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BANGLADESH
WATER DEVELOPMENT BOARD

HYDROLOGIC AND HYDRAULIC DESIGN PROCEDURES FOR DRAINAGE STRUCTURES

DESIGN DIRECTORATE-I
DACCA

DM-7/

EAST PAKISTAN
WATER AND POWER
DEVELOPMENT
AUTHORITY

HYDROLOGIC AND HYDRAULIC
DESIGN PROCEDURES
FOR
DRAINAGE STRUCTURES

Prepared By

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SUMMARY OF DESIGN PROCEDURE

1.1 GENERAL

The drainage sluice is probably the one structure that is most prevalent in East Pakistan. Although it is often small, relatively inexpensive, and usually commands a small area, its effect on the whole of the Province should not be under-estimated. Collectively it is a structure that shares an intimate relationship with a large portion of the rural population, often determining the success or failure of a farmer's crop.

Because of its prevalence and simplicity of operation it is often through the sluice that the farmer will rate the competance of his country's engineers. A highway bridge may be poorly designed and still carry the load and the traveler is none the wiser. But a farmer standing knee deep in water knowing his fields should be dry will damn the engineer.

It is essential, then, that proper attention be given to the design of the drainage sluice to insure it fulfills its desired function. This manual is an effort to improve hydraulic design methods currently in use and to present a procedure that is complete and follows a logical sequence.

Much of the basic data required for a proper design is often not available, and probably will not be available for years to come. Accordingly the design engineer will have to make certain assumptions during the course of his design. It is natural, that at time he will err. To safeguard against this eventuality he should make every effort to err on the side of conservatism. The engineer should always remember that in the final analysis the cost of a two-vent sluice is considerably less than twice the cost of a one-vent sluice.

The material throughout the following sections has been prepared primarily for the use of assistant engineers w who often will be encountering this type of design problem for the first time. A great deal of explanation and formula derivation accompanies each step of the procedure to help him thoroughly understand the underlying design principles. In some cases certain items had to be discussed out of design sequence because their subject matter was not in tenor with a particular article. Consequently the experienced engineer, who wishes to follow the basic design process without explanations, may find some difficulty because of the interruptions. To assist him a step-by-step procedure has been s summarized with references to article and page of the text.

1.2 PRELIMINARY DATA REQUIRED FOR DESIGN

The following data must be supplied to the designer, or collected by him, before he can begin his design. Some of the data are mandatory, others are helpful. Before the designer begins his studies he should check the material he has in hand to see if it is complete, the most recent available, and accurate. Nothing is more frustrating than to struggle through a design procedure and then find there are errors in the original survey or later information that casts a different light upon the whole design problem.

1.2.1 DATA SUPPLIED BY THE FIELD

a) MANDATORY :

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. Large scale topography of the structure site.
4. A center line profile of the outfall channel with frequent cross sections.
5. A description of the outfall channel condition. (see Appendix B for details).
6. Profile and cross-sections of the main drainage channel for at least one mile upstream of the structure.
7. River stage records for dry season and monsoon season periods at the outfall.
8. Measured dry season discharge at the structure site.
9. Measured, or at least estimated, flood discharge at the structure site.
10. In the case of tidal regulators, tide records for at least one year.

b) HELPFUL :

1. Rainfall recording within the basin.
2. Estimate of percentage of paddy and non-paddy lands.
3. Principal crops grown in the area, with a seasonal cropping pattern.

1.2.2 SUPPLEMENTAL DATA BY THE DESIGN ENGINEER

1. Published rainfall records.
2. Published stage - discharge records of the principal rivers in the area.
3. If possible the designer should make a personal trip to the area f r first-hand observation of conditions.

1.3 SUMMARY OF DESIGN PROCEDURE

General Procedure	References of specific application
I. HYDROLOGIC INVESTIGATIONS	
A. Preliminary	
1. Determine geographical location and size of basin from topographic maps	
2. Locate and measure length of principle drainage course.	
3. Locate the geographical center (centeroid) of the basin area. Measure length along the principle channel from point on the channel opposite the centroid to the proposed structure site.	
The basin centroid can be found by vertically suspending a card-board cutout of the basin shape successively from two or more points and finding the intersection of plumb lines from each point.	
B. Develop A Design Storm	Sec.2.3, Pg. II-3
a) Locate basin on Figure A-II-2 and find the 4-month rainfall index.	Sec.2.3.1, Pg. II-4
b) Select a storm frequency of 10 or 25 years.	
c) Multiply the 4-month index by the daily combined indices in Table II-1. This gives the accumulated point rainfall volume, in inches, for storms of 1,2,3,4 & 5 days duration. For durations longer than 5 days use the charts found in the Appendix.	Table II-3, Pg. II-7
2. Equivalent Uniform Depth of Rainfall :	Sec.2.3.2, Pg. II-5

General Procedure	References of specific application
a) Compute the reduction factor for rainfall variability. A step-by-step is indicated on Page II-6.	Table II-4, Pg. II-8
b) Multiply the point rainfall volumes from step 1-C) by the rainfall reduction factor. Separate accumulative totals into daily increments.	Table II-5, Pg. II-8
3. Rainfall Time-Distribution :	Sec. 2.3.3, Pg. II-6
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 forth.	Table II-6, Pg. II-9
b) Determine a unit time interval for breaking down the daily increments. The unit interval will be tied into the unit hydrograph.	Appendix A, Pg. A-4
1) Find the time of basin concentration.	Formula (10)
2) Find the time of lag.	Formula (7)
3) Divide the lag time by 5 and round off to the nearest whole number divisible into 24 hours. This unit time interval will be used for both unitgraph unit rainfall duration, t_r and the unit time interval for storm distribution.	
c) Locate the gaging station closest to the basin being investigated. Covert 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.	Table II-7 Pg. II-9
d) Convert unit duration point rainfall to equivalent uniform depth using the reduction factor from step 2-a)	Table II-8, Pg. II-10
e) Arrange the unit durations for the maximum day into sequence. Use the same sequences given in step 3-a).	

General Procedure	References of specific application
f) Compute the volumes and arrange the unit durations for the remaining days into sequence.	Table II-9 Pg. II-12
C. Determine Rainfall Losses	Sec. 2.5, Pg. II-12
1. 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 II-10 can be used as an average.	Table II-10, Pg. II-14
2. Antecedent Conditions : For normal design conditions use Condition II. For pumping plant design or areas under extensive irrigation systems use Condition III.	Sec. 2.6.4, Pg. II-24
3. Initial Soil-Moisture Loss :	Sec. 2.5.2, Pg. II-15
a) Paddy land : No initial loss under all conditions. It is assumed the soil is saturated.	
b) Non-paddy Land : Condition II : 0.50 Inches; Condition III : 0.25 Inches	Pg. II-24
4. Subsequent Soil-Moisture Loss :	Sec. 2.5.3, Pg. II-16
a) Paddy land and non-paddy land: Under all conditions assume a constant rate of infiltration of 0.04"/Hour or 1.00"/Day. This rate can be influenced by a high ground-water table.	Pg. II-25; Pg. II-16

General Procedure	References of specific application
5. Depression Storage:	Sec. 2.5.4, Pg. II-17
a) Paddy Land: Under all conditions the first 4" of rain falling on paddy land is assumed to go into storage	
b) Non-Paddy Land:	
Condition II: Assume a constant storage rate of 0.033"/Hour during periods of rainfall until a maximum of 1.00" is stored.	
Condition III: Use the same constant rate until 0.50" is stored.	
D. Determine Rainfall Excess(Runoff Distribution)	Sec. 2.6, Pg. II-18
1. Paddy Land: Separate the above losses from the rainfall time-distribution determined at step B-3-f, by assuming the entire basin area consists of paddy land.	Table II-11, Pg. II-21
2. Non-Paddy Land: Repeat the process assuming the entire basin area consists of non-paddy land.	Table II-12, Pg. II-22
3. Weighted Basin Average: Compute the weighted basin runoff distribution by multiplying the net runoff obtained in steps 1&2 by the respective land classification percentage.	Table II-13, Pg. II-23
E Develope A Unit Hydrograph	Sec. 2.7, Pg. II-25
The procedure for developing a unit hydrograph is given in Appendix A, An example is given on Page II-27.	Sec. A.5, Pg. A-4.

General Procedure	References of specific application
F. Prepare the Runoff Hydrograph	Sec. 2.8, Pg. II-28
1. Determine the unitgraph discharge ordinates at time intervals equal to the Unit storm duration step B. 3.b-3.	
2. Multiply the unitgraph discharge ordinates successively by the rainfall volume under each interval of the weighted basin runoff distribution	
3. Determine the total basin runoff distribution by making a summation of unit runoff occurring during each unit storm period.	
1.3.2 HYDRAULIC INVESTIGATIONS	
A. The outfall Channel	Sec. 5
1. From field data prepare a longitudinal profile of the drainage channel from a point upstream of the site to the outfall.	Fig. V-5
2. Plot channel cross-sections and determine an average bed slope.	
3. Determine the outfall channel roughness coefficient	Sec. 5.3.1 Pg. V-3
4. Determine channel Conveyance:	Sec.5.3.3 Pg. V-10
a) Determine the conveyance, K_d, for various depths of flow.	
b) Plot A curve of conveyance vs. depth of flow.	Fig. V-6
5. Compute outfall channel (Sluice Tailwater) Profiles:	Sec. 5.4 Pg. V-11
a) Low River Stages-Drawdown Curve:	
For river stages at or below the critical depth level in the channel, the channel water surface profile forms a drawdown curve. Starting with the critical depth for various discharges, compute the water surface profile up to the sluice.	Fig. V-7 & V-8

General Procedure	References of specific application
b) High River Stages-Backwater Profiles: River stages higher than the critical depth level in the channel will create a backwater water surface profile. Assuming various river stages compute backwater profiles for several discharges at each stage.	Fig. V-9, V-10, V-11
6. Prepare A Tailwater Rating Curve: Plot tailwater elevations at the sluice for varying river stages Vs. discharge.	Fig. V-12
B. Flood Routing Through the Sluice	Sec. III
1. Determine which flood routing procedure is to be used.	
a) When discharge is a function of storage level only: Use during periods of low tailwater when discharge is not controlled by tailwater level.	
b) When discharge is a function of tailwater level: Use during periods of high tailwater levels, or functioning tailwater such as in tidal zones.	Sec. 3.5 Pg. III-12
c) Discharge through Pumping Stations: Discharge is controlled neither by headwater level nor tailwater level.	Sec. 3.6 Pg. III-17
2. Prepare Basin Area-Capacity Vs. Elevation curves.	Fig. I-I Pg. III-7
3. On the basis of the inflow hydrograph, estimate required sluice discharge capacity. The estimate may be based on the capacity of similar existing sluice structures, experience gain from previous designs, or by assuming a trial discharge of 2/3 the maximum inflow and refining the assumption later.	
4. Prepare sluice discharge rating curves using the relation ~~discharge~~ discussed in Section IV.	
a) For flow conditions 3 & 5 when the control is at the entrance.	Fig. III-2
b) For flow condition 1 when the outlet is submerged	Fig. III-7
5. When the routing procedure is for discharge as a function of storage level only, prepare a 2S/t + D curve.	Fig. III-3 Pg. III-8

General Procedure	References of specific application
6. When the routing procedure is for discharge as a function of tailwater level, use the tailwater rating curve prepared at outfall channel, step 6.	Fig. III-8
7. Route the flood through the structure.	
a) Discharge as a function of storage level only.	Pg. III-10 Table III-4
b) Discharge as a function of tailwater level.	Pg. III-15 Table III-10
c) Pumping Stations.	Pg. III-17 Fig. III-12
8. From the flood routing computations pl[illegible] surface [illegible]	Fig. III-6, III-11, III-13
9. Make assessment of flood damage.	Sec. 3.4.4, 3.5.4, 3.6.2
10. 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 [illegible] capacity.	

C. This completes the hydraulic design procedures. Proceed now with the structural layout and design.

SECTION II

HYDROLOGIC INVESTIGATIONS

2.1 INTRODUCTION

The design of a drainage sluice is not a simplified procedure accomplished by applying a standard formula to certain static conditions assumed by the designer. The conditions influencing discharge are set by nature and unfortunately, are far from being static. Rather, they are quite variable from season to season, day to day, and even from hour to hour.

Hydrology has been defined as a science dealing with the occurrence and movement of water upon and beneath the land areas of the earth. And using hydrology we will be able to define the variables encountered in sluice design.

Obviously a drainage sluice is a structure whose reason for being is to drain off excess water from an area within finite boundaries. But where does the water come from? In what manner does it arrive at the sluice? How does the sluice transport the water? And where does it go? These are the questions the designer must answer; and after answering them it is his responsibility to supply a structure that will quickly and efficiently remove the expected inflow of water in a manner that will cause a minimum of damage to property and minimum of inconvenience of inhabitants with a reasonable expenditure of funds.

The first step that must be taken to properly design a sluice are to answer these questions and determine the magnitude and characteristics of the flow you are attempting to control.

Where does the water come from? The two prime sources are rainfall within the basin boundaries and inflow from sources out-sidethe basin. A study must be made of rainfall records of the area, and the intensity and frequency of certain major storms must be estimated. The size and frquency of major inflow floods are to be estimated. Then by using statistical procedures a design storm and a design flood are determined.

In what manner does the water arrive at the sluice? How much of the water is lost between the time it enters the basin and the time the flow reaches the sluice structure? A unit hydrograph is used as an aid and a means of determining the time required for runoff from a storm to concentrate at the structure and the magnitude of the peak runoff. Estimates of consumptive use, seepage and evaporation losses, etc., are made and deducted from the gross rainfall to determine the amount of rainfall excess contributing to surface runoff.

How does the sluice transport the water? Because of the varying headwater and tailwater conditions that may prevail the discharge through the sluice is not constant over a period of time. The method used to determine the rate of discharge is called flood routing and will be discussed more fully later.

Where does the water go after it leaves the sluice? Usually it will pass into an outfall channel, either natural or man made. Studies must be made of the stage-discharge relationship of the channel to determine what effect the tailwater level will have on the sluice discharge capacity. Most outfall channels will be short so that uniform depth of flow may not occur. Therefore, backwater curves must be computed to establish the stage relationship at the sluice.

Most of the text-book procedures used to solve the above problems are based on sophisticated methods supported by an abundance of data. Unfortunately much of the required data is lacking in East Pakistan at this time. The following methods used are based on the data available and the assumptions that are used are felt to be reasonable for the conditions. At such time when more information is available the methods can be easily modified.

2.2 WHERE DOES THE WATER COME FROM?

Within the region comprising East Pakistan there is a variance in average annual rainfall ranging from 6[illegible] to 255 inches per year at points only 150 miles apart. The world recorded maximum of 908

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inches per year occurred at Cherrapunji, less than 25 miles over the Assam border. This region of intense rainfall lies in the belt of the southwest monsoons which last from June till mid-October. During the monsoon the rainfall is not continuous; rather it falls in intermittent showers which may be intense and may occur several times during a day. Downpours of heavy rain are generally localized with normal rainfall intensity occurring a short distance away. Pre-monsoon rains occur during April and May, and have the characteristics of strong northwest wind with heavy thunder storms, usually of short duration.

The methods for determining peak discharges, as presented inthis paper, are applicable only to watersheds where flow originates principally from rainfall within the basin. Those watersheds having flow originating outside the basin in the form of groundwater or inflowing streams require special study.

