HIGHWAY RESEARCH BOARD

Research Report No. 11-B

SURFACE DRAINAGE

REPORT OF COMMITTEE
AND
THREE PAPERS

1950

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SURFACE DRAINAGE

COMMITTEE REPORT AND THREE PAPERS
PRESENTED AT THE TWENTY-NINTH ANNUAL MEETING
1949

HIGHWAY RESEARCH BOARD
DIVISION OF ENGINEERING AND INDUSTRIAL RESEARCH
NATIONAL RESEARCH COUNCIL

Washington 25, D. C. December 1950

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1949 PROGRESS REPORT
COMMITTEE ON SURFACE DRAINAGE

Carl F. Izzard[1], Chairman

The Committee on Surface Drainage held its annual meeting at the University of Iowa, Iowa City, on June 16-17, 1949, immediately following the Fourth Hydraulics Conference sponsored by the Iowa Institute of Hydraulic Research. Nine members of the Committee were present, including two from the Pacific coast, which has not been represented previously.

The following report gives the present status of research projects sponsored by the Committee, discusses the needs for additional research, and cites developments in hydraulic research of interest to highway engineers.

Good progress is being made on the project on scour around bridge piers and abutments which is being conducted by the Iowa Institute of Hydraulic Research as a cooperative project of the Iowa State Highway Commission and the Bureau of Public Roads. The pattern and depth of scour with certain basic pier shapes, with and without a web wall, have been observed with the pier at an angle of 0, 10, 20, and 30 degrees to the axis of the testing channel. Contours of the scour are recorded by photographing the exposed edges of thin horizontal layers of colored sand spaced 0.1 ft. apart. Tests so far have been made with no appreciable bed load movement in the channel.

An additional channel is now being constructed which will be equipped with a sand-feeding mechanism which will make it possible to observe the effect of moving bed load upon the scour around piers. No reports have been published. The Bureau of Public Roads has assigned an engineer to work part-time on this project while undertaking graduate studies in hydraulic engineering. After completing training, it is planned that this man will take charge of field investigations of scour around bridges. The laboratory project is expected to continue for several years.

Tests aimed at reducing potential scour around existing piers were made by the Rocky Mountain Hydraulic Laboratory in Colorado during the summer of 1949.

Qualitative studies of scour patterns around groups of piling and quantitative comparison of scour for single piles under various conditions of uniform sediment sizes are partially completed at Massachusetts Institute of Technology.

The investigation on hydraulics of storm drains for express highways is continuing at the University of Illinois with expanded laboratory facilities. Tests have been completed on a 1: 3 scale model of the toe-of-slope gutter and inlet grating, and of the barrier curb gutter with both grating and curb opening inlets. An analysis of the results of tests on the curb opening inlets, which had no depression of the gutter flow line, correlating these data with the results of other recent tests on curb opening inlets with a depression of the gutter flow line, is being presented at the annual meeting of the Highway Research Board. A half-scale model of an inlet catch basin proposed for the Congress Street Expressway in Chicago has been tested. A full-scale model of the toe-of-slope gutter is being constructed with a concrete surface to determine actual roughness factors for typical finishes. Full-scale tests are being made of flow through typical inlet boxes without a catch basin. Tests will be made later on a structure designed to trap sediment which may be carried into a storm drain. A very thorough analysis has been made of various statistical methods of determining the frequency and intensity of rainfall for various durations. A new method having a sound statistical basis has been developed and is being tested.

[1]Chief, Hydraulic Research Branch, Bureau of Public Roads

The Johns Hopkins University is conducting a research project on storm water drainage for the City of Baltimore, Baltimore County, and the Maryland State Roads Commission. The project as now planned will encompass two rather distinct lines of investigation: (1) a study of curb and gutter inlet capacities, and (2) a hydrologic study of storm runoff. The first has the objectives of learning the deficiencies of inlets as now designed and the subsequent improvement of present designs. The second study will attempt to adapt and test under Baltimore conditions the most advanced techniques insofar as possible. Field measurements on sample urban areas are planned.

The investigation on hydraulics of culverts at the St. Anthony Falls Hydraulic Laboratory, University of Minnesota, is still in an inactive status. The committee was unanimous in its belief that this project should be reactivated. The laboratory has recently published Bulletin No. 2, "Grate Inlets for Surface Drainage of Streets and Highways," which is a complete report of the studies by Larson.

Tapley advises that a final report on a model investigation of divided flow on street intersections is in preparation. These tests were made at Los Angeles for the City and the California Division of Highways in connection with surface drainage intercepted by the Hollywood Parkway.

The Bureau of Standards has completed tests for the Bureau of Public Roads on a 1:16 model of a vertical drop in a storm sewer tunnel designed to operate under pressure with velocities up to 25 ft. per sec. The tests indicate a loss of head of 4-1/2 ft. at the design discharge. Losses would have been much greater if the connections to the drop shaft had not been well rounded, and could have been reduced to about 1 ft. by careful streamlining of the approach bend.

Highway engineers may be interested in knowing that the Bureau of Standards publishes annually a bulletin, "Hydraulic Research in the United States," summarizing information on nature, description, status, results, and publications of hydraulic research projects.

A paper in the April 1949 *Proceedings* of the American Society of Civil Engineers entitled "Control of the Hydraulic Jump by Sills" gives basic information useful to the highway engineer in studying the problem of dissipating energy at the outlet of a culvert. A symposium in the November 1949 *Proceedings* brings together for the first time data on "High-Velocity Flow in Open Channels." Important differences between high-velocity and low-velocity flow which should be considered in the design of curves and transitions for high-velocity flow are indicated.

The Subcommittee on Hydrology of the Federal Inter-Agency River Basin Committee has published a new edition of "River Basin Maps Showing Hydrologic Stations." Copies of these maps are available on request to the U. S. Weather Bureau, Washington, D. C. The subcommittee has also recently published its Bulletin No. 3, "Summary of Current Requirements for Additional Hydrologic Stations to Meet Federal Needs," which is a report resulting from the regional conferences held in 1948.

Sixteen state highway departments and one county highway department this year are contributing approximately $100,000, matched by the U. S. Geological Survey, for various investigations of stream flow similar to those described in last year's report.[2] Statewide flood frequency studies are about ready for publication in Connecticut, Minnesota, Missouri, and Washington. A tabulation of flood frequency data for the Columbia River Basin of the State of Washington has been published in Water Supply Paper 1080. Dalrymple is presenting a paper at this meeting describing how frequency data from a number of gaging stations in a region may be correlated.

The importance of having reliable information on flood frequencies is emphasized by the fact that approximately $400,000,000 is being spent annually on the construction of drainage structures by federal, state, and local

[2] Highway Research Abstracts, Vol. 19, No. 3, pp. 20-23 (March 1949)

agencies in the United States. This estimate is based on statistics compiled by the Bureau of Public Roads which indicate that the cost of stream-crossing bridges over 20-ft. span averages 15 percent of the total cost of constructing highways and that approximately 10 percent of the total cost, or about $160,000,000 a year, goes into culverts and other small drainage structures. It is interesting to note that the small drainage structures aggregate in cost more than the total cost of all bridges over 20-ft. length up to a length of 280 ft.

The hydrologic data available for the design of small drainage structures is pitifully small when one considers the investment that is being made in such structures. For example, if we consider that culverts take in drainage areas up to 1-1/2 sq. mi., we find that only 0.2 percent of the gaging stations currently operated by the U. S. Geological Survey are on drainage areas in this size category. Only 21 percent are on drainage areas less than 100 sq. mi. The committee feels strongly that more gaging stations ought to be installed on drainage areas under 100 sq. mi., and particularly on drainage areas under 2 sq. mi.

The Soil Conservation Service is continuing its good work in collecting and analyzing runoff data on small drainage areas with particular emphasis on the effect of changes in land use. The regions to which their studies are applicable, however, constitute only parts of a few states in the East, Middle West, and Southwest. The committee hopes that the Soil Conservation Service will be able to expand its program for hydrologic research, recognizing that peak runoff rates on small agricultural areas are subject to great variations requiring intensive research to evaluate the variables. The present status of this research is described in a paper by Potter which will be presented at this meeting.

The Bureau of Public Roads has recently established a Hydraulics Branch in the Research Division which will be responsible for all research activities in hydraulics and hydrology as applied to highway engineering. The staff will also work on the development of simplified methods of hydraulic analysis and will conduct in-service training courses to familiarize field engineers with these techniques. The services of this staff will be available for consultation on difficult hydraulic problems encountered on current projects. A number of charts to facilitate hydraulic computations for flow of water through culverts, open channels, gutters, and storm drain inlets have been prepared. Copies of these charts can be obtained for trial use by highway departments on request.

REGIONAL FLOOD FREQUENCY

Tate Dalrymple
U. S. Geological Survey

SYNOPSIS

The practice of the United States Geological Survey in determining the magnitude and frequency of floods is presented. The flood frequency analyses described herein are based on the discharge records that have been obtained as a part of their nation-wide stream-gaging program. Gaging station records are obtained at specific locations and seldom are available at the site of a proposed structure. For maximum usefulness these records must be made applicable to ungaged areas.

A method of computing flood frequencies at a gaging station is first described. The annual flood method rather than the partial duration series method is recommended, due to simplicity. A definite relationship between the two has been established and values on a partial duration series basis, useful in the design of overflow type of bridge approach, can be obtained from the study of annual floods.

The next part of the paper shows how gaging station records may be combined to give a frequency relationship applicable for a region. This method provides a means for estimating flood frequencies on ungaged streams. Two curves usually result from the regional study, one showing the ratio of the mean annual flood to a flood of any selected frequency and the other showing the relationship between the mean annual flood and some function of the drainage area, usually the size. From these two curves the magnitude and frequency of a flood at any location on any stream in the region may be determined. Due to lack of basic data, results are generally restricted to areas larger than 100 sq. mi. and for frequencies less than 50 years.

An example is worked out showing application of the method to the Maumee River Basin in Ohio, Indiana and Michigan. A short bibliography relating to both statistics and floods completes the paper.

The purpose of this paper is to present a method of analyzing streamflow data to obtain information relative to the magnitude and frequency of flood discharges. Studies of flood frequencies have been found especially helpful in problems involving economic considerations such as design of bridge clearances, channel capacities and roadbed levels, where costs must be balanced against flood damage or liabilities arising from failure and interruption of services. Drainage structures are seldom designed capable of passing the maximum flood that may occur as it is not economically sound to provide for such unusual occurrences. A flood less than the maximum possible usually will be selected as the basis for design, and some consideration must be given to the frequency with which this flood will recur.

The subject of flood frequencies has attracted many investigators and much profit has been derived from their study, but the viewpoints and theories expressed have not always been consistent. There is no uniformity of opinion today as to which is the best method, and probably one is as good as the other; although results may differ somewhat this is not serious provided consideration is given to the particular method used.

The method of computing flood frequencies that is presented in this paper reflects the latest developments based on a continuing study of the subject by engineers of the Water Resources Division of the United States Geological Survey. The method has been revised several times in the past few years and probably will be again in the future. There is no static level for such a rapidly developing subject.

The discharge records collected by the Geological Survey, and other private, State and Federal agencies, are the data upon which flood frequency studies are made. These records are obtained at about 6,000 places in the United States and, where of sufficient length, furnish an excellent base for deriving a flood frequency curve. Such a curve, however, applies only to that one particular place; generally the information is wanted at an ungaged spot. Thus there are two parts to the problem: (1) computation of flood frequency at a point, as a gaging station, and (2) transferring this point data to other places or adapting it to apply over a basin or a region.

FLOOD FREQUENCY AT A GAGING STATION

The first step in beginning a flood compilation is to select the gaging stations to be included. Gaging stations to be considered for this kind of study should have at least 10 years of record, although records as short as 5 years have been used when no others were available on any nearby stream. Storage or other artificial factors which would tend to modify flood discharges significantly should be a minimum. Do not include canals, ditches, and drains in which discharges are subject to substantial control by man.

Peak stages and corresponding discharges both should be listed in chronologic order. Momentary peaks only should be listed, although mean daily discharges have been used extensively in the past. For streams with loop ratings, or those subject to rate of change or backwater effects, peak stages are not concurrent with peak discharges and they should be listed independently.

The next step is to review the station history, especially the stage-discharge rating. The determination of the proper rating and its use to obtain peak discharges requires careful study and considered judgment. As this review is perhaps the most important and valuable part of a flood-frequency investigation, it should be performed by persons experienced in field and office stream-gaging practice. The desired goal is to obtain a set of peak discharges that are consistent among themselves and as accurate as all available data permit. A common source of inconsistency arises from changes in rating curves on the basis of recent discharge measurements and neglect to revise past records. No way has been found to assure that all users will obtain revised figures of previously published records.

Where it is necessary to compute frequencies of stage occurrence careful thought must be given to the nature of the stage-discharge relation. If the stage-discharge relation has remained essentially stable throughout the period of record, then frequencies can either be computed directly from stage or preferably can be computed from discharge first, then transferred to stage by means of the stage-discharge relation. In the case of sandy channels there may be very frequent shifting of the bed, and a stage-frequency curve may be valid only for a definite period of time or, in extreme cases, may be of little value. Ice-affected streams involve further problems yet unsolved; when peaks due to ice jams are among the events included in an array of high stages, it is not possible to transform from discharge to stage frequencies.

Kinds of Flood Series - There are two kinds of flood series to be considered, annual floods and a partial duration series, or floods above a base.

An annual flood is defined as the highest momentary peak discharge in a water year. Only the greatest flood in each year is used. An objection most frequently encountered with respect to the use of annual floods is that it uses only one flood in each year. Infrequently, the second highest flood in a given year, which the above rule omits, may outrank many annual floods.

This objection is met by listing all floods above a selected base without regard to number within any given period. Such a list is called a partial duration series. The base is generally selected as equal to the lowest annual flood so that at least one flood in each year is included. In a long record, however, the base is usually raised so that on the average only three or four floods a year are included. The only other criterion to follow in the selection of the floods is that each peak be individual, i. e., be separated by substantial recession in stage and discharge. An objection to the use of the partial duration series is that the floods listed may not be fully independent events, i. e., one flood sets the stage for another.

the average interval in which a flood of given size will recur as an annual maximum. In the partial duration series, this is the average interval between floods of a given size regardless of their relationship to the year or any other period of time. This distinction remains, even though for large floods the recurrence intervals are closely the same on both scales. The two methods give essentially identical results for intervals greater than about

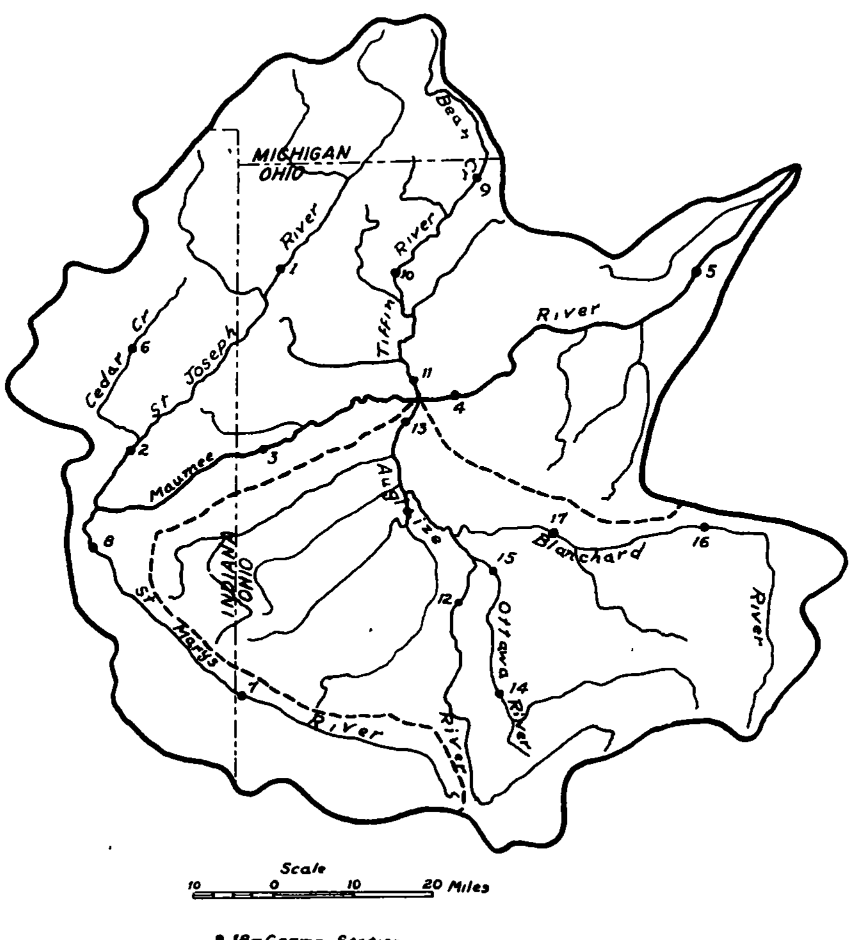

Figure 1. Maumee River Basin

There is an important distinction in meaning as between the recurrence intervals of annual floods and the partial duration series floods. In the annual flood series the recurrence interval is

ten years. As most designs are for intervals greater than this, it is apparent that, from a practical standpoint, either method is satisfactory, although the simplicity of the annual flood method makes it attractive.

From statistical principals there is a definite relationship between the values in the two series. The following tabulation shows comparative values of recurrence intervals by the two methods:

Recurrence Intervals in Years

Annual Floods	Partial Duration Series
1.10	0.41
1.25	.62
1.50	.91
1.75	1.18
2.00	1.45
2.5	2.0
5.0	4.6
10.0	9.5
15.0	14.5
20.0	20.0
100	100

Plotting Positions - Having the discharges listed, it is customary to number them in order of magnitude, numbering the largest 1. Then there is known (1) the relative distribution of floods in (2) a given period of years. The next step is to fit a time scale to the data. Published ideas on this subject are quite diverse, largely because people differ as to the proper method of treating small samples.

The formula adopted by the Geological Survey is simple to compute, is applicable both to annual flood data and the partial duration series, and gives results acceptably in conformance with some of the latest theories. Recurrence intervals for both annual floods and the partial duration series are computed from the formula $(N + 1)/M$, where N equals number of years of record and M equals relative magnitude of the event beginning with the highest as 1.

Annual Flood Peaks - List the highest observed peak in each water year in chronological order. Only complete years of stream-flow records can be included but historical flood data can often be used to advantage, particularly to assist in defining the upper end of the frequency curve. The peaks should be numbered in order of magnitude beginning with 1 for the highest and recurrence intervals, in years, computed by the formula $(N + 1)/M$.

Annual floods are now plotted on a special form developed by Powell[1] for analysis of flood frequencies by the Gumbel[1] method. The discharges are plotted to a linear scale as ordinate: the abscissa (scale of recurrence intervals) is specially graduated according to the theory of largest values.

For the general purpose of flood-frequency graphs the kind of graduations on the paper is of no great importance. However, it is desirable to have uniformity, and if a choice is to be made, the chart based on the theory of largest values has much to offer as flood discharges plotted on this chart approximate a straight-line graph.

Partial Duration Series - List all peaks whose discharge exceeds a chosen base discharge, regardless of the number of peaks occurring in a year. The number of peaks will be about three to four per year if the base is chosen from the list of annual flood peaks as a discharge (rounded upward) whose recurrence interval is 1.15 years.

