Prediction of floods

Ping Yi Lin

University of Iowa

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PREDICTION OF FLOODS

A THESIS

PRESENTED TO THE FACULTY OF THE GRADUATE COLLEGE
OF THE STATE UNIVERSITY OF IOWA IN PARTIAL
FULFILLMENT OF THE REQUIREMENTS FOR
THE DEGREE OF MASTER OF SCIENCE

BY

PING YI, LIN, B. S. IN C. E.

1925
PREFACE

Being interested in the question of flood relief, the writer has endeavored to emphasize the possibility of predicting floods with high degree of accuracy in the results, he has also considered that is the most economical method of flood protection.

In the following pages, his attempt is to study thoroughly the phenomena of flood waves in order to develop some mathematical expressions based upon physical observations and principles of hydraulics, for determining the rate of progression of flood wave in rivers.

Some examples of the velocity of flood waves in several rivers have been collected and tabulated in the latter article. He has discussed in the last chapter, the most important factors affecting floods, and also outlined procedures for estimating the probable future maximum floods.

This work has been done in a period of ten months during his residence in the State University of Iowa. Most of his time has been spent in developing the mathematical equations, and the rest has been taken in preparing the following notes.
As the writer has some difficulty in writing English, excuse must be asked for frequent mistakes in both grammar and rhetoric in his writing. However, he intends to translate and enlarge this work into Chinese in order to encourage the leading conservancy districts in China to adopt the flood prediction system as an economical protection from floods for the places where no protection is existed and also as a necessary factor of extra safety for those well protected districts, when he returns to his own country.

The writer wishes to express his hearty thanks to the leading Professors of the State University of Iowa for their valuable instructions and suggestions regarding this work and also other studies. Grateful acknowledgements are due to the Hydro-electric Plant of the State University of Iowa, and Iowa City Light and Power Company for the flood records; and especially to Mr. A. F. Koch for the kindness in presenting his personal notes and flood stages recorded at Amana, Iowa.

Ping Yi, Lin

The State University of Iowa,
Iowa City, Iowa, U. S. A.,
January, 1925.
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FLOOD PREDICTION SYSTEMS

ARTICLE I. INTRODUCTION

Flood prediction system is the most economical and easiest method of flood protection. The system is very simple merely consisting of observation stations at some selected places along the stream. The special charge of the system is to give flood warnings to certain communities, where flood damage is likely to be caused, predicting the probable maximum stage of river to be reached by the flood and the probable time of arrival of such stage; so that relief works can be timely prepared to save the damages of properties and the losses of lives by the flood.

Such a system is absolutely required for the benefit of those places on large rivers which are most liable to be exposed to danger from floods due to absence of protection works. It is also a necessity for the extra safety of preserving the life and property of communities in those well protected districts when a calamitous flood should occur far greater than that provided for. The protection works, such
as dams, levees, and etc. would fail in service, if there is no anticipation or warning to call the people to attention for preparing additional protections, and consequently the damage caused by overflow of levees would be much greater than if none were in existence. The maximum flood, which may be expected from any given drainage area, is indeterminate, while the permanent and perfect protections might be impracticable owing to the under estimation or to the limit of expenditure at that time when the investigation was made.

The flood prediction system was first established in the Seine basin by Mr. Belgrand in 1854, and has also been adopted for the other rivers in France. In India it is sometimes arranged that a telegram shall, in the low-water stage of river, be sent from the upper station when a rise of two feet occurs in twenty-four hours or any less period, with a further telegram for any such subsequent rise. In the United States the first flood prediction system was applied on the Ohio River for predicting the flood at Cairo, the junction of the Mississippi and the Ohio River. In 1880 a large number of river stations were established on the Tennessee River above Chattanooga and its tributaries; after 1896 this river is well guarded by competent river and rainfall observers, located on all of the important tributaries. During the recent years several new systems were also established in the other districts in this country; one of them
is that carried out by the Water Supply Commission of Pennsylvania for the Delaware River and other main streams within that state, in order to reduce the damage and loss of life by the floods; the act was passed by the Legislature and approved on May 23, 1913. The other new system is that arranged on South Platte River, Colorado, in connection with the investigation of "Return of Seepage Water" by the cooperation of several bodies of the public organizations.

The accuracy, perfection and great values of flood prediction have been proved in many cases. Once during the great flood of March 1876, the maximum height of flood at Paris was predicted, three days before hand, to within half an inch; thus in one place, below Paris, an embankment was raised above the predicted flood-level; and the people of the low districts had time to remove with their valuables and goods to places of safety. Recently; during the flood of June 1924 in the valley of the Iowa river, the whole town of Marengo was swept by high water of the overflow from Bear Creek, however no great damage was suffered, partly because the time of flooding was short and partly because early warnings were received at 2:00 A. M. June 28 by the night track operator at the Rock Island Railway Station. He then immediately notified the night watchman who in turn notified the merchants in the town to take care of their basement stocks. Early in that morning the citizen's Telephone Company also did some laudable service in
notifying farmers to get their live stock to safety. From above facts, the flood warning system is therefore worthy of possible encouragement; and where practicable, this system should be established on every large river which is liable to inundate and to cause great damages to the big cities and important towns located on lower parts of the river.

ARTICLE 2. PROCESSES OF FLOOD PREDICTION

There are three steps of predicting floods namely: weather observation, rainfall measurement, and river gaging.

WEATHER OBSERVATION.- One, who is familiar with the weather condition, can often forecast the occurrence of storm at a considerable time before the storm, by merely observing the outstanding weather characteristics, especially the wind direction, in his locality; but owing to the insufficient informations and various uncertainties it is difficult for one to foretell what the probable amount and extent of the storm will be.

In the United States, weather observations for daily weather forecasts are taken by U. S. Weather Bureau at about 200 stations. All observations, such as observation of air temperature and pressure, wind direction and velocity,
humidity, precipitation, sunshine, and cloudness, are made at 8 A. M. and 8 P. M. 75th. Meridian time. The various observers after taking these observations, send their reports by telegraph to the main office at Washington and also to other important stations where local predictions are required. The data are then assembled and sheets of the "Daily Weather Map" are promptly printed and distributed to the general public. From this weather map, the forecaster studies the changes and movements of the air conditions during the preceding 24 hours, and he estimates what the weather will be in the different localities the following day.

RAINFALL OBSERVATION.—When a heavy rainstorm or a cloudburst occurs over any drainage area, the probable rise of the river in the immediate district can often be predicted by measuring the intensity and estimating the amount of rainfall over the basin; but it is liable to give erroneous result owing to the uncertainty of the ratio between the amount of rainfall and the depth of runoff, the uneven distribution of rainfall, and the variation with the previous ground condition and the season of the year. At places high up on the course of a stream the rainfall is the only available indication of the occurrence of a flood, however, the interval between the observation and the arrival of the resulting flood is usually very small, while it may also afford early information to the downstream stations when an inundation will be threatening.
RIVER GAGING.—At places lower down the valley, where the main river drains enormous area of watersheds, the floods are not always caused by the actual fall of rain over the immediate vicinity of the drainage area, but usually result from the combined effect of the floods of the upper part of the main river and its tributaries. A flood in the lower part of the river, therefore, rises gradually and continues longer, and does not attain its maximum stage till a considerable time after the upper portion and the tributaries have reached their highest levels, the period depending upon the distance to be traversed and the rate of propagation of the flood. By taking the above fact into consideration, other means are available for giving timely warning of the approach of a flood, in addition to rainfall observation.