2.3 THE DESIGN STORM

It is uneconomical to design structures for the maximum storm that may ever occur. They are designed instead with the expectation that they will be over-charged once in 10, 15, or 25 years on the average. Therefore, for flood and drainage studies it is necessary to know the frequency of intense rainfall for varying periods of time.

The General Consultants to EPWAPDA have analysed the existing rainfall records of East Pakistan and have prepared a series of charts showing the relationship between probable rainfall intensity, frequency and duration. These charts are given in the IECO Master Plan, Supplement A-Climate and Hydrology, and are reproduced in the appendix of this paper. 2.3-1

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The basis of the IECO studies is the total mean rainfall that may be expected to fall during the four calender month period of maximum rainfall. Generally speaking this is the monsoon period. It is recommended, however, that the engineer use the four-month rainfall whether he is studying a pre-monsoon, monsoon, or post-monsoon period. This is because river rise in the province is not totally dependent on rainfall and a sluice may be required to pass an early monsoon storm before the rivers rise, or a late monsoon storm after the rivers have fallen.

Two other charts are included in the IECO study. One, to convert the total four-month rainfall to total rainfall over a selected number of days; and the other to convert the index frequency to a selected frequency. Since most of our studies will require a rainfall duration no longer than 5-days and frequencies of either 10-years or 25-years, the two charts can be combined in the following tabular form.

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	0.326

COMBINED RAINFALL INDEX
FOR SELECTED FREQUENCIES

TABLE II-1

The four-month rainfall index is presented in the form of an isohyetal map of East Pakistan. By finding the location of any basin in question, determining the four-month rainfall at that point, and multiplying the rainfall by the values of Table II-1, the total basin rainfall for periods from one to five days can be found. These rainfall volumes form the basis for our design storm.

2.3.1 POINT RAINFALL

Point rainfall is the quantity of rain falling at a specific point, usually as measured at a rain gaging station. We will define point rainfall as the volume of rain, at the center of a storm, equal to the product of the four-month rainfall index and the combined rainfall indices from Table II-1.

If point rainfall were used as the basis for designing a structure the computed volume of water would be unrealistically large resulting in increased project costs. 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.

2.3.2 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 question immediately arises, what is the total volume of rainfall resulting from our design storm? In order to answer the question two considerations are apparent. First, what is the storm center location overthe basin; and second, what is the rate of decrease in rainfall intensity away from the storm center?

The storm center may occur at any point over the basin, or may move along any path across the basin. We will choose to ignore this variable and, instead, 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. 2.3.3
The IECO Master Plan states that studies of storm in East Pakistan indicate, that a fairly good relationship exists between depth of rain and distance from storm center. The results of these studies are presented in the appendix as a graph showing the relationship between distance from the storm center and rainfall expressed as a percentage of point rainfall. Table II-2 shows this relationship in tabular form.

Distance From Storm Center	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
7	67.8
8	66.0
9	64.5
10	63.1

RAINFALL VARIABILITY
Table II-2

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

1. Locate the geographical center of the basin;

2. Draw concentric circles, or isohyetals, at even mile intervals around the center;

3. Planimeter the areas between the respective isohyetals within the basin area;

4. Multiply the area by the proper percentage from Table II-2;

5. Make a summation of the values from step 4 and divide by the total basin area;

6. The result is a percentage which when applied to point rainfall yields the equivalent uniform depth of rainfall.

2.3.3 <u>RAINFALL TIME-DISTRIBUTION</u>

Rainfall time-distribution is the amount of rainfall that occurs during certain hourly time periods during the total storm duration. Obviously the hourly increments of a storm cannot be predicted. However, this much we know; if the greatest hourly increment occurs during the first hour of a storm, a smaller flood results than if the greatest hourly increment occurs during the sixth hour of a storm. This is because the first hour's rain serves to satisfy any soil-moisture depletion and fill up ground depressions before any substantial runoff takes place. A heavy rainfall later in the storm has a higher percent of runoff over the same time period.

The U.S. Bureau of Reclamation points out that although a usual sequence of events produce floods, it is the unusual event or series of events that produce the great floods. It is because of the inadequate rainfall records in East Pakistan, and the inability to determine what is usual and what is unusual, that the four-month rainfall index has been recommended for pre-monsoon, post-monsoon, and monsoon studies. Also because of this it is recommended that storms of short duration not be used. Ordinarily a storm of 3-days duration will be sufficient for most basins we will encounter, but a 4 or 5-day storm should also be investigated.

At this point in our discussion we will pause and by example develop a 5-day rainfall based on the preceeding principles. Completing this we will make a rainfall time-distribution that will form our design storm.

PROCEDURE

Problem: Determine the probable 5-day rainfall from a storm with a 10-year recurrence frequency for the Sarai River Basin in Rangpur District.

A. Point Rainfall:

1. From the isohyetal map in the appendix the 4-month rainfall index is 65".

2. Using the combined rainfall indices from Table II-1, the accumulative 5-day rainfall is:

Days	Accumulative Point Rainfall
1	8.3"
2	12.5"
3	15.0"
4	16.7"
5	17.9"

Table II-3

B. **Equivalent Uniform Depth of Rainfall:**

1. Figure II.4 is a map of the basin area with its geographical center located and isohyetals drawn at one mile intervals starting one-half mile from the center.

2. Area between isohyetals have been plani-metered and the percentages from Table II-2 applied, as shown in the following table:

Average Distance From Storm Center	Area sq. Miles	Area Times percentage
1/4	0.8	0.8
1	6.3	5.6
2	10.5	8.6
3	10.5	8.1
4	7.5	5.6
5	4.8	3.5
6	2.4	1.7
	42.8	33.9

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

$$\frac{33.9 \times 100}{42.8} = 79\%$$

4. Convert accumulative point rainfall from "A" above to daily uniform depth equivalents by multiplying Table II-3 values by 79%.

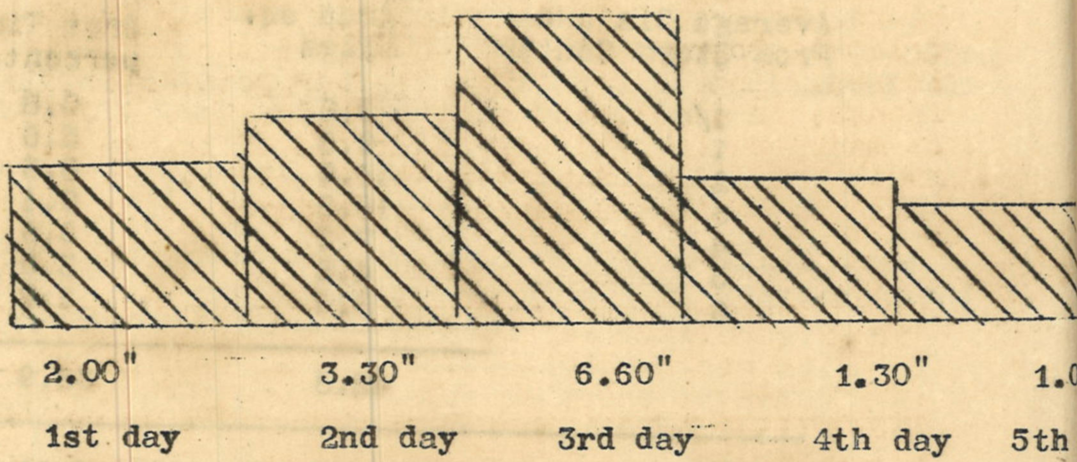
Equivalent Uniform Depth

Days	Accumulative Total	Daily Increments
1	6.6"	6.6"
2	9.9"	3.3"
3	11.9"	2.0"
4	13.2"	1.3"
5	14.2"	1.0"

Table II-5

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 earlier, these daily increments can occur in any order. We will arbitrarily arrange as 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 UNIFORM EQUIVALENT DEPTH

Figure II-6

For future calculation we will wish to know the quantity o rain falling during certain hourly intervals. As an aid fo breaking down rainfall time-distribution into intervals le than 24-hours, Table II-7 has been prepared from Master Pl rainfall data. A reasonable 24-hour rainfall time-distribu can be made for any basin by using the particular rainfall data closest in distance to the basin being studied and ad ting it to fit the calculated 24-hour basin rainfall.

	4-No. Index	Storm Duration in Hours Accumulative rainfall in Inches						
		1	2	3	6	12	18	2
Barisal	65	2.33	2.77	3.11	3.82	5.36	6.85	8.
Jessore	46	2.37	3.00	3.31	3.80	4.43	5.15	5.
Cox's Bazar	115	2.33	2.88	3.31	4.40	6.65	10.50	14.
Chittagong	86	2.43	3.07	3.58	4.56	6.67	9.00	11.
Dacca	55	3.10	4.00	5.50	5.25	6.10	6.60	7.
Bogra	58	3.40	4.23	4.70	5.47	6.30	6.90	7.
Sylhet	120	2.85	4.15	5.00	6.16	8.20	12.00	15.

24-Hour Point Rainfall Time-Distribution

Table II-7

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. Two to 6 miles; 3 hours
3. Over 6 miles; 6 hours

Our Sarai Basin example has a main drainage course length of about 20 miles, so we will use a unit interval of 6-hours. The rainfall data located closest to Rangpur District is Bogra, with a 24-hour point rainfall of 7.4 inches. Since Sarai has a 24-hour point rainfall of 8.3 inches, an adjustment must be made. The adjustment will be made on the basis of the ratio of the respective 4-month rainfall indices; that is, 65/58. The following table shows the adjustment procedure.

	Hours			
	6	12	18	24
Bogra Maximum Accumulative Point Rainfall	5.47	6.30	6.90	7.40
Sarai Maximum Accumulative Point Rainfall	6.13	7.06	7.74	8.30
Sarai Maximum Incremental Point Rainfall	6.13	0.93	0.68	0.56
Sarai Uniform Equivalent Depth	4.85	0.75	0.55	0.45

SARAI 24-HOUR RAINFALL
TIME- DISTRIBUTION

Table II-8

As pointed out for rainfall time-distribution with 24-hour increments, the increments within a 24-hour period also cannot be predicated. 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 0.55", 0.45", 4.85", and 0.75".

We have now established the rainfall sequence for the peak 24-hour period. It only remains to determine a sequence for the other 24-hour periods and our design storm is complete.

Day	6-Hour Increments
1	0.55"
	0.45"
	0.25"
	0.75"
2	0.55"
	0.45"
	1.55"
	0.75"
3	0.55"
	0.45"
	4.85"
	0.75"
4	0.55"
	0.45"
	0
	0.30"
5	0.55"
	0
	0
	0.45"

SARAI BASIN DESIGN STORM

*UNIFORM EQUIVALENT DEPTH

RAINFALL TIME-DISTRIBUTION

Table II-9

2.4 IN WHAT MANNER DOES THE WATER REACH THE SLUICE?

Rainfall runoff passing through a drainage basin is like a massive foot-race run over an obstacle course. However, instead of being interested in the winner and his time, we are concerned with the greatest number of runners who cross the finish line simultaneously and the time it takes them to do so. In Section 2.3 we prepared a design storm, which, to continue our analogy, told us the number of participants starting the race. In the following sections we will analyze the race and see what happens to the participants. To do this we will make two general classifications: those runners who become irretrievably lost within the basin and those who eventually find their way to the finish line. The first group is called rainfall losses, and the second, rainfall excess.

2.5 RAINFALL LOSSES

Rainfall losses are those portions of precipitation which upon falling to the earth's surface are either retained where they fall, pass into the soil as infiltration, or return to the atmosphere. Principal losses can be catagorized as follows:

1. Interception: Rainfall stored on the surface of basin vegetation.

2. Depression Storage: Water retained in puddles, ditches and other depressions on the soil surface.

3. Soil Moisture: Infiltration water stored in the voids between soil particles.

4. Interflow: Infiltration water that moves horizontally throught the upper soil layers and returns to the surface at another point.

5. Groundwater: Vertical passage of water that reaches the groundwater table.

6. Consumptive Use: During storm periods of long duration the vegetal covering in the basin extracts water from one of the above sources to sustain growth. This loss is replaced by future rainfall.

We will neglect interception and evaporation, which are minor losses. Interflow and groundwater will be treated with soil moisture since most soils in East Pakistan range from below average infiltration after saturation to impermeable.

2.5.1 LAND CLASSIFICATION

Use of the land within a basin will be a deciding factor in determining what losses occur. The land of East Pakistan is primarily used for agricultural purposes and can be roughly classified into four groups depending upon the extent of annual flooding.

1. Intermediate Land comprises 45% of the total cultivated land area and is flooded to a depth of 2 feet annually. Principle crops from March to July are aus rice and jute, and in the fall and winter pulses, wheat, oil seeds and sugar cane.

2. Lowland comprises 25% of the total cultivated land area and is flooded to a depth from 3 to 12 feet annually. Principle crops from March to April are a mixture of broadcast aman rice and aus rice. In the winter it lies idle.

3. Very Low Land comprises 5% of the total cultivated land area, and is deeply flooded annually. This land mosly lies in Sylhet and the principle crop is boro rice.

4. Highland comprises about 25% of the total cultivated land area and is unaffected by flooding. The principle crop is mainly transplanted aman rice and it is not cropped in the winter.

A breakdown of the land use classification for the total area of East Pakistan(excluding Chittagong Hill Tracts) is as follows:

1. Forest Area -- 8%
2. Not available for cultivation -- 25%
3. Current fallow -- 3%
4. Net area sown -- 64%

Of the total land available for cultivation, 62% is cultivated as paddy. This amounts to 42% of the total land area. Table II-10 is a district by district breakdown of these percentages.

	Percentage of Total Land Area	
District	Available for Cultivation	Cultivated as Paddy Land
Dacca	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
Average	67	42

AGRICULTURAL LAND CLASSIFICATION
Table II-10

Very little study has been made in East Pakistan relating rainfall losses to land use classification. It becomes necessary, then, to review results of studies in other geographical regions and apply them here. We realize that calculated relationships for a particular region should not be used, even for a neighbouring region, without some preliminary verification, and this becomes more important when a different geographical region is involved. However, with the total absence of verification we have no other choice. For this reason we will limit ourselves to two broad land use classifications and estimate losses on the basis of paddy land and non-paddy land.

During the study of a particular drainage basin a field survey should be made to determine the percentage of each of the above classifications. However, for the preliminary study of large basins the percentages can be taken from Table II-10 as an estimate.

2.5.2 <u>SOIL-MOISTURE-INITIAL LOSS</u>

At the begining of a storm the first rainfall striking the earth's surface infiltrates into the soil voids and replenishes any soil-moisture deficit. The amount of the deficit depends on the amount of rainfall that has occured in a period from 5 to 30 days prior to our storm. We will not be particularly concerned with antecedent conditions, other than to say they are average, because in effect we have provided our own when we made our rainfall time-distribution.

The amount of water a soil is capable of holding depends upon the texture of the soil. Soil textures in East Pakistan range from fine sand to clay, but silty loams and silty clay loams predominate. Permeability generally ranges from moderately slow to slow. Using the U.S. Soil Conservation Service classifications of hydrologic soil groups, most of these soils would come under Group C.

The Soil Conservation Service has recommended an initial abstraction of

$$Ia = 0.2 S$$

be made to cover initial interception, infiltration, and surface storage losses for this group. "S" is the difference between storm rainfall and direct runoff. Utilizing the SCS rainfall-runoff curves for land use classifications and soil groups similar to those in East Pakistan, "S" has been found to equal 2.5 inches, and I_a equals 0.50 inches.