List only the highest peak of two or more occurring within 48 hr. of each other, unless it is evident that the peaks in that period are independent, as may occur at times on flashy streams. The peaks can be considered independent if ground-water flow is reached between them. Do not list a peak unless the discharge of the trough between it and the adjacent higher peak is 25 percent or more below the discharge of the lower peak. For periods of diurnal peaks caused by snow melt list only the highest occurring during each distinct period of melting regardless of the fact that other peaks may fulfill the preceding requirements.

The peaks should be arranged in order of magnitude and assigned numbers corresponding to their position in the array beginning with the highest as 1. The next step is to compute recurrence intervals for this class of floods

[1]See list of references at the end of this paper.

by the formula (N + 1)/M where N is number of years of record and M is order of magnitude.

The data should be plotted on semi-log graph paper, using the linear scale (ordinate) for the discharge data, and the logarithmic scale (abscissa) for recurrence intervals.

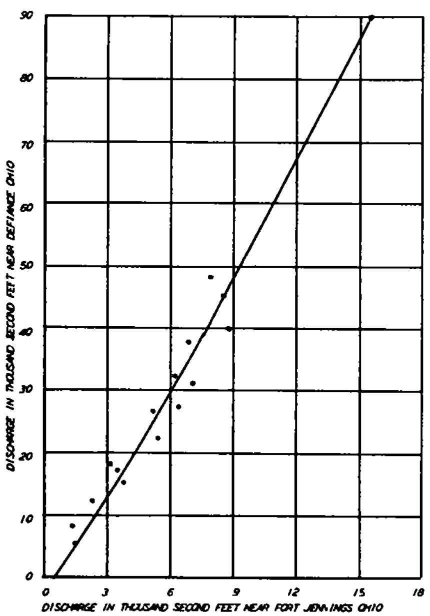

Figure 2. Discharge Correlation, Auglaize River Near Defiance vs. Auglaize River Near Fort Jennings

Historical Data - Historical floods provide probably the most effective data available on which to base flood-frequency determinations and where the data are reliable this information should be given the greatest weight in constructing the flood-frequency graph.

Historical data are particularly valuable where there is an account of all floods, above a certain stage, over a long period antedating the beginning of stream gaging. The minimum or base stage covered by a historical account is generally high, more often it corresponds to "flood" stage, that is the stage where damage begins or threatens. A list of historical floods is of the nature of a partial duration series above a high base but since there is generally only one flood of such magnitude in any one year, it may also be viewed as a partial list of annual floods. In either case, the treatment is the same. Where, as is often the case, there is only one historical flood, the "maximum known", the base is the same as the flood.

All floods, both historical and those for period of record, above the high base, should be grouped together and assigned recurrence intervals in the same manner as previously discussed but based on the period of time for which they are known to be the highest.

With a large gap in magnitude between a historical flood and the highest in the period of record and no intimation of what may have happened in between, there does not seem to by any good way of combining the historical flood with the remainder of the data. In such a case the historical flood should be simply entered at a recurrence interval equal to one plus the period for which it is known to be the greatest of record.

Fitting Frequency Graphs - Having plotted a frequency diagram there appears a need for fitting a curve to the data. The fact that most stream-flow records are less than 25 years in length does not satisfy the demand for estimates of long-term destructive floods. The use of the frequency graphs for purposes of extrapolation may be dangerous, as the linear distance from 25 to 200 years seems very short on most graphs and the error of a curve fitted by whatever method may be extremely great at its outer extension. Most frequency functions, however elaborate, merely represent flexible curves with the general characteristics inherent in random observations. The data, not the functional theory, are used to define the graph and extrapolation can only be justified when the phenomena have been proven to conform to underlying law. Since no known fitted curve can serve any use in extrapolation its main purpose would therefore seem to be merely to provide a smoothing or interpolation formula. The value of an analytically fitted function therefore seems doubtful indeed. Graphical treatment only is recommended.

An example of the details of listing and plotting flood peaks is shown in a paper in *Proceedings* of the 26th Annual Meeting of the Highway Research Board (see "Use of Streamflow records in Design of Bridge Waterways" beginning on page 163), and a similar procedure will not be repeated here.

Unless a very long record is being analyzed as at major stations on large rivers, it is not advisable, except in an emergency, to prepare a frequency curve derived from one station alone. The array of peaks at any one station is a random sample, and, as such, may be far different in character, particularly in a short record, from those of nearby stations otherwise of like characteristics. Flood frequencies should be generalized and related to a common period of record where possible. Frequency characteristics of individual stations should then be based on or related to these generalized frequency curves.

For certain engineering applications where single station analysis only is done, a simple curve of best fit may be drawn. In drawing the curve it should be remembered that the recurrence interval for the highest flood is doubtful, and those next highest probably decreasingly so, hence the topmost points cannot always be used directly. The design of the plotting paper tends to straighten out the shape of the frequency curves, but, with the above exceptions, the base data remains the best indication of the shape.

But even in such cases, if it becomes necessary to extrapolate any appreciable distance beyond the extent of the usable data, then some method other than mere extension of a curve becomes necessary. These methods might include study of ratios of the 25-or 50-yr. flood to the mean annual flood at several long-term stations. They might include a compilation and graphing of extreme discharges in an area, plotting discharge against drainage area. From an envelope curve for such data, to which some frequency might be assigned, based on composite length of experience or frequencies attributed to some of the data, a figure might be obtained which could be used either as a point towards which to extend the curve or as a value which it might approach asymptotically.

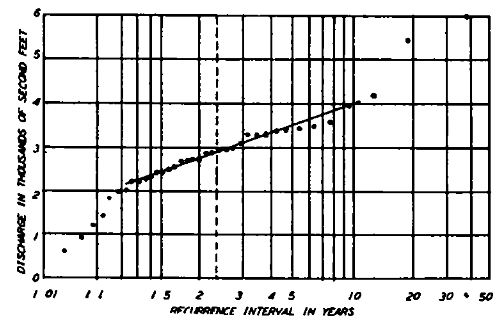

Figure 3. Preliminary Frequency Curve for Annual Floods on St. Joseph River Near Blakeslee, Ohio (1)

REGIONAL DISTRIBUTION

Flood-frequency compilations prepared in accordance with the procedure discussed above result in tables and diagrams of flood magnitude and average recurrence intervals for each station studied. These charts are not applicable directly to ungaged areas. In most cases the flood records are short and the sampling errors correspondingly large, and the records are for different periods of time. Investigations have been made of the possibilities of combining the flood data for a drainage basin or larger region, and also of relating the flood-frequency function to measurable characteristics of the drainage basin, thereby in the first instance reducing the large sampling errors, and in the second instance giving the data regional significance and so making the flood-frequency studies applicable to ungaged areas. A flood-frequency graph that is based on the combined experience of a group of stations has firmer support than one drawn to fit the data at a single station.

The objective of such analysis is to obtain all of the relevant information from the available flood data, strengthening as much as possible the flood-frequency functions, and making available adequate methods of estimating flood frequencies for ungaged areas. In general, we are concerned only with floods not exceeding 50-yr. magnitude, and no attempt will be made to extrapolate beyond this value. In studies of this kind, it is impossible to avoid use of statistical theory. In order to aid those not thoroughly familiar with statistics, a partial list of elementary textbooks is included in the list of references.

Although flood-frequency compilations often include floods other than the maximum annual, it has been demonstrated that the partial duration series and the annual flood series give essentially identical results for intervals greater than about ten years. For this reason, annual floods only are used in these studies. When it is desirable to know how often, on the average, a stream will get above a certain discharge, as when designing a low fill across a valley on a secondary road so that the higher floods will overflow the road, the frequency curve based on the partial duration series must be used. The most simple way to do this is to convert the curve based on the annual flood series by use of the relation expressed in the table presented above; results will be entirely adequate.

Combining Records - It has been suggested that the station-year method of analysis could be applied profitably to flood-frequency studies as it has been to rainfall intensity frequencies. By this method, for example, five records of 20 years each could be combined to obtain a 100-yr. record, thereby increasing the accuracy of flood predictions by reducing sampling errors. Such a method requires (1) that flood-frequency characteristics be comparable and (2) that the data be independent. Studies indicate that the first of these requirements is met by some drainage basins or regions but the second requirement is not met, at least by gaging stations with drainage areas of over 100 sq. mi. Five stations in an area essentially hydrologically homogeneous measure the same flood five times rather than measuring five floods each year; in other words, a 20-yr. period cannot yield more than a 20-yr. record, and cannot be expanded so as to become a 100-yr. period. Despite this unfortunate situation there is considerable to be gained by combining flood records in other ways.

There are two aspects to the analysis of flood frequency in a region: (1) time sampling and (2) geographic sampling. For the first, we require a sample of a large number of years, not necessarily at one station. For the second, we need a large number of stations to sample wide diversity of terrains. The two aspects need not be concurrent. For example, in a flood program one might operate a limited number of long-term base stations, supplemented by a large number of short-term or temporary stations on a large number of different kinds of streams.

Tabulation of Annual Flood Data - The first step in the flood-frequency analysis is to marshal all available annual flood data in a block table. Each station record should be identified by a number, preferably referring to a map, and the full station name and location should be given, as well as the drainage area in square miles. Four lines per record are provided for the data and computations. The calendar year data and discharge of the annual flood are listed for each water year. The date should be included in view of the possibility that the study may reveal sufficient independence between floods that some form of application of the station-year method may be utilized.

Computation of Comparable Means - Experience has demonstrated that the arithmetic average of the annual floods for one station may not be compared with the average for another station with a different length of record. The mean annual floods for each station must be comparable, and therefore must be adjusted to the same period of record. A base period may be chosen equal to any period desired for which records are available, but in general it is best

TABLE 1

DATA FOR HOMOGENEITY TEST FOR GAGING STATIONS IN MAUMEE RIVER BASIN

No	Gaging Station	Drainage Area	Mean Annual Flood, $Q_{2.33}$	10-yr. Flood, Q_{10}	Ratio $\frac{Q_{10}}{Q_{2.33}}$	$Q_{2.33}$ x 1.38	Recurrence Interval for Q of Column 7	Period of Record
1	2	3	4	5	6	7	8	9
		sq mi	sec ft	sec ft		sec ft	yr	yr
1	St. Joseph River near Blakeslee, Ohio	369	2,900	4,000	1.38	4,000	10	6
2	St. Joseph River near Fort Wayne, Ind.	1,060	8,300	10,600	1.28	11,500	21	8
3	Maumee River at Antwerp, Ohio	2,049	15,200	22,300	1.47	21,000	7	37
4	Maumee River near Defiance, Ohio	5,530	48,300	68,000	1.41[a]	66,700	9	22
5	Maumee River at Waterville, Ohio	6,314	52,500	70,800	1.35[a]	72,400	12	25
6	Cedar Creek at Auburn, Ind	93	908	1,090	1.20	1,250	45	6
7	St. Marys River near Willshire, Ohio	355	3,720	5,050	1.36	5,130	11	7
8	St. Marys River near Fort Wayne, Ind	753	7,700	10,600	1.38	10,600	10	18
9	Bean Creek at Powers, Ohio	238	2,900	3,550	1.22	4,000	30	8
10	Tiffin River at Stryker, Ohio	444	4,080	5,480	1.34	5,630	13	17
11	Tiffin River near Brunersburg, Ohio	766	6,500	8,550	1.32	8,970	14	7
12	Auglaize River near Fort Jennings, Ohio	333	5,850	8,200	1.40	8,070	9	23
13	Auglaize River near Defiance, Ohio	2,329	27,000	42,700	1.58	37,300	6	34
14	Ottawa River at Allentown, Ohio	168	2,930	4,200	1.43	4,040	8	19
15	Ottawa River at Kalida, Ohio	315	4,800	7,300	1.52	6,620	7	5
16	Blanchard River near Findlay, Ohio	343	5,600	7,850	1.40	7,730	9	22
17	Blanchard River at Glandorf, Ohio	643	8,600	12,100	1.41	11,900	9	16
	Average Ratio - - - - - - - - - - - - - - - - - -				1.38			

[a] The mean of these two was used to compute the average.

to use the period of the longest record.

The mean used for each frequency distribution is the graphical mean determined by the intersection of the visually best fitting frequency line with the mean line (the line corresponding to the 2. 33 years recurrence interval). The graphical mean is more stable and dependable than an arithmetic mean.

were continuous for the entire base period selected.

There are several ways in which recurrence intervals for a short record may be adjusted to the longer period. The most promising of these methods at present consists of computing a figure for each year of the base record for which no record was obtained. The

TABLE 2

TABULATION OF FLOOD DATA, MAUMEE RIVER BASIN

Sta No	Station and Location	Drainage Area	Water Year	1912	1913	1914	- -	1947	1948	Adjusted Mean
		sq mi								sec ft
1	St Joseph	369	Date							
	River near		cfs	3,400[a]	6,000[a]	2,700[a]	- -	2,750[a]	2,450[a]	2,900
	Blakeslee,		order	9	1	21		20	26	
	Ohio		ratio							
2	St Joseph	1,060	Date		Mar -			Apr 24	Feb 29	
	River near		cfs	9,450[a]	16,500	7,900[a]	- -	6,970	6,800	8,300
	Fort Wayne,		order	9	1	19	- -	27	28	
	Ind		ratio		1 99		- -	840	819	
3	Maumee	2,049	Date	Apr 2	Mar 27	May 13	- -	June 4	Feb 29	
	River at		cfs	19,200	40,000	14,300	- -	11,600	11,900	15,200
	Antwerp,		order	8	1	19		27	25	
	Ohio		ratio	1 26	2 63	941	- -	763	783	
- - -	- - -	- - -	- - -	- - -	- - -	- - -	- - -	- - -	- - -	- - -
17	Blanchard	643	Date				- -	June 9	Mar 23	
	River at		cfs	9,900[a]	28,000[a]	7,600[a]	- -	11,300	9,140	8,600
	Glandorf		order	12	1	21	- -	4	14	
	Ohio		ratio				- -	1 31	1 06	

[a]Computed

This method of determining the mean in effect gives greater weight to the medium floods than to the extreme floods with large sampling errors, and for this reason is not influenced adversely by the chance inclusion or exclusion of a major flood, as is the arithmetic mean.

The graphical mean annual floods may be determined by plotting records for each station on a frequency chart and drawing the best fitted curve for a short interval on either side of the 2. 33-yr. line. However, to do this, the recurrence intervals computed for each flood of a short record must be the same as they would be if the record

record then would be considered complete for the purpose of determining the graphical mean (and for making a test for homogeneity). The computed figures are not true discharges and should never be so considered; they are merely computation figures.

The computed figures may be obtained by comparison of records for the short-term station with records from a long-term station, usually by means of a correlation curve based on flood peaks, plotting the peak discharge for a flood peak at one station against the peak discharge for the corresponding flood at the other station.

When the record for each station is

complete, i. e., there is a figure, either of an actual measured discharge or a computed value, for each year of the total period, order numbers should be assigned and the corresponding recurrence interval computed for each year of record. The complete data should be plotted on the frequency chart and the short-interval curve drawn so as to obtain the 2.33-yr., or mean, discharge.

Computation of Flood Ratios - The adjusted mean floods, computed as described in the preceding paragraph, should be entered on the block table (see Table 2). Each individual flood that was actually measured (not the computed values) should be divided by the adjusted mean. The resulting flood ratio or flood magnitude in relation to the mean is then entered in the table. This expresses all floods in dimensionless terms and places them on a comparable basis, that is, all are measured in relation to the station mean flood for the standard period.

Test for Homogeneity of Records - The flood-frequency graphs prepared in flood-frequency compilations usually do not have the same slope at each station, but for several stations within a region the differences in slopes may not be great. If we can determine that the differences in slopes are no greater than might be expected from random errors, or vagaries of sampling, then we can combine several records to obtain an average flood-frequency curve or line that will be more accurate than any one of the individual lines, and that can be applied throughout a region.

The test set-up requires a study of the 10-yr. floods as estimated at each station. The curve, or line, drawn on the flood charts that have been prepared to compute comparable means should be extended to the 10-yr. recurrence interval. Each 10-yr. flood should be divided by the mean flood to get the 10-yr. ratio. An average of these ratios should be obtained. Then list for each station the length of record in years and the recurrence interval corresponding to a discharge equal to the average flood ratio times the mean flood.

Tentatively assume that each station represents a different sample from a single homogeneous record. If this is so, then the recurrence intervals will not differ among themselves by an amount greater than can be attributed to chance. A figure (see Fig. 4 of example) has been set up to test this

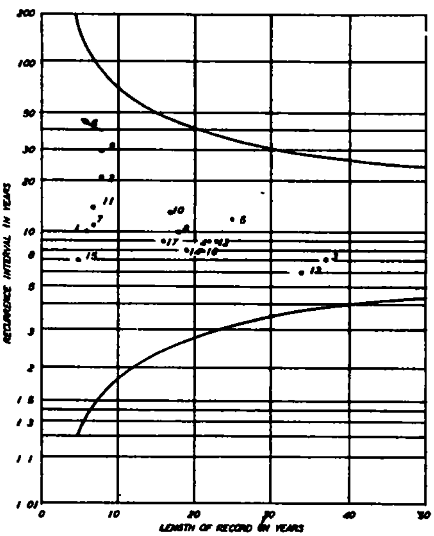

Figure 4. Homogeneity Test

supposition. It shows within what range of recurrence intervals we can expect an estimate of a 10-yr. flood to be. The range, of course, is rather great with short records, and as one would expect, the range narrows down with long records; the upper and lower limits ultimately converging on the 10-yr. interval. If all points as listed above plot within the limits, we may reasonably conclude that the records are acceptably homogeneous.

Computation of Median Flood Ratios for a Group of Stations - Where the previous tests indicate the flood characteristics at one or two stations to be quite different from the others of the group, these stations may be omitted from the others of the group, these stations may be omitted from the averaging process to be described. In general, however, don't be in a hurry to discard a record; the apparent lack of homogeneity may be due to poor record or similar reasons.

There are also cases where stations may be too homogeneous, as for example, stations close together on the same stream. In general, if two stations on the same stream do not differ by more than 25 percent in flow, the one with the shortest record may be dropped from the list. Where the records do not overlap or are only partly overlapping, the records may be combined to make one long record for the flood-frequency studies.

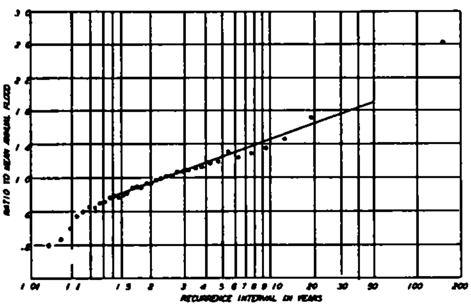

Figure 5. Composite Frequency Curve for Annual Floods in Maumee River Basin

All stations satisfying the above requirements should be grouped for the purpose of computing average recurrence intervals, applicable for the region. For every station, list each flood ratio in order of magnitude for those floods actually measured (not for the computed values); usually there will be many blank spaces in the table.

The table will show one or more flood ratios for the measured floods having an order number of one, one or more flood ratios for the measured floods having an order number of two, and so on for each order number up to the highest. For each order number the median flood ratio should be recorded; this median is the mid value of an odd number of events or the mean of the two central values of an even number of events. The recurrence interval corresponding to each order number should also be listed (see Table 3 of example).

Each median flood ratio should be plotted to its corresponding recurrence interval on a frequency chart and an average frequency curve drawn (see Fig. 5 of example). This curve, showing flood discharge in ratio to the mean annual flood, is based on all significant discharge records available and may be considered as representing the most likely flood frequency values for all places in the region.