By establishing gaging stations at suitable points on the main stream and its branches, noting the heights attained by the flood at frequent intervals, determining the period occupied by the flood in reaching different stations, comparing the corresponding rises on these gages, plotting the flood waves, and keeping records in a number of occurrences, it is possible after several instances to predict with considerable accuracy the time of arrival of the peak of a flood at a given station, and the stage likely to be reached by the river some two, three, or more days before the advent of the flood.
ARTICLE 3. PROPAGATION OF FLOOD WAVE ON RIVERS

The rate of propagation of a flood wave on large rivers differs in general, from the velocity of flowing waters; it has been observed many times that the wave velocity is somewhat greater than the average velocity of the currents. Let us imagine a river of low stage, receiving an increased supply from a suddenly swollen tributary. At first the increased flow of water from the tributary will just spread over the top of low water of the main river and extend both up and down stream from the junction to a distance of several miles. After a while the main river accordingly rises to a higher level and consequently the flood current comes down the river with a velocity greater than the original. This is no doubt that the newly-added water will originate a very long flood wave which propagating down the river must first fill up the river channel or the immediate valley to the flood line. The speed of filling up is faster than the velocity of the moving water because the transmission of pressure through the water is so rapid, and the water particles at higher level, in the moving state are still tending to raise those at lower level to an equal depth as if they were in the state of a still pool. Hence, the river, at points on its lower course, is found to be rising long before the arrival of the actual flood waters, carrying down a mass of floating bodies and suspended matters from the upper courses of the stream. This phenomenon is
called waves of translation, which cause a rise in the surface of the stream ahead of the arrival of actual flowing waters, and is important in disposing the flood water off a river.

The floods, however, of various tributaries do not generally coincide in the time of arrival at any given point on the main river, owing to variations in the period of the occurrence of the rainfall and in the nature of the water-sheds, and also owing to the differences in the distance to be traversed, and in the rates of propagation of the floods, which depend upon the descent of each tributary and the condition of its channel. The floods accordingly, from some tributaries come first and the floods of others descend later; and consequently the river, in the lower portion of its course may have a continuous flood wave consisting of a series of peaks and troughs.

On mountainous streams the phenomena of floods are not similar to those described above. During summer season, a heavy rain occurring locally in an elevated region always causes a suddenly increased flow in the channel among the foot hills, in which the water was previously shallow or even almost dry. The flood water, usually 2 or 3 feet deep, moving down the stream with a great speed due to considerable amount of fall and the increased depth, appears as a nearly vertical wall of water with much splashing and foaming in its front
accompanied with roaring, caused by the continuous falling of the faster moving water near the top over the slower moving water near the bottom. This kind of torrent in some rivers is called the "roll wave" which suddenly descends down the valley below without any warning owing to the fact that there may have been an entire absence of rain locally, or even no knowledge of its occurrence elsewhere. The same effect will be produced and is much more conspicuous when a large quantity of water released suddenly through the failure of a reservoir dam, rushes down the narrow valley. In the great flood of 1889 at Johnstown, Pa., which was caused by the failure of South Fork reservoir, a vertical wall of water with a height of about 10 feet, was seen to move down the valley, carrying on its front brush and logs mingled with spray and foam. In 41 minutes it traveled a distance of 13 miles down the descent of 380 feet, the velocity is hence 19 miles per hour.

When such a torrent enters a deeper stream with a smaller fall and consequently smaller velocity, the wall of water will begin to diminish its amplitude at certain distance above the entrance and becomes a wave of transmission in the main stream as that previously described.
ARTICLE 4. METHOD FOR DETERMINING THE RATE OF PROPAGATION OR VELOCITY OF A FLOOD WAVE

The rate of propagation of a flood wave in a river, however, which is a variable quantity depending upon the slope, stage and other conditions of the river, can be possibly determined with considerable accuracy by means of gage readings as mentioned in the previous articles. But, if, there is no record of observations in existence for a given river in question, it is more desirable to derive some mathematical equations for determining the most probable values of the velocity of a flood wave, based on the following fundamental principles:

(1) When a flood advances down a stream the river channel or the immediate valley is first filled up to the flood line by the excessive supply from the upstream.

(2) The river, at any points downstream, is found to be rising long before the arrival of the actual flood water.

(3) The flood discharge at corresponding stage of different sections is very nearly equal or the same.

Let us now suppose the flood wave to have a profile H C T as shown in Fig. 1. And consider:

H = head of the flood wave;
C = crest of the flood wave;
T = tail of the flood wave;

H', C', & T' = head, crest and tail of the flood wave in the advanced position.
Fig. 1. Diagram showing the form and the manner of advancing of a flood wave
Let $V_0=$ velocity of flood wave;
$V_1=$ velocity of flood flow at C;
$V_2=$ velocity of normal flow at H & T;
$V =$ velocity of flood flow at E;
$V + \Delta V =$ velocity of flood flow at F;
$D_1 =$ depth of flood stage at C;
$D_2 =$ depth of normal stage at H & T;
$D =$ depth of flood stage at E;
$D + \Delta D =$ depth of flood stage at F;
$A_1 =$ area of flood stage at C;
$A_2 =$ area of normal stage at H & T;
$A =$ area of flood stage at E;
$A + \Delta A =$ area of flood stage at F;
$q_1 =$ quantity of flood flow at C;
$q_2 =$ quantity of normal flow at H & T;
$q =$ quantity of flood flow at E;
$q + \Delta q =$ quantity of flood flow at F;
$\Delta L =$ distance advanced by the flood wave;
$\Delta t =$ time required by the flood wave in moving through a distance "$\Delta L$";
$Q_1 =$ total flow of flood water passing C in the time "$\Delta t$";
$Q_2 =$ total flow of normal water passing H & T in the time "$\Delta t$";
$Q =$ total flow of flood water passing E in the time "$\Delta t$";
\[ Q + \Delta Q = \text{total flow of flood water passing F in the time } "\Delta t". \]

The relations between above quantities are:

\[ q_1 = A_1 V_1, \quad q_2 = A_2 V_2, \]
\[ q = A V, \quad q + \Delta q = (A + \Delta A)(V + \Delta V), \]
\[ Q_1 = A_1 V_1 (\Delta t), \quad Q_2 = A_2 V_2 (\Delta t), \]
\[ Q = A V (\Delta t), \quad Q + \Delta Q = (A + \Delta A)(V + \Delta V)(\Delta t). \]

The quantity of water stored temporarily between E and F is,

\[ (Q + \Delta Q) - Q = (A + \Delta A)(V + \Delta V)(\Delta t) - (AV)(\Delta t), \]

which may be reduced to

\[ \Delta Q = (V\Delta A + A\Delta V + \Delta V\Delta A)(\Delta t). \]

During that time the quantity of water stored must extend and fill up the shaded portion "\Delta L" (See Fig. 1.) and we have therefore,

\[ \Delta Q = (\Delta A)(\Delta L), \]

and

\[ (\Delta A)(\Delta L) = (V\Delta A + A\Delta V + \Delta V\Delta A)(\Delta t), \]

Dividing both sides by \((\Delta A)\) into \((\Delta t)\), we obtain,

\[ \frac{\Delta L}{\Delta t} = \frac{V(\Delta A) + A(\Delta V) + (\Delta V)(\Delta A)}{\Delta A} \]

Since,

\[ \frac{\Delta L}{\Delta t} = V \]

Therefore,

\[ V_0 = V + A \frac{(\Delta V)}{(\Delta A)} + \Delta V. \]
When \( \Delta D \) approaches zero, \( \Delta A \) and \( \Delta V \) will approach zero. In limit,

\[
V_0 = V + A \quad \frac{\text{d}V}{\text{d}A} + 0
\]

Transposing, and Dividing we obtain a differential equation,

\[
\frac{\text{d}A}{A} = \frac{\text{d}V}{V_0 - V}
\]  \hspace{1cm} (1)

The integration is

\[
\log_e(A)_{A_2}^{A_1} = \log_e\left(\frac{1}{V_0 - V}\right)\frac{V_1}{V_2}
\]

Substituting the limits and changing to the form

\[
\frac{A_2}{A_1} = \frac{V_0 - V_1}{V_0 - V_2}
\]  \hspace{1cm} (2)

From which we have

\[
V_0 = \frac{V_1A_1 - V_2A_2}{A_1 - A_2} = \frac{q_1 - q_2}{A_1 - A_2}
\]  \hspace{1cm} (3)

This is a general equation for determining the velocity of a flood wave in any river under ordinary conditions. It may be expressed in the following words, namely: The velocity of a flood wave is equal to the difference between peak and nor-
mal discharges divided by the difference between areas of the high and normal water sections.

