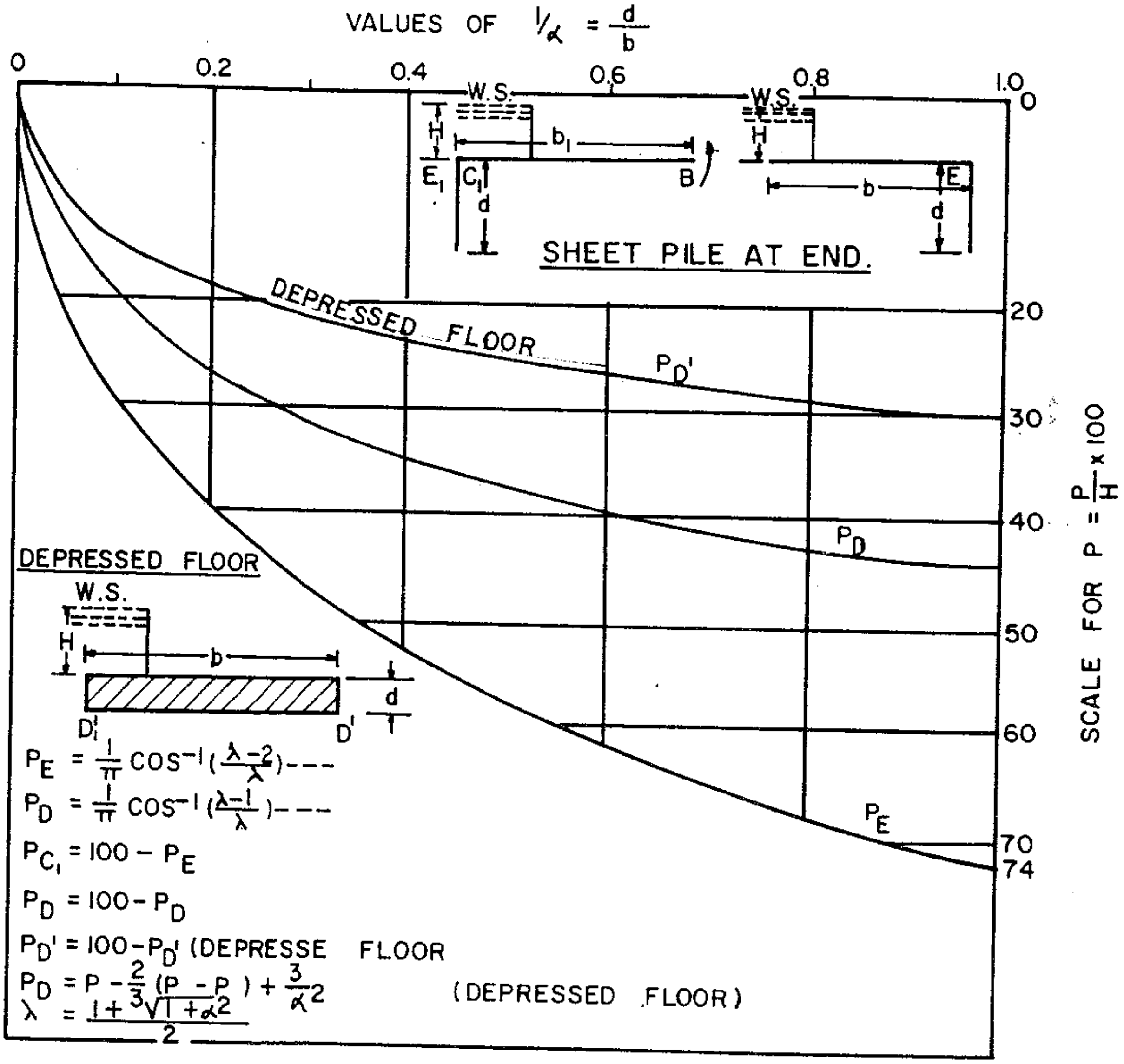
D/s floor = 29' - 0"

U/s floor = 20' - 0"

Total = 76'- 0"

Let us provide total floor length 80' - 0" with D/s floor 32' U/s floor 21', barrel 24' and glacis 3' and both U/s and D/s cutoff depth 6'




VALUES OF 1/α = d/b
0
0.2
0.4
0.6
0.8
1.0
W.S.
H
b1
E1
C1
B
d
b
E
SHEET PILE AT END.
DEPRESSED FLOOR
PD'
PD
PE
0
20
30
40
50
60
70
74
SCALE FOR P = P/H x 100
DEPRESSED FLOOR
W.S.
H
b
d
D'1
D'
PE = 1/π cos⁻¹((λ−2)/λ)---
PD = 1/π cos⁻¹((λ−1)/λ)---
PC1 = 100 − PE
PD = 100 − PD
PD' = 100 − PD' (DEPRESSE FLOOR
PD = P − 2/3 (P − P) + 3/α2 (DEPRESSED FLOOR)
λ = (1 + 3√(1 + α2))/2

__Floor thickness in relation to uplift pressure__
(Gate closed, R/s controlling)

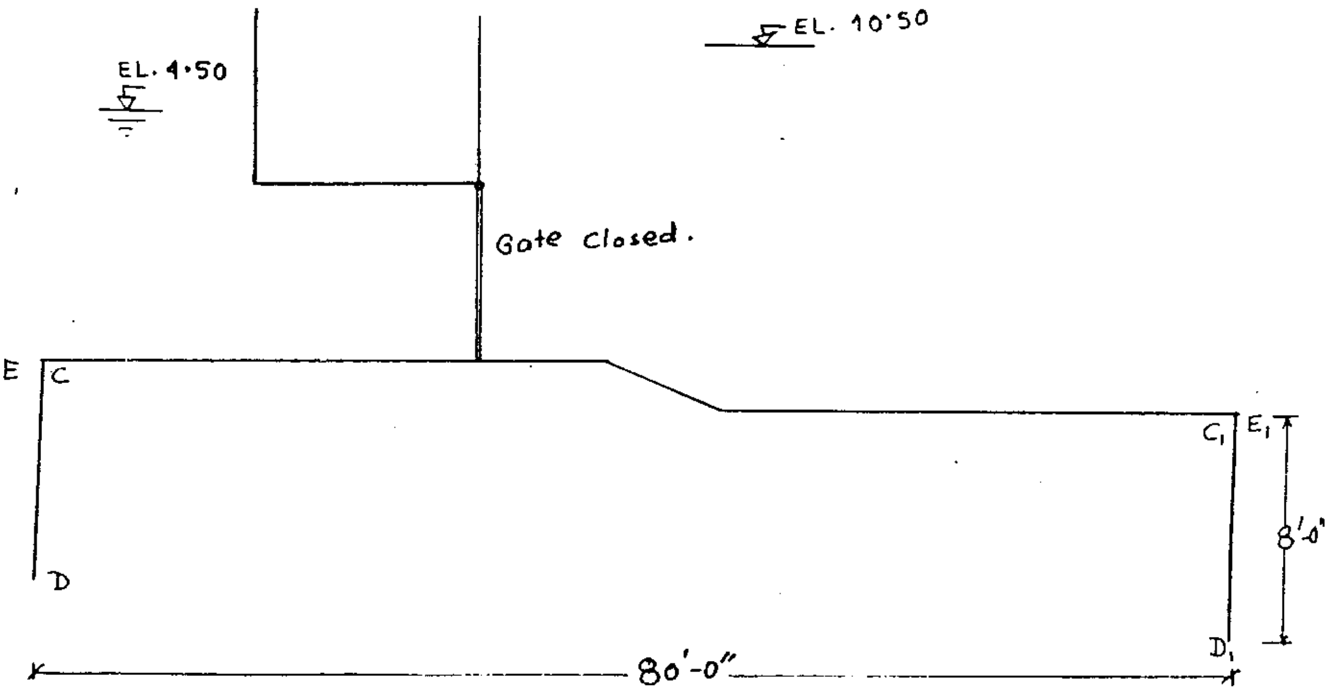
Now we have already fixed

Floor length b = 80' - 0"

R/s depth of cutoff d = 6' - 0"

C/s depth of cutoff d = 6' - 0"

Let us assume floor thickness at U/s and D/s end 1' - 6"



$$\frac{1}{\alpha} = \frac{d}{b} = \frac{6}{80} = 0.075$$

From Khosla's pressure chart. (Fig. 9)

$P_D = 16\%$ $\qquad$ $P_{D1} = 100 - 16 = 84\%$

$P_E = 26\%$ $\qquad$ $P_{C1} = 100 - 26 = 74\%$

Correction due to interference of cutoff

Let C be the correction to be applied as percentage of head.

b = distance between two piles

D = depth of cutoff the influence of which has to be determined on the neighbouring cutoff of depth d

d = depth of cutoff on which the effect of cutoff of depth D is sought to be determined.

b = total length of floor.

Correction due to interferece of C/s cutoff on to R/s cutoff

$$C = 19 \left(\frac{D}{b}\right)^{1/2} \times \frac{d + D}{b}$$

in our case D = d = 4.5

$$C = 19 \left(\frac{4.5}{78}\right)^{1/2} \times \frac{4.5+4.5}{80}$$

$$= 0.51\%$$

This correction is positive for points in the rear or back water and subtractive for points forward in the direction of flow.

Correction due to floor thickness (R/s)

Floor thickness at end is assumed 1' - 6".

