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USE OF STREAM-FLOW RECORDS IN DESIGN OF BRIDGE WATERWAYS

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SYNOPSIS

The experience of the past few years has demonstrated that results obtained by use of the common formulas for sizes of bridge waterways have not been entirely satisfactory. Some structures have been underdesigned while others have been overdesigned. There is a present need for a method of computing bridge openings that is based upon a more sound evaluation of the basic factors than is contained in the formulas that have been used in the past.

The most logical basis for design of a structure that is to provide passage for the "water traffic" is the definite record of that traffic, stream-flow records. To calculate properly the size of a bridge opening requires among other things, a knowledge of the physical characteristics of the stream and its channel, the relation of the elevation of water surface to the discharge, and the magnitude and frequency of occurrence of flood discharges. From this information the factors to be computed are: the drop in water surface between a selected cross section in the approach channel and the bridge site, the drop in water surface through the bridge, the maximum height to which water may rise just upstream from bridge, the amount of backwater caused by bridge, and the velocities that will be produced. The results of these computations, reduced to a family of curves, furnish a basis for selection of the proper size of bridge opening. The final selections should be governed by the policy controlling the designing engineer in respect to frequency, hydraulic requirements, factor of safety, and economic considerations.

The physical characteristics of the stream channel can be ascertained by a relatively simple survey. The cross sectional area of the channel and its conveyance capacity, as computed from Manning's equation, should be recorded as curves, plotting each against elevations. The relation of water surface elevation to discharge is best expressed by a curve called a "rating curve." Defining such a curve involves the measurement of the discharge of the stream at a number of elevations, and is a basic relationship of a stream-gaging station. If the bridge site is at or near such a station it is a simple problem to define the rating curve at the site, but if there is no nearby gaging station an approximate rating curve can be made by transferring from another station or by a suitable analytical process.

The selection of the discharge to be used in the design should be guided by a knowledge of flood frequencies. The present practice of the Geological Survey in computing flood frequencies provides for the systematic compilation of the data and computation of plotting positions by a simple equation. Due to simplicity, the annual flood method makes it more attractive than the partial duration series method. Annual flood data are plotted on Gumbel graphs, although for general purposes the kind of graduation is of no great importance. However,

it is desirable to have uniformity and if a choice is to be made the Gumbel chart has much to offer as flood discharges plotted against recurrence intervals on this paper approximate a straight line graph.

To illustrate the method of computing the design discharge of bridge waterways, an example has been worked out for an imaginary stream. Computations were reduced to a family of curves that can be used, in connection with the flood frequency curve, to select the proper size of bridge opening. The curves may be used to study various combinations of conditions, each of which would give a design that would pass the required flood but would contain individual factors that differ. The final design must be selected by weighing all factors, including but not depending solely upon hydraulic conditions.

In the past, the size of bridge openings have been determined generally by some formula that includes a coefficient and some power of the drainage area. The coefficient varies widely for different types of topography and is difficult to select accurately, especially for large drainage areas of greatly variable topography and culture. It may be expected that unsatisfactory results will be obtained by the use of formulas containing terms whose correct values are difficult to determine. That results have been unsatisfactory is evidenced by the experience of the past few years when many structures so designed have been damaged or destroyed while others experiencing similar floods have not been endangered in the least. Either underdesign or overdesign is wasteful; both should be avoided.

It is desirable to have a method for computing bridge openings that is based upon a more sound evaluation of basic factors than has been contained in the formulas used in the past. It is especially important to have such a method now when highway departments are starting huge programs of road building, and unusually high costs make the most economical design a basic requisite.

It is the purpose of this paper to describe briefly a procedure for evaluating the basic factors involved and how to use these factors to calculate the size of bridge opening that should be provided to satisfy the natural requirements for the discharge selected for the design. The method herein outlined yields by a rational analysis the necessary cross sectional area, the elevation of the water surface at the downstream side of the bridge, the amount of backwater, and the average velocities that will occur.

To calculate properly the size of a bridge opening requires among other things, a knowledge of: (1) the physical characteristics of the stream and its channel at, above, and below the bridge site, (2) the relation of the elevation of water surface, or stage, to the discharge, and (3) the magnitude and frequency of occurrence of flood discharges. When these three elements are known, the proper size of opening can be obtained by relatively simple computations. The results of these computations give information from which the engineer responsible for the final design may select an opening that will serve the hydraulic needs to combine with other factors that must be taken into consideration in the economic design of a bridge and its substructure.

The information that should be obtained in the field may be listed as:

Location plan,
Cross section of channel at proposed site,
Cross section of approach channel,
Profile of stream from above the approach section to below the bridge site; show slope of bed, low-water surface, and flood-water surface if possible,
Character of bed and banks,
Possibility of backwater from tributary streams or from natural features or artificial structures downstream from site,
Elevation of all flood heights obtainable, identifying each by date,
Drainage area for very small areas,
Photographs.

From this information, the factors to be computed are: (1) the drop in water surface between a selected cross section in the approach channel and the bridge site, (2) the drop in water surface through the bridge, (3) the maximum height to which water may rise just upstream from bridge, (4) the amount of backwater caused by bridge, and (5) the velocities that will be produced. For preliminary design a number of selected discharges are used in the computations and items (1 to 5) computed, assuming several conditions of contraction caused by different assumed sizes of bridge openings. The results of these computations, reduced to a family of curves, furnish a basis for selection of the proper size of bridge opening.

The selection of the size of bridge opening may not properly be a function of the Geological Survey, but it can assist the design engineer and hydraulic engineer of the highway departments by presenting as great an array of pertinent information as can be assembled. The final selection should be governed by the policy controlling the designing engineer in respect to frequency, hydraulic requirements, factor of safety, and economic consideration.

Factors to be considered, besides purely hydraulic ones, are movement of debris, drift, piers, type of bridge, velocities, and permanence of channel and control.

Debris usually will have a minor effect on large bridges but should be considered in the design of small structures. The large amount of debris carried by some streams may reduce the opening under a bridge to such an extent that it will be entirely inadequate to pass the discharge. On these streams allowance must be made for passage of debris. The laws governing the movement of debris are not well understood and the pertinent literature is meager.

Nearly all floods carry drift or floating material of some sort. Large trees may require several feet of height to safely pass under a bridge, and it may be desirable to survey the drainage basin adequately to estimate the size of drift that may be carried by the stream. Large floods at times float houses off their foundations, particularly outhouses, camp cabins and boat houses, and these also should be considered.

The size of opening required does not include area of piers, but the type and shape of piers provided will have some bearing on the necessary size of opening. Piers located in the center of a stream may catch sufficient drift to seriously contract the channel opening. Pile trestles offer considerable resistance and usually collect even small sized drift, further retarding the flow, and should be given a greater factor of safety than other types of piers.

A concrete or steel arch bridge that is designed to allow water to come well up on, or submerge, the arch will offer more resistance to the flow than a truss that is above the flood. The waterway area to be provided is exclusive of the submerged members, and the design of the superstructure should be considered when making computations.

Average velocities should be computed for each subsection of the channel when final computations are made. Whether or not velocities are excessive will depend upon the kind of material that composes the channel and on the distribution of velocities in the section.