Studies in the Soviet Union indicate that for clay and loamy soils the initial loss from wetting the soil, filling up small depressions on the surface, and infiltration is about 0.3 to 0.4 inches.

RECOMMENDATIONS

Paddy Land: During the rainy season paddy land will be under active cultivation. The farmers will have applied water to the paddy and the soil will be saturated. No initial loss will be used.

Non-Paddy Land: It will be assumed that the first one half inch of rain falling on non-paddy land will be required to satisfy the initial soil-moisture deficit.

Any substantial areas covered with standing water, such as beels, rivers, or during the monsoons, low lands, will be treated as paddy and will not have an initial loss.

2.5.3 SOIL-MOISTURE-SUBSEQUENT LOSS

During the storm period infiltration continues to take place at a slower rate than the initial soil-moisture repletion. Infiltrometer studies have shown that all but impervious clay soils have a minimum constant infiltration rate after saturation that may range from 0.05 inch/hour to greater than 1.00 inch/hour. This movement of water down into the soil must be replaced at the surface by direct rainfall or from depression storage.

We will assume an average infiltration rate of 1.00"/day (0.04"/Hour) for the relatively impervious soils in East Pakistan. This constant rate will be used for both paddy land and non-paddy land throughout the storm period starting from the time the initial soil-moisture loss has been satisfied.

In applying the above loss some judicious judgement will have to be exercised. During the pre-monsoon season the prevailing ground water table will be relatively low and the loss can be applied over the entire basin area, except possibly for some low depression areas in the Sylhet District. During the monsoon season the ground water will have risen, due to interflow from the rivers, so that these losses may not take place. The basin area should be checked for low lying land, probably used for paddy, that will become flooded to a level equal to the ground water table. These areas will not have an infiltration loss. If possible, data on ground water levels should be obtained

2.5.5

to help determine how much of an area in the basin will be at or under the ground water level. Only land 2 or 3 feet above this level should have the infiltration lo applied.

2.5.4 DEPRESSION STORAGE

After the initial soil-moisture deficits have been sati fied, rain water begins to accumulate in small surface depressions. As these fill up they overflow into other larger depressions and finally overland flow to the ba drainage system takes place. The depression retention r 2.6
is not constant throughout the storm period. The rate i greater early in the storm and decreases gradually duri the storm period.

All basins will not have the same rate of storage and any particular basin will not have the same rate during different seasons ofthe year. To simplify the computations, then, a constant rate of retention will be assu- med. This retention will only occur actual periods of rainfall. If there are periods when there is no rain, as there is with our Sarai design storm, no losses will be deducted for that period.

RECOMMENDATIONS

Paddy Land: Paddy land is bounded by small earth bunds constructed by the farmer to retain paddy water. These may be as high as six inches or more. The bunds are ofte used as walkways by the farmers and livestock for passag across a cultivated area, and consequently become uneven and broken down. We will assume that the first 4 inches of rain falling upon paddy land serves to fill up the paddy basin. Rainfall in excess of the 4 inches becomes runoff.

Non-Paddy Land: The general discussions above pertain more to non-paddy land than they do for paddy land. We will assume that the constant rate of retention is approximately 0.033 inches/hour during periods of rainfall, provided there is sufficient rainfall to satisfy this loss. For instance; Assume over a 6-hour period 0.35 inches of rainfalls. If 0.25 inches of this goes into subsequent soil-moisture loss, then there only remains 0.10 inches for depression storage, not the 0.20 inches if the full constant rate were used for 6 hours. The loss will be applied only during the first hours of rainfall until 1.00 inches of total loss has gone into depression storage, after which no further losses will be taken.

2.5.5 CONSUMPTIVE USE

During a storm period vegetation continues to grow. The water required to sustain life will mostly come from the soil-moisture stored in the upper soil levels. As this moisture is extracted from the soil additional water will infiltrate down into the voids. Paddy consumptive use will vary from about 0.25 to 0.50 inches per day, depending upon its period of growth. We will neglect this loss because we have provided sufficient moisture for replacement under Soil-Moisture-Subsequent Loss.

2.6 RAINFALL EXCESS

Total runoff volume, expressed as inches of depth over the basin area, is termed rainfall excess. Or, expressed in another form, rainfall excess is the difference between total precipitation and retention losses expressed in inches.

In the preceding sections we have estimated the various individual losses that will occur. In this section we will separate these losses from total precipitation to find rainfall excess. The procedure we will use to separate these losses will be in a tabular form. First we will assume the entire basin area consists of paddy land, and losses attributed to paddy will be extracted. Then we will assume the entire basin area consists of non-paddy land, and non-paddy losses will be extracted. Finally, we will combine paddy and non-paddy rainfall excess into a weighted basin average based upon the percentage of basin area attributed to each classification.

We are interested not only in the total volume of rainfall excess, but also the volume occuring during each 6-hour increment of the storm. The rainfall time distribution of our storm (Table II-9) will be used to accomplish this. All losses will be deducted from each 6-hour rainfall increment giving the rainfall excess for the period. The final result will be a runoff time-distribution for the basin. This should not be confused with the runoff hydrograph, which will be discussed later. The runoff time-distribution only indicates the amount of water, covering the whole basin area, available for runoff during each 6-hour period. Referring to our foot race analogy, we can say that due to some prophetic foresight we are able to determine, before the race begins, those runners who will never finish the race. So we are about to eliminate them from further consideration. The runoff time-distribution is the list of remaining hardy souls who are lined up at their respective starting posts and the intervals at which they will begin the race.

2.6.1 PADDY LAND RAINFALL EXCESS

Our Sarai example will be for a pre-monsoon storm when the ground water table is low and does not influence the infiltration rate. We will assume the following losses:

1. Initial soil-moisture loss; 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. The soils of Rangpur District are classified as interstream alluvial soils. Generally their permeability ranges from moderately slow in sandy to silty loams, to slow in silty soils.

3. Initial paddy depression storage assumed to be available; 4.00 inches.

Table II-11 indicates the computation procedure. Rainfall in inches is taken from Table II-9. Loss columns are as described above. Available paddy storage is the storage available at the end of each 6-hour period. Losses and available storage are indicated as negative numbers since they represent deficits. Net runoff is the rainfall excess if the entire basin area consisted of paddy land.

Starting at the 1st 6-hour period the losses are separate from rainfall and the result entered in the storage column. Storage is cumulative, so in the 1st 6-hour period the computations are

$$0.55'' - 0.25'' - 4.00'' = -3.70''$$

The process is continued with no runoff taking place until all available paddy storage is satisfied. This does not occur until the 3rd period of the 3rd day. At this point the computations are

$$4.85'' - 0.25'' - 0.20'' = + 4.40''$$

2.6.2 NON-PADDY LAND RAINFALL EXCESS

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

1. Initial Soil-moisture loss: 0.50 inch.
2. Subsequent soil-moisture loss will be the same as for paddy land; 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 inch is stored.

Table II-12 indicates the computation procedure. Column headings are the same as for Table II-11 and the procedure similar. As an example of how depression storage is handled, refer to the 2nd period of the 1st day. Here the rainfall is 0.45 inches, soil-moisture loss is 0.25 inch, and no previous available storage. Deducting 0.25" from 0.45" leaves 0.20" remaining for depression storage. In the 3rd period, 1st day, there is no rainfall available for depression storage at all. After the 2nd period, 2nd day, we have accumulated a total depression storage of 0.85", which leaves 0.15" to be taken from the 4th period, to complete the 1.00" storage requirement. It should be remembered that during periods when no rain falls, depression storage should not be deducted.

2.6.3 RAINFALL EXCESS--WEIGHTED BASIN AVERAGE

We do not have a detailed land classification breakdown for the Sarai Basin, but since its area is 43 square miles we will assume it is comparable to the whole of Rangpur District. Referring to Table II-10 we find that 42% of the total land area is paddy land, leaving 58% as non-paddy land. Table II-13 shows our computations for a weighted average. The net runoff columns are taken from Table II-11 and 12 respectively. The weighted runoff columns are obtained by multiplying paddy land runoff by 42%, and non-paddy land runoff by 58%. The basin weighted runoff is the sum of the paddy and non-paddy weighted runoff.

Our runoff time-distribution is now complete. Total rainfall excess for the 5-day storm is 7.10 inches, or 50% of the total storm precipitation.

Day	Hours	Rainfall Inches	Losses			Available Paddy Storage
			Soil Moisture*		Depression Storage	
			Initial	Subse quent		
1	0 -6	0.55	0	-0.25	-4.00	-3.70
	6-12	0.45		-0.25		-3.50
	12-18	0.25		-0.25		-3.50
	18-24	0.75		-0.25		-3.00
2	0- 6	0.55		-0.25		-2.70
	6-12	0.45		-0.25		-2.50
	12-18	1.55		-0.25		-1.20
	18-24	0.75		-0.25		-0.70
3	0- 6	0.55		-0.25		-0.40
	6-12	0.45		-0.25		-0.20
	12.18	4.85		-0.25		0
	18-24	0.75		-0.25		0
4	0- 6	0.55		-0.25		0
	6-12	0.45		-0.25		0
	12-18	0		-0.25		-0.25
	18-24	0.30		-0.25		-0.20
5	0- 6	0.55		-0.25		0
	6-12	0		-0.25		-0.25
	12-18	0		-0.25		-0.50
	18-24	0.45		-0.25		-0.30

SARAI BASIN-RAINFALL EXCESS

PADDY LAND

TABLE II-11

Day	Hours	Rain Fall Inches	Losses: Soil Moisture Initial	Losses: Soil Moisture Subsequent	Losses: Depression Storage	Available Non-Paddy Storage	Net Run Off
1	0- 6	0.55	-0.50	-	-0.05	0	0
	6-12	0.45		-0.25	-0.20	0	0
	12-18	0.25		-0.25	0	0	0
	18-24	0.75		-0.25	-0.20	0	0.30
2	0- 6	0.55		-0.25	-0.20	0	0.10
	6-12	0.45		-0.25	-0.20	0	0
	12-18	1.55		-0.25	-0.15	0	1.15
	18-24	0.75		-0.25	0	0	0.50
3	0-6	0.55		-0.25	0	0	0.30
	6-12	0.45		-0.25	0	0	0.20
	12-18	4.85		-0.25	0	0	4.60
	18-24	0.75		-0.25	0	0	0.50
4	0- 6	0.55		-0.25	0	0	0.30
	6-12	0.45		-0.25	0	0	0.20
	12-18	0		-0.25	0	-0.25	0
	18-24	0.30		-0.25	0	-0.20	0
5	0- 6	0.55		-0.25	0	0	0.10
	6-12	0		-0.25	0	-0.25	0
	12-18	0		-0.25	0	-0.50	0
	18-24	0.45		-0.25	0	-0.30	0

SARAI BASIN - RAINFALL EXCESS

NON-PADDY LAND

TABLE II-12

Day	Hours	Paddy Land: Net Runoff	Paddy Land: Weighted Runoff	Non-Paddy Land: Net Runoff	Non-Paddy Land: Weighted Runoff	Basin 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	0.30	0.17	0.17
2	0- 6	0	0	0.10	0.06	0.06
	6-12	0	0	0	0	0
	12-18	0	0	1.15	0.67	0.67
	18-24	0	0	0.50	0.29	0.29
3	0- 6	0	0	0.30	0.17	0.17
	6-12	0	0	0.20	0.12	0.12
	12-18	4.40	1.85	4.60	2.67	4.52
	18.24	0.50	0.21	0.50	0.29	0.50
4	0- 6	0.30	0.13	0.30	0.17	0.30
	6-12	0.20	0.08	0.20	0.12	0.20
	12-18	0	0	0	0	0
	18-24	0	0	0	0	0
5	0- 6	0.10	0.0[illegible]	0.10	0.06	0.10
	6-12	0	0	0	0	0
	12-18	0	0	0	0	0
	18-24	0	0	0	0	0
	Total	5.50		8.25		7.10

SARAI BASIN - RAINFALL EXCESS
WEIGHTED BASIN AVERAGE

TABLE II-13

2.6.4 ANTECEDENT CONDITIONS

In Section 2.5.2 we stated we would not be particularly concerned with antecedent conditions other than to say they were average. For most designs this will generally be true. However, it should be pointed out that antecedent conditions will have some effect on runoff and for certain cases this factor should be recognized. The U.S. Soil Conservation Service has three general classifications of antecedent conditions.

CONDITION I:

A condition of watershed soils where the soils are dry but not to the wilting point, and where satisfactory plowing or cultivation takes place.

CONDITION II:

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

CONDITION III:

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

We will use the same three conditions and re-define them to apply to non-paddy land in East Pakistan.

CONDITION I:

Storms occurring during the pre-monsoon months of March and April and the post-monsoon month of November. This condition is not considered applicable to the design of drainage structures.

Initial soil-moisture loss: 0.75"
Subsequent loss: 1.00"/day
Total depression storage: 1.75"

CONDITION II:

Storms occurring during the pre-monsoon months of May and June and post-monsoon month of October. This condition will be the general condition for design where drainage is restricted by high river levels during the monsoon season.

Initial soil-moisture loss: 0.50"
Subsequent loss: 1.00"/day
Total depression storage: 1.00"

CONDITION III:

Storms occurring during the monsoon months of July to September. This condition will be used where drainage is not restricted by high river levels,

such as for pumping plant design. It will als
be used in place of Condition II for areas be
cultivated under an irrigation system that kee
the land in a state of near saturation.

Initial soil-moisture loss: 0.25"
Subsequent loss: 1.00"/day
Total depression storage: 0.50"

In all cases the non-paddy depression storage is to be extracted at the rate of 0.033"/hour during the periods of rainfall, as was done in the Sarai example.

Paddy land will be treated the same for all three conditions. That is:

Initial soil-moisture loss: nil
Subsequent loss: 1.00"/day
Paddy depression storage: 4.00"

It should be remembered that paddy depression storage i not restricted by an hourly rate. The total difference between rainfall and subsequent loss goes into paddy sto rage until the total 4.00" is satisfied.

If the designer wishes to use a more extreme rate of ru off for major pumping plant design, paddy depression st age may be reduced to 3.00". Values less than this are recommended.