If C_F is the correction for floor thickness

$$\text{Then } C_F = \frac{84 - 74}{6} \times 1.5 = 2.5\% \text{ (+ve)}$$

$P_{c1 \text{(Corrected)}} = 74 + 0.51 + 2.5 = 77\%$

<u>Correction for floor thickness C/s</u>

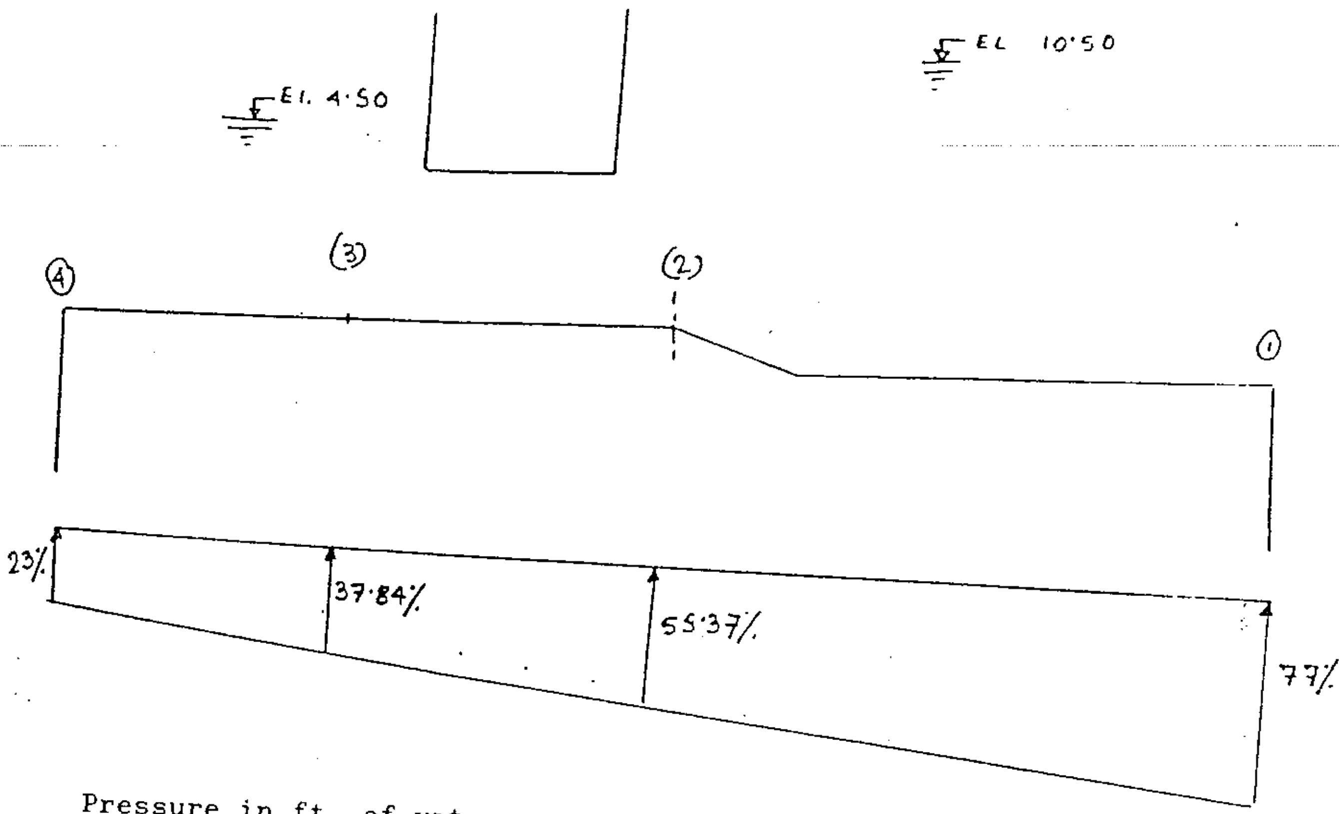
1) Correction due to interference of R/s cutoff on to C/s cutoff will be the same as before as the cutoff depth and floor length is same. But the the direction will be reverse. i.e. C = 0.51% (-ve).

2) Correction for floor thickness $= \frac{26-16}{6} \times 1.5 = 2.5\%$ (-ve)

$$P_{E \text{ (Corrected)}} = 26-0.51-2.5$$
$$= 23\%$$

The uplift pressure while the gate is open need not to calculate as the head difference is small as a result the uplift pressure will also be negligible. This happens as because while the gate is open during low tide the water level on the country side falls down discharging through the sluice as the tide level comes down resulting a small head difference.

Pressure as percentage of head difference



Pressure in ft. of water

$$P_1 = \frac{77}{100} \times 6 = 4.61 \text{ ft.}$$

$$P_2 = \frac{55.37}{100} \times 6 = 3.32 \text{ "}$$

$$P_3 = \frac{37.84}{100} \times 6 = 2.23 \text{ "}$$

$$P_4 = \frac{23}{100} \times 6 = 1.38 \text{ "}$$

Here the points (3) and (4) are critical. The uplift pressure of water at points (1) and (2) are is 4.61' and 3.32' respectively while the downward water pressure is 6' - 0". So theoretically no thickness is necessary. At points (3) and (4) the uplift water pressure due to head difference is 2.23' and 1.38' respectively which is to be balanced by equivalent weight of concrete and necessary thickness of concrete is to be provided.

Thickness of concrete

at point (3) $t_3 = \dfrac{2.23}{1.40} = 1.57$ ft.

at point (4) $t_4 = \dfrac{1.38}{1.40} = 1.78$ ft.

So our assumption of thickness at end is near to the calculated value and on safe side. We shall provide the thickness of concrete at end 1' - 6" and at glacis 2' - 0". At the barrel part the downward weight of barrel and soil will be more than sufficient to balance the uplift pressure. So at the barrel part a normal thickness of 2' - 0" may be provided.

Here the thickness is calculated only by considering the highest tide level at R/s and retention level on C/s. In the reverse case while the R/s water level becomes lowest i.e. at the lowest tide level the C/s water level will also be at very low level and the head difference will be very less, consequently the required thickness will also be very less. So the thickness as required by condition (1) is to be provided at both ends.

CHAPTER VIII

HYDRAULIC DESIGN OF FLUSHING REGULATOR

Flushing Regulator

Purpose:

Flushing Regulator is constructed for the following purposes:

a) Flushing of irrigation water

b) Prevention of early flood

c) Prevention of flood

The design of the flushing regulator is carried out through the following steps:

1) Ventage computation:

This will fix the required discharge capacity of the structure according to the irrigation water requirement and the size and no. of ventage of the structure is to be fixed accordingly.

2) Hydraulic Computation:

This will give the required dimensions of different parts of the structure. Such as the length of U/s and D/s apron, thickness of apron, depth of cutoff wall etc.

3) Structural Design:

After the completion of hydraulic analysis and hydraulic design, the structural design is to be done which will give the detail dimensions of various parts of the structure at any point, reinforcements etc.

1) Ventage Computation:

The number and sizes of ventage to be provided for the area to be irrigated depends on the crop water requirement. To find out the crop water requirement one must have clear idea of the following hydrological terms.

a) Evapotranspiration:

Evapotranspiration is the total evaporation, i.e. evaporation from all water, soil, vegetation and other surfaces plus transpiration. Climate is one of the most important factors determining the amount of water loss by evapotranspiration from the crop. A reference value of evapotranspiration (E_{To}) will be required later on which may be defined as "the rate of evapotranspiration from an extended surface of 8 to 15 cm tall green grass cover of uniform height, actively growing, completely shading the ground and not short of water".

b) Crop water requirement:

Crop water requirement (E_T) is defined as "the amount of water potentially required to meet the evapotranspiration needs of vegatative areas so that plant production is not limited from lack of water". This may also termed as crop evapotranspiration. E_T (crop) for a selected crop is given by

$$E_T = K_c \times E_{To}$$

Where K_c = Crop co-efficient

What is Kc?

Crop co-efficient K_c are presented to relate E_{To} to crop evapotranspiration or crop water requirement E_T (crop). The K_c value represents evapotranspiration of a crop grown under optimum conditions producing optimum yields.

c) Net irrigation requirement:

Net irrigation requirement for a given crop or cropping pattern is the deficit of soil water balance. The main variables composing the soil water balance to be considered include i) crop water requirements ii) contribution from precipitation iii) ground water and iv) carry over of soil water and where applicable v) in and outflow of water, either surface or sub-surface.

Net irrigation water may be expressed as

$$In = \underbrace{(Er\ (crop) + F + R)}_{\text{Losses}} - \underbrace{(Pe + Ge + N + W)}_{\text{gains}}$$

where

In = Net irrigation requirement

Er = crop water requirement or evapotranspiration(crop)

F = deep percolation

R = surface or sub-surface out flow

Pe = contribution to root zone by rainfall

Ge = contribution to root zone by ground water

N = surface or sub-surface inflow

W = change in soil water content in the root zone

For practical purposes net irrigation requirement is taken as crop water requirement i.e. In = Er (crop)

d) Effective rainfall:

Effective rainfall is only a portion of total rainfall. Part of rain may be lost by surface runoff, by deep percolation below root zone. In regions with heavy and high intensity rains only a portion can enter and be stored in the root zone and the effectiveness of rain is consequently low.

Value of E_T & K_C

The value of monthly crop water requirement were calculated by the radiation method for different zones of Bangladesh and tabulated in table (1) (Table attached). The value Kc were taken from Crop Water Requirement Irrigation & Drainage paper 24 by FAO and tabulated in Table 2.

e) Field irrigation requirement

Field irrigation requirement is the amount of water and timing of its application needed to compensate soil water deficits i.e. the amount of water to compensate the net irrigation requirement In. Irrigation is never 100% effective and allowance must be made for unavoidable or avoidable losses including deep percolation, surface runoff and other managerial and technical faults. Irrigation application efficiency Ea normally expressed in fraction or percentage of In is applied to get field irrigation requirement if.

$$If = \frac{In}{Ea}$$

$$\text{Field irrigation requirement} = \frac{\text{Net irrigation requirement}}{\text{Field application efficiency}}$$

f) <u>Project Diversion Requirement or Irrigation Supply requirement : Distribution efficiency</u>

To quantify the supply needed to meet irrigation requirement (irrigation supply requirement) at the field the efficiency of distribution system must be determined. Distribution efficiency, Ed, is determined as that portion of water released at the headworks and that received at the field inlet.

The main factors deciding distribution efficiency are method of water delivery (continuous, rotation, demand) size of project area and effectiveness of management organization. Distribution efficiency can be sub divided into conveyance efficiency, Ec, between headworks and the inlet to a block of fields, & field ditch efficiency, Eb between main canal and inlet of an individual field. Distribution efficiency is obtained from Ed = Ec× Eb.

$$E_c = \frac{\text{Field irrigation requirement}}{\text{Project diversion requirement}}$$

$$E_b = \frac{\text{Net irrigation requirement}}{\text{Field irrigation requirement}}$$

$$E_c \times E_b = \frac{\text{Field irrigation requirement}}{\text{Project Diversion Requirement}} \times \frac{\text{Net irr.requirement}}{\text{Field irr.requirement}}$$

$$E_d = \frac{\text{Net irrigation requirement}}{\text{Project diversion requirement}}$$

or

$$\text{Project diversion requirement} = \frac{\text{Net irrigation requirement}}{Ed}$$

Design discharge capacity for inlet structure

The design discharge capacity for an irrigation inlet structure is the irrigable area multiplied by the overall project duty (cfs/acres).

