PAPER No. 462

Hydrologic Evaluation of Marh-Chiniot Surface Drainage System Design

BY

Engr. Dr. N. M. Awan
Director, Centre of Excellence in Water Resources Engineering

Engr. Muhammad Saleem Bashir
M. Phil Student

Paper No.462

HYDROLOGIC EVALUATION OF MARH-CHINIOT SURFACE DRAINAGE SYSTEM DESIGN

By

Dr. N.M. Awan (*) & M. Saleem Bashir (**)

INTRODUCTION

In the last twenty years a network of surface drains has been constructed all over the country to remove excessive water due to heavy precipitation, to convey saline tubewell water, and also to act as subsurface drains during the dry season. The main problem faced by design engineers is to determine the quantity and peak of runoff from a given area to create favourable conditions for crop growth. The design capacity of a surface drain is mainly based on surface runoff which is dependent on the rainfall intensity, duration, and physiographic characteristics of the drainage area. Reliable rainfall-runoff relation is the basis of efficient design and operation of a surface drainage system.

Natural drains draining small flat irrigated areas have not been adequately guaged in the past. Therefore, reliable information about rainfall-runoff relationship is not available. Consequently most of the surface drainage system in the country has been designed on the basis of empirical formulae developed elsewhere. Many drains in the Punjab, for example, have been designed on the basis of design criterion of 1 - 2 cusecs/square mile without considering the recurrence interval of rainfall.

For SCARP Planning, WAPDA appointed Tipton and Kalbmbach Inc. (T&K), as their Consultants in early sixties. They adopted U.S. Soil Conservation service

(*) Director, Centre of Excellence in Water Resources Engineering, Engineering University, Lahore.
(**) M.Phil. Student.

and synthetic unit hydrograph methods and recommended surface drainage facilities to be designed for a storm of 24 hour duration with a recurrence interval of 10 years. Presently, WAPDA is using this criterion for designing of drainage system.

The present study was carried out to evaluate these criteria and to suggest a more reliable criterion for the design of drainage system. Observed rainfall and runoff data at the outfall of Marh-Chiniot drain has been used and analysed in this study.

SCOPE OF STUDY

Marh-Chiniot drain is located in SCARP-I. Its discharge data was available from 1967-72 with WAPDA. Rainfall data was available from 1964-80 with the Punjab Irrigation Department. Considering the representativeness of agro-climatic and physiographic conditions of the drainage area, design criteria developed for this surface drain might be applicable to other similar drainage projects in the Punjab. Since flood damage data was not available for this area, hence correlation of design storm and flood damages could not be developed.

OBJECTIVES OF THE STUDY

The primary objective of this study was to compare the observed peak discharge produced by a certain storm with estimated peak using unit hydrograph method. Other objectives were as follows:

1) To evaluate the drain design capacity for storms of various recurrence intervals.
2) To predict runoff, peak discharge and drainage coefficients for the drainage basin.

LITERATURE REVIEW

Research on rainfall-runoff relation for flat irrigated areas has passed from pure empiricism to semi-empiricism and finally to the present-day rational analysis based on hydrograph analysis and synthesis, either separately or in combination. Reliable techniques are now available to predict peak discharge from rainfall provided adequate hydro-meteorological data are available. Unfortunately this aspect does not seem to have been given the desired importance and attention despite the fact that a large number of surface drainage projects have been developed in the last two decades. As a result our designs continued to be mostly based on empirical formulas of the form $Q = CA^n$ using coefficients/constants derived elsewhere. In early sixties a gradual shift to practices of the U.S. Soil Conservation Service and the U.S. Bureau of Reclamation was noticed. Among these the Curve Number and the Dimensionless Unit Hydrograph Methods have been extensively used by design engineers in the country. Space does not permit to review all literature on the subject. It would suffice to give a detailed review of the methods used in this study.