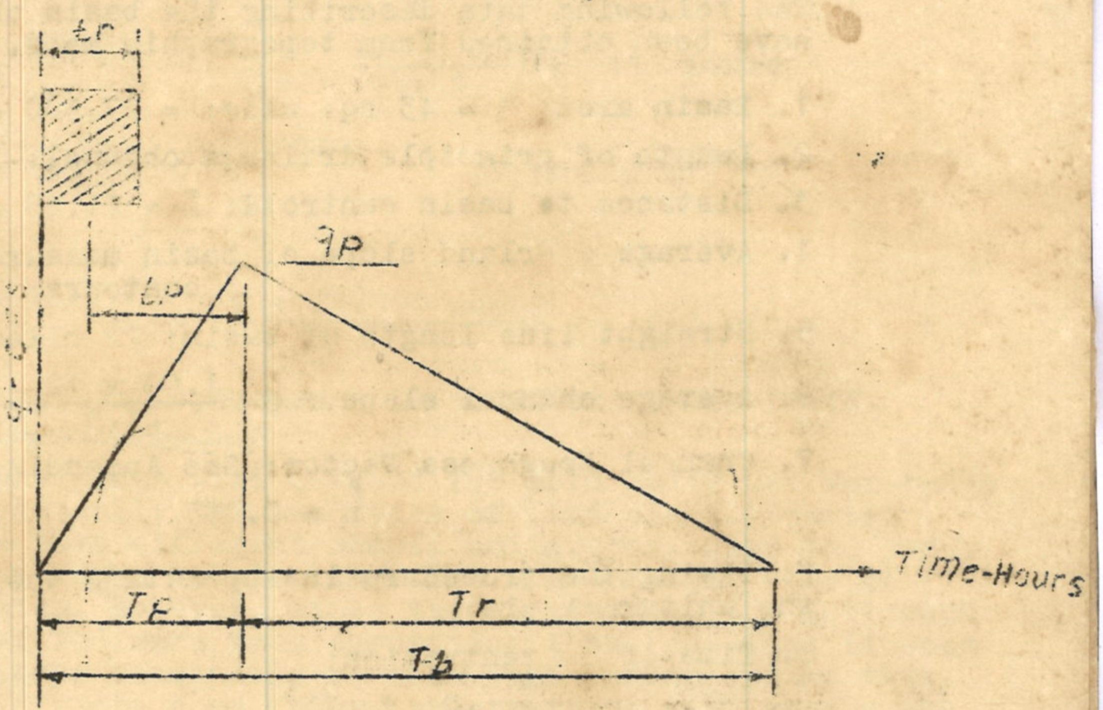
2.7 THE UNIT HYDROGRAPH

2.7.1 The unit hydrograph, or unitgraph, is a graphical prese ation of discharge with respect to time at the drainage basin outlet. Again, using our foot-race analogy, the u graph indicates the order in which the participants cro the finish line. The unitgraph does not represent the entire race, but only a selective sampling of the partic pants. After determining the order of the sample the out come of the rest of the race can be found.

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. The unit duration is the same time interval we selected for our rainfall time-distribution. Using our Sarai Basin example, the unitgraph is formed from one inch of rainfall excess /falling uniformly over the entire basin area during a unit time interval of 6 hours.

2.7.1 <u>THE UNIT HYDROGRAPH SHAPE</u>

Most of the basins we will encounter will be ungauged with scanty hydrologic data available. Therefore, rather than using the more complicated and classical curvilinear hydrograph shape we will use the simpler triangular approximation. The shape of the hydrograph and its nomenclature is as follows:



TRIANGULAR UNITGRAPH DEFINITION
FIGURE II-14

q_p = peak rate of runoff in cfs

T_p = time of rise from begining 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

2.7.2 DEVELOPING THE UNIT HYDROGRAPH

A discussion of the method used to develop the trian unitgraph is given in Appendix A. Refer to Article A for the step-by step procedure.

EXAMPLE: SARAI BASIN

The following data describing the basin characterist have been obtained from topographic maps.

1. Basin area: A = 43 sq. miles = 27,500 acres
2. Length of principle drainage channel: L = 21.5 mi
3. Distance to basin centroid: $\bar{L}$ = 11.25 miles
4. Average overland slope of basin measured between contours: 1 ft/mile
5. Straight line length of basin: 12 miles
6. Average channel slope : $S = \frac{1.00 \times 12}{21.5} = 0.56$ ft/mi
7. Channel Roughness Factor: (See Appendix B) $n = 0.055$

Following the procedure in Appendix A the elements of the unitgraph are:

8. Time of concentration:

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

$$T_c = 31\left(\frac{21.5^2 \times 0.055^2}{0.56}\right)^{0.3} = 41 \text{ hours}$$

9. Lag time to peak:

$$t_p = 0.6\ T_c = 24.6 \text{ hours}$$

10. Time of rise:

$$T_p = t_p + \tfrac{1}{2}\ t_r$$

In Section II, Article 2.3.3 a unit time interval of hours was used in developing the design storm; t_r the also equals to 6 hours.

$$T_p = 24.6 + \tfrac{1}{2} \times 6 = 27.6 \text{ hours Use 28 hour}$$

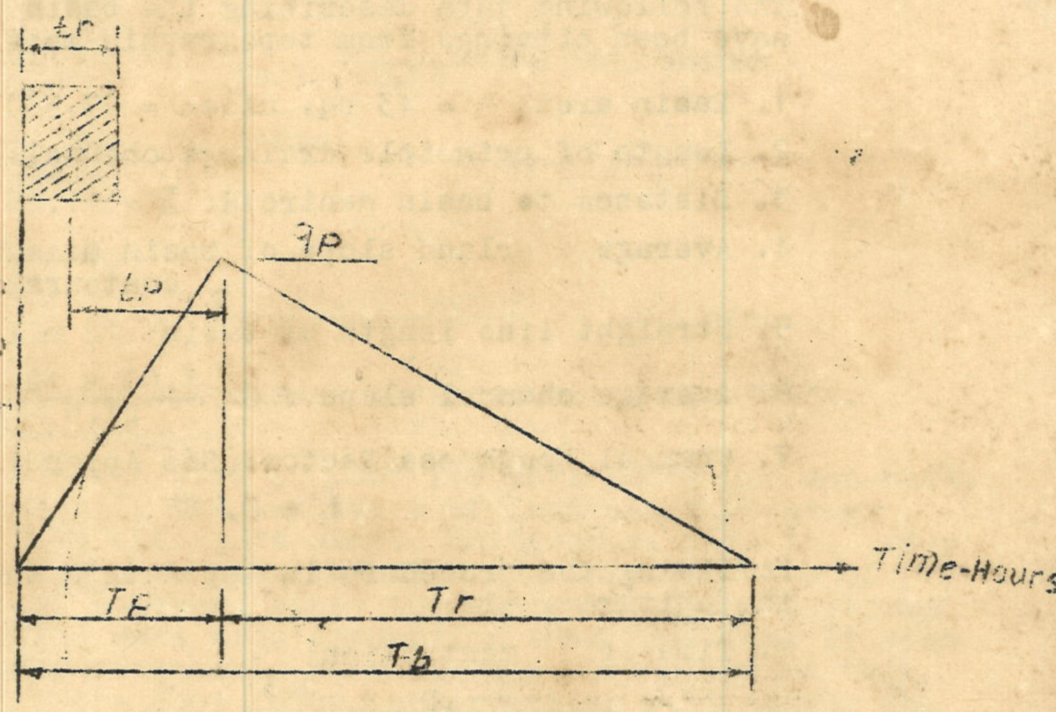
11. Coefficient C_t:

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

$$C_t = \frac{24.6}{(21.5 \times 11.25)^{0.3}} = 4.72$$

2.7.1 THE UNIT HYDROGRAPH SHAPE

Most of the basins we will encounter will be ungauged with scanty hydrologic data available. Therefore, rather than using the more complicated and classical curvilinear hydrograph shape we will use the simpler triangular approximation. The shape of the hydrograph and its nomenclature is as follows:



TRIANGULAR UNITGRAPH DEFINITION

FIGURE II-14

q_p = peak rate of runoff in cfs

T_p = time of rise from begining 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

2.7.2 DEVELOPING THE UNIT HYDROGRAPH

A discussion of the method used to develop the triangul unitgraph is given in Appendix A. Refer to Article A.5 for the step-by step procedure.

EXAMPLE: SARAI BASIN

The following data describing the basin characteristics have been obtained from topographic maps.

1. Basin area: $A = 43$ sq. miles = 27,500 acres
2. Length of principle drainage channel: $L = 21.5$ miles
3. Distance to basin centroid: $\bar{L} = 11.25$ miles
4. Average overland slope of basin measured between contours: 1 ft/mile
5. Straight line length of basin: 12 miles
6. Average channel slope : $S = \frac{1.00 \times 12}{21.5} = 0.56$ ft/mile
7. Channel Roughness Factor:(See Appendix B) $n = 0.055$

Following the procedure in Appendix A the elements of the unitgraph are:

8. Time of concentration:

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

$$T_c = 31 \left(\frac{21.5^2 \times 0.055^2}{0.56}\right)^{0.3} = 41 \text{ hours}$$

9. Lag time to peak:

$$t_p = 0.6\, T_c = 24.6 \text{ hours}$$

10. Time of rise:

$$T_r = t_p + \tfrac{1}{2}\, t_r$$

In Section II, Article 2.3.3 a unit time interval of 6 hours was used in developing the design storm; t_r then, also equals to 6 hours.

$$T_p = 24.6 + \tfrac{1}{2} \times 6 = 27.6 \text{ hours Use 28 hours}$$

11. Coefficient C_t:

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

$$C_t = \frac{24.6}{(21.5 \times 11.25)^{0.3}} = 4.72$$

12. Coefficient C_p :

From Figure II - 15 $C_p = 0.44$

13. Unitgraph Base length:

$$T_b = \frac{1.2\ T_c}{C_p}$$

$$T_b = \frac{1.2 \times 41}{0.44} = 112 \text{ hours}$$

14. Peak Discharge:

$$q_p = \frac{2A}{T_b}$$

$$q_p = \frac{2 \times 27{,}500 \text{ acres}}{112} = 491 \text{ cfs.}$$

The completed hydrograph is shown on Figure II-16.

This unitgraph will be used for both paddy and non-paddy land. Actually the two types of land classification should have different shaped unitgraphs, with the paddy land having a shorter lag time and a faster time of recession. However, since the elements of the unitgraph are derived more from assumption than from fact it does not appear reasonable to make this refinement at this time. Perhaps some future observations will supply the data will make this possible.

2.8 THE RUNOFF HYDROGRAPH

The runoff hydrograph represents the time-discharge relationship of the total design storm runoff at the basin outlet. That is, the time and number all our foot-race participitants cross the finish line. The construction of the runoff hydrograph takes advantage of the following unit hydrograph principles. (see Fig.II-14).

1. Hydrographs of the same basin with the same unit rainfall duration, t_r, have vertical ordinates, q_x, directly proportional to the depth of rainfall excess.

2. Hydrographs of the same basin with the same unit rainfall duration have the same time base, length, T_b, regardless of intensity of rainfall.

3. Several of the above hydrographs can be combined to form one composite hydrograph by the simple addition of their vertical ordinates.

By breaking our 5-day storm into 6-hour time intervals we have, in effect, created 11 sub-storm periods of rainfall excess, each proportional to the 6-hour unitgraph. 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.

This procedure can be done either graphically or by using a tabular form. The tabular form will be used here. The computations are shown on Table II-17.

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. Since the time of rise usually won't be a multiple of our chosen time interval we start from the peak to make sure we catch the maximum discharge.

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

Columns 4 to 13 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.(see Table II-13).

Column 14 is the summation of values for each 6-hour increment. These values represent the discharge ordinates of the runoff hydrograph, and are shown graphically on Figure II-18.

A check on the computations can be made using the following formulae:

1. Total volume of water under the hydrograph:

$$V = \frac{\Sigma It}{12} \qquad \Sigma I = \text{from column 14}$$
$$t = \text{6-hour time interval}$$

$$V = \frac{32,258 \times 6 \text{ hours}}{12} = 16,129 \text{ acre-feet}$$

2. Total rainfall excess, i

$$\Sigma i = \frac{12\ V}{A} \qquad A = \text{basin area, acres}$$

$$\text{or, } \Sigma i = \frac{It}{A}$$

$$\Sigma i = \frac{32,258 \times 6 \text{ hours}}{27,500 \text{ acres}} = 7.03\ ''$$

This should equal the summation of weighted rainfall excess through the 4th day in Table II-13, or 7.00".

UNIT GRAPH CFS	HRS.	DAY	RAINFALL EXCESS - 6 HOUR DURATION 0.17"	0.06"	0.67"	0.29"	0.17"	0.12"	4.52"	0.50"	0.30"	0.20"	RUNOFF HYDRO-GRAPH
0	18	1	0										0
70	24		12	0									12
176	6		30	4									34
281	12	2	48	11	0								59
386	18		66	17	47	0							130
491	24		84	23	118	20	0						245
466	6		79	29	188	51	12	0					359
421	12	3	72	28	259	82	30	8	0				479
395	18		65	25	329	112	48	21	316	0			916
350	24		60	23	312	142	66	34	795	35	0		1467
316	6		54	21	282	135	84	46	1270	88	21	0	2001
281	12	4	48	19	258	122	79	59	1745	141	53	14	2538
246	18		42	17	234	112	72	56	2220	193	84	35	3065
210	24		36	15	212	102	65	51	2110	245	116	56	3008
175	6		30	13	188	92	50	46	1905	233	147	77	2791
140	12	5	24	10	165	82	54	42	1740	210	140	98	2575
105	18		18	8	141	71	48	38	1590	195	126	95	2326
70	24		12	6	117	61	42	34	1430	175	115	84	2076
35	6		6	4	94		36	29	1270	158	105	77	1830
0	12	6	0	2	70	41	30	25	1110	140	95	70	1583
	18			0	47	30	24	21	950	123	84	63	1342
	24				24	20	18	17	792	105	74	56	1106
	6				0	10	12	13	633	88	63	49	868
	12	7				0	6	8	475	70	53	42	654
	18						0	4	316	53	42	35	450
	24							0	158	35	32	28	253
	6								0	18	21	21	60
	12	8								0	10	14	24
	18										0	7	7
	24											0	0
													32,258

RUNOFF HYDROGRAPH COMPUTATION
TABLE II-17

SECTION III

FLOOD ROUTING

3.1 INTRODUCTION

A flood wave moving through a short reach of channel of regular section will undergo little change in its configuration. However, if the flow is impeded or obstructed in any way the flood wave configuration may be modified appreciably. The determination of this modification is called flood routing. Flood routing in our case will consist of predicting an outflow hydrograph below a drainage structure from a known inflow hydrograph above the structure.

Up to this point in our study we have investigated various conditions and events taking place at relatively remote places from our main point of interest, which is our drainage structure. From these studies we have determined an event which is likely to occur at our point of interest.. a flood. In actually this event may never occur in the manner which we have described it, but from the conditions we have imposed it should approximate an actual similar event.

Having determined an event upon which we can base our design we must now introduce a structure and see what effect it will have on the event. Depending upon the structure used the effect may vary from complete blockage of flow to no effect whatsoever. Obviously the structure we will ultimately use will fall somewhere between these two extremes. It should also be obvious that the designer will have considerable latitude in choosing the final elements of his structure, and the final design will depend a lot upon his individual discretion. It should be kept in mind, then, that although the following examples will only follow the design process through once, the designer should repeat the process several times making various modifications to his design until he is satisfied with the result.

3.2 STORAGE

By placing a drainage structure across a channel the movement of the flood flow is impeded. Discharge through the structure will be somewhat less than the magnitude of inflow and a certain amount of the inflow will be held back as storage. Natural drainage channels are usually capable of holding a considerable amount of water in storage, and if a certain depth of storage outside the channel banks can be allowed, the total available storage may be several times the bank-full storage. The key to our design will be how much storage we can allow outside the channel system.

The principle purpose of an agricultural drainage is to remove excessive water from an area within a period of time sufficient to prevent water logging the soil and causing appreciable damage to plant growth. The degree of protection to be provided depends on the time permissible for removal of excess runoff water. Damage to crops due to standing water in the field depends on the age of the plant, water depth, duration of submergence and the quality of the water. Agriculture experts familiar with the situation in East Pakistan suggest the following criteri

PRE-MONSOON STORMS-APRIL THROUGH JUNE

AUS RICE, in its early growing stage in April, can survive flooding up to 12 inches in depth for periods up to 3 days with muddy water, or 6 days with clear water. Submergence over longer periods at this or greater depths will damage the plant. 3.2

In its later growing stage in late June, aus rice can survive flooding up to $2\frac{1}{2}$ feet. Damage to plants will begin at depths greater than $2\frac{1}{2}$ feet.