Now the overall project duty i.e. gross project diversion requirement per acre of irrigable area is required to be computed.

The necessary data taken for the bridge cum flushing regulator over SHAKARIA KHAL is taken as follows:

Project Area : 7000 Acres

Cultivable Area : 6000 Acres

Irrigable Area : 1800 Acres

Efficiencies and land preparation as follows:

Land preparation : Paddy Land - 7"

Others 3"

Field application efficiency: Paddy crops - 50%

Others - 75%

Conveyance efficiency: Paddy crops - 80%

Others - 70%

In our case project duty and project diversion requirement have been calculated as follows:

1. The irrigation period for the paddy is between June to November

2. Crop water requirement/Net irrigation requirement (neglecting dependable rainfall due to short duration draught)

$$E_{T(crop)} = K_c \times E_{To}$$

$$E_T = 1.25 \times \frac{5.70}{30} = 0.243"/day$$

Ref: Table (2)

The crop co-efficient used above has been taken maximum during the growing period.

Table-1
POTENTIAL EVAPOTRANSPIRATION AND DEPENDABLE RAINFALL
IN BANGLADESH

	REGION	1	2	3	4	5
JAN	ETP	3.5 in	3.4 in	3.5 in	2.1 in	2.3 in
	DR	0.0 in	0.0 in	0.0 in	0.0 in	0.0 in
FEB	ETP	5.0	4.5	4.9	3.4	4.0
	DR	0.1	0.1	0.1	0.1	0.1
MAR	ETP	7.2	6.6	6.8	5.2	5.1
	DR	0.1	0.2	0.5	1.0	1.8
APR	ETP	8.0	7.2	5.9	5.8	5.4
	DR	0.4	1.2	2.8	5.7	1.8
MAY	ETP	6.8	6.3	6.2	5.9	5.6
	DR	5.0	3.9	6.4	13.5	6.0
JUNE	ETP	5.7	5.4	5.7	5.2	5.1
	DR	7.1	7.3	8.9	15.1	15.0
JULY	ETP	6.2	5.2	5.7	5.2	5.1
	DR	9.1	9.7	10.4	9.8	16.0
AUG.	ETP	5.4	4.9	5.6	5.0	4.8
	DR	8.7	7.7	10.4	9.9	15.2
SEP.	ETP	4.8	4.3	5.0	4.3	4.4
	DR	7.2	5.7	6.9	8.0	8.8
OCT.	ETP	4.4	4.1	4.6	3.7	3.9
	DR	3.2	2.6	2.8	3.7	5.1
NOV.	ETP	3.7	3.5	3.7	2.6	3.3
	DR	0.0	0.0	0.0	0.0	0.0
DEC.	ETP	3.4	3.1	3.4	2.3	3.1
	DR	0.0	0.0	0.0	0.0	0.0

Region 1 : Rangpur, Dinajpur, Bogra, Rajshahi, Pabna
Region 2 : Kushita, Jessore, Faridpur, Khulna
Region 3 : Tangail, Dacca, Comilla, Mymensingh, Jamalpur
Region 4 : Sylhet
Region 5 : Chittagong, Noakhali, Patuakhali, Barisal
.... Potential evapotranspiration(ETP)from green grass
.... Dependable Rainfall, (DR), 75% probability of being equalled or exceeded.

3. Field irrigation requirement

$$I_f = \frac{\text{Net irrigation requirement}}{\text{Field application efficiency}}$$

$$= \frac{0.243}{0.50} = 0.487"/\text{day}$$

4. Project diversion requirement $= \dfrac{\text{Field irrigation requirement}}{\text{Conveyance efficiency}}$

$$= \frac{0.487}{0.80} = 0.61"/\text{day}$$

Table 2 gives coefficients (KC) for multiplying ET_o to find the evapotranspiration of specific crops. The given KC values are for the maximum monthly water use during the peak stage of plant growth. The value of E_{To} was taken from Table (1).

Table-2
CROP EVAPOTRANSPIRATION COEFFICIENTS
AND EFFECTIVE ROOT ZONE DEPTH

Crop	Effective Root Depth	Coefficient (KC)
Flooded rice	-	1.25
Wheat & grains	3.0 ft.	1.1
Sugarcane	3.0	1.1
Bananas	4.0	1.0
Pulses	2.0	1.1
Vegetables	2.0	1.1
Vine crops, melons	2.5	1.0

* Crop evapotranspiration is calculated by multiplying KC by EPT given in Table 1. The KC values given are for the crop at maximum vegetative growth stage. Evapotranspiration before and after will be less.

Now 0.61" of water over one acre land gives a volume

$$\text{of } \frac{0.61}{12} = 0.051 \text{ acre ft.}$$

We know 2 acre ft. = 1 cusec day

0.051 acre ft = 1/2 x 0.051

= 0.0255 cusec day

Now the area that can be irrigated by one cusec day

$$\text{i.e. duty} = \frac{1}{0.0255} \text{ Acre} = 39.21 \text{ Acres}$$

Let us take duty = 40 acres/cusec

5. Design discharge for the irrigation inlet structure

$$= \frac{\text{Irrigable area}}{\text{Overall project duty}}$$

$$= \frac{1800}{40} = 4.5 \text{ cfs}$$

Considering cultivable area

$$\text{Q division} = \frac{6000/}{40} = 150 \text{ cfs}$$

i.e. for diversion requirement of 0.61"/day/acre we require to have the capacity of 0.0255 cusec-day. If we increase the capacity 1 cusec-day we can irrigate 1/0.0255 acres.

So 39.21 acres can be irrigated by 1 cusec-day our required irrigable area 6000 acres can be irrigated by $\frac{6000}{40}$ = 150 cfs. with a factor of safety of 2.00 considering the further extension of the project the design capacity of the structure may be taken 300 cfs.

Computation for ventage requirement

The size and number of ventage may be calculated as follows:

Given

1) Design discharge = 300 cfs (Fom hydraulic analysis)

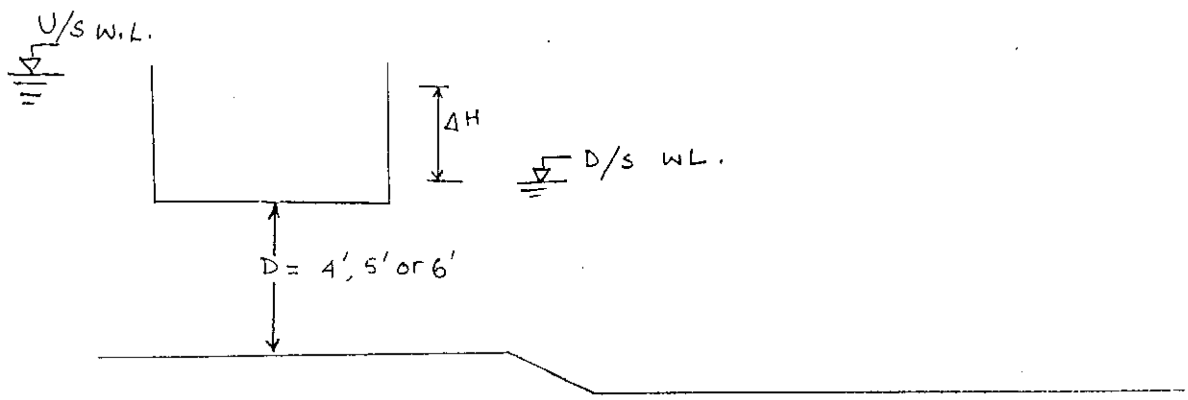
2) Bank level or Ground level : 41.00 PWD

3) The head difference between inlet & outlet under submerged condition shall be between 6" to 9".

4) When the head difference will be higher than 6", the country side water level and the design flow will be maintained by progressive closing of the water-way opening.

5) A number of curves may be drawn with head difference vs discharge for different vent sizes. From these curves required size and No. of ventage may be fixed as per required capacity.

submerged orifice flow



Box width = 3', 4', or 5'