Sherman's[5] Unit hydrograph method is well known to all designers. The method is useful when adequate rainfall and runoff data are available. Where these data are scarce or not available at all, the synthetic unit, hydrograph methods can be used provided the required physiographic data of the basin are available. Among the synthetic methods, Snyders[6] method seems to have found wide applicability in drainage design. This is despite the fact that the method was developed for hilly areas. Later on its coefficients were re-evaluated by the U.S. Corps of Engineers[8] for mountainous, foot hills and valley

areas. No attempt, however, has been made to determine Snyders coefficients for large flat irrigated areas. Laflen and Stevenson[4] in 1970, analysed the data collected at the Louisiana Agricultural Experimental Station (U.S.A). The study involved thirty two plots with slope ranging from 0.10 to 0.25 percent and row length of 500 to 1100 ft. in the direction of slope. Corn crop was grown in rows. They studied the effect of precipitation, slope, length, and antecedent soil moisture (10/12 inches) on surface runoff. The empirical relation gave peak rate of runoff as a function of maximum storm intensity for a duration of 35 minutes (inches/hour), slope (%) row length (ft) and antecedent soil moisture inches of water in the top 12 inches. This equation does not give time distribution of runoff to develop a hydrograph or unit-hydrograph.

Tipton and Kalbmbach[7] (T&K) recommended modified synthetic unit hydrograph derived in Florida, U.S.A. The relationship was of the form

$$t_p = 15.4 \left[\frac{L.L_c}{\sqrt{s}} \right]^{0.128} \qquad \text{-- (1)}$$

$$q_p = \frac{640 \times 0.345 \times A}{t_p} \qquad \text{-- (2)}$$

Where t_p = lag - time in hours

L = length of main channel in miles

L_c = length of channel from point of interest on the channel to a point of opposite the centriod of the drainage basin.

S = mean slope of the channel

q_p = unit hydrograph peak discharge in cfs.

The above relationship are for a 12 hour rainfall duration. The 12 hour rainfall duration was used for a 24 hours unit rainfall duration.

T&K recommended surface drainage channel to be designed to carry the peak runoff resulting from storms having a recurrence interval of ten years. Storms of 48 and 72 hour duration was not analysed because for storms with recurrence interval of 10 years or less, the 24 hour duration storm produced peak runoff values as great or greater than a storm of longer duration. It was further stated that flows would vary over a wide range. Hence to reduce excavation and right of way requirements to the minimum it was recommended to adopt mean annual peak (Q) discharge for the determination of an optimum drain section. The mean annual peak was calculated as follows:

$$Q = \frac{\text{Mean annual storm, inches}}{\text{10 year storm, inches}} \times \text{10 year peak}$$

A more rational approach to hydrograph analysis was used by Clarke[1]. He developed the concept of the instantaneous unit-hydrograph involving translation of incremental runoff from sub areas within a basin to the basin outflow location according to travel times and then routing this runoff through a linear reservoir to account for the storage effects of the basin and channels. The methods can be applied if its two parameters, i.e. time of concentration (t_c) and attenuation constant (R) are known. The time of concentration is estimated by measuring the time between the end of effective rainfall over the basin and the inflection point on the recession limb of the surface runoff hydrograph. When the time of concentration has been determined the basin is divided into incremental runoff producing areas that have equal travel times to the outflow location. Isochrones representing equal travel time to

the outflow location are laid out using the distance travelled per unit time to establish the location of the line. The areas between isochrones are then measured and tabulated with the corresponding travel times (from 0 to t_c) for each incremental area, thus giving time-area histogram/relationship.

The runoff from the contributing areas (between isochrones) which is to be translated to the outflow location is in units of volume (in - mi^2 or mm-km^2) and these must be converted to the proper units of discharge. This conversion is given below:

$$I_i = k\, a_i / t \qquad \text{-- (3)}$$

where: I_i = ordinate in units of discharge of the time-area runoff volumes at the end of period i.

a_i = ordinate in units of depth-area of rainfall excess of the time area runoff at the end of period i.

K = conversion factor (k = 645 for cfs and 0.278 for m^3/sec).

Δt = time period of computation interval in hours.

The routing of the translated runoff through storage at the outflow location is accomplished as follows:

$$O_i = CI_i + (I - C)\, O_{i-I} \qquad \text{-- (4)}$$

Where: O_i = outflow from the basin at the end of period i, in discharge units.

I_i = inflow from each area at the end of period i, discharge units.

C = diemensionless routing constant given by

$$C = \frac{2\Delta t}{2R + \Delta t}$$

and $$R = -\frac{O}{dO/dt} \qquad \text{-- (5)}$$

The magnitude of R can be approximately evaluated at the point of inflection of the recession line of the observed runoff hydrograph. The hydrograph that result from routing these flows from the incremental areas is the instantaneous unit hydrograph. The instantaneous unit-hydrograph can be converted to a unit hydrograph of a duration Δt by simply averaging two unit-hydrographs spaced at interval Δt apart.