High velocities in the contracted channel under the bridge are quickly reduced in the wide channel below, causing damage on the downstream side of approach fills by the release of energy. Every effort should be made to keep velocities as low as practicable in the vicinity of abutments; in this respect it would aid if the depth of the channel adjacent to the fill were reduced and the roughness was increased by planting shrubs and trees along the sides of fills.

A critical factor in the design of a bridge opening is the stage-discharge relation at the bridge site for flood stages. The discharge capacity of any structure may be decreased by later alterations downstream that will further contract the channel and require a higher water surface elevation to pass the given discharge. The building of levees, a dam, another roadway, or the growth of brush and trees, may so alter the stage-discharge relation that the bridge might be seriously endangered. The channel below a bridge should not be allowed to become choked with an excessive growth of vegetation, and the design of any future structure below the bridge should give consideration to the effect it will have on the bridge.

PHYSICAL CHARACTERISTICS OF STREAM CHANNEL

The physical characteristics of the stream channel can be ascertained by a survey of the valley at and near the bridge site. A determination of the cross section at the site, the cross section of the approach channel, and the slope of the stream, involves a small instrumental survey. Values of channel roughness, as used in Manning's formula, should be assigned to each part of the channel. To assign these values requires inspection of the site and intimate knowledge of roughness factors, that can best be acquired after much experience. However, the values of the applicable coefficient depend upon local conditions that can be easily observed and evaluated within such limits that several competent engineers should choose figures within an acceptable range.

Cross sections determined from field surveys are usually easy to obtain, but streams subject to considerable scour and fill, those that transport large amounts of debris, and those that have no fixed channel during floods, may be difficult to survey so that the cross section at the time of flood peaks can be accurately determined. Often the proper section can be defined by prodding the channel bed until a firm bottom is located at the lower limit of scour. Channels in debris, such as are found in much of the southwest, that are dry for much of the year and that shift substantially during flood periods require very careful study to select the proper section.

The conveyance capacity of a channel is defined as: $K = \frac{1.486}{n} \times R^{\frac{2}{3}} \times A$, and may be recognized as the Manning discharge formula without the slope factor. In computing conveyance, the cross section must be divided into subsections if the depth or the roughness coefficient varies across the channel; the conveyance for the channel is the sum of the conveyances for each subsection.

Curves showing cross sectional area and conveyance should be drawn for use in making later computations. These factors should be computed for several elevations from medium to high stages and smooth curves drawn through the plotted points.

The cross section of the approach channel should be located preferably near the upstream end of the probable drawdown curve. If conditions are such that flood flow in different parts of the channel will not be parallel the section should be run perpendicular to flow.

RELATION OF STAGE TO DISCHARGE

The relation of the elevation of the water surface to the discharge is best expressed by a curve called a "rating curve". Defining such a curve involves the measurement of the discharge of the stream at a number of elevations, and is a basic relationship of a stream-gaging station. If the bridge site is at or near such a station it is a simple problem to define the rating curve at the site. If there is no nearby gaging station, an approximate rating curve can be made by transferring from another station or by a suitable analytical process. The rating curve is the critical element and its definition should be the responsibility of engineers having long familiarity with and an understanding of the basic hydraulic principles involved.

At each section of a stream there will probably be different flow conditions for the same discharge, the difference depending upon channel characteristics, but usually at any one section the flood discharge will be approximately the same for stages of equal elevation. There must be some physical feature that determines what the stage shall be for a certain discharge, but this feature may, and usually does, change for different stages. The factors that regulate and stabilize the flow at some section, as at a gaging station, are called the "control".

The significance of controls must be understood if a clear picture of the problem of design of waterway openings is to be obtained. Consider that at a bridge site the control is a dam located a short distance downstream, then for the maximum peak discharge there would be a certain water surface elevation that would govern the height the bridge would have to be to pass the flood, and if the bridge was this high or higher it would safely do so. Suppose, however, that after the bridge has been built the dam is built higher; this would cause a higher water surface elevation than before for the same discharge, and the bridge might be seriously damaged or destroyed. Thus after a bridge is built it is very important to see that, if the control conditions on which the design was based change so as to adversely affect the structure, some change will be necessary in the bridge opening. The possibility of a dam, railroad, or other structure being built so as to constrict the channel below a proposed bridge should be investigated as part of the preliminary study made by the highway engineer.

The possibility for backwater should be carefully investigated, and if likely to be present it should be adequately provided for. Many roads parallel large streams and cross tributaries near their mouth where the backwater effect is considerable. A bridge opening may be entirely adequate to pass a given flood if the water has a free getaway, but if the flood occurs when the area under the bridge is mostly filled with water backed up by another stream, or by any obstruction, the capacity of the channel will be so greatly reduced that the flood cannot pass and serious injury to the bridge may result.

A rating curve for a site on a tributary just above the main stream should reflect the stage-discharge relation of the main stream just below the tributary rather than of the tributary alone. A rating curve for a site subject to backwater from a dam should be based on the flow with gates closed and the reservoir full at time of a flood. The necessary area under the bridge may be many times that required for free getaway conditions, but the larger opening must be provided to carry the flood discharges under those conditions.

MAGNITUDE AND FREQUENCY OF FLOODS

The Special Committee on Flood Protection Data of the American Society of Civil Engineers has stated that "It is important to emphasize strongly that the soundest basis for the study of floods as a guide for protective works and measures is the available authentic information regarding floods that have occurred. This information may pertain either directly to the stream under investigation or to other streams having comparable physical characteristics. Though flood formulas have a definite and valuable place in the recording of experience and in the analysis and interpretation of flood flows, they have such limitations of use that the individual investigator is safest if he bases his analysis to the fullest degree possible on original flood data and related hydrologic information."

Momentary flood peaks should be used in analysing flood events for the purpose of bridge design. Floods caused primarily by ice jams, dam failures, high tides, or high winds should be excluded from the tabulations as such events are not indicative of high natural flows; however, of course, such factors are not to be overlooked in the overall analysis of the factors affecting design.

The most useful flood information is obtained from an actual stream-flow record of considerable length and at the location of the structure to be designed. Such records are frequently not available and records for the same general region must be used instead, although a considerable loss in accuracy may result. The greatest discharge published is often the greatest one that occurred during the period that is covered by a formal record. Local information may furnish highwater marks for previous floods that are considerably higher than the published peak; the discharge corresponding to the higher elevation may be taken from an extension of the station rating curve, or may be computed by some synthetic method.

By comparing the highest discharges at several places in an area, as by plotting discharge against drainage area, low values may be adjusted to agree with the general experience. Even though records from all stations are of equal length, storm patterns may have been such that heavy flood producing rains have not been observed in certain basins while adjacent basins may have experienced such rains several times.

The selection of the discharge to be used in the design should be guided by knowledge of flood frequencies. The selection of this discharge must be based on judgment and the policy that controls the action of the designing engineer and should be determined by analysis of hydraulic factors.