For submerged orifice flow $V = C\,(2g\Delta H)^{1/2}$

$C = 0.82$

$V = 0.82 \times (2g\Delta H)^{1/2}$

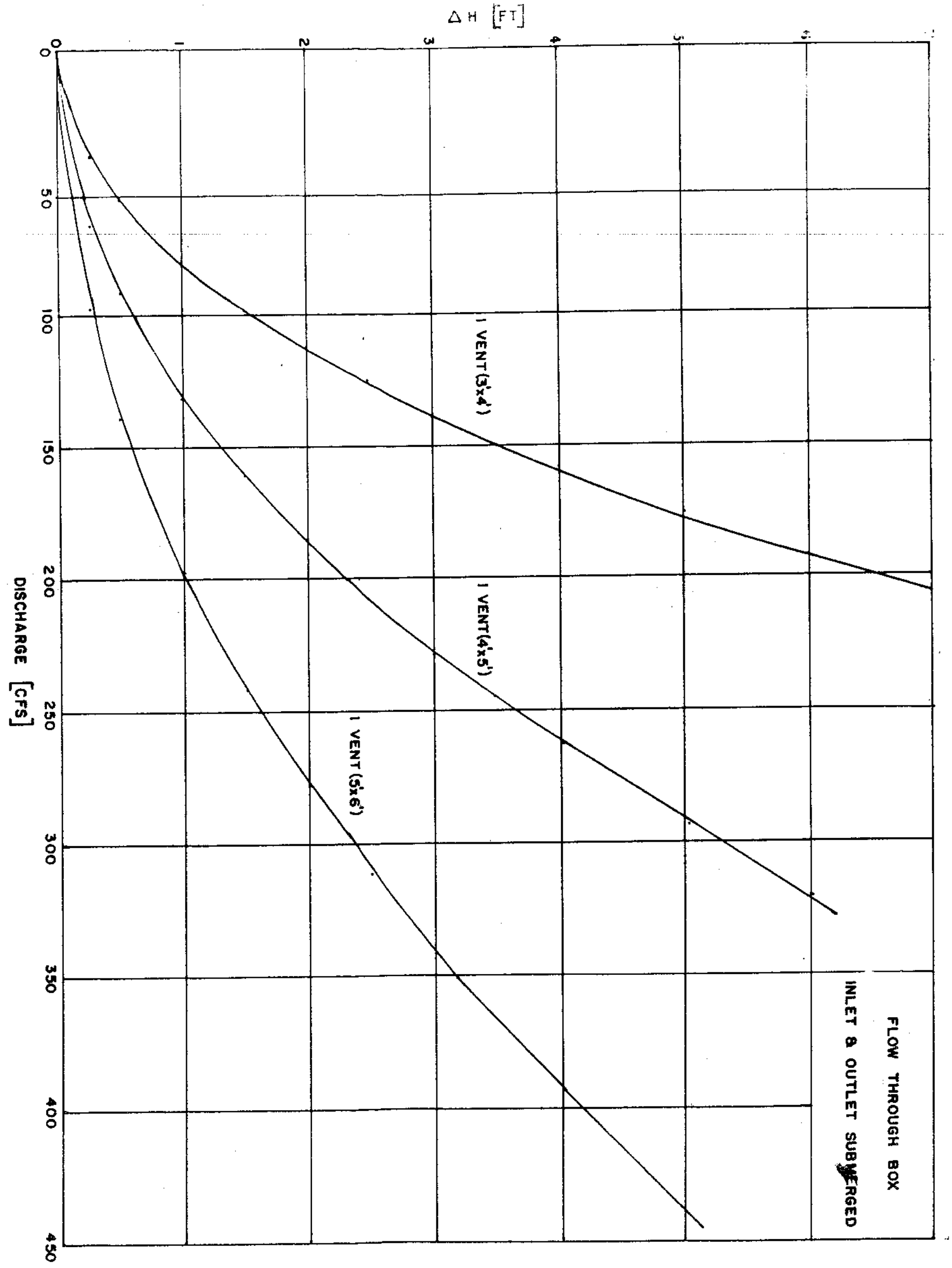
$= 6.58\,(\Delta H)^{1/2}$

$Q = A_{vent} \times V$

Submerged orifice flow : $V = C \, (2g\Delta H)^{1/2}$

$C = 0.82$ $\qquad$ $Q = A_{vent} \times V$

H Ft.	V ft/sec	Discharge Q, cfs. 3 x 4	4 x 5	5 x 6	Remarks
0.25	3.29	39.50	65.80	98.70	
0.50	4.65	55.80	93.00	139.50	
0.75	5.69	68.28	113.80	171.00	
1.00	6.58	79.0	131.60	197.40	
1.50	8.06	96.72	161.20	241.80	From the curves it is seen that a 2 vent structure of 5x6 size may serve the purpose giving Q = 171x2 = 342 cfs which is near & greater than 300 cfs.
2.00	9.30	11.16	186.0	279.0	
2.50	10.40	12.50	208.0	312.0	
3.00	11.40	137.0	228.0	352.0	
4.00	13.16	158.0	263.20	394.8	
5.00	14.71	176.52	294.20	441.30	
6.00	16.12	193.44	322.40	483.6	
7.00	17.41	208.92	384.20	522.30	
8.00	18.61	223.32	372.20	558.30	

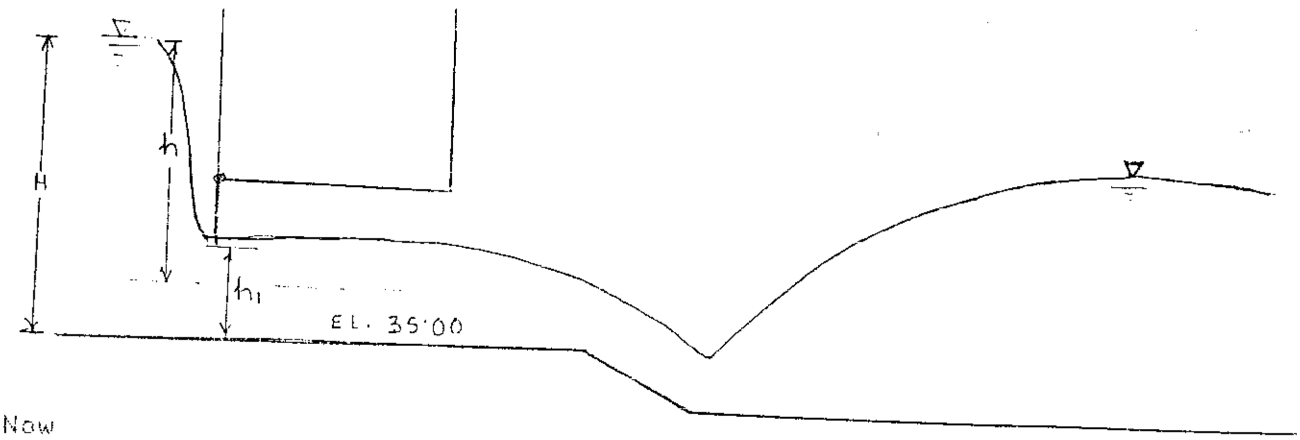


ΔH [FT]
DISCHARGE [CFS]
1 VENT (3'x4')
1 VENT (4'x5')
1 VENT (5'x6')
FLOW THROUGH BOX
INLET & OUTLET SUBMERGED

Computation of ventage opening with respect to water level elevation of R/s

Though our required design discharge is 300 cfs and this may be achieved by a head difference of 0.6 to 0.75 with full opening of the gates. But in cases when the head difference is more, the discharge is to be maintained by progressive closing and opening of the water way as the head difference between R/s and C/s rises or lowers respectively.

So it is required to calculate the gate opening at different W.L. elevation of R/s so that the discharge passed through the vents remains at 300 cfs.



Now

$$Q = Cb\,h_1 \times \sqrt{2g(H - h_1/2)}$$

or $300 = 0.60 \times 10\,h_1 \sqrt{64.4\,(H - h_1/2)}$ [C=0.60, & b = 5′ × 2=10′ - 0" i.e. width of both vents]

or $h_1\sqrt{64.4\,(H-h_1/2)} = 50.00$

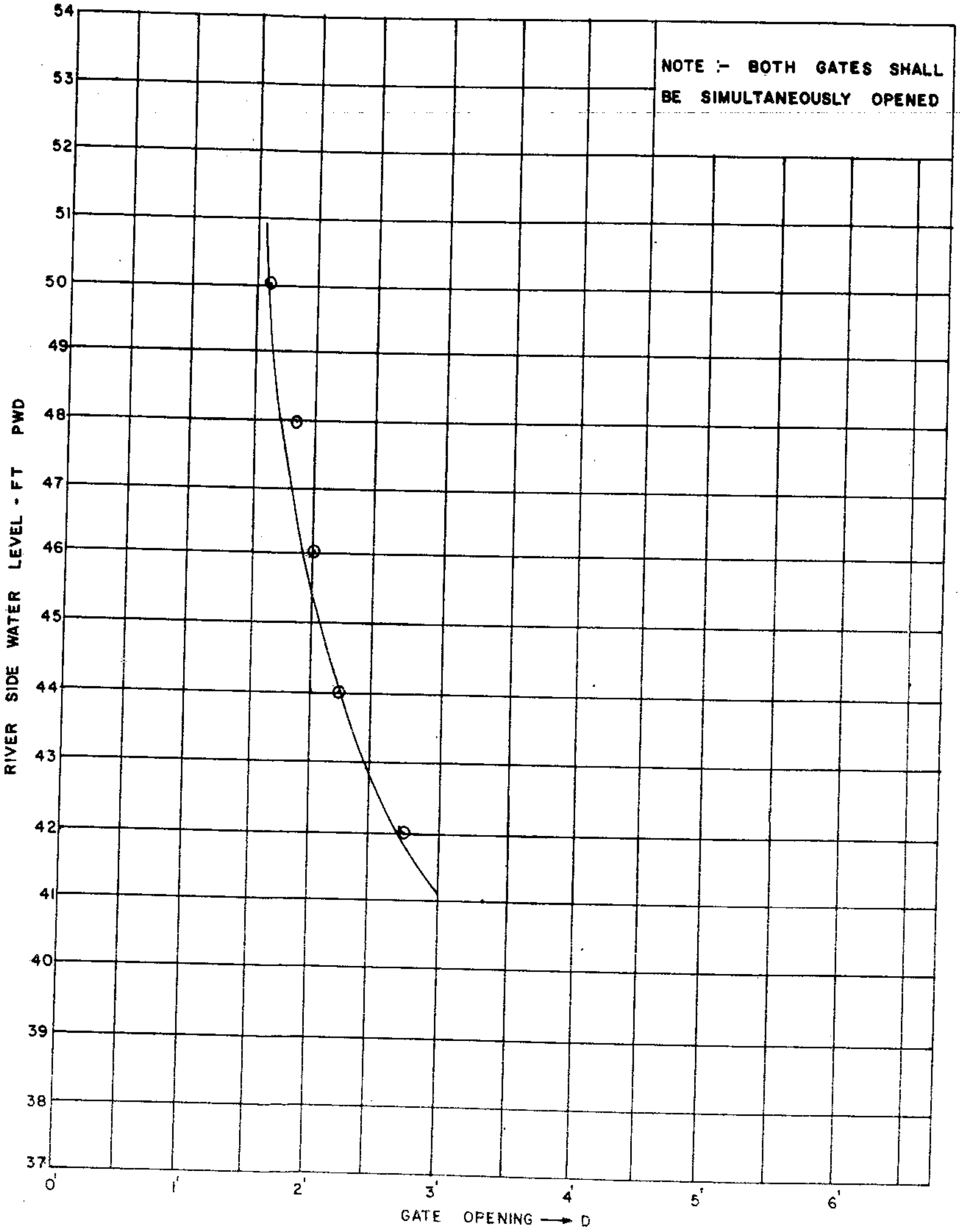
or $h_1\sqrt{(H - h_1/2)} = 6.23$

Equating for various values of H, respective values of h_1 can be computed. A curve may be drawn as river side water level elevation vs gate opening so that one can open the gate as required at different elevation of R/s W.L.