BORO RICE in Sylhet District may be subjected to flooding in its mature stage during April or early May. Depths greater than $2\frac{1}{2}$ feet will cause plant damage.

JUTE, during its early growing stage in June, can stand about 2 feet of root submergence for one or two weeks. For longer durations the submerged portion of the fibre will be damaged.

MONSOON STORMS--JULY AND AUGUST

AUS RICE, in its mature stage in late July or early August, can survive flooding up to 3 feet for periods up to 6 days. If it is submerged under 3 feet for only 2 or 3 days, a loss of 20% or 30% will occur.

TRANSPLANTED AMAN, in its early growing stage from July to August, can survive flooding up to 12 inches in depth for periods up to 3 days with muddy water or 6 days with clear water. Submergence over longer periods at this or greater depths will damage the plant.

JUTE, in its mature stage in July and August, can survive flooding up to 3 or 4 feet for periods up to 2 weeks. For any longer duration the submerged portion of the fibre will be damaged.

LATE MONSOON STORMS---SEPTEMBER AND OCTOBER

Usually this will not be a critical period for sluice or pumping plant design. Structures based on the previous criteria will be adequate for this period. An exception is along the southern fringes of the Tripura Hills. Here maximum storms may occur in October and flooding to depths greater than 2½ feet will completely destroy the rice crop. Submerged, the cropping patterns are,

1.	AUS:	Late March to mid July, or, late April to late August.
2.	BROADCAST AMAN:	Mid March to early Decemner.
3.	TRANSPLANTED AMAN:	Early July to mid November, or, late August to early December.
4.	BORO.	Early December to early May
5.	JUTE:	Late April to late August.

3.2.1 CROP DAMAGE COSTS

The criteria under Article 3.2 may prove to be severe and the designer may find it economically unjustified to eliminate all possibility of crop damage. It is here the designer must exercise his judgement. He should investigate the degree of flooding and extent of crop damage using sluices of varying discharge capacity. Then by comparing the differential damage costs and the increased structure cost required to reduce the damage he can make his judgement.

To assist in evaluating crop damage, the following criteria for rice will be used:

1. Up until late August it will be assumed that damaged aman can be re-transplanted and a total crop loss does not occur. We will assume the loss at Rs. 45 per acre, based upon the cost of transplanted seedlings and the farmer's labour.

2. After late August the farmer will no longer be able to transplant.

   a. For areas with irrigation development works in existence use 100% crop damage at the rate of Rs. 300 per acre.

   b. For areas without irrigation development works use 100% crop damage at the rate of Rs. 200 per acre.

3.3 FLOOD ROUTING PROCRDURES

Flood routing is basically a system of book-keeping involving inflow, outflow and storage. During a definite time interval (assuming there are no losses other than those already extracted).

$$I_V - D_V = S$$

Where, I_V = total volume of inflow during the period

D_V = total volume of outflow during the period

S = total volume going into storage during the period.

The equation written for a definite time period and expressing inflow and outflow in CFS is,

$$\frac{S}{t} = I - D$$

Where, S = total volume going into storage during the period measured in cubic feet

t = time of period in seconds

I = average inflow during the period in CFS

D = average outflow during the period in CFS

This equation forms the basis for a hydrologic procedure in routing in which "t" is known as the routing period.

In order to simplify the solution of the problem we will make the assumption that storage is a function of discharge alone. That is, we will assume the storage pond to be a reservoir with the water surface approximately level. Actually there will be a backwater curve within the basin channel system which we are ignoring. This will increase the storage capacity of the channel system and tend to raise the flood levels upstream of the drainage structure. However, because the country is flat, the basin relatively small and our basic data scanty, the simplification is justified.

Generally we will encounter three types of routing problems.

1. WHEN DISCHARGE IS A FUNCTION OF STORAGE LEVEL ONLY.

   This will usually occur during the pre-monsoon and post-monsoon seasons when river stages are low, and possibly during dry periods of the monsoon season. Under this condition the outflow through a sluice is not controlled by the downstream tail-water levels. A characteristic of flow through the sluice will be a hydraulic jump in the stilling pool.

2. WHEN DISCHARGE IS A FUNCTION OF TAILWATER LEVEL

This may occur either during the monsoon season when the storage level is higher than the river stage, or in the coastal regions where there is high tidal influence. A characteristic of flow through the sluice will be a drowned out hydraulic jump.

3. PUMPING STATIONS

Here discharge will not be a function of either storage or tailwater levels, except to the extent that head differential has an effect on pumping efficiency. Head differntial during the drainage cycle for a particular storm will not vary enough to substantially change the pump efficiency, so a constant pumping rate can be assumed.

3.4 DISCHARGE AS A FUNCTION OF STORAGE LEVEL ONLY

In Section IV, CONDITIONS OF FLOW THROUGH SLUICES, we have listed six different conditions of flow that may occur through a sluice, depending upon the relationship between headwater and tailwater levels. Of these six conditions we stated that conditions 2 and 6 will not occur in sluices, so we will eliminate them from further consideration.

For discharge to be a function of storage level only there must be no influence or control exerted by the tailwater. Characteristic of this condition, then, is low tailwater levels. This condition will only occur during the pre-monsoon and post-monsoon seasons when the river stages are low, and in the coastal areas during low tide. This condition will usually be the critical condition for design because maximum discharges may occur at this time. With low tailwater levels the possibility of downstream scour is greater, so extra care must be exercised in setting the stilling pool invert level.

During a typical drainage cycle the storage level will at first be low and discharge will begin under flow condition 5.(It is assumed there will be a properly designed stilling basin as part of the structure and a critical depth control will occur at the outlet. In case discharge is into a flat apron, flow condition 4 may occur instead). As the storage level rises the entrance to the sluice may become submerged and the discharge will pass into 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. As pointed out in

Section IV, we will assume the change between conditio 5 and 3 will take place when the headwater depth is 1.5 times the vent height.

3.4.1 THE BOOK KEEPING EQUATION

General

The book keeping equation expressed in Article 3.3 may be re-written in terms that are known. The subscript 1 in a term will indicate the condition existing at the beginning of time period t, and the subscript 2 will indicate the condition existing at the end of time period t.

$$(I - D) \; t = S$$

Let $S_2 - S_1 = S$ storage differential during period

$\dfrac{I_1 + I_2}{2} = I$ average inflow during period

$\dfrac{D_1 + D_2}{2} = D$ average outflow during period

Substituting the terms;

$$\left(\frac{I_1 + I_2}{2} - \frac{D_1 + D_2}{2} \right) t = S_2 - S_1$$

$$I_1 + I_2 - D_1 - D_2 = \frac{2S_2}{t} - \frac{2S_1}{t}$$

In the above equation we know I_1 and I_2 from our inflow hydrograph. We know S_1, or can assume it as a condition of the problem. Knowing S_1 we will also know D_1 from our stage-discharge relation, which we will discuss later. Transposing all known terms to the left side of the equation, we obtain,

$$\left(I_1 + I_2 \right) + \left(\frac{2S_1}{t} - D_1 \right) = \left(\frac{2S_2}{t} + D_2 \right)$$

This equation can be solved by arithmatic integration using a tabular form. But before we can proceed with the solution to the problem we will have to prepare some curves relating outflow discharge to storage.

3.4.2 STAGE-STORAGE-DISCHARGE RELATIONS

Preliminary to solving the book keeping equation, four curves are required. The inflow hydrograph curve has already been discussed under Section II. The other three curves are the area-capacity curves, the stage-discharge relation curve and the 2S/t + D curve.

AREA-CAPACITY CURVES

This curve should already be familiar to the engineer. However, at the risk of covering familiar ground its preparation will be briefly discussed. A contour map of the area is a prerequisite. Starting at the lowest contour level the areas between successive contour intervals are plainmetered. The area curve is plotted as gound elevation vs. total accumulated area at any elevation. As an example, on fig. III-1 there are 90 acres of land lying at or below elevation 68.

The capacity curve is the accumulation of volume below a particular elevation. Capacity between two contour intervals is calculated as the average of the areas at the two levels multiplied by the contour interval.

For the Sarai Basin the calculations are as follows. Since the contour intervals is one foot the average area in acres equals the incremental capacity in acre-feet.

Elevation feet	Area acres	Average area	Total Capacity acre-feet
60	5	-0	0
61	7	6	6
62	9	8	14
63	13	11	25
64	17	15	40
65	21	19	59
66	25	23	82
67	30	28	110
68	90	60	170
69	420	255	425
70	1950	1185	1610
71	4400	5175	4785
72	7100	5750	10535
73	9600	8350	18885 Total Acres ft.

STAGE-DISCHARGE CURVE

As pointed out under Article 3.4, discharge will occur under flow conditions 3 and 5. Referring to Section IV, and using the procedure discussed there, the formulae for discharge are:

Condition 5: $q = C_2 \; H^{3/2}$

where C_2 = 2.68 for a multiple vent sluice.

Condition 3: $q = C_q \; D \sqrt{2gH}$

With the height of vent equaling 6 feet,

$q = 48 \; Cq \; H^{\frac{1}{2}}$

The discharge, q, is for one foot of vent width, and a five foot vent is five times this value. Figure III-2 is the plotted curve. We have estimated 5 vents as an approximate required capacity and have also indicated this discharge on the cuve.

2S/t + D CURVE

The computations for this curve are performed in the following table. Column (2) is the storage capacity in acre-feet taken from Figure III-1. Column (3) is storage capacity in cubic-feet.(1 acre-foot = 43,560 cubic-feet). In Column (4) we have selected a time interval of 21,600 seconds (6-hours) in order to save space. A 3-hour time interval would have been better and would give better definition at the low stages. Discharge in Column (5) is taken from the stage-discharge curve, Figure III-2.
Column (5) vs. Column (6) is plotted on Figure III-3.

ELEV. FEET 1	ACRE-FEET 2	S 10^6 ft.^{3} 3	2S/t CFS 4	D CFS 5	2S/t + D CFS 6
61.5	10	.43	41	125	166
62	14	.61	56	195	251
64	40	1.74	161	550	711
66	81	3.53	327	1000	1327
67	110	4.80	445	1250	1695
68	170	7.40	685	1515	2200
69	423	18.40	1705	1750	3455
70	1700	74.10	6860	1875	8735
70.5	3000	130.80	12100	1925	14025
71	4900	213.50	19750	1980	21730
71.5	7400	322.50	29850	2035	31885
72	10500	458.00	42450	2090	44540
73	19000	829.00	76700	2290	78992

3.4.3 FLOOD ROUTING EXAMPLE

In the previous article we decided to try a sluice containing 5 vents, each 5 feet wide by 6 feet high. The design flood, represented by the runoff hydrograph in Table II-17, will now be routed through the structure. The computations are done in Table III-4. For convenience we have assumed that there is no water stored behind the structure at the beginning of the flood. Any storage level could have been assumed, but since the storage volume within the first several feet is small there will be little difference in the final result. The procedure is as follows:

1. The first values in Column (6) and (8) are known or assumed.

2. The first value in Column (5) is the value of $2S/t + D$ taken from Figure III-3 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 III-3 corresponding to the value of $2S/t + D$ on the same line.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
DAY	HOUR	I (CFS)	$I_1 + I_2$	$\frac{2S}{t} + D$ (4) + (7)	D (CFS)	$\frac{2S}{t} - D$ (5)-2x(6)	BAS W.S.
1	24	12	-	0	0*	0	60.00
2	6	34	46	46	30	-14	60.60
	12	59	93	79	50	-21	60.80
	18	130	189	168	118	-68	61.40
	24	245	375	307	225	-143	62.20
3	6	359	604	461	340	-219	62.95
	12	479	838	619	455	-291	63.55
	18	916	1395	1104	830	-556	65.30
	24	1467	2383	1827	1325	-823	67.30
4	6	2001	3468	2645	1640	-635	68.45
	12	2538	4539	3904	1770	364	69.10
	18	3065	5603	5967	1840	2287	69.70
	24	3008	6073	8360	1860	4640	69.80
5	6	2791	5799	10439	1880	6679	70.05
	12	2575	5366	12045	1900	8245	70.20
	18	2326	4901	13146	1910	9326	70.25
	24	2076	4402	13728	1915	9898	70.35
6	6	1830	3906	13804	1915	9974	70.35
	12	1583	3413	13387	1910	9567	70.25
	18	1342	2925	12492	1900	8692	70.20
	24	1106	2448	11140	1895	7350	70.18
7	6	868	1974	9324	1875	5574	70.00
	12	654	1522	7096	1855	3386	69.85
	18	450	1104	4490	1795	900	69.30
	24	253	703	1603	1200	-797	66.80
8	6	60	313	-464	0	-	60.00
	12	24	84	-	0	-	-
	18	7	31	-	0	-	-
	24	0	7	-	0	-	-
			0	-	0	-	-

I = 32258 D = 32.276

FLOOD ROUTING COMPUTATIONS

Table III-4

(8)
BASIN W.S.El.
60.00*
60.60
60.80
61.40
62.20
62.95
63.55
65.30
67.30
68.45
69.10
69.70
69.80
70.05
70.20
70.25
70.35
70.35
70.25
70.20
70.18
70.00
69.85
69.30
66.80
60.00
-
-
-
-

6. Column (8) is the basin water surface elevation taken from Figure III-2 for the corresponding values of D in Column (6).

After completing the computations the outflow hydrograph, discharge vs. time, is plotted (Figure III-5). This will be used for stilling basin design. To assist in determining the extent of crop damage, a basin water surface elevation vs. time curve is also plotted (Figure III-6). This completes the flood routing procedure.

3.4.3 ASSESSMENT OF CROP DAMAGE:

In article 3.2 we set the criteria for damage due to pre-monsoon flooding. That is, any area flooded to a depth greater that 12 inches for a period longer, than 3 days. In Article 3.2 1 we set the pre-monsoon damage assessment at Rs. 36/acre.

We can find the area subject to damage by using the basin stage- time relation curve, Figure III-6.

1. Locate the elevation on the curve that has an ordinate 3 days wide. Any land above this elevation will be flooded for a duration less than 3 days; any land below this elevation will be flooded for a duration longer than 3 days.

2. Any land one foot below this elevation will be flooded to a depth at least 12 inches for a period exceeding 3 days. From Figure III-6 we find the 3-day level is at elevation 69.5. Thus, our assumption is that any crops growing at or below elevation 68.5 will be permanently damaged.

3. From the Area-capacity curves, Figure III-I, we find there are 200 acres of land at an elevation lower than 68.5

4. The bankfull level of the drainage khal is El.67 so we can assume the 30 acres below this level is permanently beyond protection and can deduct it from the 200 acre gross area.

5. This leaves 170 acres damaged at a rate of Rs.36/ acre; or a total damage of about Rs. 6,100.

This is a small amount of damage for an area this large. The designer should consider re-routing the flood throug a 4-vent sluice and compare the increased damages with th savings in structural costs. He might also consider routing a 25-year flood through the structure if he feels there is a justification for doing so. The storm may occu other than in the pre-monsoon season, and if the sluice is able to operate he should consider investigating this condition also. It may be that this will be the controlli factor since higher crop damages will occur as the season progresses.