The question of evaluating flood frequencies has been discussed by Professor George W. Pickels in the University of Illinois Engineering Experiment Station Bulletin 296 as follows:

"In the design of any drainage structure, whether it be a small storm drain at the one extreme or a large bridge over a river at the other, a fundamental assumption must be made as to the maximum quantity of water which the structure must discharge in a unit of time. This assumption includes not only the maximum discharge to be used in the design of the structure, but also the frequency with which such a discharge may be expected to occur.

"Engineers rarely design drainage structures capable of discharging the maximum possible flood, because such a flood is not likely to occur more often, on the average, than once in several hundred, or possible a thousand, years. It is not economically sound to design structures for such unusual occurrences, unless the question of the loss of life is involved, as would be the case in the design of a spillway for a dam, where loss of life might result if the dam should fail. Whether the structure shall be designed to pass the flood which may be expected to occur, on the average, once in 10, 20, 50, or 100 years depends on the estimated damage which would result if the capacity of the structure were exceeded. When this damage has been estimated, the amount of money which may be spent justifiably to prevent such damage can be computed. The principal difficulty arises in determining the frequency with which flood flows of assumed magnitudes will occur."

The present practice of the Geological Survey in computing flood frequencies has been incorporated in instructions recently compiled by W. B. Langbein. Forms have been designed for this work which provide for the systematic compilation of the data and computation of plotting positions. Excerpts from these instructions are given.

The flood-frequency method has encountered considerable criticism, largely, it is believed, because of abuse. The method has little place in determining maximum limits of flood design (i.e. "the maximum possible flood"). With the ordinary stream-flow record, of say 25-yr. length, errors of sampling introduce large errors in judging the magnitude of floods of greater than about 15-yr. magnitude. Properly computed and conservatively interpreted, flood-frequence analysis can be a valuable hydrologic tool. The subject has been an attractive one to many students and it accordingly has benefited by their voluminous writings. The viewpoints and theories expressed, although instructive, have not always been consistent.

There are three major aspects to the problem: (1) kind of flood data to be studied, (2) plotting positions and (3) fitting frequency functions.

The kind of data to be studied are annual floods and partial duration series (floods above a base).

An annual flood is defined as the highest 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. Occasionally, 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 time period. The floods are numbered with respect to size, beginning with the highest as number 1. 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.

An objection to the use of the partial flood series is that the floods listed may not be fully independent events, i.e., one flood sets the stage for another. A related objection is that when the listed floods are so closely consecutive, the flood peaks may actually be one, as the damage was caused by the highest; the associated peaks may only have indirect or secondary effects on the losses. The differences between the two kinds of data may be largely resolved. However, it is good practice to work up flood data both ways.

For detailed plotting procedures of annual flood peaks only complete years of stream flow records can be included but historical flood data can also be included to the extent indicated below. The peaks (excluding historical data) should be numbered in order in magnitude beginning with 1 for the highest. Compute recurrence interval in years by the formula $\frac{N+1}{M}$ where N equals the number of years in the record and M equals the order of relative magnitude as assigned. Plot on Gumbel chart paper. The greatest known flood will plot at a recurrence interval equal to one plus the number of years in the period in which it occurred.

Only complete and continuous years of record can be used, except as follows:

1. Either use longest continuous period only, or use all complete years of stream gaging. No selection may be made of portion of records to be used except to the extent of using the longest continuous period.

2. In the annual floods a record may begin say in April just a few days prior to a large flood, which is not exceeded for the remainder of the year and examination of adjacent station records indicates that there was little flood activity prior to the flood. The recorded flood may then be accepted as an annual flood.

3. Fragmentary historical flood data are

selective and may not be used, except the highest known as described below.

It may be that from historical evidence, the highest flood in the period of record is also known to be greatest for many years preceding the period of record. In that event, it should be plotted at a recurrence interval equal to one plus the period for which it is known to be the highest flood.

Another situation may exist where a great flood prior to the period of record, whose discharge is known, is subsequently exceeded by a flood within a period of record. Recurrence intervals are computed as follows:

Hypothetical example: Assume a discharge of 1,000 sec.-ft. in 1850; the record begins in 1910, but the above stands as "maximum known" until 1938 when a discharge of 2,000 sec.-ft. was recorded. Hence plotting positions (up to and including the 1945 flood) would be:

max. flood in 95-yr. period	$\frac{95+1}{1}$	96 yr.	2,000 sec.-ft.
2nd highest in 95-yr. period	$\frac{95+1}{2}$	48 yr.	1,000 sec.-ft.
2nd highest in 35-yr. period	$\frac{35+1}{2}$	18 yr.	800 sec.-ft.
3rd highest in 35-yr. period	$\frac{35+1}{3}$	12 yr.	600 sec.-ft.
etc.			

Annual flood data are plotted on Gumbel graphs. The discharges are plotted to a linear scale as ordinate. The abscissa (scale of recurrence-intervals) is specially graduated according to Gumbel's "theory of largest values."

For the general purposes 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 Gumbel chart has much to offer. Flood discharges plotted against recurrence intervals on this paper approximate a straight-line graph.

For partial duration series, list all peaks, regardless of date of occurrence, above a selected base. Ordinarily this base should be chosen so that the number of peaks is at least equal to the years of record, but not more than three or four floods per year.

A peak shall be defined as a discharge which significantly exceeds the preceding and following discharges. It should be at least 25 percent greater than the adjacent troughs, and in general each peak should be associated with separate and distinct meteorologic events. Under most conditions this would mean that peaks should be separated by a period of at least a day or more.

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 by the formula $\frac{N+1}{M}$ 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.

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, however, satisfy the demand for estimates of long-term destructive floods. The tendency is to use the frequency graph for purposes of extrapolation. This tendency is dangerous, as the linear distance from 25 to 200 years seems very short on most graphs. The error of a curve fitted by whatever method may be extremely great at its outer end. Since no known fitted curve can serve any use in a long extrapolation its main purpose would therefore seem to be merely to provide a smoothing or interpolation formula. The value of any analytically fitted function therefore seems doubtful indeed. Graphical treatment only is contemplated. The Gumbel chart is recommended for annual floods, because it is based on a(á) logical a priori theory of flood occurrences. Flood data should approximate a straight line on this paper. Semi-log paper should be used for partial duration series, with discharge on the linear scale and recurrence intervals on the logarithmic scale.

Plottings of flood data by either the annual flood method or the partial duration series will show equivalent results for the higher or less frequent floods. For the lower floods the annual flood graph will be consistently below that of the partial-duration series. There is a systematic relationship between the two, however, that can be derived from basic statistical theory.

An annual flood is the maximum of all floods in a given year whereas a flood in the partial duration series is selected as exceeding a certain base and without reference to the number of other floods in the year. However, since a large flood is apt to outrank any other flood in the year in which it occurs, the recurrence intervals of great floods are closely the same on both scales.

Table 1 was computed on the assumption of floods occurring as completely independent events. However, the effect of interdependence is such as to make the items in the second column somewhat high.

There is an important distinction in meaning as between the recurrence intervals of these floods. In the annual flood series the

TABLE 1
RECURRENCE INTERVALS IN YEARS

Annual Floods	Partial Duration Series
1.10	0.41
1.25	.62
1.50	.91
1.75	1.18
2.0	1.45
2.5	2.0
5.0	4.5
10.0	9.5
15.0	14.5
20.0	20.0
100.0	100.0

recurrence interval is 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 two become sensibly numerically the same.