Extrapolation of Flood-Frequency Graphs - The primary purpose of flood-frequency studies is to provide information on the magnitude of a flood at a specified location and with a specified frequency or recurrence interval. This specified recurrence interval normally is beyond the length of the record. Involved, extensive, and apparently never-ending theoretical studies of flood frequencies have generally attempted to produce a plotting system that gives a straight line plot of frequencies so that extrapolation can be made with diminished error. The sampling errors of flood records are such that all of these attempts are more or less in vain. For example, the graph paper and plotting system herein recommended has much in its favor, both theoretically and practically, as far as the abscissa or recurrence interval scale is concerned. The use of a linear ordinate or discharge scale, however, is entirely arbitrary, having no theoretical support, and has nothing to recommend it except that many flood records approach straight lines by the use of a linear discharge scale; it may be that the ordinate scale should be logarithmic or some other function of the discharge. For these reasons long extrapolation on a flood-frequency graph is extremely dangerous.

Reasonable requests for data on flood frequencies must be met if practical value is to be given to the analysis. For many purposes, as for design of highway structures, the need is for knowledge of floods having a frequency of only about 50 years. An extrapolation to this interval usually is not great

TABLE 3

PLOTTING POSITIONS FOR MEDIAN FLOOD RATIOS, 1912-48, MAUMEE RIVER BASIN

Order Number	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	Median Ratio	Plotting Position
Col 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
1		1 99	2 63							1 71			3 33			2 86		2 63	38 0
2	1 88		1 72		1 58			1 74		1 44	1 54	1 50	1 78	1 76		1 66	1 73	1 72	19 0
3		1 28	1 48		1 54				1 26	1 39		1 45	1 67	1 72		1 46	1 45	1 46	12 7
4	1 37		1 39		1 36				1 23			1 35	1 46			1 38	1 31	1 36	9 50
5			1 39			1 18				1 26			1 44			1 33	1 27	1 30	7 60
6		1 22	1 36		1 24					1 24			1 44	1 32		1 20		1 24	6 33
7			1 36					1 22				1 26	1.40					1 31	5 43
8	1 18	1 20	1 26			1 11							1 37					1 20	4 75
9			1 23	1 18					1 13			1 21				1 14		1 18	4 22
10			1 21		1 14	1 08	1 15	1 06			1 13		1 27					1 14	3 80
11			1 13		1 10	1 08						1 19	1 19			1 10		1 12	3 45
12			1 10		1 10		1 06					1 17	1 16	1 07				1 10	3 17
13			1 08					1 05		1 07			1 16					1 08	2 92
14			1 03	1 08								1 08	1 15	1 06		1 07	1 06	1 07	2 71
15			1 03		1 04	1 01					1 04		1 02	996				1 02	2 53
16			1 02		1 02	1 01				1 00		1 06	1 01			1 03		1 02	2 38
17			993		1 02			974					989			1 00	996	994	2 24
18			993		998			961		980				955			977	978	2 11
19			941		962							923	944			964		944	2 00
20			941		949				996	973			930	915		893		941	1 90
21			927					929				885	874	894		875		890	1 81
22			914		934							885	833				872	885	1 73
23			882	895				860		881			826	884				882	1 65
24			829		880		788	852				783	815	864		826	845	829	1 58
25			783		872					804			792	853			779	798	1 52
26			763		866		736			789		750	778		798	723		770	1 46
27		840	763		848					784	861	735	775	805				794	1 41
28		819	744							767								767	1 36
29			744		789			669				670	760				687	716	1 31
30	769	801	710				629	659				656	696	758				703	1 27
31			698		720		619	644	607				667	751		589		656	1 23
32			671		697			627				615	644	744	715		523	658	1 19
33	648	731	581	622				576		490	680	601	637			511	519	601	1 15
34			574		596			476	552			547	559	693	464	502	453	550	1 12
35			494		488			455	479	355	357	395	448	423	290	275	395	409	1 09
36	317		256		240			386	297	289		256	307	314	265	214	221	277	1 06
37			222		206		152	260			154	236	194			171		200	1 03

and may be made with considerable confidence.

Estimation of Flood Frequencies on Ungaged Areas - It may be assumed that a flood-frequency distribution computed as above may be applied to all streams in the region with drainage areas within the range covered by the stations used in deriving the distribution. In order to apply the flood-frequency distribution to ungaged areas it is necessary to estimate the mean flood for each area. This involves a correlation analysis of the observed mean floods with drainage basin characteristics.

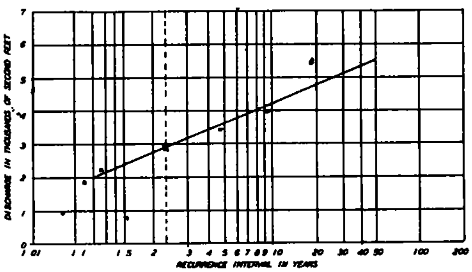

Figure 6. Frequency Curve for Annual Floods on St. Joseph River Near Blakeslee, Ohio (1)

Assuming that the region is hydrologically homogeneous the factors which affect the mean flood are area, topography, shape of the drainage basin, channel storage, and doubtless others. Of these the most important is area, the factor most readily available. Measuring the other factors is more difficult, and unless good topographic maps are available may be impossible. Channel storage undoubtedly has an important effect, but cannot be directly measured, and must be considered as indirectly accounted for by other factors, such as slope and shape.

Many methods of correlation analysis are available; Ezekiel[2] gives a full discussion of this problem. Kinnison and Colby[2] describe a method of correlating topographic characteristics with discharges which they applied successfully to New England flood data. In selecting data for this sort of analysis, emphasis should be given to diversity of terrain and to small streams. Length of record is of minor consideration, provided that the mean flood is adjusted to the base period. The main point to accent is to include a large number of basins, so as to sample as wide differences in physical characteristics as possible. The basins included may cover a much wider region than were grouped in the frequency analysis.

[2]See list of references at the end of this paper.

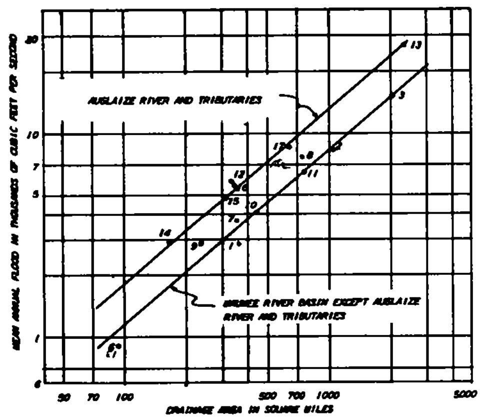

Figure 7. Mean Annual Flood, Maumee River Basin

Whatever method of correlation is used the resulting formula will not fit the data perfectly. The errors may be related to some factor not included in the correlation, such as permeability of the soils, differences in rainfall, or channel storage, as well as random errors. In some regions, area alone may furnish the best correlation that can be made; in other regions, other factors might well be included, as area of lakes and swamps.

For practical engineering use, a correlation of mean flood with drainage area may suffice. This simple correlation may require more than one curve for the region under study but eliminates tedious and lengthy computations that often may not be practicable. Plot, usually to log-log scales, mean annual flood discharge as ordinate and drainage area as abscissa. Fit a mean curve graphically to the data (see Fig. 7 of example). From this curve can be read the mean annual flood discharge for any stream in the region.

The mean annual floods must be multiplied by the flood ratios previously computed to obtain the discharge corresponding to a selected frequency. If desired, a complete frequency curve can be produced by plotting discharges for several frequencies and drawing the curve they define.

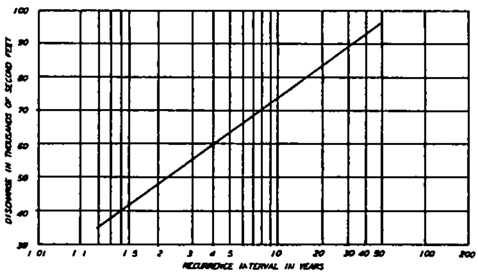

Figure 8. Frequency Curve, Main Stem Maumee River, Below Defiance, Ohio - Note: This curve is based on a drainage area of 6,000 sq. mi.; for other size of areas from 5,500 to 6,600 subtract or add 1 percent for each 100 sq. mi.

Summary of Procedure - The step by step procedure for determining regional flood frequencies is as follows:

1. Tabulate flood data for all gaging stations in the region having a record of about five years or more. List maximum annual floods.
2. Review station history and make careful study of the stage-discharge relationship.
3. Select base period for study. Usually this will be the period of the longest record.
4. Compute discharge figures for missing years for each station.
5. Number floods for each station in order of magnitude, numbering the greatest flood 1.
6. Compute recurrence intervals by formula: R. I. = (N+1)/M.
7. Plot discharge vs. recurrence intervals and draw frequency curve to get the 2.33 and 10-yr. floods.
8. Test for homogeneity.
9. List annual floods in block table for homogeneous stations, and record the mean annual, or 2.33-yr. flood.
10. Compute ratio of annual flood to mean flood for each year of actual record.
11. Tabulate flood ratios, listing ratios for each order number on one line or in one column.
12. Determine median flood ratio for each order number, and record corresponding recurrence intervals.
13. Plot median flood ratios vs. recurrence intervals and draw composite frequency curve. In general, do not extrapolate above 50-yr. frequency.
14. Plot mean annual floods vs. drainage areas. Draw curve, or curves, to show relation applicable for region.
15. Determine flood frequency for any place in region from curves of items 14 and 15.

EXAMPLE

As an example of the application of the method described in this paper, a regional flood frequency analysis is presented for the Maumee River Basin, which drains parts of Ohio, Indiana, and Michigan.

A map of the Maumee River Basin is shown as Figure 1. The numbers on map show location of gaging stations used in the analysis and they may be identified by reference to Table 1.

The procedure outlined below is referenced to the items numbered in the "Summary of Procedure" section.

1. Annual flood peaks, in cubic feet per second, were listed for the 17 gaging stations in the basin having a record of five years or more. The maximum flood peak was listed for each water year, or year ending September 30.

2. A review of the stage-discharge relation was made and discharges were recomputed for several stations where more recent high-water discharge measurements indicated the originally used rating curve, which was usually extrapolated, was in error. Slight changes were made in order to make each record consistent within itself.

3. The base period selected was

1912-48. This is the period of the longest record, that for Maumee River at Antwerp, Ohio.

4. Discharge figures for years of missing record were computed by correlating the discharge at one gaging station with that for another, beginning with the complete record for the Antwerp station. Figure 2 shows the correlation curve for Auglaize River near Defiance vs. Auglaize River near Fort Jennings.

5. Order numbers were assigned from 1 to 37, numbering the largest flood 1.

6. Recurrence intervals were computed as (N+1)/M, where N is number of years of record, in this case 37, and M is the order number.

7. Discharges were plotted to their proper recurrence intervals on frequency charts and the frequency curve drawn from below 2.33 years to the 10-yr. interval. No attempt was made to put the curve through the low or the high points, as only the middle frequencies were needed. An example of this plot is shown for St. Joseph River near Blakeslee, Ohio, as Figure 3.

8. Data for homogeneity test are listed in Table 1. Data for columns 4, 5, and 8 were taken from frequency charts as discussed under item 7; data in other columns were taken from original records or computed.

The test for homogeneity was made by plotting data in columns 8 and 9 of Table 1, as shown in Figure 4. All data plot within the limits, therefore these records are considered to be homogeneous and were combined in one group.

9. Table 2 shows in outline the block tabulation of flood data from each gaging station in the basin. The mean discharge adjusted to the base period 1912-48, listed in the last column, were taken from the individual plots as discussed under item 7, and are listed in column 4 or Table 1.

10. The ratio of annual flood to mean flood for each year of actual record is shown on the fourth line of data for each station in Table 2.

11. Table 3 shows flood ratios for all floods of record listed in Table 2, with floods having the same order number listed on one line.

12. The median flood ratio for each order number and the corresponding recurrence intervals are listed in columns 19 and 20 of Table 3.

13. Data from columns 19 and 20 of Table 3 were plotted on a frequency chart and a curve fitted graphically. This composite frequency curve shows the ratio to the mean annual flood for recurrence intervals up to 50 years and is presented as Figure 5.

In plotting data from Table 3, the highest ratio was plotted at 150 years rather than at 38 years, as is given by the data. An examination of the records shown in Table 2 shows that at every station the March 1913 flood is the highest for the period. Records for the Miami River at Dayton, Ohio, a stream that heads just across the watershed to the south from Maumee River, shows that the March 1913 flood was the greatest at least since 1805 (see: "Floods of Ohio and Mississippi Rivers," January and February 1937; Geological Survey Water Supply Paper 838, page 654); local knowledge which began about 200 years ago does not include a greater flood. The rainfall causing the March 1913 flood in this area centered over the Maumee-Miami watershed (see: "The Ohio Valley Flood of March-April 1913"; Geological Survey Water-Supply Paper 334, plate III) and it has been assumed that this flood in the Maumee Basin would be of about the same frequency as in the Miami Basin; this assumption is also supported by local knowledge. The period 1805-1948 is 144 years but this has been rounded to 150 as it is not an exact figure. The curve was not drawn to pass through this upper point as its correct position is still unknown; the 150-yr. plotting certainly is more nearly correct than the 38 years indicated by the period of continuous record.

The curve as drawn may not actually fit every location on every stream in the basin but it does reflect average conditions and is a good representation of probable values. A check on how well this average curve fits the data for the individual gaging stations used in the analysis can be made by plotting the curve with a plot of the measured discharges at each station; this has been done for the 17 stations studied and the agreement is good, especially for those stations having the longer record. As an example, the curve for St. Joseph River near Blakeslee, Ohio, is shown as Figure 6.

14. The mean annual floods were plotted against drainage areas (columns 3 and 4 of Table 1), except for the two lower stations on the main Maumee River, and are shown on Figure 7. These points define two curves, and a study of the records shows one curve, the highest, is applicable for the Auglaize River sub-basin and the other curve is applicable for the rest of the region.

It is believed the reason the curve for the Auglaize River Basin is different and greater than for the rest of the region is because the normal direction of travel of storms, from southwest to northeast, is down stream for nearly all of the tributaries. This would tend to produce greater floods in this stream, and is a condition that does not apply to the other sub-basins in the Maumee River Basin.

The curves shown on Figure 7 do not extend below 75 sq. mi. and should not be extrapolated to smaller areas. The smallest area gaged is 93 sq. mi. and to extrapolate further would be unwise. When data becomes available from smaller areas the curves can be extended downward and their value increased.

The lower Maumee River was not included with the other streams as the drainage area is much larger, and a different treatment will be shown below.

15. To determine the magnitude or frequency of recurrence of a flood in the Maumee River Basin, proceed as follows: (a) locate the stream where it is desired to determine the magnitude of, say, the 25-yr. flood; (b) measure the drainage area, in square miles, from topographic or other map; (c) from Figure 7 read the mean annual flood in cubic feet per second, corresponding to the drainage area, being careful to use the proper curve; (d) from Figure 5 determine the ratio of the 25-yr. flood to the mean annual flood; and (e) multiply the two figures obtained to give the 25-yr. flood at the point in question.

As an example, assume it is desired to know the 25-yr. flood from a 500 sq. mi. drainage area in the Auglaize River sub-basin. From Figure 7, the mean annual flood is found to be 7,200 sec.-ft.; and from Figure 5, the ratio of the 25-yr. flood is found to be 1.71; the 25-yr. flood, therefore, is 12,300 cfs.

For the main stem of the Maumee River below Defiance the drainage area varies from about 5,500 to 6,600 sq. mi. and the flood discharges vary about 1 percent for each 100 sq. mi. One frequency curve may be used for this main stem, presented as Figure 8, that will have a maximum error of about 5 percent. For more accurate use, results obtained from Figure 8 should be reduced or increased one percent per 100 sq. mi. for areas less than or more than 6,000 sq. mi.

ACKNOWLEDGMENTS

The flood frequency method presented in this paper is the result of several years of study by a number of engineers of the Water Resources Division of the United States Geological Survey. Special recognition is given to Mr. W. P. Cross, Hydraulic Engineer, Surface Water Branch, Columbus, Ohio, and Mr. W. B. Langbein, Hydraulic Engineer, Technical Coordination Branch, Washington, D. C.

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SURFACE RUNOFF FROM AGRICULTURAL WATERSHEDS

W. H. Potter[1]

SYNOPSIS

The hydrologic behavior of small watersheds is materially different from that of large watersheds Because of these differences, many analytical procedures and empirical formulae used successfully in estimating runoff from large watersheds have been found to be unsatisfactory when applied to the small watershed.

Attempts to supplement short-time runoff records with long-time rainfall records by use of: (1) direct relationships between rainfall intensity and rates of runoff; (2) rational method; (3) unit hydrograph; and (4) infiltration theory have not proved satisfactory when applied to the small watershed.

Probability studies integrate the frequency of occurrence of various watershed conditions with the frequency of occurrence of various rainfall intensities and patterns and are, therefore, especially useful in estimating peak rates of runoff from small watersheds that may be expected for various recurrence intervals.

The theory of extreme values, developed by Dr. E. J. Gumbel assumes a distribution of peak rates such that the coefficient of skew does not have to be calculated as is the case with the Hazen and Foster probability curves. For this reason the Gumbel curve is very well suited for use in probability studies of skewed distributions as those of rainfall and runoff data.

Probability studies based on short-term runoff records are reliable only if the rainfall for the short period is a good sample of a much longer period. Three normalcy tests should be applied to the rainfall and probability curves should be corrected when rainfall for the period of runoff record is found to be abnormal.

Probability studies have been completed for seven physiographic areas and the recommended peak rates of runoff for use in the design of conservation structures have been published in Technical Publications of the Soil Conservation Service. These recommended peak rates are summarized for five of the seven areas.

There is no fixed limit that can be ascribed to the maximum size of a small watershed, but it is probably safe to say that most areas smaller than 20 sq. mi. will have the characteristics of the small watershed while those larger than 100 sq. mi. can be classed as large watersheds. The area between these limits may be thought of as a transition zone.

Small agricultural watersheds present a special field for the hydrologist. Such factors as soil type, soil depth, size of watershed, shape of watershed, topography, vegetal cover, and antecedent soil moisture, have marked effects on amounts and peak rates of surface runoff from small watersheds that are discernible only to a much lesser degree on large watersheds. This is because the range of differences in any of these factors on a small watershed at the beginning of any storm is usually small. The large watershed, however, is composed of many small watersheds that may differ materially with each other in any or all of the above factors. Since the runoff from the large watershed is dependent on the runoffs from the smaller tributary watersheds the net effect of any factor on amounts and rates is comparatively small. For example, the high rates and amounts of runoff from one tributary row-crop area could be offset by the much lower values from another tributary where the vegetal cover was woods or grass.

Two other factors contribute to differences in runoff characteristics between large and small watersheds.

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In large watersheds, drainage channels are usually well defined and channel storage is appreciable, whereas in small watersheds this is seldom the case. Channel storage acts as a detention reservoir, tending to flatten the peaks of tributary stream flow and to increase the uniformity of the main-stream hydrograph. The other factor, closely allied to channel storage, is the increased time of concentration of the large watershed. Because of this, peak rates of runoff from large watersheds are not likely to be affected by short intense rainfall or by the time of occurrence of such intense rainfall with respect to the beginning of the storm period (storm pattern), both of which materially affect runoff from small watersheds.