Finally, if the designer feels that his basic informatio is such that he wishes to be conservative, he may satisfy himself that 5-vents are preferable and end his hydrologi investigations at this point. In any event, the designer should make it a practice to record in his design notes his reasons for continuing or discontinuing the investiga tion so that he has a ready reference for future use if his design is ever questioned.

3.5 DISCHARGE AS A FUNCTION OF TAILWATER LEVEL

When discharge occurs under flow conditions 1 and 4 the tailwater level forms a barrier restricting the flow. Usually (but not always) the high tailwater level is due to some outside influence, such as tide or high river stage, which is not directly related to the flood wave passing through the sluice. If the tailwater level were constant throughout the drainage cycle the problem would be relatively simple, however, this is usually not the case. Instead, the river may be rising due to storm run-off outside the basin, or fluctuating due to tidal action.

If simple equations could be established for all the variables, the flood routing could be made by mathematical integration. However, such a solution is neither possible nor practical. Many techniques have been developed to solve this problem. They vary from an arithmetical integration approach to an entirely graphical solution. For simplicity, an arithmetical trial-and-error method will be used here.

The variables we must contend with are,

1. Storm runoff
2. Basin water surface level
3. Sluice discharge
4. Tailwater level
5. River water surface level

The storm runoff and the river water surface level are two unrelated variables and can be determined independently. After the design storm and its runoff have been determined, it is assumed the storm will occur at a particular time of the year. River stage records or tidal records are collected for this period and a maximum, average, or minimum condition is chosen as the design condition. The other variables (basin W.S. level, discharge, and tailwater level) will be dependent upon these two variables.

1. The basin storage level is dependent upon runoff and discharge.

2. Discharge is dependent upon basin storage level and tailwater level.

3. Tailwater level is dependent upon discharge and river stage.

3.5.1 STAGE-DISCHARGE RELATIONS

In previous articles we prepared an inflow hydrograph and basin area-capacity curves for Sarai basin. These curves will be used again with the following exercise. In addition, a new set of discharge relations will have to be developed covering flow conditions 1 and 4 to supplement flow conditions 3 and 5 indicated on Figure III-2.

Since flow condition 1 is a sluice with the outlet submerged, the discharge depends only on the head differential through the sluice. Using the formula from Article 4.3.1,

$$Q = CA\sqrt{2g\,\Delta h}$$

$A = 30$ sq. ft. for a 5' by 6' vent

$C = 0.82$ from Table IV-3

$Q = 198\,\Delta h^{\frac{1}{2}}$ per vent

or, $Q = 990\,\Delta h^{\frac{1}{2}}$ for 5 vents.

The relation curve is plotted on Figure III-7.

The formula for flow condition 4, from Article 4.3.3, is

$$Q = C_1\,dB\sqrt{2g\,\Delta h}.$$

$B = 25$ ft. for 5-5' wide vents

$C_1 = 0.89$ from Table IV-9.

$Q = 178\,d\,\Delta h^{\frac{1}{2}}$ for 5 vents.

Both d and Δh are variables so the equation cannot be described by a single curve. Instead, a family of curves will have to be used, but since this is a lengthy task it will ultimately be easier just to solve the equation as required.

To be able to determine the flow depth, d, we must know the tailwater level at the sluice. If the sluice were discharging directly into the river this level would be approximately the same as the river stage. However, if there is an outfall channel connecting the sluice to the river the tailwater at the sluice will be higher than the river stage due to a backwater effect. Therefore, backwater curves must be computed for various river stages and sluice discharges to determine tailwater levels at the sluice.

Rather than divert our attention from the principles of flood routing, we will reserve discussion on backwater curves for Section V. We will only present the result of these studies (Figure III-8), indicating tailwater elevations for various discharges and river stages.

As we have previously pointed out, the design storm may occur at any time during the rainy season, and the river stage is not necessarily dependent upon the basin runoff. We have chosen the period from late May through early June, 1958, as being representative of Jamuna river stage during an early monsoon storm. The shape of the stage hydrograph (Fig. III-9) suggests that a storm in the area could have caused the river rise, so we will assume our basin design storm is part of a larger area-wide storm.

The reader should note that the shape of the stage hydrograph is similar to the shape of a tidal cycle. Thus, the following method of flood routing can be used in tidal zones by reducing the routing time interval to suit the tidal cycle time interval.

3.5.2 THE BOOK-KEEPING EQUATION

The book-keeping equation is derived from the same general equation discussed under Article 3.3,

$$S = (I - D)t$$

To facilitate the computation procedure we will convert the units to hours and acre-feet. Since there are 43,560 cubic-feet/acre-foot, and 3600 seconds/hour.

$$\frac{1 \text{ acre-foot}}{\text{hour}} = \frac{\text{feet}^3}{43,560} \times \frac{3600}{\text{seconds}} = \frac{\text{CFS}}{12}$$

$$\frac{\text{acre-feet}}{t} = \frac{\text{CFS} \times t \text{ (hours)}}{12}$$

and, $$S = \left[\frac{It}{12} - \frac{Dt}{12}\right]$$

where, S = acre-feet

t = hours

I and D = CFS

5.3 FLOOD ROUTING EXAMPLE

We have assumed the first day of the storm occurs on the 24th of May. We will also assume the basin storage level is equal to the river stage level, Elev. 63.65. From storage volume at this level is 35 acre-feet. These are our known data at the beginning of the storm.

The procedure for routing computations, shown in Table III-10, is as follows:

1. Select a routing time interval, column (2). This interval may be varied during the routing computations.
2. Column (3); obtain an average inflow in CFS from the runoff hydrograph, Table II-17 or Fig. II-18, and convert to acre-feet.
3. Column (4); obtain the river stage at the end of the time period from Fig. III-9.
4. Column (5); determine the tailwater elevation at the sluice by estimating an outflow discharge and interpolating between the indicated river stage curves on Fig. III-8. The discharge can be estimated by taking a value somewhat less than the inflow in Column (3).
5. Column (6); assume a trial basin storage level.
6. With the sluice invert set at elev. 60, determined, H, and h by taking the average elevations at the beginning and end of the time period from columns (5) and (6).
7. Determine which flow condition the sluice is discharging under and compute the outflow discharge, Column (10).

8. Compute the difference between inflow and outflow, column (11), and add (or substract) to the total storage at the beginning of the period, column (12).

9. Find the corresponding storage elevation from the elevation-capacity curve, Fig. III-1

10. Compare the storage elevation in column (13) with the trial elevation in column (6).

11. If they do not agree within 0.1 foot make a second trial elevation and repeat the procedure. At the same time compare the calculated outflow discharge with the assumed discharge used in the determining the tailwater elevation. If necessary, revise the tailwater level.

12. After an agreement is reached, proceed to the next time interval.

13. Continue the process until the drainage cycle is complete.

3.5. ASSESSMENT OF DAMAGE

With the storm occurring in late May and early June we will assume the damaged crop is aus rice which is entering into its later growing season. For this period we will assume the plant is about 2 feet high. With the maximum calculated flood storage level at Elev. 70.65, and the plant able to withstand 2 ft. of submergence, the upper level at which damage begins will be Elev. 68.65.

In calculating the amount of damage we will assume any submergence over 2 ft. for a longer period than 3 days will cause 100% damage. Measuring off the 3-days interval on Fig.III-11 we find this coincides with Elev. 70. Thus the 100% damage level is at Elev. 68. We will assume land lying between Elev. 68 and 68.65 will suffer 50% damage. The amount of 100% damage will be Rs. 200/ acre.

Using the area-elevation curves, Fig.III-1, the estimated damage is computed as follows:

Area below Elev. 68	90 acres
Deduct area within khals	30 acres
Net area suffering 100% damage	60 acres
Total damage = 60 acres x Rs.200/acre	= Rs.12,000
Area below Elev. 68.65	250 acres
Area below Elev. 68.00	90 acres
Net area suffering 50% damage	160 acres
Total damage = 160 acres x 50% x Rs.200/acres	= Rs.16,000

Total flood damage = Rs. 28,000

3.6 DISCHARGE THROUGH PUMPING STATIONS

Probably the simplest form of flood routing is determining the storage level of ponded flood water behind a pumping station. This is because inflow and outflow are entirely independent, and outflow is usually constant. At the same time the problem becomes more involved because of the higher cost of constructing a pumping plant compared to a simple sluice. Whereas we may be able to satisfy ourselves that a sluice is adequate after routing a single design storm with a particular recurrence frequency, a pumping plant will require several storms of various frequencies followed by extensive economic evaluation in order to justify the cost.

As with a sluice, the problem consists of determining an adequate and economic pump discharge capacity to fit the design conditions, and the key to the problem is the level to which we can economically allow flood storage to pond. We will not go into describing the process of a complete economic evaluation. As before we will restrict ourselves to the routing procedure and estimating the damage resulting from one storm.

The equation used in routing are the same as used previously;

$$S = I - D \quad \text{with all units in acre-feet;}$$

$$\text{and Acre-feet} = \frac{\text{CFS} \times t\ (\text{hours})}{12}$$

3.6.1 FLOOD ROUTING EXAMPLE

Rather than develop a new storm we will use the same pre-monsoon storm from previous examples. The reader should refer to Article 2.6.4, pages II-24 and 25, where we have stated that for pumping plants operating during the monsoon season the runoff should be evaluated for Antecedent Condition III, and the storm should have a duration of 10 to 14 days rather than the 5 days used here.

For pumping capacity we will use 1000 CFS as the first trial. In actual practice the engineer should also investigate 800 CFS, 1200 CFS, and so forth, until he has sufficient data to make an evaluation of differential damage costs.

Referring to the computations on Figure III-12, column (3) is the average inflow in CFS taken from the runoff hydrograph, Table III-17. The routing is started on the 4th 6-hour period of the 3rd day when the inflow first exceeds the 1000 CFS pumping capacity. Up to that time it is assumed the pumps have operated intermittantly and have held the storage level to Elev. 65. The Inflow minus Outflow columns are column(3) less the 1000 CFS pumping capacity. Column (6) is the accumulation of storage in acre-feet from column (5). Column (7) is the basin storage level taken from Figure III-1 for corresponding volumes from column (6).

Storage level vs. time from column (7) are plotted on Figure III-13, and the routing procedure is complete.

3.6.2 ASSESSMENT OF CROP DAMAGE

During July and August transplanted aman is in its early growing stage and can survive flooding up to 12 inches of clear water for periods up to 6 days without damage. Crops damaged during this period can be re-transplanted and a total crop loss does not occur. We will assume monsoon rain-water falling in paddy constitutes clear water.

The level of land flooded for a period of 6-days or more is Elev. 69.90 (Fig.III-13), and the level at which crop damage begins is Elev. 68.90. There are 360 acres lying at or below this level. Deducting the 30 acres of khal land below Elev. 67, and assessing damage at Rs. 45/acre, the total amount of damage is Rs. 14,850.

After late August any damaged crops cannot be successfully re-transplanted. Assuming the design storm occurs during this period and the rice has now grown to 18 inches high, the level where damage begins is Elev. 68.40. There are 165 acres lying at or below this level. Deducting the 30 acres of khal land below Elev. 67 and assessing 100% crop damage at Rs. 200/acre, the total amount of damage is Rs. 27,000.

SECTION IV

CONDITIONS OF FLOW THROUGH SLUICES

4.1 INTRODUCTION

Discharge through a sluice vent is similar to culvert discharge. The structure forms a unique type of construction and its entrance is a special type of contraction. Flow characteristics through the structure are very complicated because the flow is controlled by many variables, such as inlet geometry, size, roughness, headwater and tailwater levels, etc. Because of these variables the sluice may act as an orifice, a pipe, a weir, or an open channel, depending upon the prevailing conditions.

The Engineer must realize that flow is variable under these conditions and he must be able to recognise which type of flow will develop under a certain set of conditions. While preparing his design for a sluice he must investigate all conditions and develop a structure which will safely convey the flow under the most adverse condition.

In Section IV we will review the mechanics of flow through sluices for six basic conditions of flow. We will develop the formulae for determining discharge and the discharge coefficients to be used under various flow conditions.

4.2 TYPES OF FLOW THROUGH SLUICES

Flow through a sluice may be classified into two general types depending upon the existing headwater and tailwater levels. If the inlet and outlet are submerged the sluice vent will flow full like a pipe. If the inlet and outlet are not submerged the vent will flow partially full and will act as an open channel.

For practical purposes the two general types can be broken down into six specific types of flow:

Type 1 INLET AND OUTLET SUBMERGED

The outlet is submerged by high tailwater. The vent will flow full like a pipe with the control at the outlet. Discharge is a function of head differential between inlet and outlet water levels.

Type 2 INLET SUBMERGED, OUTLET NOT SUBMERGED, VENT HYDRAULICALLY LONG

Although the tailwater level is below the vent soffit the vent will flow full. The vent is sufficiently long for friction forces to cause the depth of flow to expand, filling the vent. This type is common in long culverts <u>but will not usually occur in sluices.</u>

Type 3 INLET SUBMERGED, OUTLET NOT SUBMERGED, VENT HYDRAULICALLY SHORT

This is the same as Type 2, except the vent is short and will not flow full. It is the usual condition occuring in sluices when the headwater depth is greater than 1.5 times the vent height. The control is at the entrance and is similar to orifice discharge with the discharge coefficient varying from 0.45 to 0.75.

Type 4 INLET NOT SUBMERGED; HIGH TAILWATER

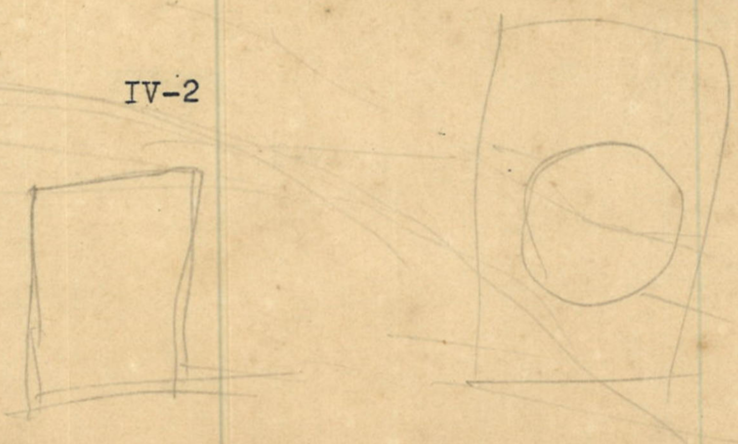
When the headwater depth is less than 1.5 times the vent height the entrance is not sealed by water and the sluice acts as a broad-crested weir. The tailwater level is higher than critical flow depth through the vent. This type is also associated with a drowned out hydraulic jump. The control is at the outlet and the discharge coefficient varies from 0.75 to 0.95.

Type 5 INLET NOT SUBMERGED; LOW TAILWATER

Type 5 is similar to Type 4 except the tailwater depth is less than critical flow depth through the vent. This type will usually occur when a stilling pool, or downstream drop, is part of the structure and the flow is made to pass through critical depth. A hydraulic jump will usually occur downstream. The control is at the outlet and the discharge coefficient varies from 3.47 to 2.46.

Type 6 INLET NOT SUBMERGED; LOW TAILWATER

This is the same as Type 5, except the vent invert slope is greater than critical. Since sluices are usually designed with a flat invert <u>this type will not occur.</u>



4.2.1 A TYPICAL SLUICE DISCHARGE SEQUENCE

The following example illustrates how sluice discharge may pass through the four conditions of flow associated with sluices (Types 1, 3, 4, 5).