The annual flood series might, for example, be used in design of a bridge which is apt to be destroyed only once in a year. In this case the flood to be considered is the highest flood in a year. Other floods, although they may exceed ranking floods in other years, will be safely passed. However, consider a highway which will be flooded but not necessarily destroyed by any flood, or if damaged, will be rapidly repaired and thus soon again exposed to risk. In this case we should employ the partial duration series.

The two methods give essentially identical results for intervals greater than about ten years. Since most designs are for intervals greater than this, it is apparent that, from a practical standpoint, either method is satisfactory, although perhaps the simplicity of the annual flood method makes it attractive.

An illustration of the annual flood method is given later in this paper.

EXAMPLE OF COMPUTATIONS

To illustrate the method of computing the design discharge of bridge waterways, an example has been worked out for an imaginary stream. The required data have accordingly been assumed. The location plan, channel cross sections, stream profiles, character of bed and banks, roughness factors, studies for possible variable backwater, and photographs are not included. Highwater elevations might have been obtained from local residents as follows:

March 1913	144.2 ft.
January 1930	119.1 ft.
May 1943	128.2 ft.
April 1944	131.3 ft.
April 1945	134.1 ft.

The assumed drainage area is 300 sq. mi.

From the field data it is necessary to compute (1) the physical characteristics of the stream channel, (2) the relation of water-surface elevation to discharge rate, and (3) flood frequencies.

(1) Physical characteristics of the stream channel may be expressed by curves showing (a) relation of elevation to cross sectional area and (b) relation of elevation to conveyance or carrying capacity of channel.

The area of the natural channel at the bridge site will probably be restricted by the bridge abutments, piers, and approach fills on the flood plain. In these computations three area curves have been used, one for the natural channel, one assuming vertical abutments 60 ft. apart, and the other assuming abutments 38 ft. apart; the roadway back of the abutments will be carried on a fill. The three area and three conveyance curves for the channel at the bridge site are shown as Figures 1 and 2; the area and conveyance curves for the approach channel are shown as Figure 3.

(2) The stage-discharge relationship may be expressed by the rating curve, plotting elevations against discharge rates. To define this curve the discharge corresponding to a number of elevations must be known.

The highwater profile shows an elevation of 111.8 at the bridge site and a fall of 0.06 ft. in 100 ft., or a slope of 0.0006. The conveyance at an elevation of 111.8 is 90,000, therefore, the discharge is $90{,}000\ X\ \sqrt{(0.0006)}$ = 2,160 cu. ft. per sec. (or sec.-ft.), as $Q = K\sqrt{S}$.

The elevation of floods in 1913, 1930, 1943, 1944 and 1945 are known. At a nearby gaging station having a drainage area of 465 sq. mi., the peak discharges of these floods

TABLE 2

Date	Elevation	Discharge
	ft.	*sec.-ft.*
March 1913	144.2	24,200
January 1930	119.1	5,040
May 1943	128.2	10,100
April 1944	131.3	12,900
April 1945	134.1	12,000
	111.8	2,160

TABLE 3
ANNUAL FLOOD DATA

Year	Month	Day	Discharge	Order	Recurrence Interval	Year	Month	Day	Discharge	Order	Recurrence Interval
			sec.-ft.	*M*	*yr.*				*sec.-ft.*	*M*	*yr.*
1916	Jan.	8	9,220	8	3.88	1931	Apr.	18	2,220	29	1.07
1917	Apr.	17	4,660	26	1.19	1932	Jan.	15	4,820	25	1.24
1918	Feb.	19	7,370	12	2.58	1933	May	27	6,030	17	1.82
1919	Mar.	3	9,940	7	4.43	1934	Mar.	31	3,240	28	1.11
1920	Apr.	12	5,600	22	1.41	1935	May	6	4,610	27	1.15
1921	Mar.	28	5,900	19	1.63	1936	Feb.	1	8,390	11	2.82
1922	Apr.	22	6,320	16	1.94	1937	Jan.	19	10,500	4	7.75
1923	May	2	5,740	20	1.55	1938	Apr.	16	8,420	10	3.10
1924	Mar.	18	5,980	18	1.72	1939	Mar.	26	10,400	5	6.20
1925	Mar.	13	5,490	23	1.35	1940	Mar.	4	6,830	15	2.07
1926	Apr.	30	8,600	9	3.44	1941	June	13	1,410	30	1.03
1927	Mar.	6	10,700	3	10.3	1942	Apr.	15	7,160	14	2.21
1928	Dec.	4	5,630	21	1.48	1943	May	8	10,100	6	5.17
1929	Feb.	21	7,320	13	2.38	1944	Apr.	21	12,900	1	31.0
1930	Jan.	9	5,040	24	1.29	1945	Apr.	10	12,000	2	15.50

Mean annual flood for period 1916–45: 7,080.

have been determined as 30,200, 6,300, 12,600, 16,100, and 15,000 sec.-ft. respectively, and these can be corrected to the bridge site by considering that the discharge will differ as the square root of the drainage areas. This factor is $\sqrt{(300/465)}$ or 0.80, and the peak discharges at the bridge site are accordingly estimated as 24,200, 5,040, 10,100 12,900 and 12,000 sec.-ft. These discharges and those computed from highwater marks are listed in Table 2 and have been used to define the rating curve shown as Figure 4.

(3) Flood frequencies have been computed on the basis of annual discharges computed at the nearby gaging station. Discharges at the gaging station have been reduced according to the drainage area factor shown above. In Table 3 are listed maximum annual discharges from 1916 to 1945; also shown are the computed plotting positions.

Discharges from Table 3 were plotted and are presented as Figure 5. The frequency curve shown has been extended to the 100-year value.

To provide the design engineer with a wide range of values from which to select the proper size of opening, computations have been made for discharges from 12,000 to 20,000 sec.-ft. Computations for each selected discharge were made of: (1) the drop in water surface from the selected approach section to the bridge, (2) the drop in water surface through the bridge, (3) the amount of backwater that would be caused by the bridge, and (4) mean velocities under the bridge. All computations are shown in Table 4.