In the case of more or less improved rivers the channel sections may be considered as composed of rectangles with uniform breadth \( b \) without any serious effects in calculating the velocities and discharges. So, when a bank-full flood comes down the stream the high water section is \( A_1 = bD_1 \) and the normal water section is \( A_2 = bD_2 \), in which \( D_1 \) and \( D_2 \) are the average values of their depths. Substituting these in equations (2) and (3) we obtain

\[
\frac{bD_2}{bD_1} = \frac{V_o - V_1}{V_o - V_2} \quad \text{or} \quad \frac{D_2}{D_1} = \frac{V_o - V_1}{V_o - V_2} \quad (2-A)
\]

and

\[
V_o = \frac{b(V_1D_1 - V_2D_2)}{b(D_1 - D_2)} = \frac{V_1D_1 - V_2D_2}{D_1 - D_2} \quad (4)
\]

In order to determine the flood wave velocity \( V_o \), it is necessary to obtain first the values of depths and velocities of both the high and normal waters for equation (4), in addition to the above the areas of the representative sections of the river if equation (3) is used.

The first step is to establish gages at different stations and locate their zeros with respect to certain datum plane, so that the stages of water surfaces at each station may be determined by reading the gage frequently.
Fig. 2-A Cross-Section of the Iowa River at S. U. I. Gage Station.
Fig. 2-B. Discharge, Mean Velocity, Area, and Mean Depth Curves, Iowa River at S. U. I. Gage Station, Iowa City, Iowa.
The cross-section of the river at each station should be next determined during low water season by using sounding lines or rods for the river bed under water surface, and using levels to accomplish the points on or above banks.

The cross-section can be easily plotted and the areas at the different depth also computed. The area curve showing the relation between the gage height and the area of the cross-section is very useful for computing equation (3). While the mean depth curve indicating the same relations is also to be constructed for the computations from equation (4).

The velocity of a flowing stream is generally dependent upon:

1. Surface slope of the stream, which is not very different from the river bed in most cases;
2. Roughness of the bed, which affects the coefficient of velocity and discharge;
3. Hydraulic radius, which may be considered nearly equal to the mean depth in wide streams.

The mean velocity of a stream is the average rate of motion of all the filaments of water in passing the cross-section. Velocity can be measured usually with a current meter without difficulty at low and medium stages, but it is impracticable at flood stages, owing to rapid changes of stage, swift currents, and obstructions of surface drift.

It is generally advisable to use float method in
measuring the swift current especially when there is much drift serving as floats. The floats should be placed in the stream at several positions of various distance from the shores and the average velocity of the floats can thus be obtained. To get the mean velocity of the whole cross-section some coefficient should be applied varying from 80 per cent for shallow streams to 95 per cent for deep streams. A most probable magnitude of the coefficient of 90 per cent may be used for the flood velocity in ordinary rivers. If there is no measurement available, the velocity at high water stages may be obtained without great discrepancy by extending the mean velocity curve from medium stages to flood stages. In this case the area above the level of flood plain should be considered to include only the section above the ordinary channel of the stream, as the mean velocity curve plotted for medium and low stages, applies only to this channel and not to the overflow channel. The velocity and discharge on the sides of the flood plain is comparatively small and may be neglected for the error introduced by it into the total discharge will not be great.

It may be also practicable to determine the slope of the river at a selected reach and to compute the velocity by means of the slope formula, which is generally expressed as:

\[ V = CD^mS^n \]  

(5)
where

\[ V = \text{mean velocity of stream}; \]
\[ C = \text{a constant}; \]
\[ D = \text{mean depth of cross-section}; \]
\[ S = \text{slope}; \]
\[ m = \frac{1}{2} \text{ for Chezy's Formula}; \]
\[ = \frac{2}{3} \text{ for Manning Formula}; \]
\[ = 1 \text{ for Approximate Formula}; \]
\[ n = \frac{1}{2} \text{ for all cases}. \]

The velocity formulas may thus be written:

\( (a) \quad V = CD^{1/2}S^{1/2} \) \hspace{1cm} (5-A)

\( (b) \quad V = CD^{2/3}S^{1/2} \) \hspace{1cm} (5-B)

\( (c) \quad V = CD^{1/2}S^{1/2} \) \hspace{1cm} (5-C)

The above formulas are only applicable for the normal and maximum river stages, as the surface slope in both cases is nearly equal to that of the river bed, and the flow at these stages is also considered to be steady uniform. The velocity at intermediate points between \( D_1 \) and \( D_2 \) is of course different from either \( V_1 \) and \( V_2 \). This has no connection with the present discussion and hence it will be determined in later article.

The solution of these velocity formulas can be precisely obtained by simple multiplications with aid of numerical tables for finding one-half and two-thirds powers of numbers. The nomograph shown in Fig. 3 affords more rapid solution of these formulas and less liability of making
errors, and gives results which are sufficiently accurate for practical purposes, while speed is more important in giving flood warnings.

As an example, take the data as follows:

The distance between two flood prediction stations (1) and (2) = 100 miles.

The mean depth of flood stage at station (1), \( D_1 = 20 \) feet.

The mean depth of normal stage at station (2), \( D_2 = 10 \) feet.

The surface slope of both flood water and normal water considered to be approximately equal to the river bed \( S = 3 \) feet per mile or \( 0.00057 \).

Find at what time the flood crest will reach station (2), considering the channel section of the river is uniform throughout its length.

Procedure of solution:

(1) Use a roughness coefficient \( n = 0.035 \).

(2) From King's Handbook of Hydraulics, Table 76, pages 207 to 209, find the coefficient of Chezy Formula \( c = 70 \) for the average of above conditions.

(3) Use the nomogram shown in Fig. 3; and set a straight edge at \( c = 70 \) and \( s = 3 \) feet per mile, find a point on the velocity scale.
(4) With the point on the V scale and D = 1, find a point on the slope scale.

(5) With the point on the s scale and D = 20, find $V_1 = 7.6$ feet per second.

(6) With the same point on the S scale and D = 10, find $V_2 = 5.4$ feet per second.

(7) Substitute values of D and V in equation (4), and find the velocity of the flood wave.

$$V_0 = \frac{V_1 D_1 - V_2 D_2}{D_1 - D_2}$$

$$= \frac{7.6(20) - 5.4(10)}{20 - 10}$$

$$= \frac{98}{10}$$

$$= 9.8$$ feet per second.

Following the same procedures the results obtained with Manning's and the Approximate formulas are shown, together with that calculated from Chezy formula, in the following table.
TABLE I

COMPUTATIONS FOR VELOCITY OF FLOOD WAVE

CONDITIONS: \( D_1 = 20' \), \( D_2 = 10' \), \( S = 3 \text{ ft./mi.} \), and \( n = .035 \).

<table>
<thead>
<tr>
<th>Formula</th>
<th>C</th>
<th>( V_1 )</th>
<th>( V_2 )</th>
<th>( V_0 )</th>
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<tr>
<td>Chezy</td>
<td>70</td>
<td>7.62</td>
<td>5.40</td>
<td>9.84</td>
</tr>
<tr>
<td>Manning</td>
<td>40</td>
<td>7.18</td>
<td>4.52</td>
<td>9.84</td>
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<td>7.55</td>
<td>3.73</td>
<td>11.37</td>
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<td>Average</td>
<td></td>
<td></td>
<td></td>
<td>10.35</td>
</tr>
</tbody>
</table>

The average velocity of flood wave is 10.35 feet per second or \( 10.35/1.468 = 7.06 \) miles per hour, and the time required by the flood wave to traverse from station (1) to station (2) is,

\[
t = \frac{100}{7.06} = 14.15 \text{ hours.}
\]
ARTICLE 5. SHORT METHOD FOR COMPUTING VELOCITY OF FLOOD WAVE

Let the ratios of $V_2$ to $V_1$ and $D_2$ to $D_1$ be represented respectively by

$$x = \frac{V_2}{V_1} \quad \text{and} \quad y = \frac{D_2}{D_1}$$

Then

$$V_2 = xV_1 \quad \text{and} \quad D_2 = yD_1$$

Substituting in equation (4)

$$V_0 = \frac{V_1D_1 - (xV_1)(yD_1)}{D_1 - yD_1} = \frac{V_1 - xy}{1 - y} \quad (6)$$

By $V = CD^mS^n$

$$\frac{V_2}{V_1} = \frac{C_2D_2^mS_2^n}{C_1D_1^mS_1^n}$$

Considering $C_1$, $C_2$ and $S_1$, $S_2$ to be nearly equal in values, then

$$x = \frac{V_2}{V_1} = \frac{(D_2/D_1)^m}{y^m}$$

Or

$$x^{1/m} = y$$

Substituting again $x^{1/m}$ for $y$ in equation (6)

$$V_0 = \frac{V_1 - xx^{1/m}}{1 - x} = \frac{V_1 - x^{1+1/m}}{1 - x^{1/m}} \quad (7)$$

Let the fraction in the right member be represented by $J$.