Q = 300 cfs. Invert level : 35.00

W.L. Elevation R/s	H [ft]	h1 [ft]
42	7	2.75
44	9	2.25
46	11	2.00
48	13	1.80
50	15	1.60

Note: The highest W.L. Elevation on the R/s is 50.00 PWD. (information collected from local people)


NOTE :- BOTH GATES SHALL BE SIMULTANEOUSLY OPENED
RIVER SIDE WATER LEVEL - FT PWD
54
53
52
51
50
49
48
47
46
45
44
43
42
41
40
39
38
37
0'
1'
2'
3'
4'
5'
6'
GATE OPENING → D

Hydraulic Design

Data Given

1. Average bed level of canal : 35.70
2. Bed width of canal : 55'-0"
3. Bank level or Ground level : 41.00
4. Highest flood level : 50.00 PWD from public information(ever experienced
5. Highest flood level : 14.528 M PWD.(From hydrograph)
6. Safe exit gradient : 1/7
7. Silt factor f = 0.40 (in absence of soil report)

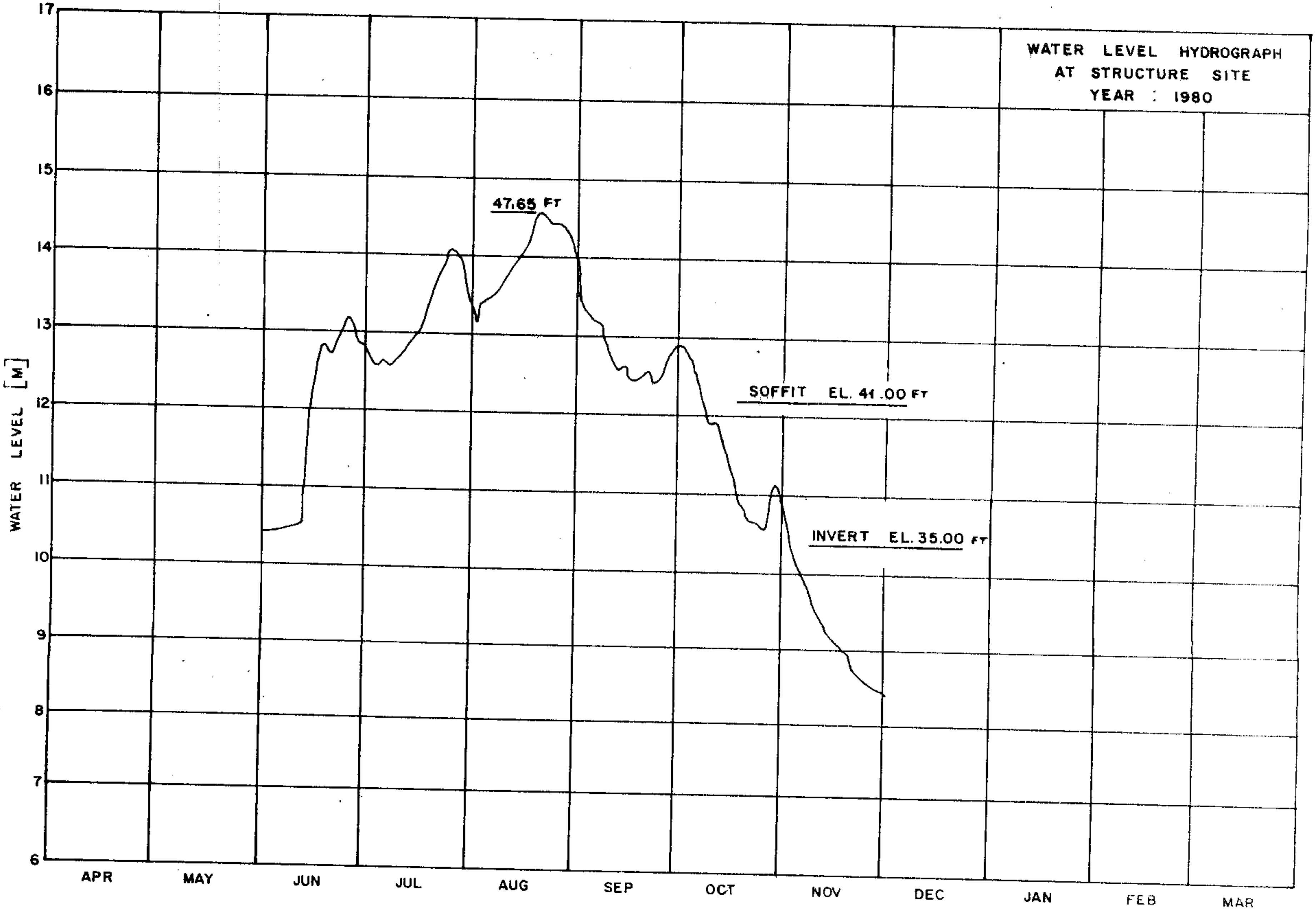
   $$f = 1.76\sqrt{dm}$$

   where dm = average particle size in mm.

8. Crest width of Embankment : 14'-0"
9. Side slope C/s : 1:2

   R/s : 1:3

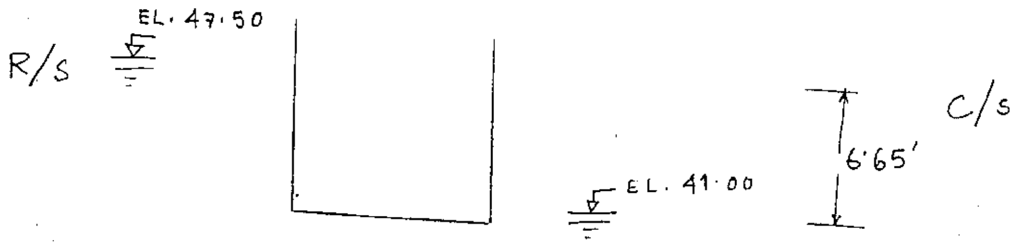
10. Proposed Crest Elevation : 53.00 PWD.

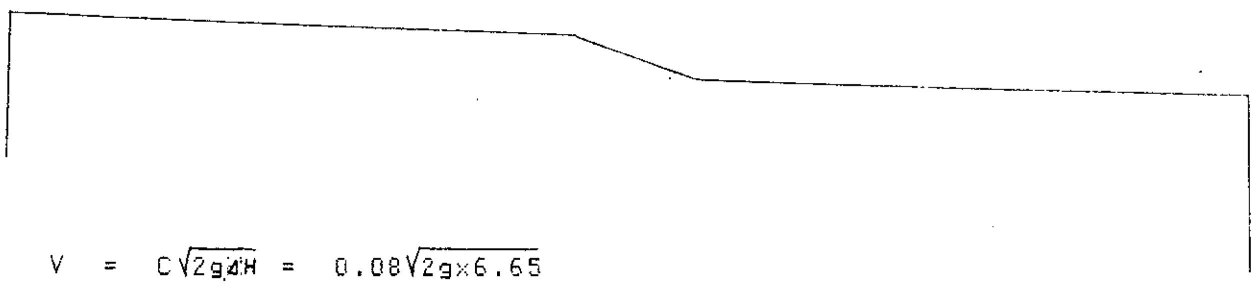

WATER LEVEL HYDROGRAPH
AT STRUCTURE SITE
YEAR : 1980
WATER LEVEL [M]
6
7
8
9
10
11
12
13
14
15
16
17
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC
JAN
FEB
MAR
47.65 FT
SOFFIT EL. 41.00 FT
INVERT EL. 35.00 FT

Floor length by hydraulic jump

Discharge: Though the required design discharge is 300 cfs. But for floor length computation the discharge to be considered at the time of maximum head difference with the gate full opening. This may happen due to ignorance of local people. The maximum W.L. on the R/s is 47.65 PWD. At this time we may consider the W.L. on the country side at 41.00 PWD the bank level of the canal, then the head difference is 6.65.

Here the maximum W.L. of the river has been taken from hydrological year book(H.Graph attached).