(1) In computing the drop from the selected approach section to the bridge for each selected discharge, shown in column 1 of Table 4, the elevation of the water surface at downstream side of bridge, which is fixed by downstream control conditions, was obtained from the rating curve of Figure 4 and is shown in column 2; for each of the channel conditions listed in column 3 the corresponding cross sectional areas, taken from curves of Figure 1 are shown in column 4; the mean velocity, column 5, is the discharge divided by the cross sectional area; the velocity head, $V^2/2g$, is listed in column 6, and the entrance loss,

TABLE 4

COMPUTATIONS FOR FLOW FROM APPROACH SECTION TO BRIDGE SECTION

Dis-charge	Drop from Approach Section to Bridge — Bridge Section (Downstream Side) — Water Sur. Elev.	Area — Abut-ments at	Area — Sq. ft.	Vel.	Vel. Head $V^2/2g$	En-trance Loss	Con-vey-ance	Slope	Elev. En-ergy Grad	Approach Section (150 feet above Bridge) — Water Sur. Elev.	Area	Vel.	$V^2/2g$	Con-vey-ance	Slope	Elev. En-ergy Grad.	Frict. Loss	Drop in Water Sur.	Drop through Bridge — Frict. Loss thru Bridge [col. 9 × 30]	Drop thru Bridge [col. 7 + col. 20]	Upst. Side Br. — Water Sur. Elev.	Upst. Side Br. — Elev. Energ. Grad. [col 6 + col. 22]	Backwater — Nor-mal Drop be-tween Sect.	Back water Col. 19 − Col. 24
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
sec.-ft.	*ft.*			*ft. per sec.*	*ft.*	*ft.*	*K*	*S*	*ft.*	*ft.*	*sq. ft.*	*ft. per sec*	*ft.*	*K*	*S*	*ft.*	*ft.*	*ft.*	*ft.*	*ft.*	*ft.*	*ft.*	*ft.*	*ft.*
12,000	130.6	Natural	2,090	5.74	0.51		426,000	0.00079	131.1	130.7	2,090	5.74	0.51	427,500	0.00079	131.2	0.12	0.12	0.02	0.02	130.6	131.1	0.12	0.0
		45–105	1,640	7.32	.83	0.08	398,000	.00091	131.4	131.2	2,160	5.56	.48	444,000	.00073	131.7	.12	.55	.03	.11	130.7	131.5		.4
		56– 94	1,180	10.2	1.62	.16	323,000	.00138	132.2	132.1	2,280	5.26	.43	477,000	.00063	132.5	.14	1.49	.04.	.20	130.8	132.4		1.4
14,000	133.2	Natural	2,450	5.71	.51		515,000	.00074	133.7	133.3	2,440	5.74	.51	522,000	.00072	133.8	.11	.11	.02	.02	133.2	133.7	.11	0.0
		45–105	1,790	7.82	.95	.10	461,000	.00092	134.2	133.9	2,530	5.53	.48	546,000	.00066	134.4	.12	.69	.03	.13	133.3	134.3		.6
		56– 94	1,280	10.9	1.85	.18	346,000	.00164	135.0	135.0	2,680	5.22	.42	590,000	.00056	135.4	.14	1.75	.05	.23	133.4	135.2		1.6
16,000	135.5	Natural	2,780	5.76	.52		606,000	.00070	136.0	135.6	2,770	5 78	.52	615,000	.00068	136.1	.10	.10	.02	.02	135.5	136.0	.10	0.0
		45–105	1,930	8.29	1 07	.11	518,000	.00095	136.6	136.3	2,880	5.56	.48	644,000	.00062	136.8	.12	.82	.03	.14	135.6	136.7		.7
		56– 94	1,360	11.8	2.16	.22	375,000	.00182	137.7	137.6	3,080	5.19	.42	698,000	.00053	138.0	.15	2.11	.05	.27	135.8	138.0		2.0
18,000	137.7	Natural	3,120	5.77	.52		702,000	.00066	138.2	137.8	3,120	5.77	.52	707,500	.00065	138.3	.10	.10	.02	.02	137.7	138.2	.10	0.0
		45–105	2,060	8.74	1.19	.12	578,000	.00097	138.9	138.7	3,270	5.50	.47	746,000	.00058	139.2	.11	.95	.03	.15	137.9	139.1		.8
		56– 94	1,450	12.4	2.39	.24	402,000	.00200	140.1	140.1	3,500	5.14	.41	806,000	.00050	140.5	.15	2.37	.06	.30	138.0	140.4		2.3
20,000	139.9	Natural	3,470	5.76	.52		801,000	.00062	140.4	140.0	3,490	5.73	.51	802,000	.00062	140.5	.09	.10	.02	.02	139.9	140.4	.10	0.0
		45–105	2,190	9.13	1.30	.13	640,000	.00098	141.2	141.0	3,660	5.46	.46	846,000	.00056	141.5	.11	1.08	.03	.16	140.1	141.4		1.0
		56– 94	1,530	13.1	2.67	.27	430,000	.00220	142.6	142.6	3,940	5.08	.40	918,000	.00047	143.0	.15	2.69	.07	.34	140.2	142.9		2.6
15,000	134.3	38–112	2,030	7.39	.85	.08	453,180	.00110	135.1	134.8	2,650	5.67	.50	582,000	.00067	135.3	.13	.56	.03	.11	134.4	135.2	.10	.5

taken as 0.10 the velocity head (entrance coefficient of 0.90), is in column 7; column 8 shows the conveyance as taken from curves of Figure 2; column 9 lists the energy or friction slope, computed by dividing the discharge by the conveyance and squaring the result; column 10 shows the elevation of the energy gradient at downstream side of bridge, and is column 2 plus column 6.

For the approach sections similar data were listed as for the bridge section, as shown in

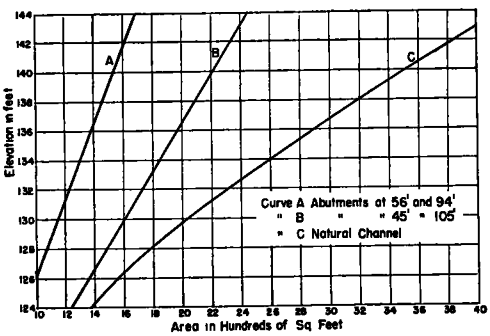

Figure 1. Area Curves for Cross Section at Bridge Site

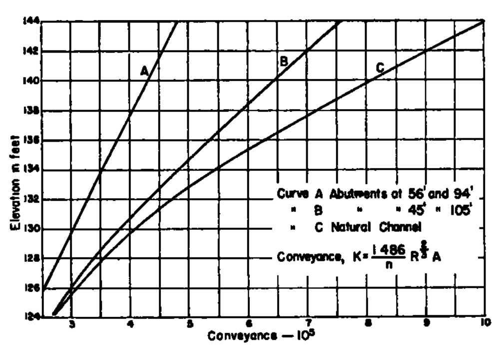

Figure 2. Conveyance Curves for Cross Section at Bridge Site

columns 11–17, the main difference being that the water surface elevation, shown in column 11, was computed by the "cut-and try" method.

The friction loss from the approach section, to the bridge section, shown in column 18 was computed as the square root of the product of the slopes at each section multiplied by the distance between sections, or $h_f = \sqrt{S_A S_B} \times L$.