Then equation (7) may be written, $V_0 = JV_1$
(a) when \( m = 1/2 \)

\[
V_0 = V_1 \frac{1 - x^3}{1 - x^2} = V_1 \frac{1 + x + x^2}{1 + x}
\]

\[
V_0 = V_1 \left(1 + \frac{x^2}{1 + x}\right)
\]  
(8)

(b) when \( m = 2/3 \)

\[
V_0 = V_1 \frac{1 - x^{5/2}}{1 - x^{3/2}} = V_1 \frac{1 + x^{1/2} + x + x^{3/2} + x^2}{1 + x^{1/2} + x}
\]

\[
V_0 = V_1 \left(1 + \frac{x^{3/2} + x^2}{1 + x^{1/2} + x}\right)
\]  
(9)

(c) when \( m = 1 \)

\[
V_0 = V_1 \frac{1 - x^2}{1 - x}
\]

\[
V_0 = V_1 (1 + x)
\]  
(10)

From equations (8), (9), and (10) we see that the velocity \( V_0 \) of a flood wave in each case is greater than that of flood flow \( V_1 \).
Since the value of velocity is more difficult to obtain than that of depth, it is better to express equation (7) in terms of depth or "y" the ratio of depth. Hence equation (7) may be changed into,

\[
V_0 = V_1 \frac{1 - y^m}{1 - y} = V_1 \frac{1 - y^{1+m}}{1 - y} \quad (11)
\]

or

\[
V_0 = KV_1
\]

in which \( V_1 \) may be obtained from \( V = C D^m S^n \) as shown on the nomgram (Fig. 3.).

(a) When \( m = 1/2 \)

\[
V_0 = V_1 \frac{1 - y^{3/2}}{1 - y} = V_1 \frac{1 + y^{1/2} + y}{1 + y^{1/2}} \]

\[
V_0 = V_1 (1 + \frac{y}{1 + y^{1/2}}) \quad (12)
\]

(b) When \( m = 2/3 \)

\[
V_0 = V_1 \frac{1 - y^{5/3}}{1 - y} = V_1 \frac{1 + y^{1/3} + y^{2/3} + y + y^{4/3}}{1 + y^{1/3} + y^{2/3}}
\]

\[
V_0 = V_1 (1 + \frac{y + y^{4/3}}{1 + y^{1/3} + y^{2/3}}) \quad (13)
\]

(c) When \( m = 1 \)

\[
V_0 = V_1 \frac{1 - y^2}{1 - y}
\]

\[
V_0 = V_1 (1 + y) \quad (14)
\]
To solve equations (8), (9) and (10), and (12), (13) and (14), values of flood wave velocity functions $J$ and $K$ are computed and placed in Table 2, from which curves showing the relations between the ratio $x$ and $J$, and $y$ and $K$ are plotted in Figs. 4 and 5.

By the help of these curves and the nomogram in Fig. 3, the velocity $V_o$ of flood wave can be more rapidly determined with considerable accuracy. Take the same example as

### Table 2

VALUES OF FLOOD WAVE VELOCITY FUNCTION

<table>
<thead>
<tr>
<th>$x = 2$</th>
<th>$m = 1$</th>
<th>$m = 2$</th>
<th>$m = 3$</th>
<th>$y = 1$</th>
<th>$m = 2$</th>
<th>$m = 3$</th>
<th>$m = 1$</th>
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</thead>
<tbody>
<tr>
<td>$V_1$</td>
<td>0.0000</td>
<td>1.0000</td>
<td>1.0000</td>
<td>0.0000</td>
<td>1.0000</td>
<td>1.0000</td>
<td>1.0000</td>
</tr>
<tr>
<td>$V_2$</td>
<td>1.0091</td>
<td>1.0294</td>
<td>1.0540</td>
<td>1.0759</td>
<td>1.0955</td>
<td>1.1154</td>
<td>1.1350</td>
</tr>
<tr>
<td>$V_3$</td>
<td>1.0322</td>
<td>1.0786</td>
<td>1.1230</td>
<td>1.1708</td>
<td>1.2186</td>
<td>1.2664</td>
<td>1.3142</td>
</tr>
<tr>
<td>$V_4$</td>
<td>1.0691</td>
<td>1.1380</td>
<td>1.2070</td>
<td>1.2869</td>
<td>1.3678</td>
<td>1.4487</td>
<td>1.5306</td>
</tr>
<tr>
<td>$V_5$</td>
<td>1.1100</td>
<td>1.2030</td>
<td>1.2970</td>
<td>1.3920</td>
<td>1.4871</td>
<td>1.5822</td>
<td>1.6773</td>
</tr>
<tr>
<td>$V_6$</td>
<td>1.1667</td>
<td>1.2730</td>
<td>1.3860</td>
<td>1.4990</td>
<td>1.6120</td>
<td>1.7251</td>
<td>1.8382</td>
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<td>$V_7$</td>
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<td>1.3380</td>
<td>1.4430</td>
<td>1.5500</td>
<td>1.6570</td>
<td>1.7641</td>
<td>1.8712</td>
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<tr>
<td>$V_8$</td>
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<td>1.4240</td>
<td>1.5300</td>
<td>1.6370</td>
<td>1.7440</td>
<td>1.8511</td>
<td>1.9582</td>
</tr>
<tr>
<td>$V_9$</td>
<td>1.3560</td>
<td>1.5040</td>
<td>1.6040</td>
<td>1.7040</td>
<td>1.8040</td>
<td>1.9040</td>
<td>2.0040</td>
</tr>
<tr>
<td>$V_{10}$</td>
<td>1.4260</td>
<td>1.5840</td>
<td>1.6867</td>
<td>1.7867</td>
<td>1.8867</td>
<td>1.9867</td>
<td>2.0867</td>
</tr>
</tbody>
</table>
Fig. 4. Flood Wave Velocity Functions Based on Ratio of Velocities
Fig. 5. Flood Wave Velocity Functions Based on Ratio of Depths
on Page 18: \( D_1 = 20' \), \( D_2 = 10' \), \( S = 3 \) feet per mile.