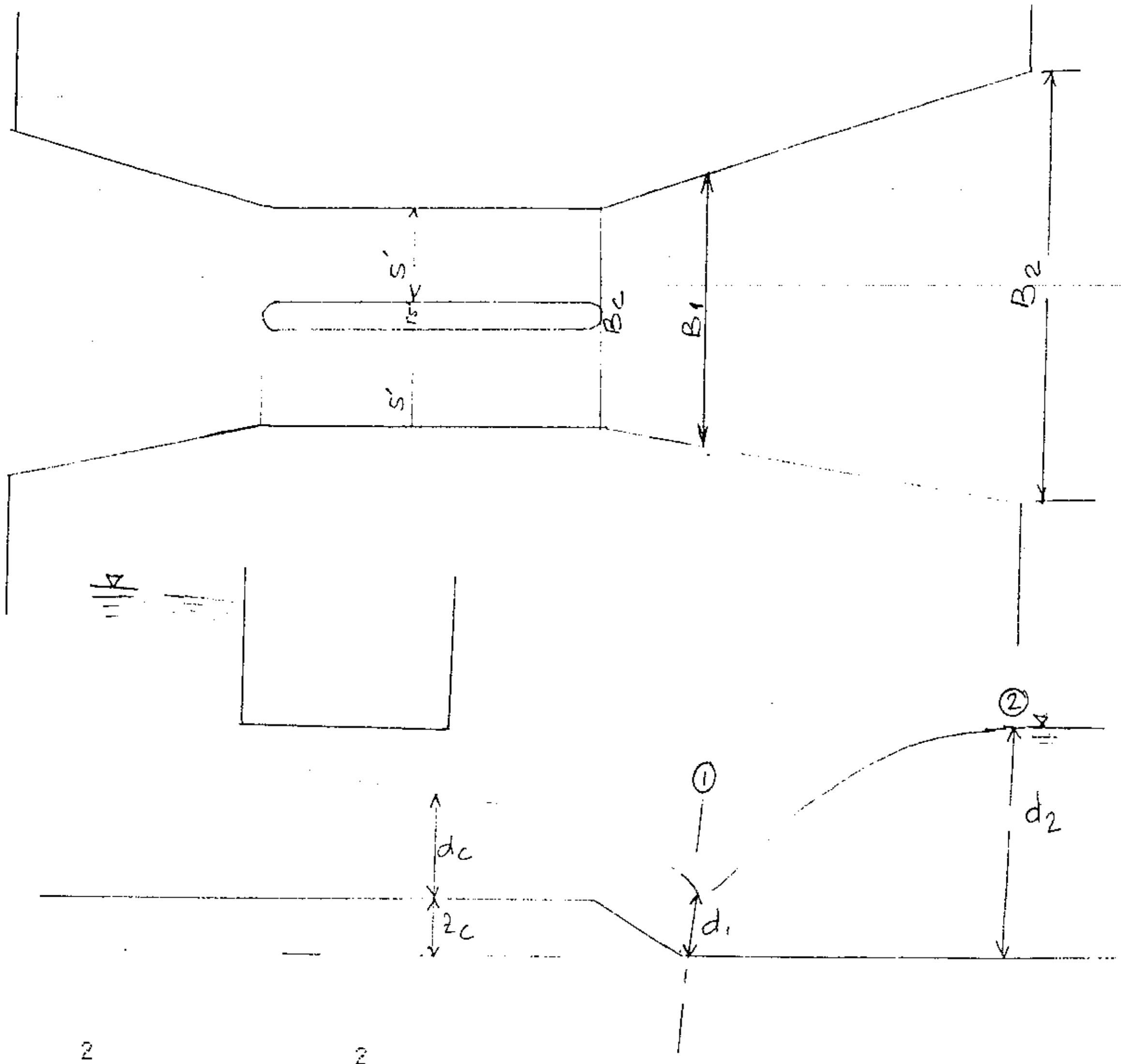


$$V = C\sqrt{2g\Delta H} = 0.08\sqrt{2g \times 6.65}$$

$$= 16.56 \text{ ft/sec.}$$

$$Q = AV = 2 \times 5 \times 6 \times 16.56 = 993.60 \text{cfs.}$$

$$\text{discharge/ft width } q = \frac{993.6}{10} = 99.36 \approx 100 \text{ cfs/ft.}$$



$$dc = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{100^2}{32.2}\right)^{1/3} = 6.77 \text{ ft}$$

$$Vc = \frac{q}{dc} = \frac{100}{6.77} = 14.77 \text{ ft/sec.}$$

BC = 5x2	=	10.00′
Pier = 1.5x1	=	1.5′
		11.50

$$B_1 = 11.50 + 2 \times 3 \text{ ton } \phi$$

$$= 11.50 + 2 \times 3 \text{ ton } 12^{\circ} = 12.77 \text{ ft.}$$

Applying B.E. at pt (1) & (c)

$$d_c + \frac{V_c^2}{2g} + Z_c = d_1 + \frac{V_1^2}{2g} \quad \ldots\ldots(1)$$

At the critical state of flow velocity head is equal to half the hydraulic depth.

At point (1) the discharge/ft width, q_1 is given by

$$q_1 = \frac{100\times10}{12.77} = 78.30 \text{ cfs}$$

From equation (1) at pt (1)

$$1.5\, d_c + Z_c = d_1 + \frac{v_1^2}{2g} \qquad q_1 = v_1\, d_1$$

$$1.5 \times 6.77 + 1 = d_1 + \frac{q_1^2}{d_1^2 \times 2g} \qquad \text{or } v_1^2 = \frac{q1^2}{d1^2}$$

$$\text{or } 11.15 = d_1 + \frac{95.20}{d_1^2}$$

By trial d_1 = 3.50 ft.

$$\text{Now } V_1 = \frac{q_1}{d_1} = \frac{78.30}{3.50} = 22.37 \text{ ft/sec}$$

$$\text{Froude No. } F = \frac{V_1}{\sqrt{gd_1}} = \frac{22.37}{\sqrt{32.2\times3.50}} = 2.10$$

Length of jump

We know $\frac{d_2}{d_1} = \frac{1}{2} [(1 + 8F^2)^{0.5} - 1]$

or $\frac{d_2}{3.50} = \frac{1}{2} [(1 + 8 \times 2.10^2)^{0.50} - 1]$

or $\frac{d_2}{3.50} = 2.51$

or $d_2 = 2.51 \times 3.50 = 8.79$ ft.

Length of jump $= 6.9(d_2 - d_1)$

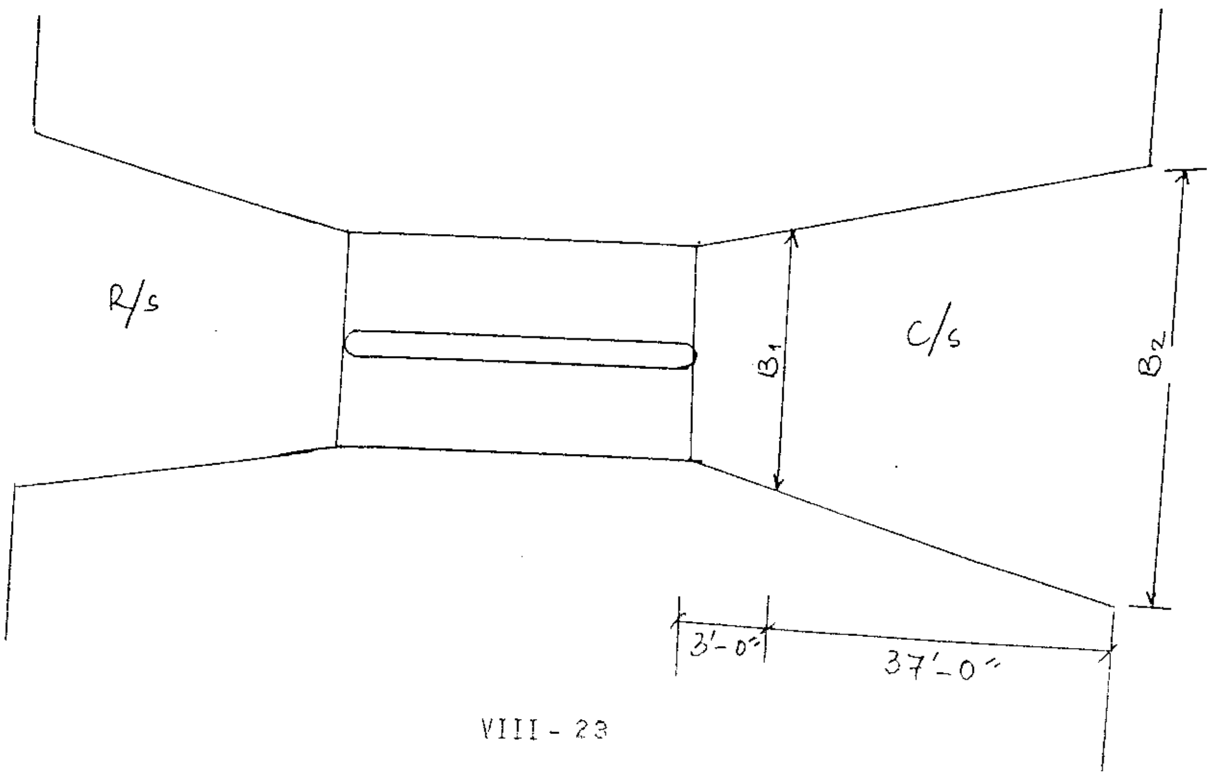
$= 6.9(8.79 - 3.50)$

$= 36.50$ ft.

$= 37$ ft.

We may provide d/s floor length 37 + 3 = 40 - 0"

Cutoff depth from scour



$B_2 = B_c + 2 \times 40 \tan 12^o$

$= 11.50' + 17.00$

$= 28.50$ ft.

Let us fix B_2 = 30-0"

$q_2 = \frac{100 \times 10}{30}$ = 33.33 cfs/ft. width at point (2)

Scour depth $R = 0.91 \left(\frac{q_2^2}{f}\right)^{1/3}$

$= 0.91 \left(\frac{q_2^2}{0.4}\right)^{1/3}$

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$= 0.91 \left(\frac{33.33^2}{0.4}\right)^{1/3}$

= 12.8 - 0"

Design scour depth

U/s = 1.25 R = 1.25 x 12.8 = 16.00

D/s = 1.50 x R = 1.50 x 12.8 = 19.2

Scour level U/s = U/s W.L. - 16.00

= 47.65 - 16.00

= 31.65 = 32

U/s depth of cutoff requirec = 35 - 30

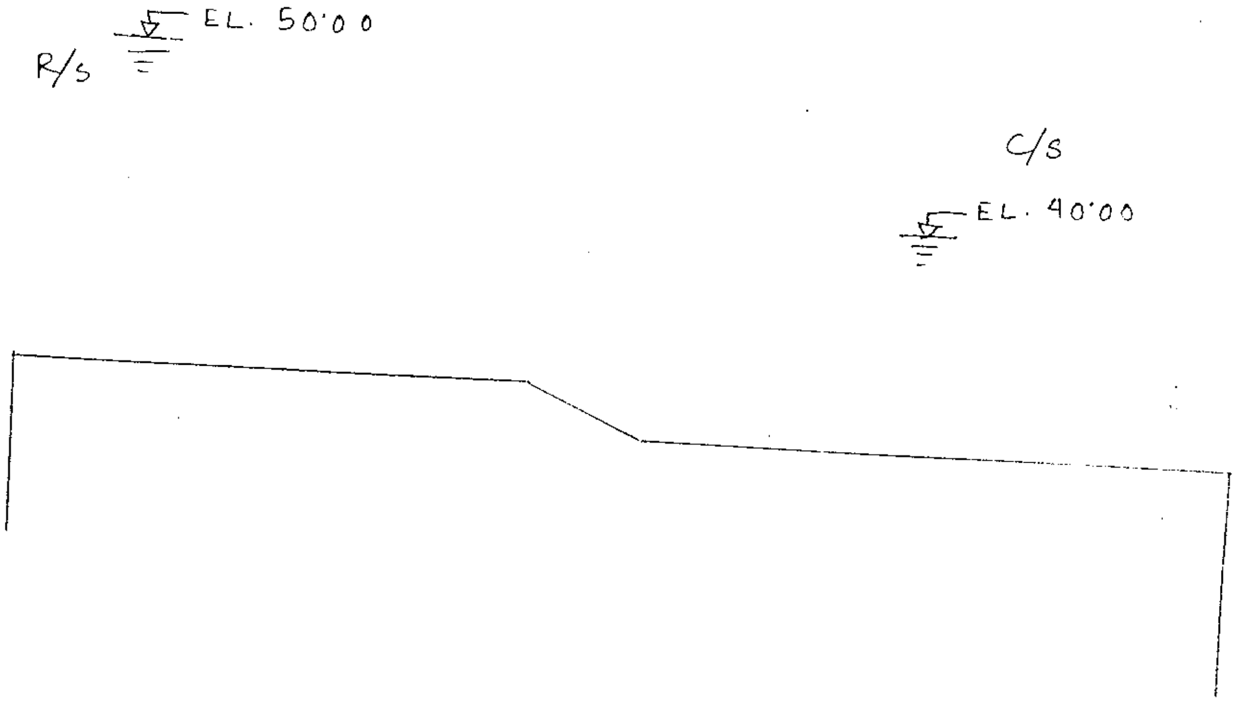
= 5 ft.