The drop in water surface between sections, column 19, is equal to the head required at the bridge less the head furnished by the approach conditions or the velocity head at

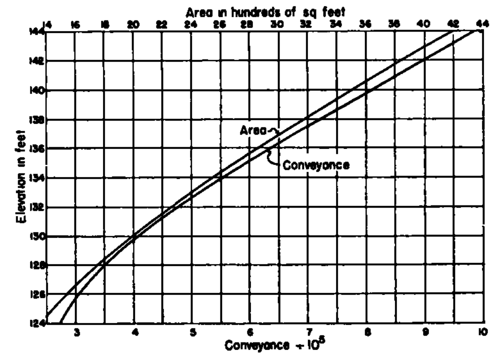

Figure 3. Area and Conveyance Curves for Cross Section of Approach Channel

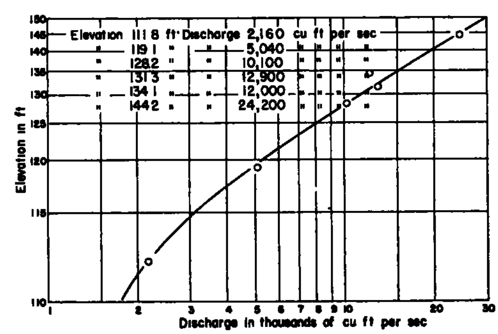

Figure 4. Rating Curve for Bridge Site

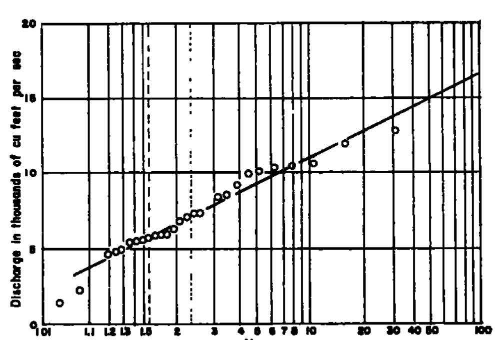

Figure 5. Recurrence Interval, in Years, for Annual Floods

bridge plus the entrance loss plus the friction loss minus the velocity head at the approach section (drop = columns 6 plus 7 plus 18 minus 14). This computed drop in water surface must be the same as the difference assumed when values in column 11 were first recorded, or the drop also equals column 11 minus column 2; the assumed and computed drops were made to agree by the "cut-and try" method.

(2) The drop through the bridge has been computed as the slope at the bridge multiplied by the width of abutments (assumed as 30 ft.) plus the entrance loss. The friction loss, drop through bridge, water surface upstream side of bridge, shown in column 23, indicates the highest elevation to which the water surface would rise if all of the velocity was converted to static head.

Values computed in Table 4 were plotted in Figure 6 and curves drawn to show the backwater effect caused by different sized openings with different discharges. These curves furnish the basis for analysis of hydraulic conditions and the selection of the size of bridge opening required.

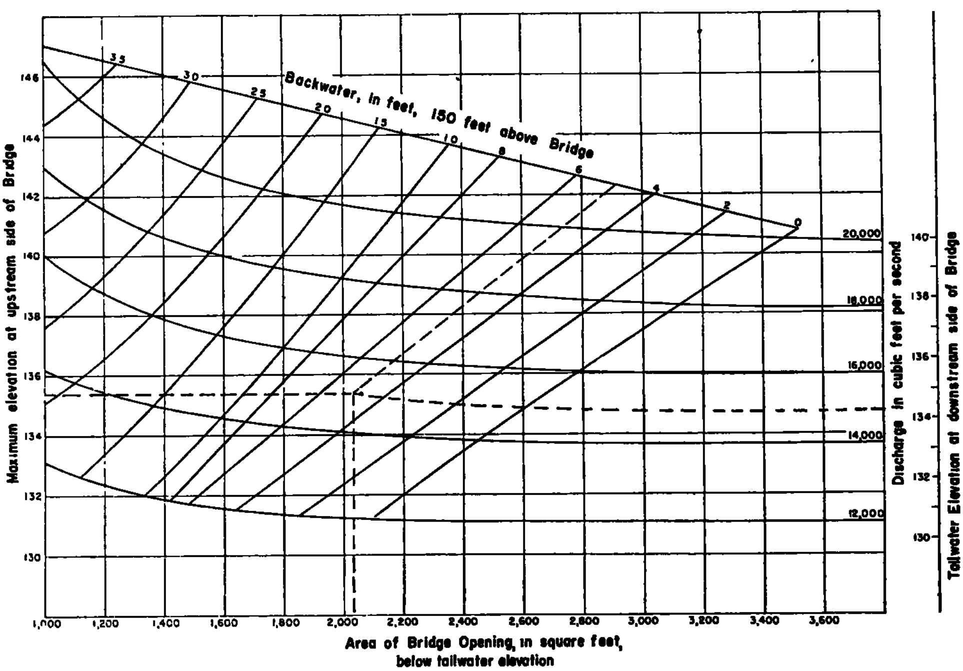

Figure 6. Design Curves

elevation at upstream side of bridge, and elevation of energy gradient are shown in columns 20 to 23.

(3) The backwater at the selected approach section caused by the bridge, shown in column 25, was computed as the drop between sections less the drop that normally occurs in the water surface in the natural channel.

(4) Mean velocities at the bridge are shown in column 5 of Table 4. The maximum velocity in a section can be roughly calculated as 1½ times the mean, but the actual velocity at a point may depend upon very local conditions and may differ greatly from this figure.

The elevation of the energy gradient at the

The flood-frequency curve of Figure 5 and the curves of Figure 6 furnish reasonably adequate data for selecting the proper size of bridge opening. As an example of use of curves, assume a bridge is to be built to pass the "50-year flood" and that backwater caused by the construction of the bridge should not exceed 0.5 ft. Figure 5 shows the "50-year flood" to be 15,000 sec.-ft. For this discharge Figure 4 shows that the water surface elevation at the downstream side of bridge will be 134.3 ft. By following the dashed line on Figure 6, representing 15,000 sec.-ft. at 134.3 ft. elevation, from the right margin to an intersection with the 0.5 ft. backwater line, the maximum elevation of water at upstream side of bridge and fill is shown on left margin of sheet as 135.3 ft. and the required cross sectional area under the bridge and below 134.3 ft. is shown on bottom of sheet as 2,030 sq. ft.

For comparison, suppose the bridge is to be so built as to pass to same flood with 2.0 ft. of backwater. Figure 6 shows that the required cross sectional area would be only 1,240 sq. ft., and the maximum height of water, and the minimum height to which approach fills would need to be built, would increase to 137.2 ft. The elevation of the bridge seat might be approximately the same in each case, as this is controlled by downstream channel conditions, the differences in effective area of cross section being obtained by changing the span length. An increase in the length of span would result in a decrease in the length and height of fill required for approaches; the problem thus becomes one of relative costs of bridge and fill.

The curves may be used to study various combinations of conditions, each of which would give a design that would pass the required flood but would contain individual factors that differ. The final design must be selected by weighting all factors, including but not depending solely upon hydraulic conditions.

After selection of the design figures, computations were made to show how this discharge would pass through the reach. Assuming that abutments will be 74 ft. apart (at 38 and 112 ft.) and that bridge will be of one span, computations show that there will be an area of 2,030 sq. ft. below elevation 134.3 ft. Computation of percentage of conveyance in each subsection of channel allows an estimate to be made of mean velocity in each subsection, as the discharge in each subsection will be proportional to its conveyance; these figures are shown in Table 5.