The velocities corresponding to \( D_1 \) and \( D_2 \) may be measured or calculated from velocity formulas. Suppose the measured mean velocities at \( D_1 \) and \( D_2 \) to be \( V_1 = 7 \) ft./sec. and \( V_2 = 5 \) ft./sec. The ratio \( V_2/V_1 = 0.715 \) for which Fig. 4 gives: \( J = 1.295 \) for Chezy Formula, \( J = 1.432 \) for Manning Formula and \( J = 1.512 \) for the Approximate Formula. These values substituted in equations (8), (9) and (10) give:

\[
\begin{align*}
V_0 &= 7(1.295) = 9.06 \text{ feet per second;} \\
V_0 &= 7(1.432) = 10.04 \text{ feet per second;} \\
V_0 &= 7(1.512) = 10.60 \text{ feet per second.}
\end{align*}
\]

The average of these values is 9.90 feet per second or 6.75 miles per hour. If velocities are not measured, we should first determine the flood velocity \( V_1 \) with nomogram in Fig. 3 and should use the function \( K \) instead of \( J \). Here, now, \( D_2 /D_1 = 0.50 \), for which Fig. 5 gives: \( K = 1.294 \) for \( m = 1/2 \), \( K = 1.384 \) for \( m = 2/3 \), and \( K = 1.500 \) for \( m = 1 \). Substituting these values into equations (12), (13) and (14), we have then:

\[
\begin{align*}
V_0 &= 7.62(1.294) = 9.84 \text{ feet per second;} \\
V_0 &= 7.18(1.343) = 9.84 \text{ feet per second;} \\
V_0 &= 7.55(1.500) = 11.37 \text{ feet per second,}
\end{align*}
\]

which are same as indicated in Table I. The average velocity \( V_0 \) is therefore 10.35 feet per second or 7.06 miles per hour.
ARTICLE 6. METHODS OF PREDICTING HEIGHT OF FLOOD WAVE

The determination of height of a flood wave propagating down an ideal river with a uniform channel section is very simple, because a flood wave does not change its form without change of the river's condition; but, however, absolute prediction is impracticable in ordinary rivers, which are subject to varying cross-section, contracting in some places and widening in others, while any method used for flood prediction merely affords early information when a large flood is threatening, and for this purpose an attempt at great accuracy is not necessary.

In rivers which have few tributaries whose discharges can be ignored or neglected without great effect to the discharge of the main stream, the most convenient method of predicting the flood height is by plotting a mean curve from numerous observations, indicating the mean gage relation between two stations, as shown in Fig. 6. Thus a mean flood height at the lower station, corresponding to a given height at the upper station, can be readily read off from the curve.

Another method of predicting flood heights is by determining the discharges at upper stations located on all the large tributaries, and computing the probable height at the lower station from these discharges. The processes consist in determining the mean discharge curve at the
Fig. 6. Diagram for Predicting Flood Height

Ohio River Flood Cincinnati Gage Height in Feet

Fig. 6. Diagram for Predicting Flood Height
different stations by numerous discharge measurements, and also in plotting the profiles of the flood waves or time discharge curves of the floods. From the curves, discharges are taken at such a time that the combined discharge in the main river will be a maximum at the lower station, and from its discharge curve the height, which the flood will attain, is determined. If the maximum discharge is far off the curve, the height may be also solved by trials from the discharge formula, \( Q = CAD^mS^n \) or by deriving a special equation containing the term of area in function of depth, as,

\[
Q = VA = (CD^mS^n)(C'D^a) = CqD^{a+m}S^n \tag{15}
\]

in which

- \( Q \) = discharge,
- \( V \) = velocity,
- \( A \) = area of cross-section,
- \( D \) = maximum depth of the section,
- \( S \) = slope of water surface, assumed to be equal to the fall of river bed,
- \( C \) = coefficient of velocity,
- \( C' \) = coefficient of area,
- \( C_q \) = coefficient of discharge,
- \( a \) = exponent for the area,
- \( m, n = \) same as those on page 17.

In all cases, predictions are liable to be more
or less upset if rain falls in the tract between the upper and lower gages; therefore gage stations should not be too far apart, and rainfall observation stations are also necessary to be established at suitable points in the region along the river course, in order that correction could be made when it is required.

One of the chief difficulties in river discharge problem is to estimate runoff from the amount of rainfall. The coefficient of runoff varies greatly with the condition of the ground, the topography of the drainage area, and the season of the year, as well as other minor conditions. But, however, to make an approximate estimation some empirical formulas are of useful; they have been deduced for average conditions by various authorities. Such formulas may be found from books on sewages, they have generally a common form of expression, which is given on page 85.
CHAPTER II
PROFILE OF FLOOD WAVES

ARTICLE 7  PROFILE OF ROLL WAVE

It has been mentioned in the preceding Chapter that a flood comes down a shallow mountainous stream or a dry creek, appearing as a nearly vertical wall of water. The following description of this phenomenon is extracted from an article "On the Effect of a Rapidly Increasing Supply of Water to a Stream" by James B. Francis, Trans. Am. Soc. C. E. 1889, Vol. 21.

"In the Chezy formula for the flow of water in channels, viz.

\[ V = \frac{C}{\sqrt{RI}} \]

\( V \) = velocity in feet per second;
\( C \) = co-efficient depending on the nature of the bed of channel;
\( R \) = the hydraulic mean depth;
\( I \) = the slope or descent per foot in length.

In this case \( C \) and \( I \) may be taken as constant. The depth of the stream, and consequently the value of \( R \), will increase with quantity of water flowing, and by the formula, the velocity will increase as \( R \) increases.

"The increased velocity due to the increased quantity
will be first felt near the point of supply, and later in the stream below, and the effect will be that after a sufficient time the upper parts of the stream having a greater velocity, will overtake the lower, combining with them and flowing on together, with a depth and velocity due to the greater quantity.

"The effect will be most distinctly indicated in a stream having a rapid descent, and may be illustrated as follows:

"In a stream having a rocky bed, for which $C=50$ suppose the descent to be 1 foot in 100 feet, or $I=0.01$; and suppose the section and wetted perimeter to be such that $R=1$. Then, by the above formula $V=5$ feet per second.

"In 100 seconds from a given moment suppose the supply of water, and with it the depth of the stream, to be increased so that $R=4$. Then the velocity will be increased to $5\sqrt{4}=10$ feet per second.

"Put $x=$ the time from the admission of the additional supply in which the increased current, will overtake the original current, which has had a start of 100 seconds, $100(5) + 5x = 10x$, from which we find $x = 100$ seconds.

"The distance from the point of admission of the additional supply to the point where it will overtake the stream flowing 5 feet per second will be $100(10) = 1,000$ feet.

"After the lapse of 100 seconds, suppose another ad-
ditional supply is admitted from the same source, increasing the depth so that \( R = 9 \), and the velocity becomes \( 5\sqrt{9} = 15 \) feet per second.

"For the time \( x \) from the admission of the second additional supply, when it will overtake the first additional supply, which has a start of 100 seconds, we have \( 100(10) + 10x = 15x \), and \( x = 200 \) seconds.

"The distance from the point of admission of the additional supply when it will overtake the preceding will be \( 200(15) = 3,000 \) feet.

"After the lapse of another 100 seconds, another additional supply is admitted, increasing the depth so that \( R = 16 \), and the velocity becomes \( 5\sqrt{16} = 20 \) feet per second.

"Similarly, the time \( x \) from the admission of the third additional supply when it will overtake the second additional supply, which has a start of 100 seconds, will be \( 100(15) + 15x = 20x \); and \( x = 300 \) seconds, and the distance from the point of admission, where it will overtake the preceding, will be \( 300(20) = 6,000 \) feet.

"After the lapse of another 100 seconds, another additional supply is admitted, increasing the depth so that \( R = 25 \) and the velocity becomes \( 5\sqrt{25} = 25 \) feet per second.

"The time \( x \) when it will overtake the preceding will be \( 100(20) + 20x = 25x \); and \( x = 400 \) seconds, and distance from the point of admission when it will overtake the preceding
will be $400(25) = 10,000$ feet.

"If there is no further increasing in the supply there will be no further increase in the value of $R$ or in the velocity. If the supply diminishes the value of $R$ will diminish also at the point of supply, the effect of the preceding additions will not be reduced, even if the supply should entirely cease, and $R$ become zero at the point of supply, and from the moment when the current due to the last increase in the supply has overtaken the preceding currents and of course become the head of the torrent, it will continue to flow down the channel with same depth and velocity so long as the descent and form of the channel continue uniform, and for the length of time that the maximum supply is continued, after which it will begin to be reduced in depth and velocity."

From above discussion, the velocity of a roll wave would be constant, if the channel condition is unchanged and the maximum influent is steadily supplied. This can also be proved from equation (3) or rather (4), by making $V_2$ and $D_2$ equal to zero, then it may be written

$$V_0 = \frac{V_1 D_1 - 0}{D_1 - 0} = V_1$$

That is the velocity of a roll wave is same as that of the maximum flow.
Hence the velocity is constant at any section along the wave. The surface curve at different points in the front portion changes its gradient according to the depth at each point, and consequently the wave adjusts itself into a form as shown in Fig. 7.