METHODOLOGY AND DATA

The drainage basin of Marh-Chiniot drain (Fig.I) is bounded on North by the Jhang Branch canal, Jaranwala drain and Rakh Branch canal on the South, Upper Gugera Branch on the east and Chenab river in the North West. The total basin area is 371 square miles and total length from the remotest point is65 miles. The drain discharges into the river Chenab two miles downstream of the Chiniot town. Statistics of the drainage system are given in Table-1.

TABLE-I

STATISTICS OF THE DRAINAGE SYSTEM

Name of drain	Length mile	Basin area (mi^2)	Peak (cfs)
1. Marh Chiniot Drain.	37.142	371	752
2. Kubria Branch Drain.	14.40	40	80
3. Marh Drain	15.76	81	352
4. Sallar Drain	11.78	100	206
5. Sukheke Branch Drain.	12.36	40	160

The normal monthly rainfall is shown in Table-2.

TABLE-2

NORMAL MONTHLY RAINFALL IN THE BASIN

Source: Meteorological Department and Agriculture University, Faisalabad.

Month (Rabi)	Rainfall (inches)	month Kharif)	Rainfall (inches)
1. October	0.59	7. April	0.57
2. November	0.47	8. May	0.62
3. December	0.89	9.June	0.91
4. January	1.02	10. July	3.97
5. February	0.80	11. August	4.10
6. March	0.96	12. September	1.50

Crops: The major crops in the area are sugarcane, rice, cotton and fodder during the Kharif season. About 29% of the area is covered by rice. Fodder covers 17%, sugarcane 10% and cotton 13%.

Soils: The soils have been classified into four soil series on the basis of their subsoil textural characteristics upto 6 ft. The basic unit in the classification system is the 'Soil Series' which consist of soils having similar textural variation - both vertical and horizontal - within the soil profiles. The distribution of the soil series which fall in the project area is given in Table-3.

TABLE-3

SOIL CLASSIFICATION

Textural Group	Soil Series	Percentage
1. Medium textured soils.	Buchiana	40
2. Moderately coarse	Farida	30
3. Coarse	Jhang	25
4. Moderately Fine	Chauarkana	5

Rainfall Stations: There are nine standard non-recording rain gauging stations maintained by the Punjab Irrigation and Power Department in the study area. These are listed in Table-4.

TABLE-4

RAINGUAGE STATIONS

S.No.	Station	Within Basin or outside	Areal coverage %	Data period
1.	Sagar	Within	6	1964-80
2.	Ajnianwala	Within	7	1964-80
3.	Manuana	Within	20	"
4.	Marh	Outside	23	"
5.	Jandoke	Outside	4	"
6.	Sallarwala	Outside	11	"
7.	Hinduana	Within	6	"
8.	Chiniot	Within	5	"
9.	Kot Khuda Yar	Outside	8	"

The average density of rainguages comes to 41 sq. miles per rain guage. This is better than the minimum of 230-360 square miles per guage prescribed by WMO as normal tolerance in respect of flat areas in temperate, mediterranean and tropical zones.

Storm Selection: Daily rainfall data during the Summer Maonsoon was selected for the study. The daily data was analysed to determine the maximum precipitation during any 24, 48 and 72 hours in each year for each station and the period of record. Depth-duration-frequency relationship is shown in Table-5 for each station. It is plotted in Fig-2 for 24 hour duration rainfall.

TABLE-5

RETURN PERIOD OF RAINFALL AT VARIOUS RAIN CAUGE STATIONS

Station	Return period (years)	Daily Rainfall inches		
		24 hr.	48 hr.	72 hr.
1	2	3	4	5
Sugar	2	2.3	3.0	3.1
	3	3.0	3.9	4.1
	4.	3.6	4.5	5.0
	5.	4.2	5.1	5.8
	10	6.2	7.3	8.5
Ajnian Wala	2	2.7	3.4	3.7
	3	3.2	4.0	4.4
	4	3.5	4.3	4.8
	5	3.7	4.5	5.0
	10	4.2	4.9	5.7
Nanuana	2	2.9	3.6	4.0
	3	3.6	4.5	4.9
	4	4.2	5.1	5.6
	5	4.7	5.7	6.3
	10	6.1	8.1	10.7
Marh	2	2.6	3.0	3.10
	3	3.3	3.8	4.2
	4	3.8	4.5	5.1
	5	4.3	5.1	5.9
	10	6.2	7.1	7.9
Jandoke	2	2.0	2.8	3.2
	3	2.2	3.3	3.6
	4	2.5	3.5	4.1
	5	2.9	3.8	4.5
	10	4.1	4.9	5.4
Salar Wala	2	2.7	3.4	3.5
	3	3.0	4.0	4.2
	4	3.1	4.2	4.5
	5	3.2	4.3	4.7
	10	4.2	4.8	6.2
Hinduana	2	2.0	2.4	2.6
	3	2.3	3.0	3.2
	4	2.6	3.3	3.6
	5	2.9	3.6	4.0
	10	3.6	4.4	5.0
Chiniot	2	2.5	3.0	3.3
	3	2.8	3.4	3.9
	4	3.0	3.7	4.3
	5	3.1	3.9	4.6
	10	3.5	4.1	5.2