Suppose we have a poldered basin with one sluice discharging to an adjacent river. It is near the end of the monsoon season and the basin is flooded from the catchment of monsoon rains. Up till now there has been no release of water because the river flood stage has been higher than the basin storage level. We will assume the storage level is higher than 1.5 times the vent height.

When the river stage falls to slightly below the storage level the gates are opened and discharge begins. Since the river stage is higher than the vent soffit <u>Type 1</u> flow will take place and the vent flows full.

As the river stage continues to fall the storage level will also drop, but usually at a lesser rate. When the river stage drops below the soffit level, the outlet is no longer submerged. Air will enter from the outlet end at the soffit line and will push into the vent causing a separation between the soffit and the water surface in the vent. At this point one of two things may happen.

If the headwater depth is still greater than 1.5 times the vent height the entrance remains sealed by water and flow <u>Type 3</u> takes place. However, if the headwater depth has dropped to less than 1.5 times the vent height the air within the vent will be able to push past the entrance breaking the water seal. <u>Flow Type 4</u> will then take place. In either case there will be an intermittant period of Type 3 flow for some brief or extended duration, depending on the relative inlet and outlet water levels. If there is a stilling basin as part of the sluice structure a characteristic of Type 4 flow will be a drowned out hydraulic jump.

As the river stage continues to fall the storage level may lag behind sufficiently so that enough energy is available for the flow to pass through critical depth. Flow Type 5 then takes place and a hydraulic jump will occur downstream.

We wish to emphasize at this point that the above sequence of flow types may not necessarily take place. The determining factor is the relationship between headwater and tailwater levels at any instant, and they are not directly dependent on each other. The <u>Sluice Capacity</u> is the controlling influence.

FOR INSTANCE:

If the river stage drops rapidly and the sluice capacity is small, the headwater level will remain relatively high and flow may pass from Type 1, to 3 to 5.

If the river stage drops slowly and the sluice capacity is relatively large, flow may pass from Type 1 to 4 and never enter into the type 6 range.

4.2.2 PROBLEMS DURING FLOW TYPE TRANSITIONS

In passing from one flow type to another there is a period when the water cannot decide what it wants to do. This is always a period of turbulence with the flow shifting back and forth from one type to the other. The engineer should try, as much as he can, to help the water pass through this difficult stage.

When flow passes from Type 1 to Type 3, air enters the vent chamber intermittantly with loud sucking noises. Inside the chamber some of the air becomes mixed with the flow and is carried out. A partial vacuum is formed and fresh air tries to enter along the soffit. When it does the vacuum collapses. At times this phenomenon may be severe enough to damage the structure. The engineer can reduce the severity by installing an air vent in the chamber allowing air to enter from some place other than along the soffit.

Similarly, when the flow tries to make the transition from Type 3 to 4, the water seal at the entrance intermittantly forms and breaks. This is not as destructive as from Type 1 to 3, but the rate of discharge is reduced. Rounding the inlet soffit lip and providing an air vent will help the water pass through this phase.

When the flow passes from Type 4 to 5 a hydraulic jump will intermittantly form and collapse downstream. This is undesireable because undulating waves are formed that travel downstream. These waves will have the tendency to erode the outfall channel banks, endanger boat traffic if it is present, and overtop canal banks if there is insufficient freeboard. A well designed stilling pool that will encourage the formation of a hydraulic jump will reduce this effect.

4.3 DISCHARGE FORMULAE AND COEFFICIENTS

The formulae and coefficients for determining discharge under the various types of flow can be derived from the basic orifice and weir formula,

$$Q = CA\sqrt{2g\,\Delta h}$$

The discharge coefficient, C, varies according to the conditions and the form of the equation, and is directly related to the coefficient of entrance loss, K_e. In actual practice boundary resistance, upstream flow conditions, and other factors also influence C; however this influence is often indeterminate so we will choose to ignore them for design purposes.

In the following articles we will develop the discharge equations and coefficients for the four types of sluice flow with examples showing their use.

4.3.1 FLOW CONDITION TYPE 1

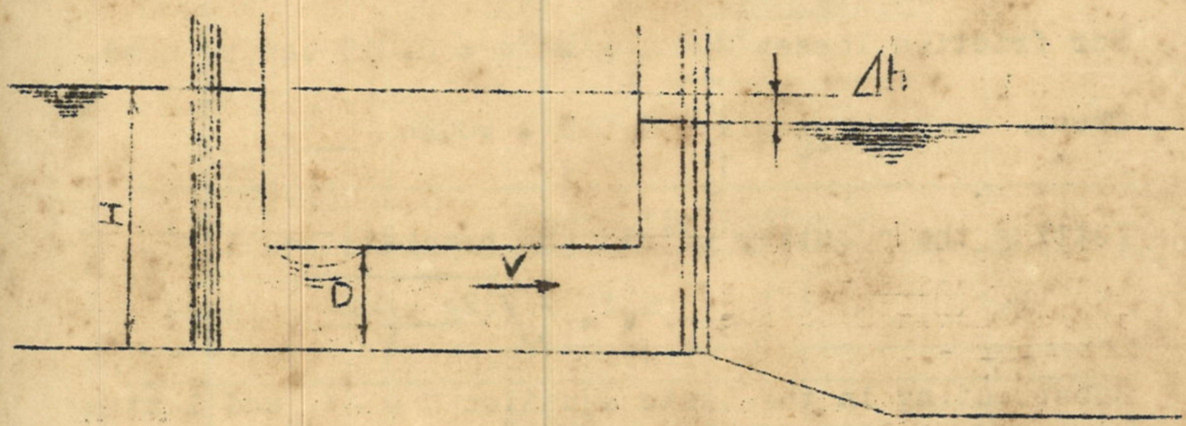


FIGURE IV-2

BERNOULLI'S EQUATION

Under condition 1 the discharge can be solved by writing Bernoulli's equation between the headwater and tailwater.

$$\Delta h = h_e + h_f + h_o$$

Where h_e = Entrance Loss

h_f = Friction Loss

h_o = Outlet Loss

These losses can be defined with respect to the velocity head:

$$h_e = K_e \cdot \frac{V^2}{2g} \; ; \quad h_o = K_o \cdot \frac{V^2}{2g} \; ; \quad h_f = \frac{Ln^2V^2}{2.208\,(r)^{4/3}}$$

$$\Delta h = K_e \cdot \frac{V^2}{2g} + \frac{Ln^2V^2}{2.208\,(r)^{4/3}} + K_o \cdot \frac{V^2}{2g}$$

$$\Delta h = \frac{V^2}{2g}\left[K_e + K_o + \frac{Ln^2 2g}{2.208\,(r)^{4/3}}\right]$$

The entrance and outlet losses can be treated as sudden contraction and sudden expansion losses, and, if the entrance is considered to be sharp-cornered, the coefficients of loss are;

$$K_e = 0.5$$

$$K_o = 1.0$$

For friction losses Manning's $n = 0.015$ can be used.

Thus, $$\Delta h = \frac{V^2}{2g}\left[1 + 0.5 + .0066\,\frac{L}{(r)^{4/3}}\right]$$

Letting the quantity within the brackets equal C^1,

$$V = \sqrt{\frac{2g\,\Delta h}{C^1}}\,.$$

Substituting in the basic equation $Q = AV$, and letting

$$\left[\frac{1}{C^1}\right]^{\frac{1}{2}} = C\,,$$

We obtain the basic orifice equation

$$Q = CA\sqrt{2g\,\Delta h}\,.$$

Using the same approach, A tabular solution for C can be found in King's Handbook of Hydraulics, 4th Edition, page 3-43.

The expressions for C in the following Table are:

For concrete box culverts with rounded-lip entrance

$$C = \left[1.05 + \frac{.0045L}{(r)^{1.25}} \right]^{-\frac{1}{2}}$$

For concrete box culverts with square - cornered entrance

$$C = \left[1 + 0.4r^{0.3} + \frac{.0045L}{(r)^{1.25}} \right]^{-\frac{1}{2}}$$

Type 1 - CULVERT DISCHARGE-ENTRANCE AND OUTLET SUBMERGED-SLUICE FLOWING FULL.

$$Q = CA\sqrt{2g \Delta h}$$

L	Hydraulic Radius-r				
Feet	.8	1.0	1.2	1.4	1.6
	Rounded lip entrance				
10'	.95	.96	.96	.96	.96
20'	.92	.94	.94	.95	.95
30'	.90	.92	.93	.94	.94
40'	.88	.90	.92	.93	.93
50'	.86	.89	.90	.91	.92
	Square lip entrance				
10'	.84	.83	.83	.82	.82
20'	.82	.82	.82	.82	.81
30'	.80	.81	.81	.81	.81
40'	.79	.80	.80	.80	.80
50'	.77	.78	.79	.79	.79

TABLE IV-3

4.3.2 FLOW CONDITION TYPE 3

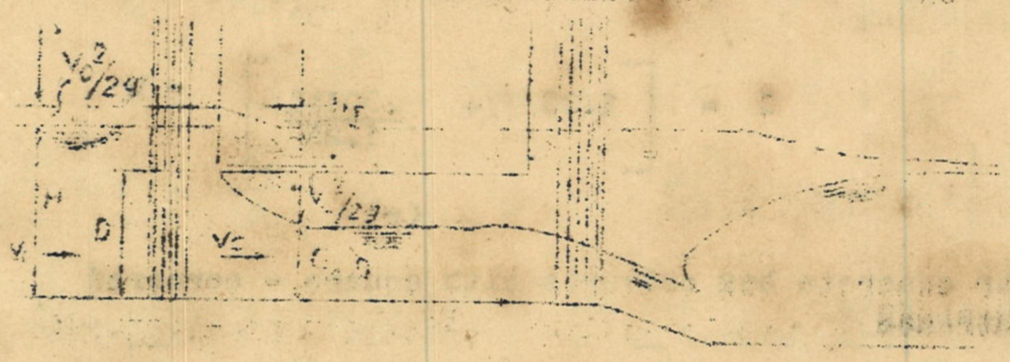


FIGURE IV-4

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 1.5 D will be used as the limit.

a. ORIFICE DISCHARGE

The flow is similar to discharge through an orifice, and with a multiple vent sluice it can be assumed that the bottom contraction is suppressed and side contractions partly suppressed.

The orifice formula for discharge per unit width is,

$q = CD\sqrt{2gh}$ with "h" measured to the orifice centerline.

"King's Hand Book of Hydraulics" lists several coefficients of discharge on pg. 3-38, table 30. For the range of head usually found with sluices, C = 0.60 can be used.

EXAMPLE:

Assuming in the above sketch that,

H = 9', D = 6', and h = 6'

$q = 0.60 \times 6' \times \sqrt{2g \times 6'}$

$q = 70.8$ cfs/ft width

b. SLUICE GATE DISCHARGE

An alternative method has been developed in Rouse "Fluid Mechanics for Hydraulic Engineers", pg. 314-315. Referring to Fig. IV-4,

$$\left[\frac{Vo^2}{2g} + H\right] = \left[C_cD + \frac{V_c^2}{2g} + h_f\right]$$

The energy loss "h_f" is small and will be neglected.

Also: $q = V_oH = V_c C_c D$

Substituting for V_o and V_c;

$$\left[\frac{q^2}{2gH^2}\right] + H = C_cD + \left[\frac{q^2}{2g\ C_c^2\ D^2}\right]$$

Collecting terms:

$$q = \frac{C_c\ D\sqrt{2g(H-C_cD)}}{\sqrt{1 - C_c^2\ D^2/H^2}},$$

Letting "C_q" be a coefficient of discharge, and introducing a velocity coefficient, C_v,

$$Cq = \frac{C_c \cdot C_v}{\sqrt{1 + C_c\ D/H}}$$

A curve for "C_c" and "C_q" is shown on Figure IV-5.

And $q = C_q \cdot D\sqrt{2gH}$

Again, this formula is reliable only for values of H 1.5D.

Solving again the problem under (a)

EXAMPLE:

$H = 9'$; $D = 6'$ and $D/H = 0.67$

From the Fig.IV-5 $C_c = 0.622$, and $C_q = 0.492$

$q = 0.492 \times 6' \sqrt{2g \times 9'}$

$q = 71.1$ cfs/ft. width

The depth of flow at the contraction is $0.622 \times 6' = 3.73'$

4.3.3 FLOW CONDITION TYPE 4

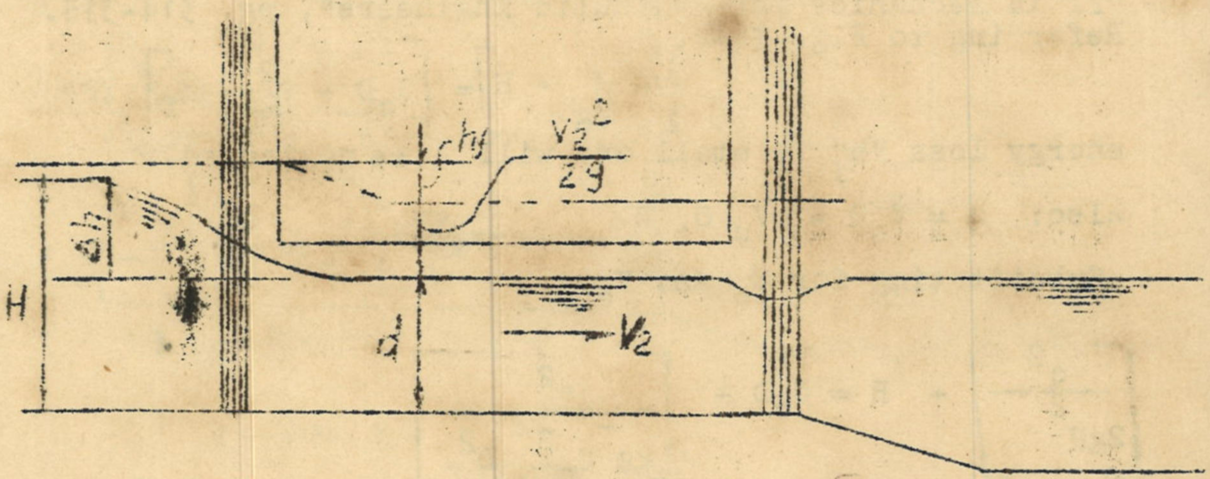


FIGURE IV-6

Under Type 4 flow with the entrance not submerged the discharge is similar to open channel flow over a broad-crested weir.

The assumption is made that the U/S velocity head is negligible.

Head loss $h_f = K_e \left[\frac{V_2^2}{2g}\right]$

$$\Delta h = \left[\frac{V_2^2}{2g}\right] + K_e \left[\frac{V_2^2}{2g}\right]$$

and $\Delta h = (H-d)$

$$(H-d) = (1 + K_e) \left[\frac{V_2^2}{2g}\right]$$

Since $V_2 = \frac{q}{d}$

$$\left[\frac{q^2}{d^2}\right] = \frac{(H-d)\ 2g}{(1 + K_e)}$$

And $q = \frac{1}{\sqrt{1 + K_e}}\, d \sqrt{2g(H-d)}$; Let $C_1 = \left[\frac{1}{\sqrt{1 + K_e}}\right]$

or $q = C_1 d \sqrt{2g\ \Delta h}$

TABLE IV-[illegible] Lists some commonly used values of K_e and C_1.

EXAMPLE:

Usually for flow under these conditions we know the upstream depth "H" and have a tailwater rating curve for the outflow channel. Therefore both "Q" and "d" are unknown and the problem is solved by trial.