As a consequence of these differences in runoff, many procedures and empirical formulae used successfully in estimating runoff from large watersheds have been found to be unsatisfactory when applied to the small agricultural watershed.

Prior to 1931, data on runoff from small agricultural watersheds were practically nonexistent. During the period, 1931-34, the Soil Conservation Service established eight conservation experiment stations where measurements were begun of the runoff from watersheds ranging in size from 1 to 35 acres. During the period, 1937-39, runoff measurements were started at three experimental watersheds located at Coshocton, Ohio, Hastings, Nebraska, and Waco, Texas. The watersheds at each of these locations ranged in size from one to approximately 5,000 acres and were representative of various types of vegetal cover and tillage practices. Since 1939, runoff measurements from additional watersheds have been made for various periods of time, for the most part from watersheds of from 1 to 300 acres.

PROCEDURES BASED ON RAINFALL VERSUS RUNOFF RELATIONSHIPS

Since all hydraulic structures to control surface runoff are so designed that their capacities may be expected to be exceeded once in some specified period, analysis of hydrologic data to be of maximum use should be expressed in terms of recurrence intervals. Because of the short periods of runoff records, early analysis of these data attempted to establish relationships between rainfall and runoff. This relationship would then have been applied to long-time Weather Bureau records to supplement the short runoff records. These attempts can be grouped under four headings; (1) direct relationship between amounts and intensity of rainfall with corresponding amounts and intensity of rainfall with corresponding amounts and rates of runoff; (2) rational method; (3) unit hydrograph, and (4) infiltration theory.

Direct Relationships - Maximum average rainfall intensities for the estimated time of concentration of a watershed are determined for each storm and plotted against the corresponding peak rates of runoff. This procedure results in a wide spread of the plotted points as is shown in Figure 1. Here, the peak rates of runoff from a 2,000-acre mixed cover watershed were plotted against 90-min. rainfall intensity for the 8-yr. period, 1939 to 1946. It will be noted that there were 11 occurrences of rainfall intensity between 0.70 and 0.90 in. per hr. with resulting peak rates of runoff varying from a few thousandths of an inch an hour to 0.18 in. per hr.

The reason for this wide variation in the value of peak rates for a given rainfall intensity is that factors affecting runoff, such as type and density of vegetal cover, initial soil moisture, and storm pattern differ materially from storm to storm. Figure 1 shows how this variation could be reduced by considering three runoff conditions. The maximum condition curve would indicate the relationship between rainfall intensity and peak rates of runoff at times when the moisture content of the soil at the time of the beginning of the storm was high; when the vegetal cover was poor or nonexistent, as after harvest or seeding; and when the storm pattern was such that the maximum intensity occurred near the end of the storm. Likewise, the minimum condition curve would represent the relationship when soil moisture was low; when the crop was making most rapid growth and of a type that afforded good cover; and when the high intensity rainfall occurred near the beginning of the storm.

For these reasons, it was found that estimates of runoff conditions for storms that had occurred prior to the period of runoff record could not be made with any degree of accuracy. Since the same difficulties were experienced when attempts were made to establish relationships between amounts of rainfall and amounts of resultant runoff, this method of analysis was

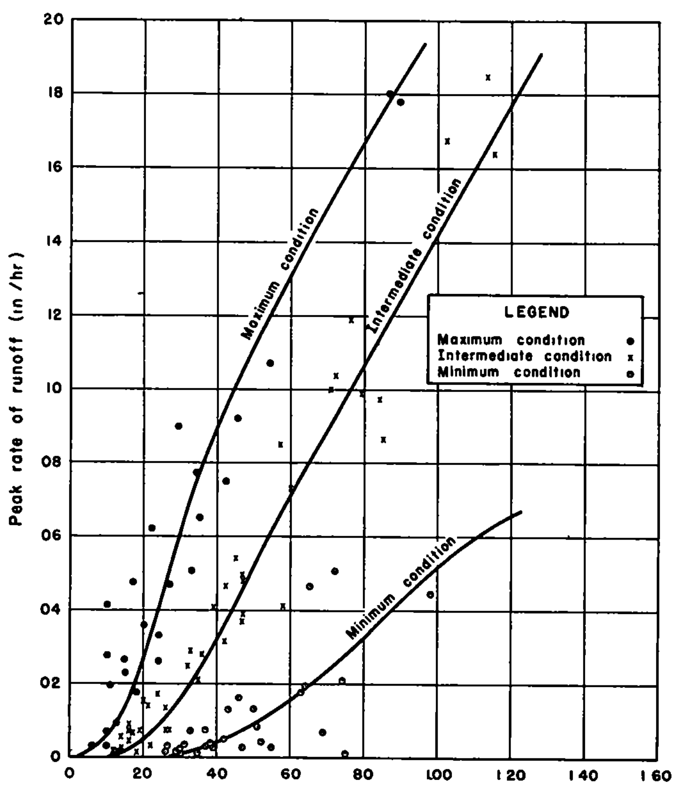


Figure 1. Preliminary Curves Showing Relationship Between Rainfall Intensity and Peak Rates of Runoff for W8

Soil moisture varies not only from day to day but also, for any one day, from one location in a watershed to another and vertically throughout the depth of the soil profile. Likewise, crops are rotated from year to year and from field to field and vary greatly as to times of planting, cultivating, and harvesting as well as to density of foliage and area protection afforded.

found to be unsatisfactory.

The Rational Method -The rational method assumes that the difference between maximum rainfall intensity for the time of concentration of a watershed and the resultant peak rate of runoff can be expressed as a constant. An examination of Figure 1, shows how far this comes from being true (14)[2]. For intensities of from 0.80 to 1.00 in. per hr. the values of C vary as much as 470 percent. If the values of C are limited to maximum runoff conditions, it then becomes necessary to determine the proportion of storms of given intensity that would occur when runoff conditions were maximum. Referring again to Figure 1, it will be noted that five storms occurred with maximum 90-min. intensities between 0.80 and 1.00 in. per hr., yet, only two of these occurred when runoff conditions were maximum. The difficulties in determining the proportion of storms that might be expected to result in maximum runoff have already been discussed in the preceding paragraph.

If the frequency of peak rates of runoff determined by using C for maximum runoff conditions is taken to be the same as that of the maximum rainfall intensity, then for any frequency the peak rates of runoff will be too high. For small structures, such as terrace outlets, culverts, and spillways for farm ponds where the design frequency is usually from 10 to 50 yr., the use of these high values of peak rates may result in structures that are greatly overdesigned. Suppose, for example, that it was desired to build a highway culvert that could be expected to be overtopped on an average of once in 10 yr. Suppose that the culvert was designed for a peak rate determined by substituting the once in 10-yr. rainfall intensity and C for maximum runoff conditions in the rational formula $Q = CIA$. If the rainfall intensity that occurred once in 10 years resulted in maximum runoff only once in 10 occurrences, then the culvert would have been overdesigned and could be expected to be overtopped on an average of only once in 100 years. If the structure had been the spillway for a large flood-control dam and due to an error in frequency it had been built for a capacity that could be expected to be exceeded once every 1,000 years instead of once in 500 years, the percent difference in Q would have been small in comparison with the difference between the once in 10-yr. and the once in 100-yr. values of our example. This is because in a frequency curve the rate of change in Q decreases rapidly as the recurrence interval increases. This is illustrated in Figure 2, which shows a typical frequency curve for a 176-acre cultivated watershed. In our culvert ex-

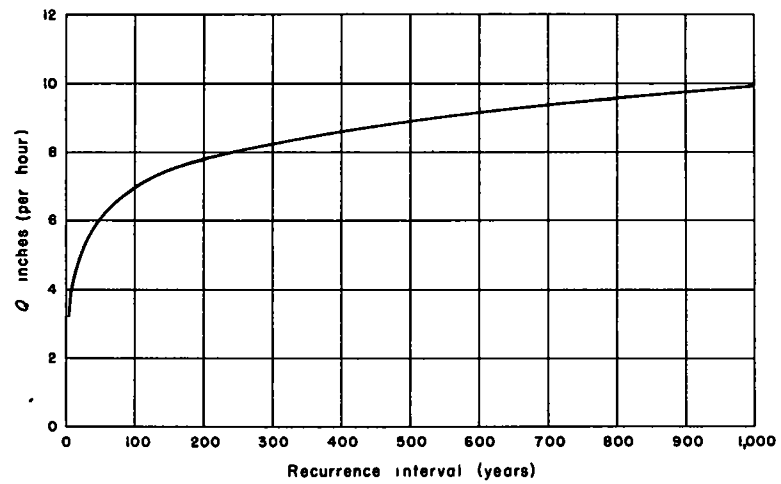


Figure 2. Frequency Curve for Watershed W-1 (1938-1948), Soil Conservation Service Blacklands Experimental Watershed, Waco, Texas

[2]Figures in parentheses refer to references listed at the end of the paper.

ample, the precent error introduced by the use of the once in 100-yr. Q instead of the one to be expected once in 10-yr. would have been

$$\frac{7.0 - 4.1}{4.1}$$

or approximately 71 percent. In the case of the flood-control spillway, the error would have been

$$\frac{9.9 - 8.9}{8.9}$$

or only 11 percent.

From the foregoing discussion, it should be concluded that the rational method will give accurate results only if the values of C are taken for maximum runoff conditions and modified to compensate for the difference between rainfall and runoff frequencies. It provides a logical framework upon which to base estimates of Q in areas where no runoff measurements are available but its use should be restricted to those who are thoroughly familiar with rainfall and runoff relationships within the physiographic region in which it is to be applied.

The Unit Hydrograph -The same difficulties that have been discussed in connection with the two preceding analytical procedures were encountered when attempts were made to apply the unit hydrograph in the analysis of runoff data from small agricultural watersheds.

For a given storm pattern of unit duration, the unit hydrograph theory assumes that the runoff characteristics of a watershed are constant and may be expressed in the form of a hydrograph of 1 in. of runoff. The following discussion will show that this assumption is not satisfactory when applied to a small watershed.

For the same unit storm, the excess rainfall or runoff can vary considerably (Fig. 1), depending on runoff conditions. When runoff conditions are maximum, the rainfall excess will also be maximum as will the depth and hence the velocity of both overland and channel[3] flow. When runoff conditions are minimum, the velocity of runoff will also be minimum. Other factors being equal, the time lag or time between the occurrence of the center of the unit storm period and the peak rate of runoff will be dependent on the velocity of the surface runoff. It would be expected, therefore, that the unit hydrograph for a storm occurring when runoff conditions were maximum would have a short time lag and a sharp peak. Conversely, the same storm occurring when runoff conditions were minimum would have a longer time lag and a flatter peak.

For runoff conditions other than maximum, the effect of vegetal cover is to decrease rainfall excess and hence reduce the velocity of both overland and channel flow. For any watershed condition, vegetal cover reduces the velocity of overland flow, due to the impediment offered by its stems. In small watersheds, the ratio of length of overland flow to the length of channel flow is large. A reduction in the velocity of overland flow, therefore, materially increases the total travel time of runoff and has an appreciable effect on the shape of the unit hydrograph. Since the ratio of overland flow to channel flow becomes smaller as the length of channel flow increases, the effect of a reduction in the velocity of overland flow on total travel time is probably negligible on a large watershed.

Because vegetal cover reduces the velocity of overland flow, watersheds that are planted to row crops in some years have different unit hydrographs than in other years when the crop may have been changed to close-growing vegetation, such as grass. Also, watersheds in which meadows are rotated from field to field have different unit hydrographs, depending on the position of the meadow fields with respect to the watershed boundaries.

If it were assumed that peak rates

[3]Channel flow for the purposes of this paper is defined as flow in that portion of a channel that is not cultivated.

of runoff were determined by developing a unit hydrograph for a storm period in which the intensity distribution was such as to produce maximum runoff and that cover, location of grassland, and soil moisture on the watershed were also optimum for maximum runoff, then it would still be necessary to determine the frequency of occurrence of such a set of conditions. As pointed out in the discussion of the rational method, the assumption that the frequency of the unit storms are the same as the frequency of maximum runoff results in considerable error when used in connection with hydraulic structures designed for a failure of once in 10 to 50 years.

Infiltration Theory - The infiltration theory assumes that the runoff from a watershed can be determined by the establishment of relationships between such factors as rainfall rate, transmission velocity, infiltration rate, percolation rate, and soil moisture. It is not the purpose of this paper to review the procedures that have been developed by various engineers and hydrologists but to point out that the application of the theory has been tested on only a few very small single-cover watersheds. It may well be that future development of this theory will produce a simple and useful tool for the analysis of runoff data. Much work, however, remains to be done before this can be accomplished, not only in testing the application of the theory to a much wider range of field conditions, but also in unifying and simplifying the present procedures.

PROBABILITY STUDIES

The use of probability studies to determine peak rates of runoff was explored because it was felt that this type of analysis should integrate the frequency of occurrence of various watershed conditions with the frequency of occurrence of various rainfall intensities and patterns.

Classes of Probability Curves - In general, probability curves used in connection with hydrologic data may be divided into two classes. The first class considers all occurrences above a predetermined base. The probable frequency of occurrence is computed in terms of the average number of times that a given value will be equaled or exceeded in some interval of time. Thus, a frequency of occurrence of once in 5 years would mean that a designated value could be expected to be equaled or exceeded by an average of one occurrence every 5 years.

In the second class, only the maximum value per unit of time is considered. The frequency of occurrence is computed in terms of the average number of time units during which a given value may be expected to be equaled or exceeded in some time period. Thus, when one year is taken as the time unit, a frequency of occurrence of once in five years would mean that there would be an average of one year every five years during which a designated value could be expected to be equaled or exceeded.

It has been found that peak rates of runoff determined by the two classes of probability curves are practically identical for recurrence intervals of 10 years or more (2). Since hydraulic structures to control runoff from small agricultural watersheds are designed for frequencies of failure of not more than once in 10 years, and since the work involved in computing the probability curve is much less for the second class curve, probability studies of peak rates of runoff have been limited for the most part to maximum annual values. Probability curves as used in the balance of this paper refer, therefore, to curves of this class.

Hazen and Foster Probability Curves - Foster's Types I and III probability curves developed by H. A. Foster (4) in 1924 and the Hazen curve developed by Allen Hazen (12) were probably the best known and most widely used probability curves prior to 1941. These curves are similar in that they are all fitted curves based on a normal distribution of peak rates. Since rainfall and runoff phenomena do not occur as normal distributions but are considerably skewed, these curves are then corrected in accordance with coefficients of skew computed from the original data.

In 1936, J. J. Slade, Jr. (23), tested the significance of these skew coefficients and showed conclusively

TABLE 1

DEPARTURE FROM MEAN Q FOR MIXED COVER WATERSHEDS

Watershed Area in Acres	Mean Q in Cubic Feet per Second [a]	Area of Application				
		Upper Miss Valley Loessial Areas	Central Great Plains of Kansas and Nebraska [b]	North Appalachian Region	High Plains of Colorado and New Mexico [c]	Coastal Plains of N.J., Del and Md. [d]
		Percent of Area in Grass				
		50	20	50-75	50	25-50
		Departure from Mean Q in Percent of Mean				
10	37	- 16	+ 8	- 40	+ 14	+ 35
20	61	- 15	+ 3	- 31	+ 16	+ 30
30	83	- 16	- 1	- 25	+ 16	+ 25
40	102	- 16	- 3	- 21	+ 16	+ 24
50	121	- 16	- 6	- 17	+ 16	+ 22
100	198	- 14	-10	- 14	+ 17	+ 21
500	586	- 10	-14	- 25	+ 26	+ 23
1,000	926	- 8	-15	- 31	+ 28	+ 26
1,500	1,136	- 1	- 8	- 29	+ 38	
2,000	1,370	- 1	- 7	- 31	+ 39	
2,500	1,566	+ 1	- 8	- 33	+ 40	
3,000	1,750	+ 3	- 9	- 35	+ 41	
3,500	1,918	+ 3	- 8	- 36	+ 41	
4,000	2,088	+ 3	- 8	- 37	+ 41	
4,500	2,255	+ 4	- 8	- 37	+ 41	
5,000	2,390	+ 5	-10	- 37	+ 42	

[a]All values are for a recurrence interval of 10 years.

[b]For watersheds with a meander factor of 1 00

[c]Average of recommended values for 4 soil and slope conditions

[d]Average of recommended values for 3 soil and slope conditions

"that skewness is never a truly significant characteristic when the sample from which it is computed has less than about 140 items . . . and that it is quite meaningless to use this measure when there are 50 or fewer items." Since the length of record for most small agricultural watersheds is 10 years or less, it is obvious that the Foster or Hazen curves could be of little use in making probability studies of the peak rates of runoff that might be expected from these areas.

Gumbel Probability Curves-During the period 1941-45, E. J. Gumbel (5 to 10) published several papers in the various technical journals in which he developed a new concept of probability as applied to rainfall and runoff data. He computed the frequency for the highest and lowest peak rates in a sample of N years as those of the modal or most frequent values of an infinite number of samples of N years. These frequencies of the highest and lowest values he expressed in terms of N. Frequencies of intermediate peak rates could then be obtained by prorating the difference between the frequencies of the highest and lowest values.

In 1943, R. W. Powell (21) developed

a special graph paper on which peak rates of runoff plotted in accordance with Gumbel's frequencies would approach a straight line.

The form of distribution of peak rates assumed by Gumbel is such that the coefficient of skew does not have to be calculated, as is the case with the Hazen and Foster curves, but is implied as a constant of 1.139. For this reason, his probability curve is very well suited for use in probability studies of skewed distributions as those of rainfall and runoff data.

To compute a probability curve of peak rates of runoff by the Gumbel method, maximum annual peak rates are arranged in ascending order of magnitude. Frequencies of the highest and lowest values are determined from Tables 1 and 2 (9, 10, 18) and the frequencies of intermediate peaks by prorating between these extreme values. Peak rates are then plotted against corresponding frequencies on special probability paper. For each plotted point a corresponding value is determined on the linear or "reduced variate" scale. Using these values as X and the peak rates as Y, a least squares straight line is then computed. Figure 3 shows an example of such a curve.

A simplification of this procedure was developed by the author (18) that involves only the calculation of mean Y, ($\bar{Y}$), and the coefficient of variation, (C_V). A series of curves was developed to express the relationship between $Y/\bar{Y}$ and C_V for various values of N and for various recurrence intervals. Knowing N and C_V, values of $Y/\bar{Y}$ may be selected from these curves for desired recurrence intervals. These values multiplied by $\bar{Y}$ give the peak rate that may be expected to be équaled or exceeded for each recurrence interval.

USE OF GUMBEL PROBABILITY CURVE IN ANALYSIS OF RUNOFF DATA

A probability curve is reliable only insofar as it is a representative sample of what has taken place in the past and of what may be expected to take place in the future. If we were dealing with a long runoff record of say 50 or 75 years from a watershed in which the physiographic features, land use, and tillage practices had been fairly constant, it would probably be safe to say that our sample was representative of past runoff conditions. And, if there was no reason to suspect that any of these factors would be materially changed, it would be reasonable to assume that the sample also was representative of future runoff conditions. If, however, our runoff record was for only 8 or 10 years, it is obvious that no such assumptions could be made with any degree of safety. Since most of the records of runoff from small agricultural watersheds are included in this latter range, it is evident that some test of their normalcy must be made before they can be used to estimate future runoff values.