U/s depth of cutoff reqd. = 19.20 - 8.79

= 10.41

Use 12'-0"

Floor length & cutoff depth by exit Gradient



Consideration of head difference

In calculating the floor length and depth of cut off by exit gradient it is better to assume that the gate is closed. R/s W.L. is at its maximum elevation & C/s W.L. is at or near to bank level. We shall consider C/s W.L. is at 1' below the bank level i.e. at El. 40.00 and R/s W.L. is at maximum elevation 50.00. (information collected from local people, ever experienced)

In this case $H = 10$ ft.

Taking depth of cutoff d/s = 12 -0"

& $G_E = 1/7$

$$G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}} \quad \text{or} \quad \lambda = \left(\frac{H}{G_E\, d\, \pi}\right)^2$$

$$= \left(\frac{10 \times 7}{12 \times 3.14}\right)^2$$

$$= 3.45$$

Again $$\lambda = \frac{1 + \sqrt{1+\alpha^2}}{2}$$

or $$1 + \sqrt{1 + \alpha^2} = 2 \times \lambda$$

or $$1 + \alpha^2 = (2\lambda - 1)^2$$

or $$\alpha = \sqrt{(2\lambda - 1)^2 - 1}$$

$$= \sqrt{(2 \times 3.45 - 1)^2 - 1}$$

$$= 5.82$$

Now $$\alpha = \frac{b}{d}$$

$$5.82 = \frac{b}{12}$$

$b = 12 \times 5.82 = 70-0"$

On the other hand during winter season while the water level on the river side remains at low stage the gate remains closed and the irrigation is to be done by the stored water on the canal. During this period we may consider the water level on the country side at El. 41.00 while that on the river side at floor level of El. 35.00 resulting a head difference of 6 - 0".

Considering the depth of cutoff 5 on R/s

and G.E = 1/7

$$\frac{G}{E.} = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}}$$

$$\text{or} \quad \lambda = \left(\frac{H}{\frac{G}{E.}\, d\, \pi}\right)^2$$

$$= \left(\frac{5 \times 7}{5 \times 3.14}\right)^2$$

$$= 4.96$$

$$\text{again} \quad \alpha = \sqrt{(2\lambda - 1)^2 - 1}$$

$$= \sqrt{(2 \times 4.96 - 1)^2 - 1}$$

$$= 8.86$$

$$\text{Now} \quad 8.86 = \frac{b}{d}$$

$$\text{or} \quad 8.86 = \frac{b}{5}$$

$$\text{or} \quad b = 44.3'$$

শফি [illegible]
উপ-বিভাগীয় প্রকৌশলী
নকশা সার্কেল-২
বাপাউবো, ঢাকা-১২১৫

The length required by the structure is given by

b = D/s floor length + U/s floor length + Glacis + barrel & operating devices

= 37 + 24 (2/3rd of D/s floor) + 3 + 26

= 90 - 0"

We may fix the floor lenght 90 , U/s cutoff depth 5 and d/s cutoff depth 12 -0".

Floor Thickness in relation to uplift pressure

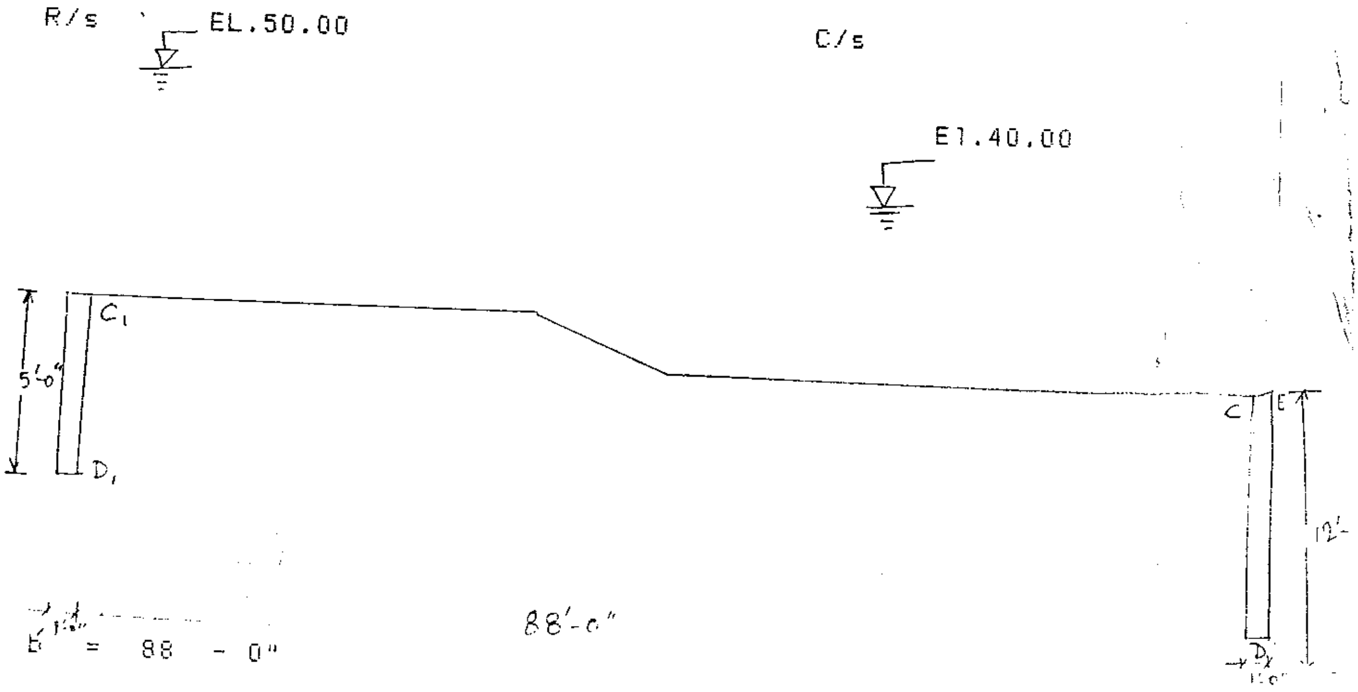
Now we have already fixed

Floor length b = 90 - 0"

depth of cutoff D/s, d = 12 - 0"

depth of cutoff U/s = 5 -0"

Let us assume floor thickness at U/s & D/s end 2 -0"



b' = 88 - 0"

b = 90 - 0"

d = 12 - 0"

D = 5 -0"

$$1/\alpha = \frac{d}{b} = \frac{5}{90} = 0.055$$

From Khosla's Pressure chart

$P_D = 13\%$ $\quad P_{D_1} = 100 - 13 = 87\%$

$P_E = 21\%$ $\quad P_{C_1} = 100 - 21 = 79\%$

<u>Correction due to interference of cutoff</u>

Let C be the correction to be applied as percentage of head.

b' = distance between two piles

D = depth of cutoff the influence of which has to be determined on the neighbouring cutoff of depth d

d = depth of cutoff on which the effect of cutoff of depth D is sought to be determined

b = total length of floor

Then $c = 19\sqrt{\frac{D}{b'}} \times \left[\frac{d + D}{b}\right]$

<u>Effect of u/s cutoff on to the d/s cutoff</u>

In this case D = 3 ft. d = 10 ft.

$$c = 19\sqrt{\frac{3}{88}} \times \left[\frac{10+3}{90}\right]$$

$= 0.50\%$ (-ive)

This correction is positive for points in the rear or back water and subtract for points forward in the direction of flow.

Effect D/s cutoff on to the u/s cutoff

In this case D = 10 ft. d = 3 ft.

$$C_1 = 19\sqrt{\frac{D}{b}} \times \frac{d+D}{b}$$

$$= 19\sqrt{\frac{10}{88}} \times \frac{3+10}{90}$$

$$= 0.92\% \quad (+ive)$$

Correction due to floor thickness (R/s)

Floor thickness at end is assumed 2'-0".

If C_F is the correction for floor thickness

$$\text{Then } C_F = \frac{87 - 79}{5} \times 2 = 3.2\% \ (+ve)$$

$$P_{C_1} = 79 + 3.2 + 0.92 = 83\%$$
(corrected)

Correction for floor thickness C/s

i) Correction due to interference of cutoff will be due to the interference of R/s cutoff on to the C/s cutoff. Its direction is negative i.e. C = 0.50 (- ve)

ii) Correction for floor thickness = $\frac{21 - 13}{12} \times 2 = 1.33\%$ (- ve)

P_E (Corrected) = 21 - 1.33 - 0.50 = 19.17%

Again while the water stored on the canal closing the gate as we assumed before, water level on the canal at El. 40.00 and R/s W.L. at El. 34.00, we may get the uplift pressure as follows:

$$\text{Here}\quad \frac{1}{\alpha} = \frac{d}{b} = \frac{12}{90} = 0.13$$

From Khosla's chart

$P_D = 22\%$ $\qquad$ $P_{D_1} = 100-22 = 78\%$

$P_E = 34\%$ $\qquad$ $P_{C_1} = 100-34 = 66\%$

<u>Correction due to thickness of floor</u>

If C_F is the correction due to floor thickness then

$$C_{F(c/s)} = \frac{78-66}{12} \times 2 = 2\%$$

$$C_{F(R/s)} = \frac{34-22}{5} \times 2 = 4.8\%$$

Correction due to interference of pile

If C is the correction due to interference of file.

$$C_{(c/s)} = 19 \sqrt{\frac{3}{88}} \times \frac{3+10}{90}$$

$= 0.50\%$ (+ive)