Let $D_1 =$ maximum depth at crest of the wave;  
$D =$ depth at any section;  
$H =$ elevation of water surface above the head of the wave;  
$B =$ elevation of bottom above the same point;  
$S =$ slope of bottom;  
$S_o =$ friction slope, a slope required to maintain $V$ constant at various depth $D$;  
$L =$ horizontal length of the wave measured from the head;  
$dD =$ difference in depth between two consecutive sections;  
$dH =$ difference in elevation of water surface between two consecutive sections;  
$dB =$ difference in elevation of bottom between two consecutive sections;  
$dL =$ horizontal distance between two consecutive sections;  
$dF =$ head required to overcome friction in distance $dL$. 
Fig. 7. Typical Profile of Roll Wave
For steady uniform flow, the fundamental velocity formula is

\[ V = CD^{1/2} \]  \hspace{1cm} (5)

For non-uniform flow the above formula is not correct, but we may write

\[ V = CD^{1/2} S_{o}^{1/2} \]  \hspace{1cm} (16)

In which \( S_{o} \) is neither the slope of the surface nor of the bottom but is an ideal slope which will be just sufficient to maintain velocity constant, or to overcome friction. It may be called friction slope for the given depth and velocity. Then

\[ S_{o} = \frac{V^{2}}{C^{2}D^{2m}} \]

while the slope of the surface at the crest of the wave is obviously

\[ S = \frac{V^{2}}{C^{2}D^{2m}} \]

Since \( D \) is always less than \( D_{l} \), \( S_{o} \) increases from the crest toward the toe of the wave.

As \( V \) is constant throughout the length no change of velocity head will exist in this case. In a short length, \( dL \), the only head consumed in overcoming friction will be

\[ dH = dF = S_{o}dL = \frac{V^{2}}{C^{2}D^{2m}}dL \]

From Fig. 7

\[ H = B + D \]  \hspace{1cm} (17)
Then,

\[ dH = dB + dD = SdL + dD \]

\[ = \frac{V^2}{c^2 D^m} dL + dD \]

Therefore,

\[ \frac{V^2}{c^2 D^m} dL = \frac{V^2}{c^2 D^m} dL + dD \]

From which

\[ dL = \frac{c^2 D^m}{V^2} \frac{D^m}{D^m - D^m} dD \]

or

\[ dL = \frac{1}{S} \frac{D^m}{D^m - D^m} dD \]  \hspace{1cm} (18) \]

Let \( Z = D/D_1 \), \( D = D_1 Z \), and \( dD = D_1 dZ \)

Then,

\[ dL = \frac{D_1}{S} \frac{Z^m}{1 - Z^m} dZ \]  \hspace{1cm} (19) \]

This is general differential equation for profile of the roll wave. By using standard methods of integration its integral equations are derived for different values of "m" in the following pages.
(a) When \( m = 1/2 \) equation (19) becomes,

\[
dL = \frac{D_1}{S} \frac{Z}{1 - Z} \quad \text{(19-A)}
\]

Dividing the denominator into numerator, we obtain,

\[
dL = \frac{D_1}{S} \frac{1}{1 - Z} \quad \text{d}Z
\]

Integrating, we obtain,

\[
L = \frac{D_1}{S} (-Z + \log_e \frac{1}{1 - Z}) + C
\]

When \( L = 0, \ Z = 0, \) and \( C = 0 \)

Therefore,

\[
L = \frac{D_1}{S} \left( \log_e \frac{1}{1 - Z} - Z \right) \quad \text{(20)}
\]

From

\[
H = D + SL
\]

= \( D_1 Z + SL \)

Then

\[
H = D_1 \log_e \frac{1}{1 - Z} \quad \text{(21)}
\]
(b) When \( m = 2/3 \) equation (19) may be written

\[
dL = \frac{D_1}{S} \frac{z^{4/3}}{1 - z^{1/3}} dZ \quad (19-B)
\]

Let \( u = z^{1/3}, \ z = u^3, \) and \( dZ = 3u^2 du \)

Then

\[
dL = \frac{3D_1}{S} \frac{u^6}{1 - u^4} du
\]

Decomposing the fraction as follows:

\[
dL = \frac{3D_1}{S} \left( -u^2 + \frac{1}{4} \frac{l}{u^4} + \frac{1}{4} \frac{l}{1 - u} + \frac{1}{2} \frac{l}{1 - u^2} \right) du
\]

Integrating

\[
L = \frac{3D_1}{S} \left( - \frac{u^3}{3} + \frac{l}{4} \frac{1}{u} + \frac{l}{1 - u} \right) + c
\]

Determining the constant \( c = 0 \) and substituting \( z^{1/3} \) for \( u \),

we obtain

\[
L = \frac{D_1}{S} \left( \frac{3}{4} \log_e \frac{1 + z^{1/3}}{1 - z^{1/3}} + \frac{3}{2} \tan^{-1} z^{1/3} - Z \right) \quad (22)
\]

By

\[
H = D_1 Z + SL
\]

Then

\[
H = D_1 \left( \frac{3}{4} \log_e \frac{1 + z^{1/3}}{1 - z^{1/3}} - \frac{1}{2} \tan^{-1} z^{1/3} \right) \quad (23)
\]
When \( m = 1 \) equation (19) takes the form

\[
dL = \frac{D_1}{S} \frac{Z^2}{1 - Z^2} dZ \quad (19-C)
\]

Decomposing the fraction

\[
dL = \frac{D_1}{S} \left( \frac{-1}{2} + \frac{1}{1 + Z} + \frac{1}{2} \frac{1}{1 - Z} \right) dZ
\]

Integrating

\[
L = \frac{D_1}{S} \left( -Z + \frac{1}{2} \log_e \frac{1 + Z}{1 - Z} \right) + c
\]

When \( L = 0, \ Z = 0, \) and \( c = 0 \)

Therefore

\[
L = \frac{D_1}{S} \left( -\log_e \frac{1 + Z}{1 - Z} \right) \quad (24)
\]

By

\[
H = D_1 Z + SL
\]

Then

\[
H = \frac{D_1}{S} \log_e \frac{1 + Z}{2 \frac{1}{1 - Z}} \quad (25)
\]
TABLE 3

PROFILE OF ROLL WAVE

\( D = 1, \ S = 0.01 \)

<table>
<thead>
<tr>
<th>( Z )</th>
<th>(a) ( m = 1/2 )</th>
<th>(b) ( m = 2/3 )</th>
<th>(c) ( m = 1 )</th>
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</thead>
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<td>( H )</td>
<td>( L )</td>
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<td>361.52</td>
<td>4.6052</td>
<td>261.73</td>
</tr>
</tbody>
</table>
Fig. 8. Computed Profile of Roll Wave
ARTICLE 8. VOLUME OF ROLL WAVE

Let \( D \) be the depth at \( E \) and \( dL \) be the length of the roll wave between \( E \) and \( F \), Fig. 9. Then

\[ dQ = D dL, \text{ for } b = 1 \text{ foot wide.} \]

Substituting \( D_1 Z \) for \( D \)

\[ dQ = D_1 Z dL \]

From equation (19)

\[ dL = \frac{D_1}{S} \frac{Z^{2m}}{1 - Z^{2m}} dZ \]

Therefore

\[ dQ = \frac{D_1^2}{S} \frac{Z^{1+2m}}{1 - Z^{2m}} dZ \]  \hspace{1cm} (26)

(a) When \( m = 1/2 \)

\[ dQ = \frac{D_1^2}{S} \frac{Z^2}{1 - Z} \]  \hspace{1cm} (26-A)

Dividing the denominator into the numerator, we obtain,

\[ dQ = \frac{D_1^2}{S} \frac{1}{1 - Z^2} - \frac{1}{1 - Z} dZ \]

Integrating

\[ Q = \frac{D_1^2}{S} \left( \log_e \frac{1}{1 - Z^2} - Z - \frac{Z^2}{2} \right) \]  \hspace{1cm} (27)
Fig. 9. Diagram for Determining the Volume of Roll Wave
(b) When \( m = \frac{2}{3} \)

\[
dQ = \frac{D_1^2}{S} \frac{Z^{7/3}}{1 - Z^{1/3}} \, dZ
\]  