(contd.)

TABLE-5 (concluded)

1	2	3	4	5
K.K. Yar	2	2.6	2.10	2.8
	3	2.7	3.0	3.8
	4	3.4	3.7	4.5
	5	3.9	4.4	5.2
	10	5.4	6.2	7.0

This is based on "station year" method. Runoff was collected for 24, 48 and 72 hour rainfalls. These rainfall depths were read from Fig-2 for 24 hours and similar figures for other durations.

Drain Discharge: Automatic stage recorder was installed by WAPDA near Chak Jhumera-Chiniot road crossing at RD 20 of Marh-Chiniot Drain. Stage recorder charts were collected and discharge was read from the rating table for a given stage. Six hourly discharge values were read for the construction of discharge hydrograph.

The runoff record was short and rainfall record less than 24 hours was not available. Further, some stream flow had to be discarded for the following reasons:

a) multiple peak
b) high oscillations in the recession
c) peculiar shaped rising and recession lines of the hydrograph
d) extreme cases of high peaks (over-topping of banks due to multiple storms).

Finally, three hydrographs of isolated storms were selected and an average unit-hydrograph was derived. This unit-

hydrograph was then used to predict flood peak and hydrographs for rainfalls of various durations and frequencies. The computed and observed hydrographs were then compared to determine the reliability of the hydrograph techniques in drainage design. Point rainfall was used in deriving discharge hydrographs. A comparison of point rainfall and weighted average rainfall is shown in Table-6.

TABLE-6

COMPARISON OF POINT AND WEIGHTED AVERAGE RAINFALL VALUES.

Sr.	Return Period (years)	Point Rainfall (inches)			Weighted Average rainfall (inches)		
		24 hr.	48 hr.	72 hr.	24 hrs.	48 hr.	72 hr.
1	2	2.4	3.0	3.3	2.6	3.3	3.6
2	3	3.0	3.7	4.2	3.0	3.8	4.4
3	4	3.3	4.1	4.8	3.4	4.2	4.8
4	5	3.7	4.5	5.2	3.6	4.6	5.3
5	10	4.7	5.5	6.5	4.4	5.4	6.6

It is obvious from the table that there is no major difference of point and weight values for a particular recurrence interval.

Storm duration: In the absence of recording rainfall gauge data, it was not possible to determine the storm duration. Comparison with the Faisalabad data showed storm duration to vary from 6 to 12 hours. However, because it was difficult to derive inferences from the available daily rainfall, 24-hour duration was assumed in this analysis. Attempt will be made to extend the relationship (Table-6) of rainfall depth-duration downward to determine 6-hour

and 12-hour duration rainfall and evaluate the error introduced by the above assumption.

Values of observed rainfall and runoff are given in Table-7.

TABLE-7

OBSERVED RAINFALL AND RUNOFF

Date	Rainfall (inches)	Direct Runoff (inches)	Peak (cfs)	Return period (years)
12.6.68	1.56	0.09	462	1.28
28.6.68	0.82	0.007	69	1.00
4.6.68	1.00	0.018	133	1.11
13.8.68	1.63	0.17	781	1.72
12.8.70	1.65	0.16	525	1.51
26.8.70	1.30	0.043	285	1.10
2.7.71	3.06	0.566	920	3.00

The above table indicates that total runoff is a very small fraction of the observed rainfall. The average slope of the area is 1:3834. Therefore, considerable amount of rainfall infilterates into the soil because of low grade and medium to coarse textured soils.

Unit-hydrograph Routing: The drainage basin was divided into five sub-basins having the characteristics shown in Table-8.