In formulating a test of normalcy for short runoff records, two assumptions were made: (1) that if the rainfall for the period of runoff record is representative of that for a much longer period, then the runoff sample is also representative of that period; (2) that if there is no reason to suspect that the physiographic features, land use, or tillage practices of the watershed will be materially changed, then the runoff record is also a representative sample of what might be expected in the future.

The three tests of rainfall selected were: (1) comparisons of monthly and annual amounts of rainfall; (2) comparisons of maximum average rainfall intensities for various time intervals; and (3) comparisons of monthly and annual number of excessive storms. These rainfall factors were selected for test because they not only have a material effect on peak runoff rates but also are readily available from long-term Weather Bureau records. It is recognized that other factors, such as rainfall pattern, soil moisture, and infiltration rate, also affect runoff. However, until such time as long-term records of these additional factors become available, normalcy tests must necessarily be limited to the three tests more fully described as follows:

Rainfall Amounts - (19) Comparisons of monthly and annual rainfall can be made by computing probability curves for these values for the period of runoff record and comparing values for like recurrence intervals with those derived from long-term Weather Bureau records. Records of monthly and annual record may be compared to similar curves computed from long-term Weather Bureau records.

Prior to 1935, rainfall intensity data from first-order Weather Bureau Stations were included in the Annual Report of the Weather Bureau and beginning in 1935 in the Meteorological Yearbook. Prior to 1935, these data were published as accumulative amounts for each

TABLE 2

RECOMMENDED *Q* FOR CULTIVATED AND PASTURE AREAS EXPRESSED AS A PERCENT OF *Q* FOR MIXED COVER

	Area of Application				
	Upper Miss Valley Loessial Areas	Central Great Plains of Kansas and Nebraska	North Appalachian Region	High Plains of Colorado and New Mexico[a]	Coastal Plains of N J, Del. and Md [b]
Q(Cultivated Area) / Q(Mixed Cover Area)[c] *Percent*	130	116	170	113	
Range in Watershed Size, *Acres*[d]	2-100	2-400	5-10,000	2-250	
Q(Pasture Area) / Q(Mixed Cover Area)[c] *Percent*	65	72	60	82	51
Range in Watershed Size, *Acres*[d]	2-100	2-200	5-10,000	2-5,000	10-200

[a]Average of recommended values for 4 soil and slope conditions

[b]Average of recommended values for 3 soil and slope conditions

[c]All values are for a recurrence interval of 10 years

[d]Above ratios applicable only for indicated range in watershed size

rainfall from the beginning of record to 1930 may be found for all stations in "Summary of the Climatological Data for the United States, by Sections," United States Weather Bureau, Bulletin W. Similar data for the period subsequent to 1930 are published in the current year books of "Climatological Data."

Rainfall Intensity -(19) A comparison of maximum rainfall intensities for various time intervals can be made in a manner similar to that described for rainfall amounts. Probability curves based on highest average intensities for each time period for each year of runoff 5 min. of excessive rainfall. Since changes in rainfall intensity did not necessarily coincide with these 5-min. division points, maximum intensities computed for various time intervals from Weather Bureau tabulations for this period are usually less than the true maximum. Yarnell (24) found this difference to be 8 to 10 percent of the computed figure for 5-min. periods and 4 to 5 percent for periods of one hour.

Number of Excessive Storms - (19) As originally defined by the Weather Bureau, an excessive storm is one in which the amount of rain that fell during any 5-min. period was equal to or greater than 0.25 in. or in which the amount that fell during any period in excess of 5 min. was equal to or greater than 0.25 in. plus 0.01 in. for each minute in excess of 5. In 1935, the amounts defined by this definition were increased for the Southern States, including North Carolina, South Carolina, Georgia, Florida, Alabama, Mississippi, Tennessee, Arkansas, Louisiana, Texas, and Oklahoma, and San Juan, Puerto Rico. Under the new definition a storm was not considered excessive unless the amount of rain in inches that fell during any time period of five or more minutes was equal to or greater than twice the time period in minutes expressed in hundredths of an inch plus 0.30 in. Care must, therefore, be exercised in making comparisons of excessive rainfall in these States to be sure that all storms are classified in accordance with the same definition.

Average monthly and annual numbers of excessive storms can be obtained from Yarnell's (24) 30-yr. totals and compared with similar averages for the period of runoff record. Since Yarnell's totals were based on Weather Bureau intensities published prior to 1935, and since these intensities were less than the true maximums, it follows that the number of excessive storms based on these lesser values must also be less than the actual number. As a result of comparisons made from experiment station records at eight different locations (19), it was concluded that Yarnell's 30-yr. totals must be increased by 16 percent to approximate the actual total.

Correction for Abnormal Rainfall -If the rainfall for the period of runoff record is found to be materially different from that for a long-time period, then the peak rates computed from a probability curve should be adjusted to what they would have been had the runoff record been a good sample of past conditions (20). A methodology for making such a correction was developed by the author for an 8-yr. period of record on two

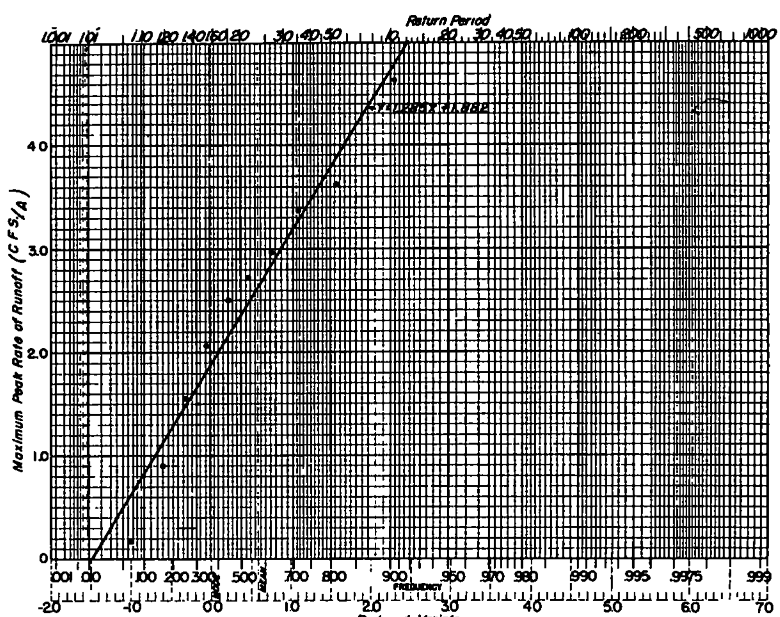


Figure 3. Frequency of Maximum Peak Rates of Runoff from Watershed D-3 at Conservation Experiment Station, Bethany, Mo., 1933-1942

watersheds located at the Central Great Plains Experimental Watershed near Hastings, Nebraska. A relationship was established between the three rainfall factors tested and the peak rates as computed by a probability curve. It was found that the amount of annual rainfall multiplied by the annual number of excessive storms determined the frequency of maximum runoff conditions, whereas the intensity of the rainfall determined the magnitude of the runoff peak for any runoff condition. Although much work remains to be done in testing this method for other physiographic and meteorologic conditions, it is felt that this method or some modification can be perfected for adjusting probability curves for abnormal rainfall.

The runoff records of small experiment station watersheds can usually be supplemented by U. S. Geological Survey records of larger watersheds with physiographic features and land use similar to those at the experiment station. To eliminate, insofar as possible, all factors other than size of watershed that might account for differences in peak rates of runoff, probability curves for these larger watersheds should be computed for the same period of record as that of the experiment station. These probability curves should then be corrected to compensate for differences between rainfall factors that existed at the large watersheds during the period of record and those previously determined for the experiment station from long-time Weather Bureau records. In making these corrections the same procedures may be followed as those developed for correcting the probability curves of small experiment station watersheds.

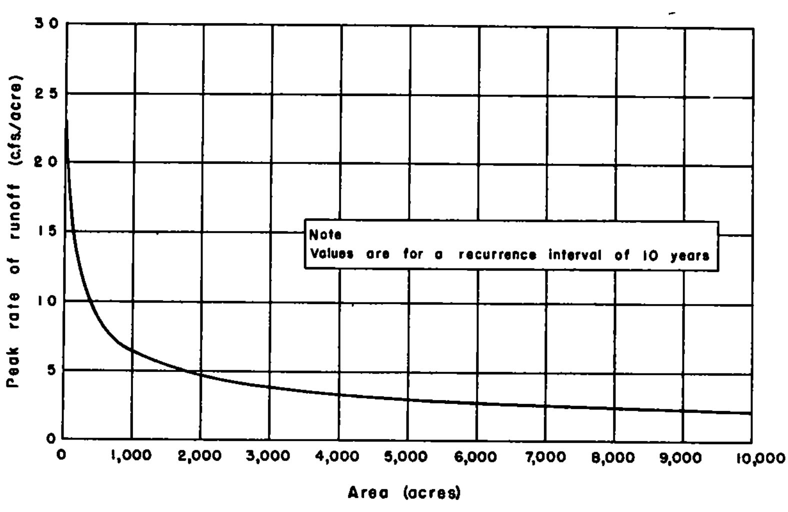


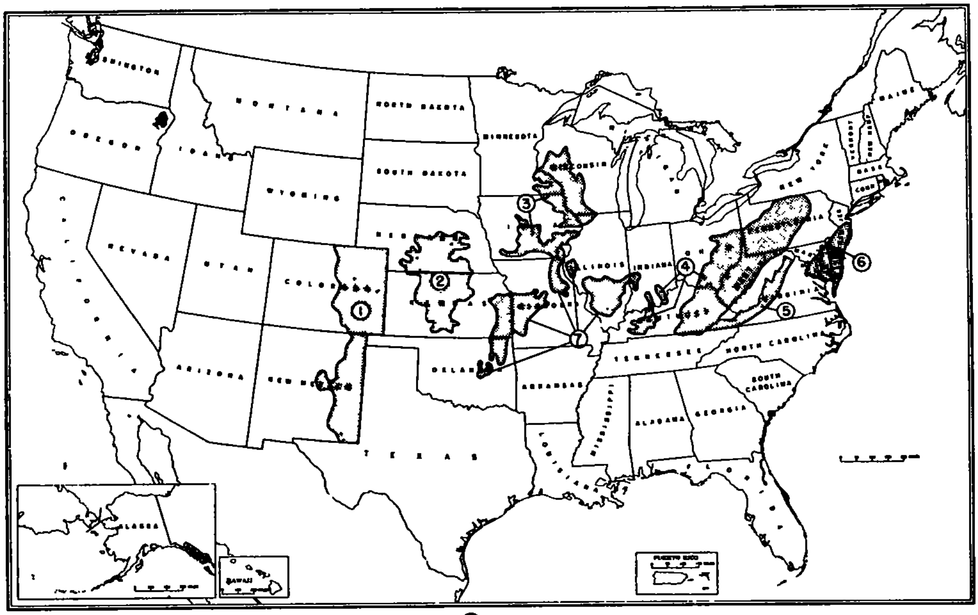
Figure 4. Relationship Between Peak Rates of Runoff and Area for Mixed Cover Watersheds in the North Appalachian Region

Area of Watershed Versus Peak Rate of Runoff - For the same recurrence interval, the peak rate of runoff per unit area decreases as the area of the watershed increases. Figure 4 shows that this decrease is very rapid for watersheds of less than 2,000 acres. The next step then in the analysis of the runoff data from the watersheds at any experiment station is to determine the relationship between the peak rates for any recurrence interval and the size of watershed.

The corrected peak rates for a once-in-10-yr. recurrence interval can now be plotted against the corresponding size of watershed on log-log paper. As the corrected probability curves for all watersheds are now representative of runoff as it would have occurred if each watershed had been subjected to the same rainfall (that determined for the experiment station from long-term Weather Bureau records) and as the physiographic features and land use for all watersheds are similar, any difference in peak rates can be ascribed to the effect of watershed size. The relationship between peak rates and watershed size can, therefore, be expressed by computing a statistical curve from the coordinates of the plotted points. This relationship is assumed constant for all recurrence intervals and when applied to the probability curves of the experiment station watersheds gives the peak rate that may be expected from any size watershed for any desired recurrence interval.

Area of Application -One other step remains to be taken in our analysis; namely, that of determining the area of application for the computed peak rates of runoff. This involves the outlining of the area or areas in which physiographic features such as parent geologic formation, principal soil types, topography, and land use are similar to those of the experimental watersheds.

This last step was made possible when the Soil Conservation Service in 1943 printed a map of the United States showing "Basic Land Resource Areas." This map was developed primarily on physiographic features determined by conservation surveys and divides the country into nine physiographic regions, 68 major subdivisions, and numerous minor subdivisions. Revisions are made from time to time as additional surveys are completed. Detailed descriptions of each subdivision have been prepared and may be used in conjuction with the map to select areas where physiographic features, insofar as they affect runoff, are similar to those of the experimental watersheds.



(1) High Plains, (2) Central Great Plains, (3) Upper Mississippi Valley Loessial Areas, (4) North Appalachian Region, (5) Ridges and Valleys of Virginia, (6) Coastal Plains (7) Claypan Prairies

Figure 5. Areas of Application for Recommended Peak Rates of Runoff

Although physiographic and cultural features of the area selected generally will be similar to those of the experimental watersheds, small local areas can usually be found within the area of application where these features are materially different. No attempt was made to delineate all of these local exceptions and it is left to the judgment of the field technician as to whether or not recommended peak rates should be increased or decreased because of these differences.

SUMMARY OF RUNOFF STUDIES COMPLETED TO DATE

Analysis of runoff data has been completed for seven physiographic areas and the recommended peak rates of runoff for use in the design of conservation structures have been published in Technical Publications of the Soil Conservation Service (Fig. 5), (1, 3, 11, 13, 16, 17, and 22). The procedures used in these analyses in general followed those outlined in the preceding pages of this paper (15). In all cases, it was found that the three rainfall factors experienced during the period of runoff record was equal to or greater than those determined from long-term Weather Bureau records. No corrections were applied to compensate for rainfall differences as the methodology for accomplishing this had not yet been developed. It is hoped that later reports will include such corrections.

It should be pointed out that all of these publications are preliminary reports that will be revised as more data become available. Also, that all of the reports are not based on the same completeness of data. The peak rates for the High Plains of Colorado and New Mexico; the Coastal Plains of New Jersey, Delaware, and Maryland; the Ridges and Valleys of Virginia, and the Claypan Prairies were determined primarily from small watersheds of from 2 to 300 acres. Additional data on peak rates from watersheds having a greater range in size are needed in these areas to more accurately establish the peak rate versus area relationship.

In the Ridges and Valleys of Virginia, the small experimental watersheds are underlain by shattered limestone and result in extremely low rates of runoff. It is felt that a large proportion of the water absorbed by the limestone on these small watersheds may appear as base flow runoff on larger areas. To test this possibility and to establish a more reliable peak rate versus area relationship, three additional watersheds ranging in size from 500 to 6,000 acres have been located on Bell Creek near Staunton, Va. Data from these watersheds are being collected as a cooperative project of the Soil Conservation Service and the U. S. Geological Survey.

In Table 1, the peak rates for a mixed-cover watershed and a recurrence interval of 10 years are expressed as percent departures from the mean for all physiographic areas shown on Figure 5 with the exception of the Ridges and Valleys of Virginia and the Claypan Prairies. The Ridges and Valleys of Virginia was not included because it was felt that insufficient data existed to determine accurately the effect of the underlying limestone on runoff peaks. Peak rates for the Claypan Prairies were determined only for recurrence intervals of 25 and 50 years. As the land use for the watershed is essentially the same for each area (mixed cover), the departures from the mean shown in the table can be ascribed to differences in rainfall and physiographic factors.

For the same five years, Table 2 shows the peak rate for a cultivated and pasture watershed for a 10-yr. recurrence interval expressed as a percentage of that for a mixed-cover watershed. Since rainfall and physiographic factors were the same for all three types of land use at any one area, the indicated increase or decrease in peak rates for that area can be ascribed to the effect of land use. The reduction in peak rates that may be affected by a change in land use is dependent to some extent on existing rainfall and physiographic factors. This is evidenced by the difference in the percentages shown in Table 2 for different areas.

CONCLUSIONS

It is hoped that this paper will help direct the attention of the engineer and hydrologist to the differences in the hydrologic behavior of large watersheds as compared to that of the small agricultural watershed.

It is felt that a recognition of these differences will lead to the development and increased use of such analytical procedures as the probability study, as a means of more accurately integrating the effects of the many factors that affect surface runoff.

It is felt that the use of such improved analytical procedures will make possible more accurate and economical designs of small hydraulic structures.

And, finally, it is hoped that greater cooperative efforts among interested Federal and State agencies will relieve the present sparsity of runoff data from small agricultural watersheds.

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TENTATIVE RESULTS ON CAPACITY OF CURB OPENING INLETS

Carl F. Izzard[1]

SYNOPSIS

This paper presents a method of estimating the capacity of a curb opening inlet as a function of the depth of flow in the approach gutter, the depression of the inlet lip below the flow line of the gutter, and the length of the opening. The method not only indicates the flow which would be entirely intercepted by a given inlet but also the percentage of the total flow in the gutter which would be intercepted as the discharge is increased beyond that point. This method is based on a theoretical analysis which treats the curb opening as a weir with the head decreasing along the crest. Despite a number of simplifying assumptions in the analysis, experimental data from three independent investigations verify, within practical limits, the validity of the theory. The study substantiates the belief of practical engineers that depression of the gutter flow line is necessary to achieve reasonably efficient operation with the curb opening type of inlet. It also proves that the capacity of the inlet varies with the grade and also with the cross section of the street as it affects the depth of flow. The second part of the paper includes suggestions as to how the engineer might apply the results of this study to the design of curb opening inlets, especially for express highways. A brief discussion is included of the relative capacities of curb opening and grate inlets.

Recent examination of the plans for an express highway disclosed that the curb opening inlets, although well designed hydraulically, were so widely spaced that only 15 to 50 percent of the gutter flow would be intercepted. The result was that the stormwater would accumulate to a depth of about 1 ft. at the low points in the grade in three successive underpasses. The storm sewers collecting discharge from these inlets would flow at less than one quarter of their designed capacity while the gutters would be overloaded.

Recognizing the need for information on hydraulic capacity of curb opening inlets on grades, a study has been made of available experimental data as summarized in the first part of this paper. The results are expressed in equations developed by mathematical analysis. The second part of the paper utilizes graphs to enable direct and quick solution for the hydraulic capacity of any length of inlet depending on the depth of flow in, and cross section of, the approach gutter, and the depression of the gutter flow line at the inlet. A comparison is also made between the hydraulic characteristics of grate inlets and curb opening inlets. Suggestions are made on computation of inlet spacing.

There have been three investigations since 1945 which provide useful data on curb opening inlets adaptable to use on express highways. The work by Conner in 1945 has been abstracted in the *Proceedings* of the Highway Research Board (1)[2]. These tests were made on a roadway cross section with an 8-in. parabolic crown. Only the discharge which would be completely intercepted by the inlet was reported.

The Corps of Engineers have reported tests (2) on curb opening inlets installed on a cross section having a transverse slope of 1-1/2 percent where at least 50 percent of the flow in the gutter went past the inlet. These tests and those by Conner both included a depression of the gutter flow line at the location of the curb opening.

The only tests on curb opening inlets

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[2]Figures in parentheses refer to the list of references at the end of the paper.

having no depression of the gutter flow line have been made within the past year by the University of Illinois in cooperation with the Illinois Division of Highways and the Bureau of Public Roads. The final results of these tests will be published in the form of a bulletin of the University of Illinois Engineering Experiment Station.