Using a sluice with 2-5' wide vents, with H=7.2', and the tailwater rating curve below; assume C = 0.82.

$$q = C_1 d \sqrt{2g \Delta h}$$

$$q = 0.82d \sqrt{2g(7.2-d)}$$

$$q = 6.57d \sqrt{7.2-d}$$

Trial 1) Assume d = 5.25' and solving the equation,

q = 48.2 cfs/ft and Q = 482 cfs.

Plot this point on the rating curve.

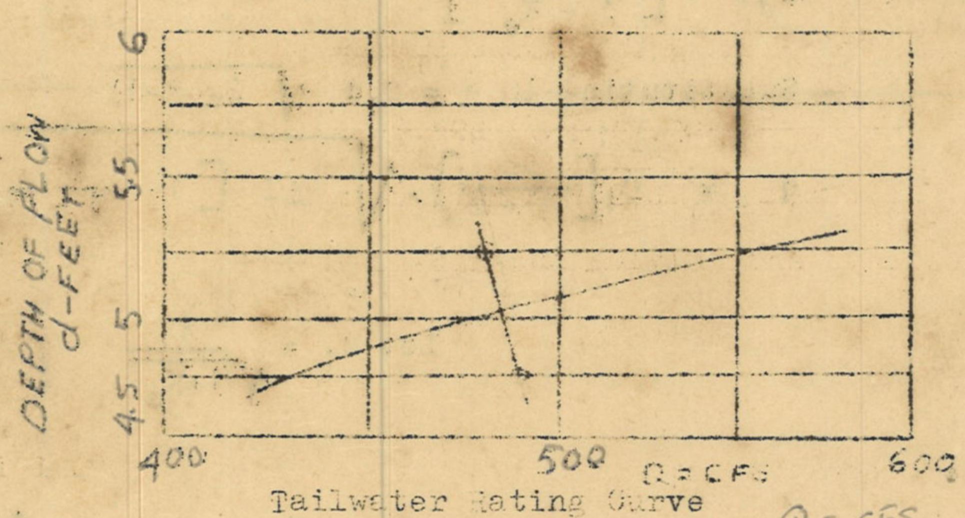
Trial 2) Assume d = 4.75' and solving the equation,

q = 49.0 cfs/ft and Q = 490 cfs.

Plot this point and draw a line to trial 1.

The intersection is at d = 5.0' and Q = 488 cfs.

Check Result: $q = 6.57 \times 5' \sqrt{7.2' - 5'} = 48.8$ cfs/ft.



Tailwater Rating Curve

Q = CFS

Fig. IV-7

4.3.4 FLOW CONDITION TYPE 5

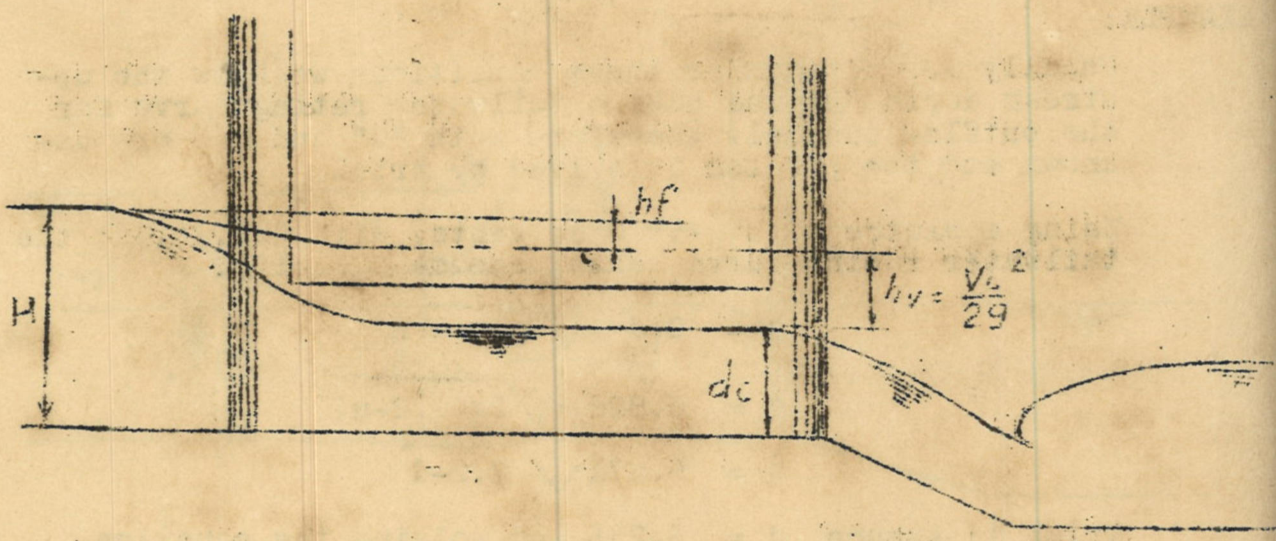


Figure IV-8

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

At the point of critical depth,

$$h_v = \tfrac{1}{2} \cdot d_c$$

$$h_f = K_e h_v = \tfrac{1}{2} K_e \cdot d_c$$

$$H = \left[d_c + \tfrac{1}{2} d_c + \tfrac{1}{2} K_e \cdot d_c\right] = \left[\tfrac{1}{2} d_c (3 + K_e)\right]$$

$$d_c = \left[\frac{2H}{3 + K_e}\right]$$

Substituting in $q = C_1 d \sqrt{2g(H-d)}$

$$q = C_1 \left[\frac{2H}{3 + K_e}\right] \cdot \sqrt{2gH \left[1 - \frac{2}{3 + K_e}\right]}$$

$$\text{Let } C_1 = \frac{1}{\sqrt{1+K_e}}$$

$$Q = \sqrt{\frac{2g\left[1 - \frac{2}{3 + K_e}\right]}{1 + K_e}} \cdot \left[\frac{2}{3+K_e}\right] \cdot H^{3/2}$$

$$\text{Let} \quad C_2 = \sqrt{2g\left[1 - \frac{2}{3+K_e}\right]} \cdot \left[\frac{2}{3+K_e}\right]$$

$$Q = C_2 H^{3/2}$$

TABLE - IV-9

COEFFICIENTS OF DISCHARGE-
SLUICE FLOWING PARTIALLY FULL

✓

Type of entrance	K_e	C_1	C_2
Warped ~~War/ed~~ transition	0.10	0.95	2.94
Cylinder quadrant	0.15	0.93	2.86
Simplified straight line	0.20	0.91	2.80
Straight line transition	0.30	0.89	2.68 ✓
Square ended transition	0.50	0.82	2.45
Well rounded entrance	0.30	0.89	2.68
Square entrance	0.50	0.82	2.45

SECTION V

DRAWDOWN AND BACKWATER PROFILES

5.1 INTRODUCTION

Natural streams and drainage khals have an irregular alignment, an irregular slope which changes with stage, irregular cross sections, and variable roughness. Thus, when applying the principles of open channel flow to natural streams it becomes necessary to divide the stream into reaches which have relatively uniform characteristics throughout, and to obtain an average slope and an average cross section for the reach. The more uniform the channel reach the more accurately the flow characteristics can be determined.

Unfortunately, the streams and khals passing through the low, flat lands of East Pakistan are highly irregular and meandering. Those which carry highly variable flood discharges are constantly shifting and changing shape, and the stream at the beginning of a flood season may have no resemblance to the same stream at the end of the season. Never-the-less, the engineer must do the best he can with the tools at hand, always keeping in mind the limitations of his methods.

In order to facilitate the calculation of profiles we will make the following assumption and simplification:

1. We will ignore changes in velocity head. In natural streams there is usually much turbulence and it is doubtful if much velocity head is regained.

2. Where possible we will keep to a minimum the number of different streams cross sections used. In most cases only one or two will be all that are needed for outfall channels. We will assume that sluice outfall channels will be maintained, and, if necessary, improved to give a more uniform section.

5.2 EQUATION OF GRADUALLY VARIED FLOW

When flow through a channel is restricted by a dam, high tailwater, or other obstruction, and the resulting depth of flow at that point is greater than the natural depth of flow in the channel, the slope of the water surface upstream gradually increases until it becomes the same as the natural slope of the channel. The water surface profile between the two points is called a backwater curve.

When a sudden drop or widening occurs in a channel, and the resulting depth of flow is less than the natural depth of flow in the channel, the slope of the water surface upstream gradually decreases until it becomes the same as the natural slope of the channel. The water surface profile between the two points is called a drawdown curve.

The two types of profiles are illustrated in Figure V-1, and the same analysis can be used for both. As mentioned in Article 5.1, the change in velocity head will be ignored.

From Fig. V-1,

$$d_1 + Z = d_2 + H$$

$$d_1 - d_2 = H - Z$$

Let $H = S.L$

$$Z = S_oL$$

$$d_1 - d_2 = \Delta d$$

$$(1) \quad \Delta d = L(S-S_o)$$

The discharge of open channels by the Manning formula is

$$Q = \frac{1.486}{n} a \; r^{2/3} \; S^{1/2}$$

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

$$Q = K_d . S^{\frac{1}{2}}$$

$$\text{or, (2)} \quad S = \left[\frac{Q}{K_d}\right]^2$$

Equations (1) and (2) will be used to determine the flow profile.

The slope, S, in equation (1) is the average water surface slope of the reach. A close approximation of S (average) can be found by using the average K_d

$$K_d \text{ (average)} = \tfrac{1}{2} (Kd_1 + K_{d_2})$$

5.3 THE K_d FACTOR

For any given cross section, a curve can be plotted which gives values of K_d corresponding to different depths of flow. The use of this curve greatly simplifies the solution of backwater and drawdown curves by reducing the number of computations involved.

Informations required to be able to compute K_d factor are an estimate of the channel roughness and reasonably accurate channel cross sections.

5.3.1 MANNING'S ROUGHNESS COEFFICIENT

The greatest difficulty in applying the Manning formula to flow problems is the determination of the roughness coefficient. There is no precise way of doing this other than by estimating the resistance to flow in a channel. An experienced hydraulic engineer may be able to do this simply by visual observation of the stream, but those of us who are less gifted must rely upon tables and the experience of others.

The value of the roughness coefficient of a channel is not constant; it varies with stage, season and location along the channel. Thus, when we speak of a channel having a particular n value we are referring to what at best can be described as an average value.

The factors which exert the greatest influence upon the roughness coefficient are:

1. Surface roughness
2. Vegetation
3. Channel irregularity
4. Channel alignment
5. Silting and scouring
6. Obstruction
7. Size and shape of channel
8. Stage and discharge
9. Seasonal change
10. Suspended material and bed load.

It should be obvious then, that the value of n can be highly variable. In selecting a value for design conditions, a basic knowledge of these factors will be found very useful. Rather than discuss each factor at this time, the reader is referred to the discussion in Open Channel Hydraulics by Chow, pages 101-106.

Two tabular methods for estimating n will be presented here. (1) A table of typical n values for channels of various types, and (2) a table of values for specific channel characteristics that can be used to modify a basic n value.

ROUGHNESS COEFFICIENTS FOR TYPICAL NATURAL STREAMS

Type of Channel & Description	Maximum	Normal	Minimum
Minor Streams of the Plain:			
1. Clean, straight, full stage, no rifts or deep pools.	.025	.030	.033
2. Same as above, but more stones and weeds	.030	.035	.040
3. Clean, winding, some pools and shoals.	.033	.040	.045
4. Same as above, but some weeds and stones.	.035	.045	.050
5. Same as above, lower stages, more ineffective slopes, and sections.	.040	.048	.055
6. Sluggish reaches, weedy, deep pools.	.050	.070	.080
7. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush.	.075	.100	.150

Table V-2

The second method is a procedure for estimating the value of n by applying correction values, representing several primary factors affecting the roughness coefficient, to a basic n value for a straight, uniform, smooth channel.

The formula for evaluating n is,

$$n = (n_o + n_1 + n_2 + n_3 + n_4) n_5$$

VEGETATION: n_4

Low - Dense growth of flexible turf grasses and weeds where the average depth of flow is 2 to 3 times the height of vegetation.

Medium - Turf grasses where the average depth of flow is 1 to 2 times the height of vegetation; stemmy grasses or weeds with moderate cover where the depth of flow is 2 to 3 times the height of vegetation; or, bushy growths moderately dense along channel side slopes with no significant vegetation along the channel bottom.

High - Turf grasses where the average depth of flow is about equal to the height of vegetation; heavy brush about 1 year old intermixed with weeds along side slopes.

Very High- Turf grasses where depth of flow is ½ the height of vegetation; or, dense growth of reeds along the channel bottom.

DEGREE OF MEANDERING: m_5

The degree of meandering depends upon the ratio of meander length to the straight length of channel.

Minor - Ratios of 1.0 to 1.2

Appreciable - Ratios of 1.2 to 1.5

Severe - Ratios of 1.5 and greater.

VALUES FOR COMPUTATION OF
ROUGHNESS COEFFICIENTS

Channel Conditions			Values
Material Involved	Earth	n_o	.020
	Fine Gravel		.024
	Rock Cut		.025
	Coarse Gravel		.028
Degree of Irregularity	Smooth	n_1	.000
	Minor		.005
	Moderate		.010
	Severe		.020
Variations of Cross Section	Gradual	n_2	.000
	Alternating Occasionally		.005
	Alternating Frequently		.010 - .015
Relative Effect of Obstructions	Negligible	n_3	.000
	Minor		.010 - .015
	Appreciable		.020 - .030
	Severe		.040 - .060
Vegetation	Low	n_4	.005 - .010
	Medium		.010 - .025
	High		.025 - .050
	Very High		.050 - .100
Degree of Meandering	Minor	m_5	1.000
	Appreciable		1.150
	Severe		1.300

Table V-3

EXAMPLE:

Determine a design roughness coefficient for the Sarai Basin outfall channel. It is assumed the channel will be reasonably maintained. Using the method of computation by formula the channel conditions are:

Material involved---earth.	n_o =	.020
Degree of irregularity--minor	n_1 =	.005
Variation of cross-section--alternating occasionally	n_2 =	.005
Obstructions--negligible	n_3 =	.000
Vegetation--low	n_4 =	.005
Meandering--minor	n_5 =	1.00

Total value of n = 0.035

Using the method of selection by typical channels, Table V-2, the description under type 3 (clean, winding, some pools and shoals) seems to fit. The table indicates a range of values from .033 to .045 with a normal value of .040. The lower values of n would occur during the higher stages of flow, which will also be the critical period for stilling basin design. Preferring to err on the safe side, we will select an n value on the low side and use n = 0.035 for a design value. This will give a lower than actual tailwater depth that will help ensure the hydraulic jump is not sweep out of the pool.

5.3.2 CHANNEL CROSS SECTIONS

Channel cross sections can be obtained either from a good large scale contour map, or by survey parties in the field. The second method is preferrable. Since it is the usual practice for surveys of the site to be made, the design engineer should instruct the crew to take the cross sections at the same time.

For short channels a few hundred feet long a minimum of 3 sections should be made. For channel several thousand feet long, sections at intervals of about 500 feet should be made with intervening sections taken if there is a drastic shape change within the reach. Figure V-4 and 5 are the results of the Sarai outfall surveys. Plotting the deep point in the channel, as indicated on Fig. V-5, does not give a clear indication of an average bed slope. However, by estimating an average bed level, as illustrated under the typical section, much better agreement can be had. The average bed slope will be used as the value for S_o in formula (1), page V-2.