TABLE 5
VELOCITY DISTRIBUTION

Sub-Section	Area	$R^{2/3}$	$\frac{1.486}{n}$	Conveyance	Percent of Total	Discharge	Mean Velocity
	sq. ft.			*K*		*sec.-ft.*	*ft. per sec.*
1	96	2.310	42.46	11,100	2.5	375	3.4
2	260	5.585	24.77	35,969	8.0	1,200	4.6
3	1,318	5.500	49.53	359,042	79.0	11,850	9.1
4	260	5.585	24.77	35,969	8.0	1,200	4.6
5	96	2.310	42.46	11,100	2.5	375	3.4
Total	2,030			453,180	100.0	15,000	7.4

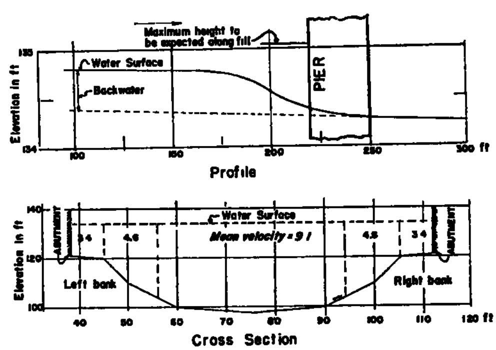

Figure 7. Water Surface Profile and Cross Section at Bridge Sites

Computations of flow from the approach section to the bridge section are shown as the last line of Table 4. Pertinent data were plotted as Figure 7 to show profile of water surface and sectional view of required bridge opening with velocities shown for different parts of the channel.

DISCUSSION

Rowe, Winslow, and Williams[1]: The following discussion has been prepared by a subcommittee[1] of the Committee on Surface Drainage of Highways and is intended to represent a summary of the views of the individual members of the main committee.

The comments received by the committee indicate that the primary purpose of this paper has been in some instances misunderstood. As explained by the author, it was his intention to present a number of hydrologic and hydraulic techniques which are worthy of consideration (along with structural and economic factors) in the overall design of bridge waterways. In many parts of the country these techniques will not be applicable because of special regional characteristics or meagerness of basic data. It is the purpose of this discussion to present possible alternative methods in solving hydrologic and hydraulic problems and not to indorse or condemn any of the techniques presented.

The practicing highway engineer may take exception to the statement that the results of the empirical practices used up to the present time have been unsatisfactory. He no doubt feels that he has done well with the limited data available but will be the first to admit that the law of compensating error was perhaps working for his benefit. If no bridges were ever damaged by flood, the designs as a whole would be uneconomical. The fact that some failed and many succeeded may simply indicate that the design techniques had a reasonable basis but that some bridges were subjected to rare floods. The objective must not be to design flood-proof bridges but to build for economic flood resistance.

In discussing frequency methods the author assumes that the reader is quite familiar with the statistical analysis of hydrologic data. For the uninitiated, a bibliography would have been helpful. The plotting formula and special paper recommended are taken from the studies of Professor E. J. Gumbel[2] and Professor Ralph W. Powell[3]. The method presented is only one of many for obtaining essentially the same result. A widely accepted plotting formula for flood frequency f is

$$f = \frac{2N}{2M - 1}$$

The symbols have the definitions given in the paper.

The detailed discussion of the difference between annual floods and all floods above a base is considered too academic for the designing engineer. For his purpose the frequency of all floods which are independent events is significant in evaluating the economic aspects of his design. Any plotting paper which reduces to a minimum the curvature of a mean line through the data is useful. Log-log paper is superior to semi log paper if the range in discharge magnitudes is very great. Semi-log paper is very satisfactory for plotting the frequency of gage-heights. An excellent bulletin discussing flood-frequency analysis and presenting a compilation of flood data for Ohio has been recently published[4].

The example showing the computation of a discharge of 2160 sec-ft from conveyance values and observed slope is misleading as regards the accuracy that may be obtained from field observations. It states that in the assumed example the fall is 0.06 ft in 100 ft. Such a result might have to be averaged over a reach of several thousand feet in which the floodmarks might vary several tenths of a foot from the mean plotted profile. There should be no implication that a reliable slope could be obtained in 100 ft except in the observed drop-down curve through an existing bridge structure. In the latter case a different analysis would be used.

In transferring the results of discharge observations from one size of drainage area to another, that is from 465 sq mi to 300 sq mi, the paper suggests using the 0.5 power of the drainage-area ratio. Engineers in different parts of the country may take exception to this relation and advocate others based on

[1] R. Robinson Rowe, California Division of Highways, Sacramento, California.
W. S. Winslow, State Highway & Public Works Commission, Raleigh, North Carolina.
Gordon R. Williams, Knappen Engineering Co., New York, N. Y.

[2] "Floods Estimated by Probability Methods," *Engineering News-Record*, Vol. 134, p. 833, June 14, 1945.

[3] "A Simple Method of Estimating Flood Frequencies," *Civil Engineering*, pp. 105–106, Feb. 1943.

[4] William P. Cross, "Floods in Ohio—Magnitude and Frequency," Bulletin No. 7, Ohio Water Resources Board, Columbus, Ohio.

their experience. For example in southern California the exponent of the drainage-area ratio may be 0.7, in the Pacific Northwest it may be only 0.23, and in still another place it may be 1.0. Considerable success has been achieved by generalizing directly from a number of discharge frequency curves for different areas to estimate equivalent frequency for an ungaged area. For example, after frequency curves for several gaged areas have been drawn, the results may be combined by plotting discharge in sec-ft per sq mi against drainage area in sq mi with frequency as a parameter.

The paper implies that considerable data on stream-flow and flood heights can be obtained in the general vicinity of many proposed structures. Actually such data are available for only a very small percentage of proposed projects. It is therefore necessary to resort to much more approximate methods until such time as more data become available.

In the detailed hydraulic analysis the author assumes that a range of discharges from 12,000 to 20,000 sec-ft will receive consideration by the bridge designer. Actually that may not be the case, as the design capacity of the bridge will be selected from a consideration of the estimated discharge-frequency relation and the importance of the proposed structure. For example, the selected discharge might correspond to a 10-yr frequency (11,000 sec-ft) for a relatively minor (or temporary) structure or a 75-yr frequency)16,000 sec-ft) for an important (or permanent) structure. Having selected the design discharge, the designer would make the bridge opening and approaches adequate to withstand the computed hydraulic conditions.