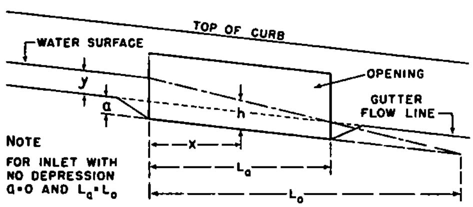

Figure 1. Diagrammatic Sketch of Typical Curb Opening Inlet

MATHEMATICAL DEVELOPMENT AND EXPERIMENTAL VERIFICATION

This analysis covers only the case where the curb opening inlet is located in a gutter on a continuous gradient. Under this condition the velocity of flow in the gutter is normal to the plane of the opening and is assumed to be wholly ineffective in inducing flow through the opening in the curb. It is also assumed that the energy due to the fall along the length of the inlet is dissipated in friction so that this fall has no effect on flow into the inlet. With these simplifying assumptions the basic theory is simply that water flows over the lip of the inlet in a manner similar to flow over a broad-crested weir with the head at the upstream end of the inlet being equal to the depth of the water in the approach gutter if there is no depression, or equal to the depth in the approach gutter plus the amount by which the inlet lip is depressed below the gutter flow line extended. The final simplifying assumption is that the head varies in direct proportion to the distance from the beginning of the inlet, becoming 0 at the downstream end of the inlet for the case where there is no depression and the inlet is just barely intercepting all the flow. If the inlet lip is depressed below the gutter flow line, then the head at the downstream end of the inlet when all the flow is just barely being intercepted is assumed to be equal to the amount of the depression.

We can therefore set up equations for the flow into the inlet as follows: Let h = head in feet on the inlet lip at a point x ft. from the beginning of the opening, y = depth of flow in feet in the approach gutter, a = depression in feet of the inlet lip below the gutter profile extended.

Taking the length of the inlet as L_a (see Fig. 1), then from the assumptions in the previous paragraph,

$$h = a + y - \frac{x}{L_a}\, y = y\left(\frac{a}{y} + 1 - \frac{x}{L_a}\right) \qquad (1)$$

If the opening is considered to be a weir then the depth of flow across the inlet lip is critical, and velocity of flow into inlet at point x is

$$v = \left[g\left(\tfrac{2}{3}\,h\right)\right]^{1/2}$$

(g being the acceleration of gravity). The flow through an elemental strip dx becomes

$$dQ = q\, dx = v\left(\tfrac{2}{3}\,h\right) dx.$$

Substituting values of v and h above, and integrating between limits x = 0 and x = L,

$$Q = \left(\tfrac{2}{3}\right)^{3/2} g^{1/2}\, y^{3/2} \int_0^L \left(\frac{a}{y} + 1 - \frac{x}{L_a}\right)^{3/2} dx$$

$$= 1.23\, L_a y^{3/2}\left[\left(\frac{a}{y} + 1\right)^{5/2} - \left(\frac{a}{y} + 1 - \frac{L}{L_a}\right)^{5/2}\right] \qquad (2)$$

Equation (2) is the discharge in cu. ft. per sec. which would be intercepted by an inlet of length L provided the theoretical assumptions are reasonably correct. It cannot be applied directly because the length L_a is unknown. The equation can be simplified if we first consider the case where the inlet has no depression, that is a = 0.

Inlet Without Depression - For the case where a = 0, equation (2) becomes

$$Q = 1.23\ L_o y^{3/2}\left[(1 - \frac{L}{L_o})^{5/2}\right] \qquad (3)$$

This equation still does not give a direct solution unless the value of L_o is known, L_o being the length at which the inlet with no depression would just barely inter-

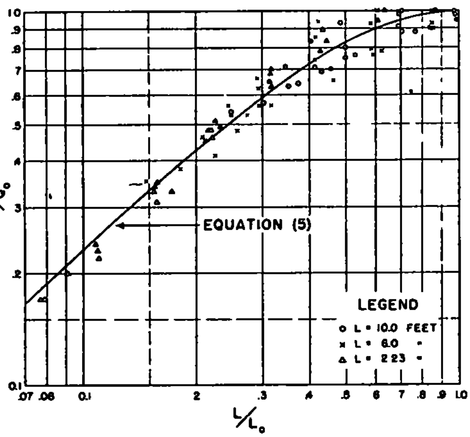

Figure 2. Illinois Data on Curb Opening Inlets Having No Depression

cept all of the flow in the gutter. If we make $L = L_o$, then equation (3) reduces to

$$Q_o = 1.23\ L_o y^{3/2} \qquad (4)$$

which is the tneoretical capacity of an inlet of length L_o with no depression when it is just barely intercepting the total flow in the gutter. As will be shown later, accurate measurement of the discharge for this case is difficult. Therefore, in order to utilize more of the experimental data it was found desirable to set up an equation for Q/Q_o by dividing equation (3) by equation (4).

$$\frac{Q}{Q_o} = 1 - (1 - \frac{L}{L_o})^{5/2} \qquad (5)$$

The Illinois experimental data included tests on inlets with no depression 2.23 ft., 6 ft., and 10 ft. long on grades from 1/8 percent to 4 percent. The tests were made on a 1:3 scale model, all dimensions here being prototype values. The roadway cross section is shown in Figure 6.

The Illinois experimental data were found to fit equation (5) reasonably well if

$$L_o = \frac{Q_o}{0.7y^{3/2}} \qquad (6)$$

This equation will be noted as identical to equation (4) except that the numerical coefficient has been changed from 1.23 to 0.7. The values of L/L_o plotted against the values of Q/Q_o shown in Figure 2, together with a curve drawn for equation (5), indicate that the equation is a reasonable approximation of the entire range of data. The scatter of these points for high values of the discharge ratio indicates why it is difficult to evaluate the numerical coefficient by the use of equation (4) alone.

Inlet with a Depression - When a curb inlet with the lip depressed a ft. below the flow line of the gutter is just barely intercepting the total flow in the gutter, the capacity, which we will call Q_a, can be approximated by the equation:

$$\frac{Q_a}{L_a} = 0.7(a + y)^{3/2}\left[1 - (1 - \frac{y}{a + y})^{5/2}\right] \qquad (7)$$

This equation will be observed as similar to equation (4) except that the numerical coefficient has the same value as in equation (6), (a + y) is substituted for y and the term in brackets is added. The latter term corrects for the fact that the head at the downstream end of the inlet reduces to a instead of reducing to zero as in equation (4).

From Figure 1 it will be found by similar triangles that

$$\frac{y}{a + y} = \frac{L_a}{L_o}$$

where L_o is the value of x when h = 0, having the same significance as in equation (4). If L_a/L_o is substituted in equation (7) for

$$\frac{y}{a + y}$$

the term in brackets will be observed as being identical with the right hand side of equation (5). Consequently this term is actually the ratio of Q_a/Q_o, Q_o being the capacity of an inlet of length L_o when the approach depth is (a + y). If we substitute Q_a/Q_o for the term in brackets, equation (7) becomes

$$\frac{Q_o}{L_a} = 0.7(a + y)^{3/2} \qquad (8)$$

Equation (8) is roughly in agreement

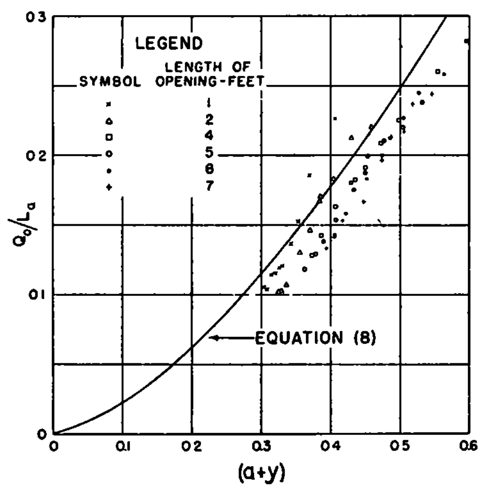

Figure 3. North Carolina Data on Curb Opening Inlets with 3-in. Depression

with the experimental data reported by Conner (3) as shown in Figure 3. In these tests, a = 0.25 ft., L varied from 1 ft. to 7 ft., and gutter grades from 0.5 percent to 10 percent. The gutter cross section is shown in Figure 6. As previously noted, Conner reported only a discharge Q_a at which the inlet barely intercepted the entire flow.

A limited number of tests (2) were made by the Corps of Engineers, St. Paul District, on inlets 3 ft. and 8.92 ft. long with a = 0.167 ft. on grades of 0.75 percent and 1.0 percent with some tests of the longer inlet on a 2 percent grade. These tests were made on a 1:2 scale model. Prototype dimensions are used in this paper. The flow intercepted was never greater than 50 percent of the total flow. The cross section is shown in Figure 6.

From this case it is necessary to set up an equation for the ratio Q/Q_a in which Q is the flow intercepted and Q_a is the flow in the gutter. If L in equation (2) is made equal to L_a, then Q/Q_a becomes

$$\frac{Q}{Q_a} = \frac{(\frac{a}{y} + 1)^{5/2} - (\frac{a}{y} + 1 - \frac{L}{L_a})^{5/2}}{(\frac{a}{y} + 1)^{5/2} - (\frac{a}{y})^{5/2}} \qquad (9)$$

This equation is plotted in Figure 8 for

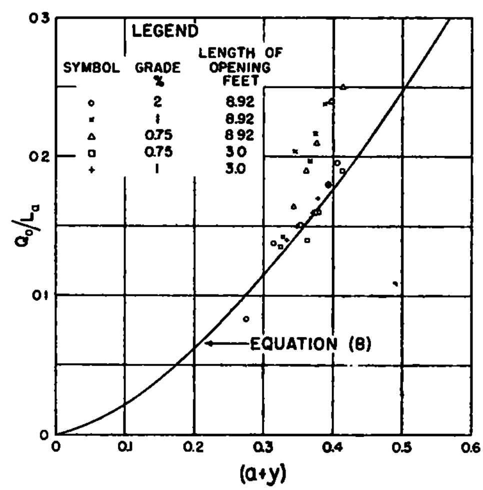

Figure 4. Corps of Engineers Data on Curb Opening Inlets with 2-in. Depression

different values of the parameter $\frac{a}{y}$ to facilitate solution.

From the experimental values of a, y, and Q/Q_a reported by the Corps of Engineers (including values of $\frac{a}{y}$ up to 1.6), L/L_a in equation (9) was determined directly from Figure 8. Then, dividing the length of inlet L by L/L_a, the theoretical length L_a was computed at which the entire gutter flow would be intercepted. This converted the data into the same variables appearing in equation (7). The final step was to compute Q_o which was done in the same manner as described for the North Carolina data. Dividing Q_o by L_a gave the experimental points plotted in Figure 4. Equation (8) is plotted for comparison.

Discussion of Experimental Data - In

order to compare the data from the three independent investigations, all of the points plotted in Figures 2, 3, and 4 are combined in Figure 5. The curve gives the values computed by equation (8) and also the values given by equation (6), which is simply the special case where a = 0. From the scientific viewpoint the deviation of the experimental points from the curve is too great to ignore and suggests that there are other variables which are affecting

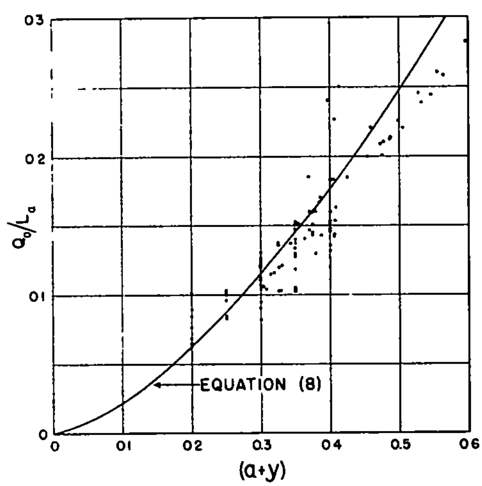

Figure 5. Composite Plot of Data from Figures 2, 3, and 4

the experimental results. The scatter is probably due in only a minor extent to experimental errors because in each investigation the data for the most part are consistent for each combination of length of inlet and roadway grade. It would probably be possible to derive a more complicated expression for the basic relationship than is given by equation (8), but it is not believed that there is a sufficient range of data in any of the investigations to justify that effort at the present.

The influence of changes in a and y upon the capacity of an inlet to intercept 100 percent of the gutter flow can be visualized from Figure 7 which is plotted from equation (7). The ordinate is the average capacity in cubic feet per second per foot of length and the abscissa is the depth (feet) in the approach gutter for the indicated values of the depression.

For an inlet of length L_a, the capacity Q_a in cubic feet per second may be read directly by multiplying the ordinate scale in Figure 7 by L_a. Thus the scale on the right hand margin, which is 10 times the scale for Q_a/L_a, will give the capacity of a 10-ft. inlet directly in cubic feet per second.

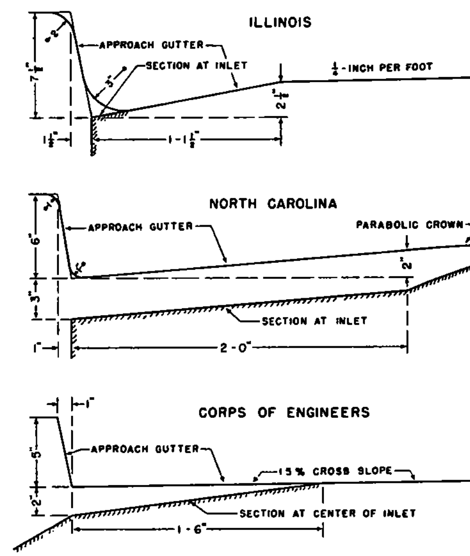

Figure 6. Sections Through Approach Gutter and Through Inlet for the Three Investigations

Balanced Capacity - The capacity of an inlet in relation to the gutter capacity can be directly compared by superimposing on Figure 7 a curve for gutter capacity. The dotted line in Figure 7 is such a curve of Q_a (right margin) plotted against y for a gutter having a longitudinal grade of 1 percent and a cross slope of 2 percent (z = 50), assuming a roughness coefficient n = 0.015. The intersection of this curve with the inlet capacity curve for a given value of a gives a value of Q_a which may be termed the "balanced capacity." If the discharge increases beyond the balanced capacity, part of the gutter flow will not be intercepted. The actual interception in that case cannot be read from the inlet capacity curve since it applies only to the condition where 100 percent of the flow is being intercepted, but can be determined from Figure 8 as will be shown. If the gutter flow is less than

Where it is possible to increase the cross slope of the gutter (for example, on a parking lane) the efficiency of a given inlet is greatly increased. This is illustrated in the third column of the

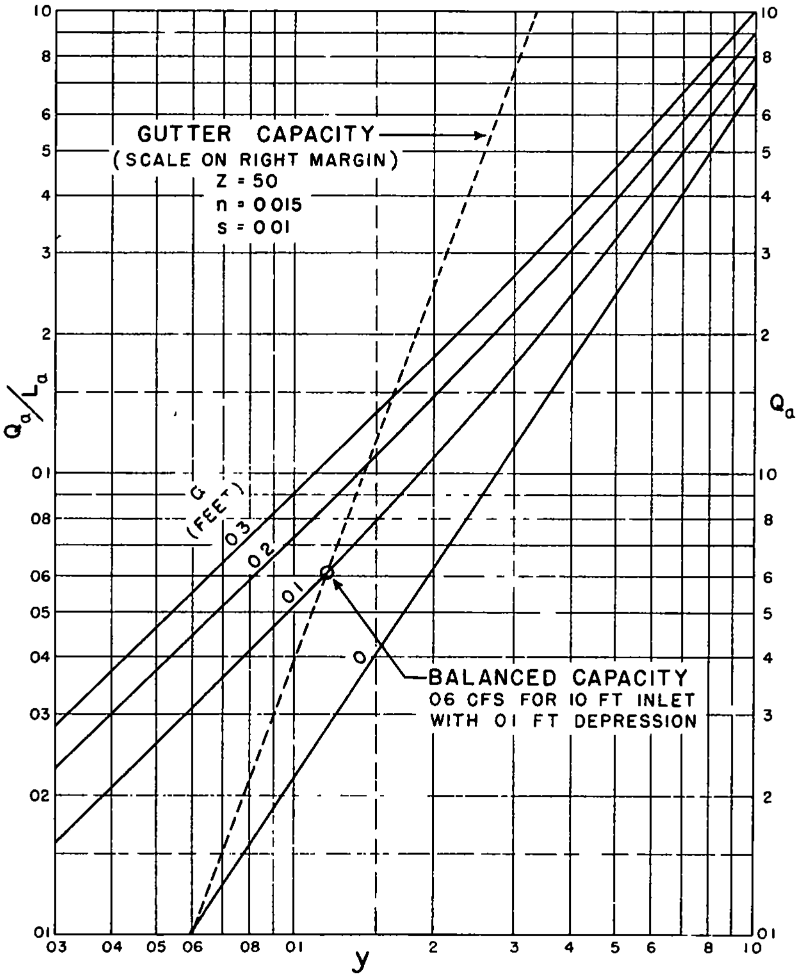

Figure 7. Average Interception per Foot of Curb Opening as Function of Depth Flow in Approaching Gutter and Depression of Flow Line when Entire Flow is Just Barely Intercepted

the balanced capacity, all the flow will be intercepted in a length less than the full length of the inlet. Thus the inlet might be considered inefficient unless it was desired to provide reserve capacity to handle greater storm intensities.

To obtain reasonable efficiency from a curb opening inlet, a depression of at least 0.1 ft is essential, as can be observed by studying Figure 7 or Table 1 based on that figure.

table which gives the balanced capacity of the same 10-ft. inlet when the cross slope is increased from 2 percent to 4 percent.

When the capacity of an inlet to intercept all the flow is exceeded, the actual interception may be determined from Figure 8 which provides a graphic solution for equation (9). depending on the ratios a/y and L/L_a The latter is the ratio of the actual length of the inlet to the theoretical length required to intercept the entire gutter flow as determined from Figure 7. The actual interception in cubic feet per second with water flowing past the inlet will always be greater than the balanced capacity because of the increased head.

From examination of Figure 8 it will be apparent that for low values of a/y an inlet may be considerably shorter than the theoretical length L_a and still intercept a high percentage of the gutter flow. When the depression is several times the depth of flow in the approach gutter, however, the partial interception ratio Q/Q_a is almost equal to the length ratio L/L_a.

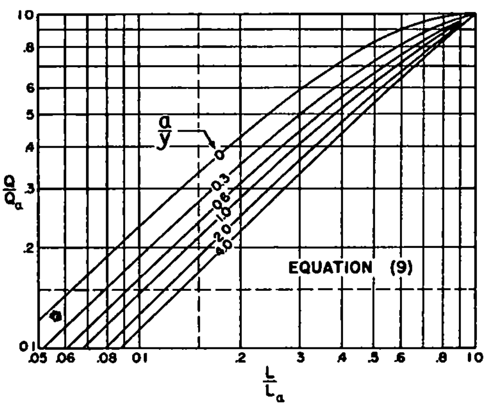

Figure 8. Partial Interception Ratio as Function of Length of Inlet Relative to Length Required to Intercept Entire Flow and Depression Relative to Depth of Flow in Approach Gutter

For convenience the symbols used in the foregoing analysis are tabulated below, followed by a summary of the characteristics of curb opening inlets.