(26-B)

Let \( u = Z^{1/3}, \quad Z = u^3, \quad dZ = 3u^2 \, du \)

Then

\[
dQ = \frac{3D_1^2}{S} \frac{u^9}{1 - u} \, du
\]

Decomposing the fraction into separate parts

\[
dQ = \frac{3D_1^2}{S} \left( \frac{1}{2} \frac{u}{1 + u^2} + \frac{1}{4} \frac{1}{1 - u} - \frac{1}{4} \frac{u^5}{1 + u} \right) \, du
\]

Integrating

\[
Q = \frac{3D_1^2}{S} \left( -\log_\varepsilon \frac{1}{4} \frac{1}{1 + u^2} + \frac{1}{2} \frac{1}{1 - u} - \frac{1}{6} \frac{u^6}{1 + u} \right)
\]

Therefore

\[
Q = \frac{3D_1^2}{S} \left( -\log_\varepsilon \frac{1}{4} \frac{1}{1 + Z^{2/3}} + \frac{Z^{2/3}}{2} - \frac{Z^2}{6} \right)
\]

(28)

(c) When \( m = 1 \)

\[
dQ = \frac{D_1^2}{S} \frac{Z^3}{1 - Z^2} \, dZ
\]  

(26-C)

\[
= \frac{D_1^2}{S} \left( \frac{1}{2} \log_\varepsilon \frac{1}{1 - Z} - Z \right) \, dZ
\]

Integrating

\[
Q = \frac{D_1^2}{S} \left( \frac{1}{2} \log_\varepsilon \frac{1}{1 - Z} - \frac{Z^2}{2} \right)
\]

(29)
ARTICLE 9. PROFILE OF TRANSLATION WAVE

Waves of translation in rivers, in most cases, are invisible, because the phenomenon is too extended in horizontal direction, and the surface gradient is too small for the eye to see the flood propagating down the river as a wave; but in a diagram with the vertical scale much exaggerated showing the profile of the progressing flood wave it appears in the form of a moving ridge, which corresponds to the hydrographs plotted on an automatic gage record.

Hence, if a hydrograph is transferred to a distance-depth curve, it will obviously represent the profile of the passed flood wave; and on the other hand, if we can develop the complete profile of a flood wave whose crest has just reached a prediction station, not only the maximum gage height at the lower stations can be predetermined but also all the stages can be plotted before the flood arriving at there.

Referring to Fig. 10, the following notations will be used in deriving equations for theoretical profile of the flood wave:

\[ H = \text{head of the flood wave.} \]
\[ C = \text{Crest of the flood wave.} \]
\[ T = \text{Tail of the flood wave.} \]
\[ H_b, C_b & T_b = \text{Head, crest and tail of the flood wave projected on the river bottom.} \]
Fig. 10. Typical Profile of Translation Wave
$D_L$ = Depth at crest of the wave.
$D_H$ = Depth at head or tail of the wave.
$D$ = Depth at any section of the wave.
$H$ = Elevation of water surface above head of the wave, and
$\gamma$ = Elevation of water surface below tail of the wave.
$S$ = Slope of river bed.
$S_0$ = Friction slope corresponding to the given depth $D$.
$L$ = Horizontal length of the wave.
$dB = SL$ = Rise or fall of river bed.
$dL$ = Horizontal length between two consecutive sections.
$dD$ = Difference in depth between two consecutive sections.
$dH$ = Difference in elevation of water surface between two consecutive sections.
$dB$ = Difference in rise or fall of river bed between two consecutive sections.
$dF$ = Head required to overcome friction in distance $dL$.
$dK$ = Change of velocity head in distance $dL$.

In order to determine the velocity at various depth in the flood wave, equation (1) may be use of to advantage by changing $A$ to $D$ and $dA$ to $dD$, and thus it may be written,

$$\frac{dD}{D} = \frac{dV}{V_0 - V} \quad (1)$$

Integrating between the limits $D$ and $D_L$, and $V$ and $V_L$, we obtain,
\[
\log_e \frac{D}{D_1} = \log_e \frac{V_0 - V_1}{V_0 - V}
\]

or

\[
\frac{D}{D_1} = \frac{V_0 - V_1}{V_0 - V}
\]

From which

\[
V = V_0 - (V_0 - V_1)\frac{D}{D}
\]

By equation (11)

\[
V_0 = V_1 \frac{1 - y^{1+m}}{1 - y}
\]

or

\[
V_0 = K V_1
\]

Where

\[
y = \frac{D_2}{D_1}
\]

and

\[
K = \frac{1 - y^{1+m}}{1 - y}
\]

Then

\[
V = \left[ K - (K - 1)\frac{D_1}{D} \right] V_1
\]

By

\[
V_1 = CD_1^{m} S^{1/2}
\]

Therefore

\[
V = \left[ K - (K - 1)\frac{D_1}{D} \right] CD_1^{m} S^{1/2}
\]

This is a general formula for the velocity of non-uniform flow at any depth D between D_1 and D_2 in a flood wave.

If we assume S_o be the friction slope as before, then, we may write

\[
V = CD^{m} S_o^{1/2}
\]
and

\[ S_0 = \frac{-v^2}{c^2 D^{3m}} \]

\[ \frac{[K - (K - 1)D_1/D]^{2}D_1^{2m}S}{D^{2m}} \]

In addition to the friction head we have to consider

\[ dH_v = \frac{(V_F - V_E)^2}{2g} \]

the change of velocity head existing between F and E, Fig. 10. Then the total head consumed in a short distance \( dL \) is

\[ dH = dF + dH_v \quad (32) \]

From Fig. 10

\[ H = B + (D - D_2) \]

\[ = SL + (D - D_2) \]

Then,

\[ dH = SdL + dD \quad (33) \]

In equation (38) the value of \( dH_v \) is too small in comparison with that of \( dH \), hence for the sake of simplicity we may neglect the change of velocity head without serious error in computing the profile of such a long flood wave.

Therefore,

\[ dH = dF = S_0 dL \]

\[ = [K - (K - 1)D_1/D]^{2}(D_1/D)^{2m}SdL \quad (34) \]

Equating (33) and (34) we obtain
which gives
\[
\frac{dL}{dD} = \frac{1}{S \left[ K - (K - 1)\frac{D_1}{D} \right] \left( \frac{D_1}{D} \right)^{2m} - 1}
\] (35)

This is general differential equation for the profile in the front of the flood wave.

In the back part of the wave Fig. 10 shows
\[
H = B - (D - D_2) = SL - (D - D_2)
\]
\[
dH = SdL - dD
\] (36)

Equating (36) and (34) we obtain similarly,
\[
\frac{dL}{dD} = \frac{-1}{S \left[ K - (K - 1)\frac{D_1}{D} \right] \left( \frac{D_1}{D} \right)^{2m} - 1}
\] (37)

Let \( Z = D/D_1 \), \( D = D_1 Z \), and \( dD = D_1 dZ \)

Then equations (35) and (37) become
\[
\frac{dL}{dZ} = \frac{\frac{1}{D_1} \frac{Z^{2m+2}}{S \left[ KZ - (K - 1) \right]^{2m+2} - Z^{2m+2}}}dZ
\] (38)
(a) When \( m = 1/2 \)

\[
\frac{dL}{Z} = \frac{1}{S} \frac{Z^3 dZ}{[KZ - (K - 1)]^2 - Z^3} \quad (36-A)
\]

\[
= \frac{1}{S} \left\{ -1 + \frac{[KZ - (K - 1)]^2}{[KZ - (K - 1)]^2 - Z^3} \right\} dZ
\]

Expanding the numerator and factoring the denominator in the fraction we obtain

\[
dL = \frac{1}{S} \left\{ -1 + \frac{K^2Z^2 - 2K(K - 1)Z + (K - 1)^2}{(1 - Z)[Z^3 - (K^2 - 1)Z + (K - 1)^2]} \right\} dZ
\]