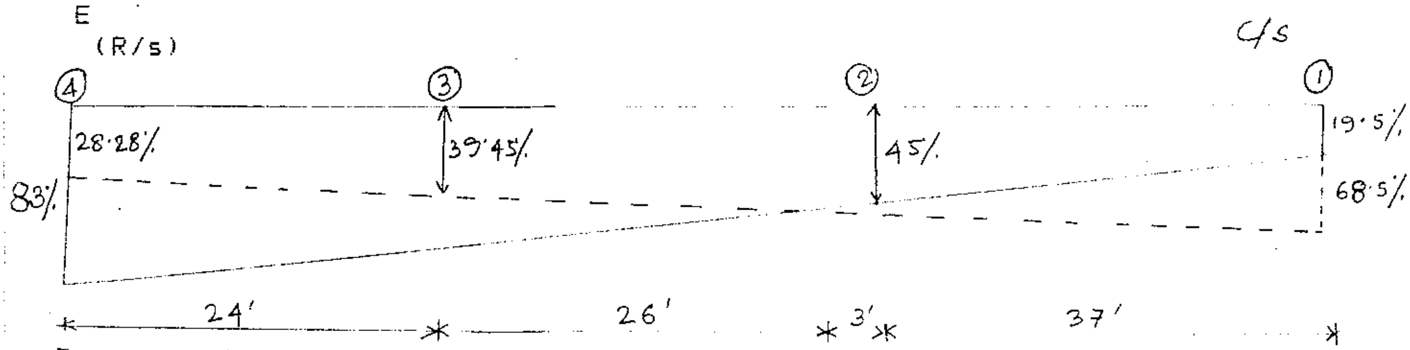
$$C_{(R/s)} = 19 \sqrt{\frac{10}{88}} \times \frac{3+10}{90}$$

$= 0.92\%$ (-ive)

Correction for floor thickness

$P_{C_1 (c/s)} = 66+2+0.50 = 68.50\%$

$P_{E (R/s)} = 34-4.8-0.92 = 28.28\%$



Pressure in ft of water

$$P_1 = \frac{19.17}{100} \times 10 = 1.92 \text{ ft. of water}$$

$$P_2 = \frac{45}{100} \times 10 = 4.5 \text{ " " "}$$

$$P_3 = \frac{39.45}{100} \times 6 = 2.36 \text{ " " "}$$

$$P_4 = \frac{28.89}{100} \times 6 = 1.75 \text{ ft} \quad " \quad "$$

Here the critical uplift pressure will be at point (3) & (4) while C/s W.L. controlling. Because at point (1) & (2) the uplift pressure is less than the downward vertical pressure of water & the part betn pt. (2) & (3) there exist barrel, top slab, soil etc. which will counter balance the upward pressure exerted by head difference. On the other hand while R/s W.L. controlling the critical point is at point (1) & (2).

Required Thickness of floor

The uplift pressure exerted by difference of head is to be counter balanced by equivalent wt. of concrete i.e. sufficient thickness of floor is to be provided.

Submerged unit wt. of concrete $= \dfrac{\gamma_{conc} - \gamma_{water}}{\gamma_{water}}$

i.e. specific weight of concrete $= \dfrac{150 - 62.40}{62.40}$

$= 1.40$

So the required thickness of concrete to be provided at

Pt. (4) i.e. at end $t_4 = \dfrac{1.75}{1.40} = 1.25'$

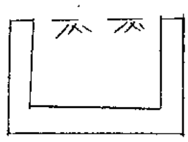
Pt. (3) $t_3 = \dfrac{2.38}{1.40} = 1.70'$

$$\text{Pt. (2)}\quad t_2 = \frac{4.5}{1.40} = 3.21'$$

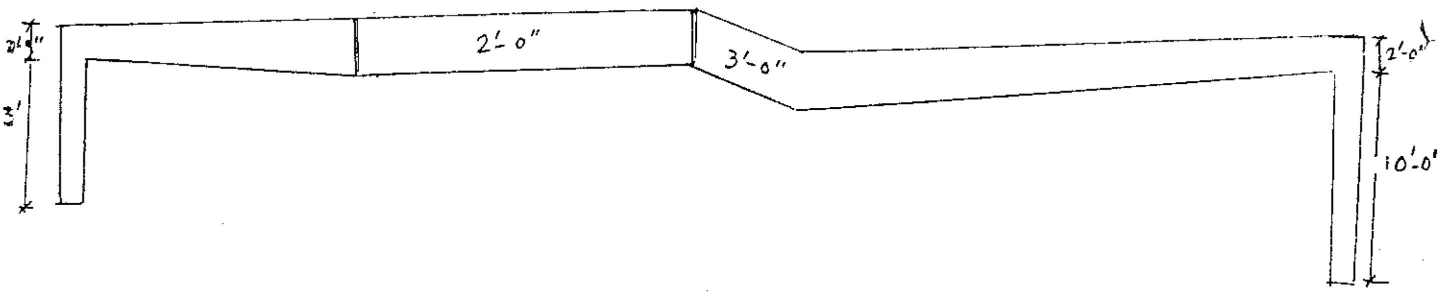
$$\text{Pt. (1)}\quad t = \frac{1.92}{1.40} = 1.37'$$

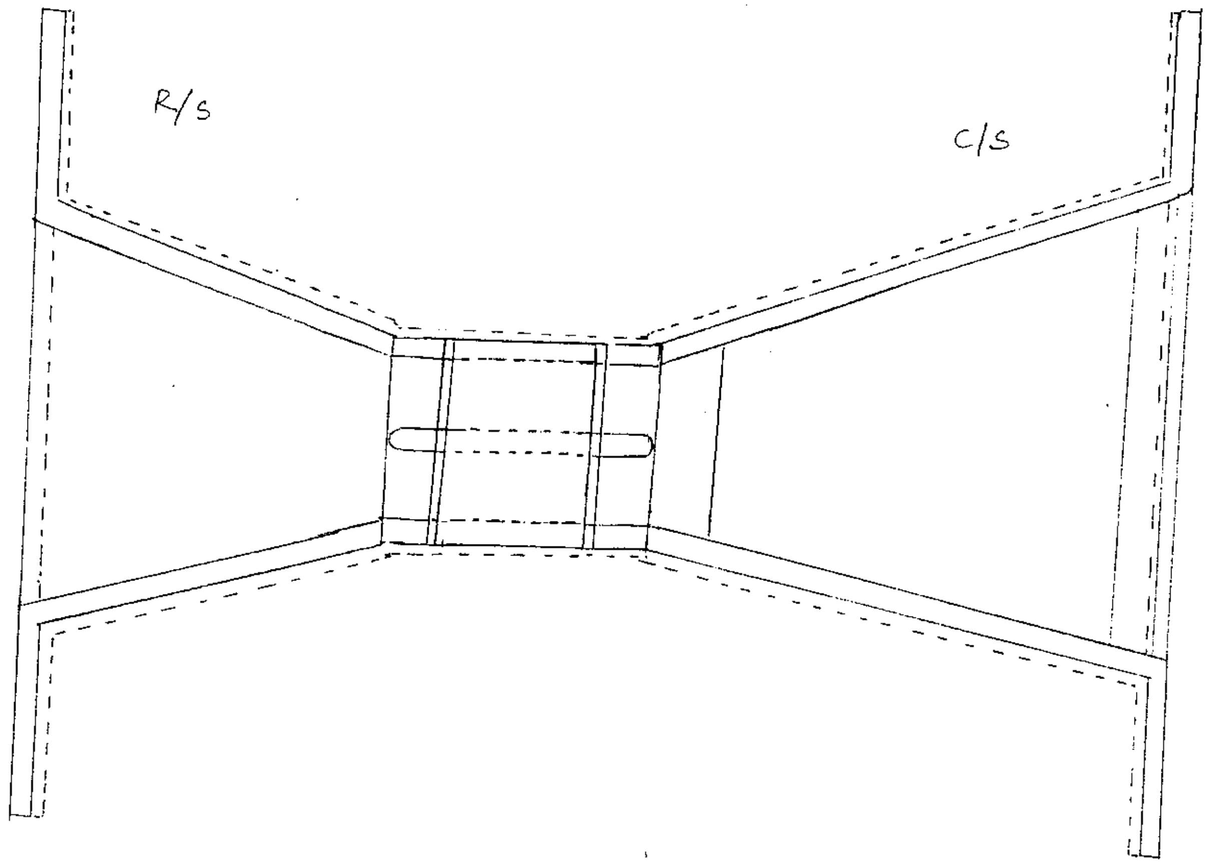
So thickness of floor at end 2-0" at glacies 3'-0' on both sides and at barrel part 2-0" may be provided.

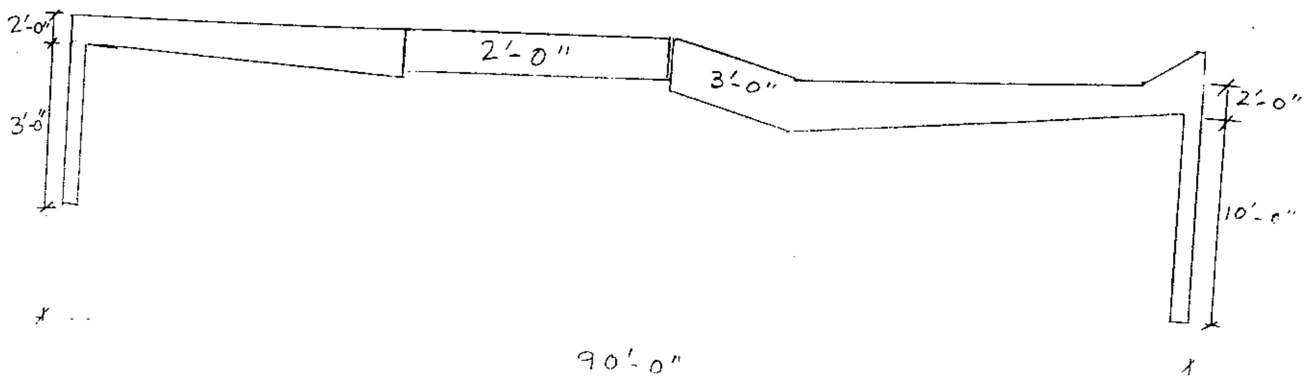
R/s



C/s




R/S
C/S


2'-0"
3'-0"
2'-0"
3'-0"
2'-0"
10'-0"
90'-0"