TABLE-8

CHARACTERISTICS OF SUB-BASINS

Sub-basin	Area (mi^2)	L Mile	Elevation difference (ft.)	Tc* (hrs)
1	40	16	22	19
2	60	14	14	20
3	90	17	16	23
4	81	17	18	22
5	100	18	19	23

* T_c is time of concentration and was computed using

Table-8 (concluded)

(7)
Kiprich formula $T_c = \left[\frac{11.9\ L^3}{H}\right]^{0.385}$

Peak discharges for each sub-basin was computed by the Snyder method using coefficients derived from observed hydrographs and derived unit-hydrographs therefrom. These coefficients are show in Table-9.

TABLE-9

CALCULATION OF SNYDERS C_t AND C_p

(n = 0.38 was kept constant)

Date of storm	L	L_c	S	t_p	C_t	C_p
9.8.1972	65	30	1.365×10^{-4}	24	0.25	0.38
27.8.1971	65	30	"	22	0.23	0.42
12.6.1968	65	30	"	28	0.29	0.59
Average					0.26	0.46

The derived unit hydrographs were then routed using a velocity of 1.5 ft/sec. The resultant hydrograph is shown in Fig-3. It is evident from the figure that peak produced synthetically for two storms given in Table-9 matched very closely to the observed peak flows as given below:

Date	Observed peak (cfs)	Snyder's peak (cfs)
9.8.72	547	587
12.6.68	444	482

Clark's Instantaneous Unit-Hydrograph Method: Hydrographs were computed using Clark's method discussed earlier. The computed hydrographs of various durations are shown in Fig-4. Clark's 4-hour unit-hydrographs matched closely with the observed average unit-hydrograph (24 hour duration). Snyder's coefficients C_t and C_p were computed from the

Clark's derived unit-hydrographs. These are given in Table-10.

TABLE-10

COMPUTED VALUES OF C_c AND C_p FROM CLARK'S UNIT-HYDROGRAPHS (n=0,38)

Storm duration (hours)	L	L_c	S	t_p	C_t	C_p
2	65	30	1.36×10^{-4}	20	0.21	0.46
4	65	30	"	32	0.33	0.60
6	65	30	"	36	0.37	0.60
Average					0.30	0.55

It may be noticed that these values of C_t and C_p compare well with the Snyder's values shows in Table-9.

Tipton and Kalmback's Modified Synthetic Method: Peak flows estimated by this method are shown below:

Area (mi^2)	L	L_c	Slop	t_p	q_p
371	65	30	1.364	72	1138

The peak discharge computed by this method is only 25 percent of the observed peak discharge. The reason for such a low value could be due to erroneous selection of coefficients. Therefore, this method cannot be used for computing design hydrographs for estimating design capacity of surface drainage system.

Drainage Coefficients: The existing system was designed with runoff factor of 2 cusecs per square mile. The full

supply discharge at the outfall of the drain was estimated to be 752 cusecs. On the otherhand, the observed discharge data in Table-7 indicates that the full supply discharge occurs only once in 1.32 years.

During the observation period a storm of 3.06 inches occurred. The recurrence interval of this storm was 3 years and produced a peak of 920 cfs which remained constant for 48 hours. Peak could not pass due to smaller design capacity of the drain. With this storm a peak of 2376 cfs could have been produced had the drain not over-spilled its banks and runoff was not ponded upstream in the basin. The area was relieved of flood after five days. This caused great damage to the crops.

It has further been estimated and stated by the Punjab Irrigation Department[2] that the estimated discharge of 3000 passed through the drain in 1973 as against designed peak of 752 cusecs on the basis of a drainage coefficient of 2 cusecs/square mile. This means that the runoff encountered was 8 cfs per square mile. The 24-hour rainfall of 1873 had recurrence interval of about three years. These facts indicate that design capacity of the drain was under-estimated.

The hydrograph peaks and drainage factors for various recurrence intervals are given in Table-11.

TABLE-11

RAINFALL, RUNOFF AND DRAINAGE FACTOR

Frequency (years)	Rainfall (in)			Peak (cfs)			Drainage factor		
	1-day	2-day	3-day	1	2	3	1	2	3
2	2.4	0.6	0.3	1478	1704	1704	4	5	5
3	3.0	0.7	0.5	2376	2544	2544	6	7	7
4	3.3	0.8	0.4	2904	2940	2940	8	8	8
5	3.7	0.8	2.3	3744	3960	4032	10	11	11

The above table indicates that design capacity should be double the existing value even for a return period of 2 years for a storm of one-day duration.