NOMENCLATURE

Gutter

Symbol	Unit	
Q_a	cfs.	flow in gutter; also inlet capacity when entire gutter flow is just barely intercepted
y	ft.	depth of flow in approach gutter for normal cross section
z		ratio of width to depth of flow in gutter for uniform cross slope
s	ft./ft.	longitudinal slope of gutter
a	ft.	depression of inlet lip below normal gutter flow line at face of curb

Inlet

L	ft.	clear length of curb opening inlet
L_a	ft.	clear length of curb opening inlet required to intercept entire gutter flow for inlet with lip depressed a feet
L_o	ft.	same as L_a for special case when gutter flow line is not depressed
x	ft.	distance from upstream end of inlet (see Fig. 1)
h	ft.	head on inlet lip at distance x from upstream end of inlet
v	ft. per sec.	velocity of flow into inlet through elemental strip under head h
q	cfs.	discharge through elemental strip into inlet
Q	cfs.	discharge intercepted by an inlet of length L
Q_o	cfs.	discharge just barely intercepted by an inlet of length L_o having no depression
g	ft. per sec.^{2}	acceleration of gravity = 32.2

TABLE 1

Depression a ft	Balanced Capacity of 10-ft Inlet on 1% Grade Cross Slope 2% cfs	Cross Slope 4% cfs	Increase Due to Change in Cross Slope %
0	0 1	0 3	200
0 1	0 6	1 1	80
0 2	1 0	1 8	80
0.3	1 5	2 4	60

SUMMARY OF HYDRAULIC CHARACTERISTICS OF CURB OPENING INLETS

The flow intercepted by a curb opening inlet on a continuous grade depends primarily on: (1) the length of opening; (2) the depression of the gutter flow line; (3) the cross section of the approach gutter; and (4) the depth of flow in the approach gutter, y. Other factors may have some effect but are not covered in this paper.

1. To estimate the flow which a given inlet will intercept, the depth of flow in the approach gutter must first be known. This may be determined from Figure 9 or otherwise computed. For a standardized cross sections stage-discharge curves for various grades are convenient, or a single curve of depth as a function of $Q/s^{1/2}$ may be used.

2. The gutter flow will be completely intercepted if that flow divided by the length of the inlet does not exceed the value of Q_a/L_a read from Figure 7 for the known depth of flow in the approach gutter and the known depression of the gutter flow line at the inlet.

3. If the actual gutter flow divided by the length of inlet is less than the amount read from Figure 7 then the entire gutter flow will be intercepted in a length less than the full length of the inlet.

4. If the actual gutter flow divided by the length of inlet is greater than the amount read from Figure 7, only part of the flow will be intercepted.

5. The length of inlet necessary to completely intercept a given gutter flow may be determined from Figure 7 by dividing the known gutter flow by the value of Q_a/L_a for the known values of y and a. This length is designated L_a and the flow so intercepted is called the balanced capacity.

6. When the balanced capacity is exceeded, the ratio of the flow intercepted to the total flow in the approach gutter may be read from Figure 8 using the two ratios L/L_a and a/y. L is the actual length of the inlet and L_a the theoretical length found under 5.

7. The balanced capacity of a given inlet decreases as the grade of the gutter increases in a manner similar to that indicated by any one of the curves for inlet capacity in Figure 10.

8. The balanced capacity of a given inlet on a given grade increases with the cross-slope of the gutter.

9. A curb opening inlet with no depression of the gutter flow line is inefficient; the flow intercepted may be markedly increased without changing the length of opening if the gutter flow line is depressed by an amount exceeding the anticipated depth of flow in the approach gutter.

10. After the balanced capacity is exceeded the actual flow intercepted will increase and the rate of flow past the inlet will also increase, these rates being determinable from Figure 8 by the method described under 6 above.

ESTIMATING CAPACITY OF A CURB OPENING INLET

(This portion of the paper is intended to indicate to the design engineer how the research data may be applied to practical problems.)

Gutter Capacity - From the foregoing analysis it will be evident that the capacity of a curb opening inlet cannot be estimated without knowing the depth of flow in the approach gutter. Accordingly, the first step is to compute the stage-discharge relationship for the given gutter section. A simplified equation based on the Manning formula has been presented in a discussion of

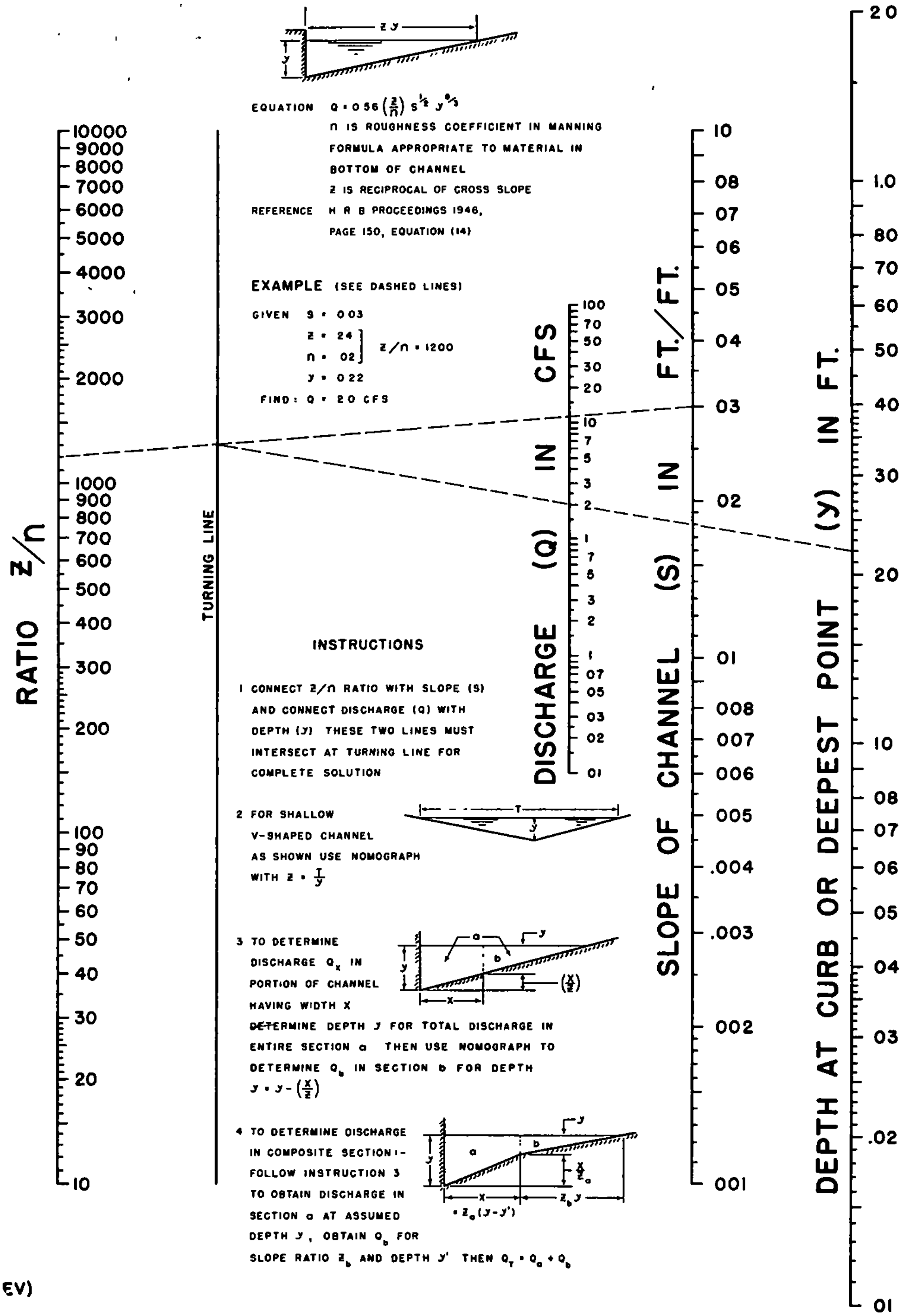

Figure 9. Nomograph for Flow in Triangular Gutters

Larson's paper (4) and is shown in the nomograph, Figure 9. The validity of this equation has been verified, with certain limitation, in a master's thesis by Larson (6). For curved cross sections the gutter capacity for each depth over the required range can be computed by dividing the flow prism into subsections about 1 ft. wide and computing the velocity in each by the Manning formula, using the average depth as the hydraulic radius. For cross sections of gradually varying depth it is not permissable to compute the hydraulic radius for the entire cross section.

as the balanced capacity. It can be found by simultaneous solution of equation (7) and the equation for gutter capacity. This is most easily done by finding the intersection of the two curves on logarithmic graph paper. The best technique to use will depend on whether the solution is desired for many values of a and L_a or for a specific inlet. For the latter case the inlet capacity curve and the gutter capacity curve can be plotted as for the example in Figure 7.

For the general case it is convenient to plot the inlet capacity curves with the ordinate Q_a/L_a and the gutter capacity

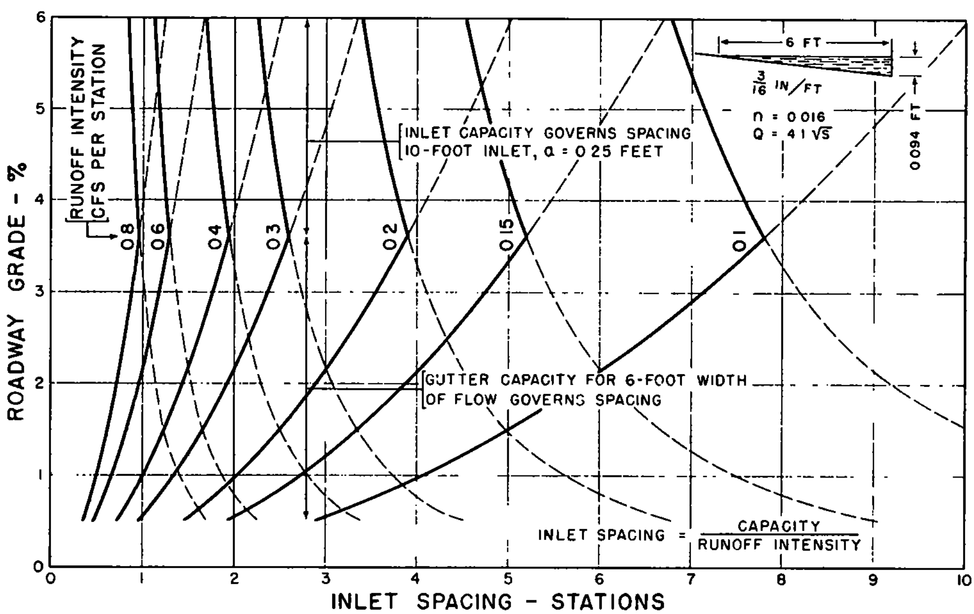

Figure 10. Typical Working Chart for Determining Inlet Spacing on Various Grades, Applicable only to 10-ft. Curb Opening with 3-in. Depression and Gutter with 3/16-in. per ft. Cross-Slope

The stage-discharge, or gutter-capacity, curve need be computed for only one longitudinal grade since the discharge for a given depth at any other grade will be in proportion to the square roots of the respective grades. The curve may be plotted as a conveyance curve with $Q/s^{1/2}$ as a function of depth.

Complete Interception - The capacity of a curb opening inlet just barely intercepting the entire gutter flow is defined

as a conveyance curve with ordinate $Q/s^{1/2}$ on the same sheet of logarithmic cross-section paper. The position of the latter curve is immaterial as it will be shifted anyway. Take the given value of roadway grade s and for any assumed depth y_1 find the corresponding discharge Q_1 in the gutter, using the conveyance curve. Divide this by L_a (which is equal to the actual length of inlet at balanced capacity) and plot the point (Q_1/L_a, y_1) on the inlet capacity graph. Shift the conveyance curve to pass through this point.

If the cross slope of the gutter is a straight line, the conveyance curve will be a straight line (as in Fig. 7) and the shifted position is simply a line through the plotted point parallel to the original conveyance curve. Otherwise the curve can be shifted easily by tracing it on an overlay. A whole series of conveyance curves can be drawn on the overlay to cover the expected range in superelevation of the roadway.

Once the conveyance curve is correctly positioned, Q_a/L_a may be read at the intersection with a given inlet capacity curve and multiplied by L_a to determine the balanced capacity. This result should be regarded as an approximate estimate which may differ from the true value by as much as 20 percent plus or minus. Considering the uncertainties in estimation of peak rates of runoff, this possible error is not serious.

If the balanced capacity is exceeded, the head builds up and actual interception will be greater than the balanced capacity. It can be estimated by the method given under "Partial Interception."

When the depth of flow and discharge in a given gutter are known, the length of inlet required to completely intercept this flow may be estimated from Figure 7 (or a similar plot) by dividing Q_a by the value of Q_a/L_a for this depth and any desired depression. This method may also be used to estimate the length of opening in a curb needed to discharge the entire flow through an open spillway across a roadway shoulder. In that case the curb line should turn out gradually, utilizing the full length of the opening to complete a 90-degree change in direction. It is also applicable to estimating the length of opening in a curb necessary to divert flow to a grate placed in a sump offset back of the curb line, the depression of the gutter flow line in that case usually being zero, unless the pavement surface is warped.

Partial Interception - The dimensionless graph Figure 8 may be used to estimate the partial interception ratio Q/Q_a. The first step is to find by the method described above the theoretical length of inlet L_a which would intercept the entire gutter flow Q_a. The ratio of the actual length of inlet L to the length L_a, and the ratio a/y, locate a unique point in Figure 8 from which Q/Q_a is read. The actual interception is then obtained by multiplying Q_a by this ratio (see Table 3 for examples). The accuracy of this estimate is subject to the same comments as given above for complete interception.

The length of inlet required to obtain a given partial interception ratio when a and y are known may be estimated by finding the ratio L/L_a from Figure 8 and the length L_a for complete interception from Figure 7. Then the length required to intercept the given portion of the total flow is the product $L_a(L/L_a)$.

Inlet Spacing - The techniques described above may be used to analyze probable operating characteristics of existing curb opening installations or to design new installations. A complete discussion of design procedures and criteria is beyond the scope of this paper, but a few suggestions may be useful in illustrating applications as they have been developed.

On express highways where the contributing drainage area is clearly defined the rate of runoff may be estimated by the usual "rational" formula $Q = CiA$. The runoff coefficient may be taken as $C = 0.9$ so long as surfaces are impermeable, or, if in grass, are steeply sloping. If relatively flat areas of turf are involved the coefficient for such areas may be reduced to perhaps 0.5 more or less, depending on the drainage characteristics of the soil. Then a weighted coefficient would be used to compute the runoff for the entire area contributing to a given inlet. Usually the time of concentration for an inlet may be taken as 5 min. if the maximum instantaneous peak runoff is desired, or may be arbitrarily increased to some duration such as 20 min. during which the width of water on the pavement would cause marked interference with traffic flow.

Once the peak rate of runoff for a given inlet has been estimated the size of the inlet could be adjusted to intercept that flow but as a practical matter the designer will wish to standardize on a limited number of inlet lengths. Consequently, the procedure should be to limit the size of the drainage area so that the peak flow will not exceed the balanced capacity of the inlet which will vary with the cross-section and the roadway grade. The simplest way to do this is to express the peak runoff for the typical section of the roadway under consideration in terms of cubic feet per second per station. From the "rational" formula

$$q = \frac{Ci \overset{\text{width}}{(\text{of d. a.})} \overset{\text{length}}{(\text{of d. a. in station})}}{432}$$

Then from a graph of balanced capacity for a given inlet on various roadway grades, the spacing may be computed as

$$L_g = \frac{Q_a}{q} = \frac{\text{balanced capacity in cfs.}}{\text{runoff in cfs. per station}}$$

the answer being in stations. To speed up computations where gutter section and inlet design are fixed, a graph such as Figure 10 may be plotted which gives inlet spacing directly for various roadway grades and various runoff intensities.

Inlets may be spaced for full interception at runoff intensities of frequent occurrence, thus operating at partial interception for less frequent storms. The designer may quickly prepare alternate designs for various frequencies to arrive at a decision as to the premium measured in increased cost which is justified by the higher degree of protection against flooded roadways.

Inlet Spacing Controlled by Gutter Capacity - On high-speed expressways carrying large volumes of traffic it is advisable to limit the extent to which the water flowing in the gutter encroaches upon the travel lane, especially for the frequent storms (reference 4, page 27). In that case it will be found where the gutter is immediately adjacent to the travel lane, rather than on the outside edge of the shoulder, that inlet spacing will be governed by the width of flow on the roadway except on the steep grades. The solution of this problem is easy as the capacity of the gutter for a fixed width of flow from the equation in Figure 9 becomes

$$Q = 0.56 \left(\frac{z}{n}\right) y^{8/3} s^{1/2} = \text{constant } (s^{1/2})$$

the constant being 4.1 for the example in Figure 10. For other types of sections the conveyance curve for the fixed depth determines the constant. Then the spacing of inlets in stations becomes

$$L_g = \frac{\text{limiting gutter flow}}{\text{runoff in cfs. per station}}$$

The curves in Figure 10 which converge on the origin give inlet spacing for various runoff intensities when the limiting width is 6 ft. as indicated by the inset diagram. It will be noted that up to a 3.6 percent grade the gutter capacity controls inlet capacity with allowable spacing increasing to a maximum at that grade. For steeper grades the balanced capacity of the given inlet controls, this capacity decreasing slowly as the grade becomes steeper than 3.6 percent.

Charts such as Figure 10 can be drawn up quickly as the computations are simple, and will save time for the designer when the same gutter and inlet standards are used repeatedly. Attention is called to the fact that this chart may be used for drainage areas which vary in width, provided only that an average width for each area is used in estimating the runoff intensity in cubic feet per second, per station.

On superelevated curves the balanced capacity of the inlets, and the gutter capacity for a given width, will increase, thus increasing allowable spacing over that on tangent sections. If enough superelevated sections are involved, a chart may be prepared for each rate of superelevation. Charts showing balanced capacity as a function of roadway grade for each rate of superelevation may suffice.

Other Considerations for Inlet Spacing - Care should be taken to locate inlets of adequate capacity at points where the crown begins to reverse so as to minimize the quantity of water flowing across the pavement from one side to the other.

Since it is important to avoid excessive ponding at low points in the grade line, any inlet near the beginning of the vertical curve ought to be designed to intercept all of the flow reaching that point, at least for frequent storms. This inlet would then also take off the coarser sediments being transported which otherwise would begin to deposit in the gutter as the velocity diminished with the decreasing grade. (Under some circumstances the engineer might prefer to clean up the sediment in the gutter rather than to permit it to accumulate underground.) The criterion for limiting depth will roughly indicate the need for additional inlets on the vertical curve, but it should be pointed out that the conveyance curve and Figure 9 are not strictly applicable when the slope is changing, and probably should not be used for grades less than about 0. 5 percent approaching the low point on the vertical curve. Experimental data are needed to define flow criteria in such cases. Except for the drawdown of the water surface as flow approaches an inlet, depths on flat grades are likely to be greater than that computed for the roughness coefficient applicable on steeper grades.

Form of Computations - A form of tabulation which has been found convenient in analyzing the interception by inlets on a given project is shown in Table 2. Constants used in this table were z = 64, n = 0. 016, L = 6 ft., and a = 0. 25 ft.