TABLE A
APPROXIMATION OF BACKWATER—COMPARISON WITH TABLE 2 (TEXT)

Q	A_2	V_2	$\frac{V_2^2}{2g}$	$\frac{1.1\,V_2^2}{2g}$	Downstream Water Surface	Trial Upstream Water Surface	A_1	V_1	$\frac{V_1^2}{2g}$	Backwater		Upstream Water Surface	
										H[1]	Col 25 Table 2	[2]	Col 11 Table 2
(45–105 Abut)													
12,000	1,640	7.3	0.83	0.91	130.6	131.0	2,120	5.7	0.50	0.4	0.4	131.0	131.2
14,000	1,790	7.8	0.94	1.04	133.2	133.7	2,500	5.6	0.49	0.55	0.6	133.75	133.9
16,000	1,930	8.3	1.07	1.17	135.5	136.1	2,650	5.6	0.49	0.68		136.2	
						136.2	2,860	5.6	0.49	0.68	0.7	136.2	136.3
18,000	2,060	8.7	1.17	1.29	137.7	138.5	3,240	5.5	0.48	0.81	0.8	138.5	138 7
20,000	2,190	9.1	1.28	1.42	139.9	140.8	3,640	5.5	0.47	0.95	1.0	140 85	141.0
(56–94 Abut)													
12,000	1,180	10.2	1.62	1.78	130.6	132.0	2,270	5.1	0.40	1 38	1.4	132.0	132.1
14,000	1,280	10 9	1.85	2.03	133.2	134.8	2,670	5.2	0.42	1.61	1.6	134.8	135.0
16,000	1,360	11.8	2.16	2 38	135.5	137.5	3,070	5.2	0.42	1.96	2.0	137.5	137.6
18,000	1,450	12.4	2.39	2.63	137.7	140.0	3,500	5.1	0.41	2.22		139.9	
						139.9	3,480	5.2	0.41	2.22	2.3	139.9	140.1
20,000	1,530	13.1	2.67	2.94	139.9	142.5	3,950	5.1	0.40	2.54		142.4	
						142.4	3,900	5.1	0.41	2.53	2.6	142.4	142.6
(37–113 Abut)													
15,000	2,060	7.2	0.81	0.89	134.3	134.7	2,650	5.7	0.50	0.39	0.4	134.7	134. 8

[1] $\left(H = 1\,1\,\frac{V_2^2}{2g}\;\frac{V_1^2}{2g}\right)$

Note that backwater agrees within 0.1 ft in all cases and that upstream water surface computed by approximate method is 0.1 to 0.2 ft lower, which could be accounted for if normal drop in water surface over 150 ft reach as shown in Col. 19 were added in.

[2] Downstream water surface plus H.

The hydraulic analysis assumes that the amount of backwater caused by the bridge is the limiting criterion in the hydraulic design. At many locations the velocity through the bridge, erosion of the bottom and banks upstream and downstream, energy conditions at the downstream end and other hydraulic characteristics may be more important than backwater alone. However, when it is desired to evaluate the backwater effect alone, a much simpler analysis can be made with sufficiently accurate results as most of the backwater at a bridge is caused by the change in velocity head between the upstream section and the constricted section through the piers and abutments plus an entrance loss which is usually expressed in terms of the velocity head in the constricted section. Friction loss through an ordinary highway bridge can be neglected as indicated by the small values in Col. 20 of Table 2 (text). The modified computation is given in Table A and comparative results with Table 2 are shown in the last four columns.

MR. DALRYMPLE, *Closure.*—A short bibliography of papers concerned with statistical analysis of hydrologic data is attached to this discussion. Only a few of the many papers on this subject are listed.

Flood discharge records in the United States are generally of such short duration that an accurate determination of recurrence intervals long enough to be of interest to the highway engineer is not possible. Whatever figure is obtained, by any method, will be subject to considerable uncertainty, therefore the most simple procedure is preferable. The simplicity of the Gumbel method, using annual flood peaks, makes it attractive for this reason alone, regardless of other meritorious features. Recurrence intervals computed from the formula $\frac{N+1}{N}$ are easier to compute and have almost the same value as Gumbel's plotting positions.

It is believed that computing discharge by use of conveyance values may be done with an acceptable accuracy. The conveyance can be accurately computed after proper field observations and measurements. The slope (of energy gradient) is more difficult to obtain, but reliable values can be computed from a discharge measurement, as $S = \left(\frac{Q}{K}\right)^2$, or from a slope curve based on a series of discharge measurements.

Highwater marks may be used to determine the surface slope, but adjustments for changes in velocity head must be made to obtain the desired energy slope. The elevation of highwater marks should be determined over a reach long enough so that an appreciable fall will be measured. Highwater elevations should be plotted and the slope line drawn only for a reach having a uniform slope. It has been found a dangerous procedure to determine the slope from only two highwater elevations, regardless of how well they are defined, as there might be a decided break in the slope between the two points. In the example, the slope was determined from eleven highwater elevations obtained in a reach of 200 ft; this slope gave a fall of 0.06 ft per 100 ft. How long the reach need be depends upon the steepness of the slope and the closeness with which the points plot to a straight line.

The drainage area coefficients assigned by various authorities have ranged from 0.3 to 0.8; the coefficient probably varies due to different drainage area characteristics. In this paper a coefficient of 0.5 was arbitrarily used, and it was not intended to suggest that this is the proper coefficient for every location. As the paper was not primarily concerned with this phase of the problem no effort was made to select an exact value.

The method suggested of basing frequency curves for ungaged areas on records for gaged areas, plotting discharge in second-feet per square mile against drainage area in square miles with frequency as a parameter, appears to be a good one. However, in practice it will often be found that the data plot so erratically that no well defined curve is indicated, and some other method must be used. The method to be used in a particular case depends upon the data available at the site under study.

If one engineer made the complete bridge design he probably would first select the design discharge then make computations only for this figure. However, if the hydraulic factors are computed by a hydraulic engineer for study by whoever is responsible for the final design, he likely would not know what frequency will be assigned to the structure and, therefore, should provide data covering a range of discharges. This would be particularily true if, for instance, the hydraulic data were furnished by the Geological Survey.

It is recognized that backwater will not always be the limiting factor in the hydraulic design but it was thought best not to complicate this paper by giving consideration to other factors. Computations were made in considerable detail to show the factors involved and their relative magnitude but shorter methods would undoubtedly be used in practice.

The main purpose of this paper was to demonstrate that discharge records can often be used advantageously in the selection of the size of a bridge waterway. After this fact is recognized the details of the method must be worked out, modified and revised from time to time on the basis of experience and increased knowledge of flood behavior.

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A METHOD OF COMPUTING LIVE LOADS TRANSMITTED TO UNDERGROUND CONDUITS

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SYNOPSIS

More than twenty years ago the Iowa Engineering Experiment Station conducted experiments which indicated rather definitely that the load transmitted to an underground conduit by a truck wheel applied at a roadway surface may be safely computed by the Boussinesq formula for a point load applied at the surface of a semi-infinite elastic solid. Up until about 1929, the usual procedure for computing live loads on such structures was to subdivide the top of the conduit into a number of small sub-areas and calculate the load on each sub-area by means of this formula. The summation of loads on all the sub-areas gave the total load on the section of conduit caused by a wheel load at the roadway surface. In 1929, Dr. D. L. Holl integrated the Boussinesq formula to obtain the pressure over a finite rectangular area in the undersoil. The result of this integration was a rather lengthy expression whose evaluation was not difficult, but was rather tedious.

In 1935 Dr. N. M. Newmark integrated the Boussinesq formula for the purpose of evaluating the pressure at a point in the undersoil due to a uniformly distributed load applied over a rectangular area at the soil surface. This problem was directed toward the determination of pressure at various points in the undersoil for the purpose of estimating the probable settlement of buildings and other structures resting on soil foundations. In connection with this solution, Newmark presented a table of ratios which greatly simplified the solution of his formula.

Although Holl's problem and that of Newmark were directed toward different