TABLE 2

COMPUTATIONS OF FLOW INTERCEPTION BY EXPRESSWAY INLETS

Station	Grade s ft per ft	Discharge Q_a cfs	$\frac{Q}{s^{1/2}}$	Depth y ft	Width zy ft	$\frac{Q_a}{L_a}$	$L_a = \frac{Q_a}{Q_2/L_a}$	$\frac{L}{L_a}$	$\frac{a}{y}$	$\frac{Q}{Q_a}$	Interception $Q = \frac{Q}{Q_a} \times Q_a$	$Q_c = (Q_a - Q)$
800+22	3 7	4	21	18	12	14	29	21	1 4	28	1 1	2 9
805+50	5 0	4	18	17	11	13	31	19	1 5	25	1 0	
	with Q_c	6 9	31	20	13	16	43	14	1 2	19	1 3	5 6
809+50	1 5	3	24	18	12	15	17	35	1 4	44	1 3	
	with Q_c	8 6	70	27	17	22	39	15	0 9	22	1 9	6 7
813+20	0 4	3	47	23	15	19	13	46	1 1	58	1 7	
	with Q_c	9 7	154	36[a]	23	30	32	19	0 7	29	2 8	6 9

[a]Over curb

The computations were made from two curves plotted on logarithmic graph paper, (1) average discharge per ft. of inlet for a = 0. 25 as in Figure 7, and (2) a conveyance curve for the gutter. The latter is faster to use than the nomograph Figure 9 when many computations are needed. The computations may be checked using Figures 7, 8, and 9.

There are two lines of computations for the last three inlets. The first line gives the flow interception for the runoff from the individual drainage area and the second line the flow intercepted when the "carryover" Q_c from the inlets above is added.

Comparing Grate and Curb Opening Inlets - An inlet grate with efficient openings will intercept substantially all the water flowing within the width of the grate. The water flowing on the roadway outside this width can be assumed to flow past the inlet, although if the grate has an appreciable length a small quantity may flow in along the edge. A grate may be assumed to have efficient openings if the bars are parallel to the direction of flow in the gutter and the unobstructed openings are long enough so that the water has a chance to fall through the openings before hitting the downstream end. The required length can be estimated by the formula

$$X = \frac{V}{4}(y)^{1/2}$$

where V is the mean velocity in the approach gutter for the width of the grate and the depth y. Other arrangements of bars can be reasonably efficient as has been demonstrated by Larson (5), but experimental tests are necessary to determine the characteristics of such inlets. On the other hand, the flow intercepted by a grate with parallel bars can be estimated directly, following the procedure prescribed under instruction 3 of Figure 9.

The partial interception ratio for a grate inlet may be approximated by using the curve for a = 0 in Figure 8, substituting for L/L_a the ratio of width of grate to total width of flow (zy) in the approach gutter. (This is due to the fact that the function for flow in a fixed width relative to total width of flow gives numerical values almost identical to the function expressed by equation (9).) Thus, if the grating is half as wide as the water surface, it will intercept about 0.83 percent of the gutter flow.

Table 3 gives the flow intercepted by an inlet grate 2 ft. wide in comparison with the capacity of curb opening inlets 4 ft., 7 ft., and 10 ft. long having a 2-in. depression of the gutter flow line. The approach gutter is assumed to be on a 1 percent grade with a cross slope of 1/4 in. per ft. and a roughness coefficient n = 0.015.

The second column of the table gives the width of flow in the gutter at the discharge indicated in the first column.

TABLE 3

COMPARISON OF COMPUTED FLOW INTERCEPTION BY GRATE AND CURB OPENING INLETS

Roadway grade 1 percent, cross-slope ¼ in per ft (z = 48), n = 0 015

Grate 2 ft by 2 ft with bars parallel to curb

Curb openings of indicated lengths, depression of gutter flow line a = 2 in

Flow in Gutter: Discharge Q_a	Flow in Gutter: Width zy	L_a Length of Curb Opening Required for 100 Percent Interception [a]	Partial Interception Ratio Q/Q_a: Curb Opening Inlet[b] L = 4	L = 7	L = 10	Grate Inlet[c]
ft	ft	ft				
0 038	2.0	1 3	1 00	1 00	1.00	1.00
0 50	5 3	6.9	67	1 00	1.00	72
1 0	6.9	10 5	49	77	97	60
1.5	8.0	13 5	41	.64	83	54
2 0	8 9	16 0	.36	58	.74	50

[a]Computed from Figure 7

[b]Computed from Figure 8

[c]Computed from Figure 9

In order to intercept the entire flow, the inlet grate would have to be as wide as the flow in the gutter. The comparable length of curb opening necessary to intercept 100 percent of the flow is given in the third column. Note that the 2-ft. grate will completely intercept a discharge of only 0. 038 cu. ft. per sec. A 7-ft. curb opening inlet, on the other hand, will completely intercept approximately 0. 5 cu. ft. per sec. and will intercept more water than the 2-ft. grate at all of the indicated rates of flow in the gutter.

Larson (4) found that the use of curb openings in combination with efficient grate inlets provides practically no increase in capacity, unless inlets are affected by backwater. However, a curb opening placed upstream from a grate inlet has been found useful (7) in taking off the trash brought down at the beginning of runoff which otherwise would lodge on the grate. A curb opening back of a grate located at the low point in the roadway grade will provide an outlet if the grate becomes clogged.

Attention is called to the fact that the relative capacity of grate and curb opening inlets of the sizes used in Table 3 will change as the grade changes. In general, as the grade becomes steeper, grate inlets, if of sufficient length, will increase in interception capacity while curb openings decrease in capacity. Conversely, on grades less than that used in Table 3 the grate becomes less efficient and the curb openings more efficient.

The principal objective of this paper has been to provide the designer with some tools he may use in making more intelligent designs for storm drain inlet systems. He will bear in mind, however, that the construction and maintenance problems and costs may overshadow purely hydraulic considerations. Simplicity in structural design is important if low unit costs are to be obtained. For this reason, and because curb openings are less susceptible to clogging by debris than are grate inlets, some cities are using curb opening inlets almost exclusively. Another point to remember is that the efficiency of a given inlet on an old street with a steep crown may be greatly reduced if the same inlet is used on a flat crown typical of express highway sections. This paper provides a means of roughly evaluating the relative capacities under such conditions.

ACKNOWLEDGEMENTS

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DISCUSSION

STIFEL W. JENS[1] - Preliminary studies and designs of drainage facilities for the depressed portion of an expressway through an old part of St. Louis have had the great advantage of the recent research on inlet design as reported by Conner (1)[2] (3), the Corps of Engineers (2), Larson (4) (5), and by Mr. Izzard in this paper, "Tentative Results on Capacity of Curb Opening Inlets." The portion of the proposed St. Louis Expressway to be built in cut (some with turfed 3: 1 slopes, and part with vertical retaining walls) involves about 1,700 ft. of 6-lane pavement with a 4-ft. dividing land strip and 3-ft.-wide concrete gutters (see Fig. A for cross sections) immediately adjacent to the outer edges of the pavement; ramps (about 4,800 lin. ft.) will be 24 ft. wide with gutters along both edges. Turf shoulders are 10 ft. wide (including the concrete gutter). On tangents, crown slopes are 1/8 in. to the foot. Curved portions of the ramps have superelevations up to a maximum of 7 percent transverse slope. Longitudinal slopes range from 1.3 percent to 6 percent, the latter on the ramps.

The Missouri State Highway Department is designing the expressway and had selected the gutter section with the following as criteria: (a) depth of flow at the pavement edge is to be limited to a maximum of 1 in., which with the 1/8 in. per ft. cross-slope results in a maximum sheet of water 8 ft. wide from edge of running slab towards the center of the pavement; (b) no grating is to extend into the running slab; (c) except in emergency, wheels are not expected to run in the concrete gutter; (d) any depression below the gutter flow line is undesirable.

With the probable final design rainfall frequency of 20 years, criterion (a) would involve infrequent short-duration flooding of the slab. This criterion, along with design runoff, determines the spacing of inlets, assuming that an inlet grating will intercept all the flow coming to it. Criterion (b) means a "carryover" of any flow along the pavement edge, with ultimate interception at some downstream inlet where the "carryover" has re-entered the gutter proper upstream, or interception at the low-point inlet. While these studies have been for preliminary designs, it is anticipated that the gutter may be painted or otherwise colored to indicate that it is not a part of the running slab. A foot-wide corrugated strip of white concrete along the edge of the lane next to the gutter might be desirable to discourage careless use of the gutter as part of the outer traffic lane.

Criterion (d) recognizes that on all high-speed urban expressways there will be occasional crowding or other careless driving which may force a car to run its right-hand wheels in the gutter for a short distance. It is realized that normally the driver of an automobile will run with his right wheels 1-1/2 to 2 ft. from the edge of the pavement, and under such usual conditions a warping of the outer 1-1/2 ft. along the edge of the pavement to create a depression of

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[2]See list of references in Mr. Izzard's paper.

2 or 3 in. below the gutter flow line in front of a side opening curb inlet is tolerable. This writer asks if experience has suggested that such a relatively sudden drop of a speeding car's right-hand wheels does not cause any loss of control with attendant serious consequences. Analytically, it would seem to have inherent potentialities for grave troubles, if not tragedies.

Table A lists 10 of the 46 inlets required under preliminary design and gives the depth (column 3) and width (column 6) of flow in the assumed gutter for the design runoffs (column 5). With but two exceptions, the design flow has been confined to the gutter proper. Columns 8 and 10 give the curb opening lengths required for complete interception of the design flow for conditions of a continuous gutter with no depression in front of the inlet opening (column 10), and for a 2-in. deep depression (column 8). Figure A gives for this table the same information as Mr. Izzard's Figure 7 and all computations are based upon the formulas in his paper, assuming n to be 0.017. Inlet spacing ranged from 70 ft. to 330 ft. varying with width and character of area drained, and with longitudinal grade; the primary consideration in spacing was the gutter capacity. Attention is directed to the fact that side openings of 13 ft. minimum are required to give 100 percent interception without any depression in the gutter opposite the curb opening. If a 2-in. depression was permissible, a

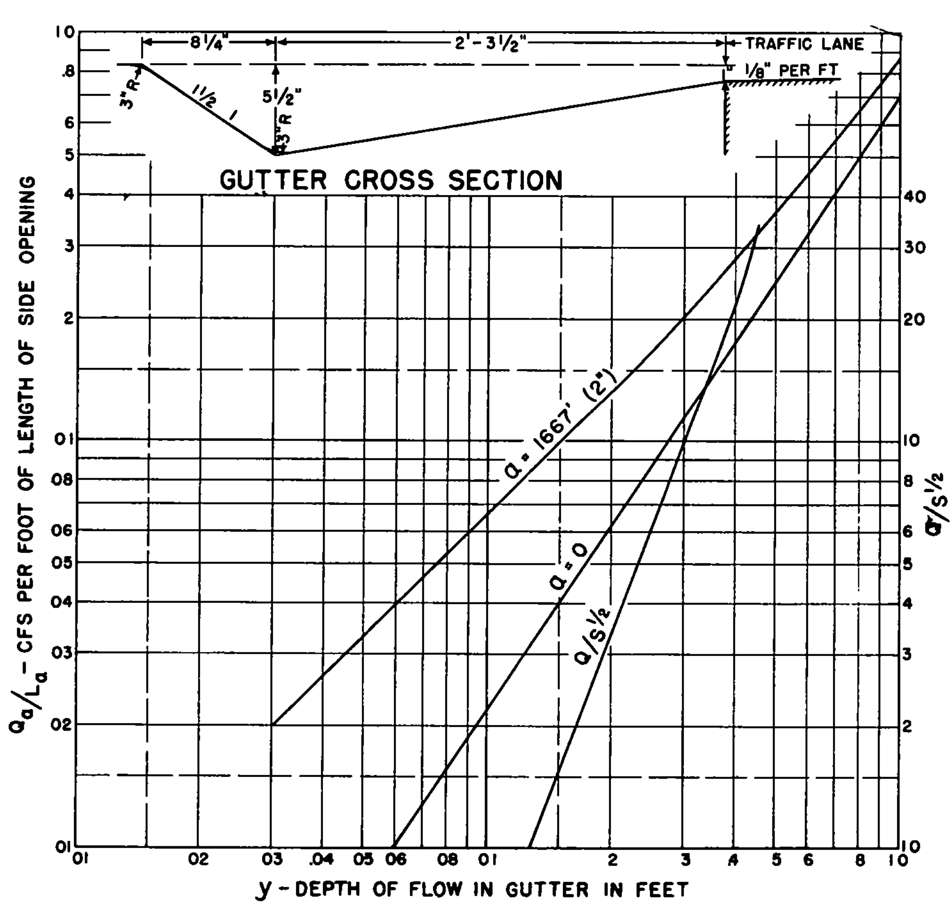

Figure A. Gutter Cross-Section and Inlet Capacity Curbs great many of the inlets could be satisfactory with only 8-ft.-long openings.

The grated inlet under preliminary designs has longitudinal slots with a clear opening of 2 ft. 3-1/2 in. by 1 ft. 9/16 in.; the concrete or brick-supporting structure has a 2-ft. inside dimension. The maximum gutter velocity of 9.5 ft. per sec. indicates a slot length of about 16.5 in. for a free-fall drop of 6 in. A minimum slot length of 2 ft. above the clear opening of the supporting structure was adopted to allow for some clogging due to debris. Preliminary designs included provision of combination grated and side opening inlets at the low points in the expressway cut, in recognition of the fact that side opening inlets have a distinct advantage with respect to debris such as leaves, paper, and other street litter. Discussions with Mr. Izzard have resulted in the decision to incorporate in final design combination curb opening grated inlets not only at the low points, but also either side of such low points, placed such that they will be near the beginning of the vertical curve ahead of where the velocity slows down and loses its debris-transporting power. It is expected that this will result in the dropping of any transported debris in the approach inlets, and the delivery of relatively clean runoff to the low point inlet.

Because the St. Louis Expressway drainage will involve combined sewers, each inlet requires a water-sealed trap on the outlet connection to the sewer. This trap would be required for either grated or side opening inlets.

TABLE A

URBAN EXPRESSWAY INLETS

LENGTHS OF CURB OPENING INLETS REQUIRED FOR COMPLETE INTERCEPTION OF GUTTER FLOW

		Flow in Gutter				Curb Opening for 100 Percent Interception			
Inlet No	Gutter Slope	y	z	Q_a	zy	Q_a/L_a	L_a	Q_a/L_a	L_a
						a = 2 in.		a = 0	
(1)	(2)	(3) ft	(4)	(5) cfs	(6) ft	(7)	(8)	(9)	(10)
1	4 1	.352	7 61	3.15	2.68	246	13 ft	148	21 ft
2	6 0	.348	7.61	3 80	2.72	242	16	.144	26
3	2 8	.360	7 61	2.75	2 74	252	11	152	18
4	2.2	.351	7 61	2 31	2 67	245	9	147	16
5	2 2	.440	10 83	4.40	4.77	317	14	206	21
6	2 2	358	7 61	2 32	2 72	250	9	151	15
7	1 7	363	7.61	2 19	2 76	.253	9	.155	14
8	2.2	355	7 61	2 36	2 70	247	10	150	16
9	FUTURE			2 00		INTER-RAMP AREA			
10	2 2	.350	7 61	2 26	2 66	243	9	.146	15

Note. Col (3) obtained by entering $Q_a/s^{1/2}$ curve of Figure 11, Cols (7) and (9) read from a = 2 and a = 0 curves for the depths y in Col (3)
Cols (8) and (10) result from dividing Col. (5) by Cols (7) and (9) respectively Lengths given to nearest full foot

To achieve complete interception of design runoffs, the inlets for the depressed portion of the Third Street Inter-regional Highway in St. Louis have been designed as grated inlets, which it is believed will render efficient drainage service at a lesser cost than side-opening curb inlets for the particular conditions of this project.

The practicing engineer concerned with highway, expressway, and street drainage owes much to Mr. Izzard and the Bureau of Public Roads for the major part they have played in stimulating and participating in the recent research in the principles and practices of overland flow and the rational design of inlets. Mr. Izzard's paper is another noteworthy contribution to the clarification of the application of recent research to actual design.

CARL F. IZZARD, *Closure* - It is gratifying to know that the principles of inlet design described in this paper have already been used in developing the design for a major project. The customary lag between the release and the application of research data appears to have been notably shortened. However, some of the experiments from which the conclusions were developed were conducted nearly five years ago, but did not receive as much recognition by design engineers as they deserved, partly because the reported results applied to specific inlets, leaving the designer in some doubt as to how other inlets of different dimensions might operate.

Mr. Jens asks if experience has suggested that a relatively sudden drop of two or three inches created by warping the edge of the pavement in front of a side opening curb inlet might result in loss of control of the vehicle. Definite observations to answer this question are lacking. Even without a depression of the edge of the pavement, the writer is inclined to question the advisability of the steep pitch of the gutter cross section (Fig. A) from the viewpoint of traffic safety. This gutter drops 4-1/2 in. below the edge of the pavement in a distance of slightly more than 2 ft. Such a gutter is excellent for carrying water but may possibly prove unduly hazardous to vehicle operation. Both of these questions can be answered only by systematic observation of vehicle operation under such conditions. This is a research problem for the traffic engineer. The problem does bring out the necessity for compromise between ideal drainage designs and requirements of traffic safety.

Since the original paper was written, new experimental data have become available from the Storm Drain Research Project being conducted at the Johns Hopkins University and jointly sponsored by the City of Baltimore, Baltimore County, and the Maryland State Roads Commission. Preliminary examination of the initial report dealing with inlets in a gutter having no depression bears out the validity of the conclusions presented in this paper and sheds more light on certain refinements in design which may prove significant. It is hoped that a report from the Johns Hopkins University project may be presented at the next annual meeting of the Highway Research Board.

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No. 10-D	Committee Report and Manual of Recommended Testing Procedures on Load Carrying Capacity of Roads as Affected by Frost Action (1950) 18 pp	.45
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*Research Reports not listed on this page are out of print and not available.

NATIONAL RESEARCH COUNCIL

The National Academy of Sciences is a private organization of eminent American Scientists, chartered under a special act of Congress in 1863 to "investigate, examine, experiment, and report on any subject of science or art." The Academy maintains the National Research Council as its operating agency.

The Council, organized with the cooperation of the scientific and technical societies of America, enjoys the voluntary services of more than 2600 scientists making up over 400 standing committees, boards, and panels in all fields of the natural sciences; its membership includes representatives of business and industry. The Council provides advisory and administrative services for research, and attempts to stimulate and coordinate research effort.

DIVISION OF ENGINEERING AND INDUSTRIAL RESEARCH

The National Research Council operates through eight divisions covering fundamental and applied natural sciences, as well as matters of international relations in scientific research. The Division of Engineering and Industrial Research is concerned with the stimulation and correlation of research in a wide variety of fields in engineering and the applied sciences.

HIGHWAY RESEARCH BOARD

The Highway Research Board is organized under the auspices of the Division of Engineering and Industrial Research of the National Research Council. Its purpose is to provide a national clearing house for highway research activities and information. The membership consists of 42 technical, educational, industrial, and governmental organizations of national scope. Associates of the Board are firms, corporations, and individuals who are interested in highway research and who desire to further its work.

The purposes of the Board are: "To encourage research and to provide a national clearing house and correlation service for research activities and information on highway administration and technology, by means of: (1) a forum for presentation and discussion of research papers and reports; (2) committees to suggest and plan research work and to correlate and evaluate results; (3) dissemination of useful information and (4) liaison and cooperative services."

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