Assume

\[
\frac{K^2Z^2 - 2K(K - 1)Z + (K - 1)^2}{(1 - Z)[Z^3 - (K^2 - 1)Z + (K - 1)^2]} = \frac{a}{1 - Z} + \frac{bZ + c}{Z^3 - (K^2 - 1)Z + (K - 1)^2}
\]

According to the method of indeterminate coefficients find

\[
a = \frac{1}{3 - 2K},
\]

\[
b = \frac{1 - K^2(3 - 2K)}{3 - 2K} = \frac{(K - 1)^2(2K + 1)}{3 - 2K},
\]

\[
c = -\frac{2(K - 1)^3}{3 - 2K}
\]
Substituting in equation (39),

\[
\int dL = \frac{d1}{s} \left\{ -\int dZ + \frac{1}{3-2K} \int \frac{dZ}{1 - Z} \right. \\
\left. + \frac{(K-1)^2}{3-2K} \int \frac{(2K+1)Z - 2(K-1)}{Z^2 - (K^2-1)Z + (K-1)^2} dZ \right\}
\]

Integrating,

\[
L = \frac{d1}{s} \left\{ -Z + \frac{1}{3-2K} \log_e \frac{1}{1 - Z} \\
+ \frac{(K-1)^2(2K+1)}{2(3-2K)} \log_e [Z^2 - (K^2-1)Z + (K-1)^2] \\
+ \frac{(K-1)^2[(2K+1)(K+1) - 4]}{2(3-2K)(K+1)^2 - 4} \\
\log_e \frac{2Z - (K^2-1) - (K-1)(K+1)^2 - 4}{2Z - (K^2-1) + (K-1)(K+1)^2 - 4} \right\} + c
\]

(40)
(b) when $m = \frac{2}{3}$

$$dL = \frac{\frac{1}{2}D_1}{S} \frac{z^{10/3}}{[KZ - (K - 1)]^{2/3} - z^{10/3}} - dZ \quad (38-B)$$

The process of integration for this equation is too cumbersome. If it is desired, a direct step-method may be employed to obtain its solution. In doing this, equation (38-B) should be changed into the following form:

$$L_1 - L_2 = \frac{\frac{1}{2}D_1}{S} \left( \frac{z_1 - z_2}{(K - 1)^{2/3} - 1} \right) \left[ \frac{1}{2} \left( \frac{z_1 + z_2}{2} \right)^{4/3} - 1 \right]$$

$$= \frac{z_1 \pm z_2}{2} = Z \quad (41)$$

where $L_1 - L_2 = dL$, $z_1 - z_2 = dz$, and

$$\frac{z_1 \pm z_2}{2} = Z$$
(g) When \( m = 1 \)

\[
dL = \int_{-\infty}^{\infty} \frac{Z^4 dZ}{S \left[ KZ - (K-1) \right]^2 - Z^4} \tag{38-C}
\]

\[
= \int_{-\infty}^{\infty} \left\{ -1 + \frac{\left[ KZ - (K-1) \right]^2}{\left[ KZ - (K-1) \right]^2 - Z^4} \right\} dZ
\]

\[
= \int_{-\infty}^{\infty} \left\{ -1 + \frac{1}{2} \frac{KZ - K + 1}{KZ - (K-1) - Z^2} \right\} dZ
\]

Integrating,

\[
L = \int_{-\infty}^{\infty} \left\{ -Z - \frac{K}{4} \log_e \left[ (K-1) - KZ + Z^2 \right] \right. \\
- \frac{K^2 - 2K + 2}{4 \sqrt{K^2 - 4(K-1)}} \frac{2Z - K - \sqrt{K^2 - 4(K-1)}}{2Z - K + \sqrt{K^2 - 4(K-1)}} \\
+ \frac{K}{4} \log_e \left[ (K-1) - KZ - Z^2 \right] \\
+ \frac{K^2 + 2K - 2}{4 \sqrt{K^2 - 4(K-1)}} \frac{\sqrt{K^2 - 4(K-1)} - 2Z + K}{\sqrt{K^2 - 4(K-1)} + 2Z - K} \right\}
\]
Simplifying,
\[
L = \frac{\frac{1}{S}D_1}{S} \left\{ -Z + \frac{K (K-1) - KZ - Z^2}{4 \log_e (K-1) - KZ + Z^2} \right\}
\]
\[
+ \frac{K^2}{2(2-K) Z - (K-1)} \log_e \frac{1 - Z}{Z - (K-1)} \right\} + c \quad (42)
\]

For an example equation (40) may be used to compute the profile of a flood wave. Let us assume the data as follows:

\[D_1 = 20', \quad D_2 = 10', \quad \text{and} \quad S = 2 \text{ ft. per mile}\]

Then,
\[y = \frac{D_2}{D_1} = \frac{10}{20} = 0.50, \quad \text{for which Fig. 5 gives,}\]

\[K = 1.294\]

Substituting the value of \(K\) in the coefficients in equation (40), we obtain,
\[
L = \frac{\frac{1}{S}D_1}{S} \left\{ -Z + (2.427 \log_e \frac{1}{1 - Z} \right\}
\]
\[
+ \frac{0.376 \log_e (Z^2 - 0.674Z + 0.086)}{2Z - (1.014)} \right\} + c
\]
\[\text{Or,} \quad L = \frac{\frac{1}{S}D_1}{S} \left[ g(Z) \right] + c \quad \text{(40-A)}\]

Taking origin at certain depth near the head of the wave, such as \(D = 10.1'\). Then, \(L = 0\) when \(Z = 10.1/20 = 0.505\).
Substituting $L = 0$ and $Z = 0.505$ in equation (40-A) and by using a natural-logarithm table we obtain,

$$c = -\frac{\frac{D}{x}}{S} (6.744)$$

Therefore,

$$L = \frac{\frac{D}{x}}{S} [\phi(z) - 6.744]$$

$$= \frac{\frac{D}{x}}{S} [\phi(z) - 6.744]$$

$$= \frac{D}{10} [\phi(z) - 6.744] \text{ in miles}$$

By the above equation the length at various depth is computed in Table 4 and the profile of the front portion of the flood wave is then plotted in Fig. 11. The profile shows that the length becomes infinite when either $D = 10'$ or $D = 20'$; this is due to the fact that the water surface at both the maximum and normal stages is parallel to the river bed.
TABLE 4

PROFILE OF TRANSLATION WAVE

<table>
<thead>
<tr>
<th>D</th>
<th>Z</th>
<th>log_eA</th>
<th>log_eB</th>
<th>log_eC</th>
<th>( \phi(Z) )</th>
<th>L</th>
<th>Miles</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10.1</td>
<td>0.505</td>
<td>1.706</td>
<td>7.403</td>
<td>8.140</td>
<td>6.744</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>11</td>
<td>0.55</td>
<td>1.938</td>
<td>8.468</td>
<td>9.185</td>
<td>9.041</td>
<td>2.297</td>
<td>22.97</td>
</tr>
<tr>
<td>12</td>
<td>0.60</td>
<td>2.224</td>
<td>8.800</td>
<td>9.418</td>
<td>9.842</td>
<td>3.098</td>
<td>30.98</td>
</tr>
<tr>
<td>13</td>
<td>0.65</td>
<td>2.548</td>
<td>9.000</td>
<td>9.537</td>
<td>0.362</td>
<td>3.618</td>
<td>36.18</td>
</tr>
<tr>
<td>14</td>
<td>0.70</td>
<td>2.922</td>
<td>9.149</td>
<td>9.613</td>
<td>0.984</td>
<td>4.240</td>
<td>42.40</td>
</tr>
<tr>
<td>15</td>
<td>0.75</td>
<td>3.364</td>
<td>9.269</td>
<td>9.667</td>
<td>1.550</td>
<td>4.806</td>
<td>48.06</td>
</tr>
<tr>
<td>16</td>
<td>0.80</td>
<td>3.905</td>
<td>9.368</td>
<td>9.705</td>
<td>2.178</td>
<td>5.434</td>
<td>54.34</td>
</tr>
<tr>
<td>17</td>
<td>0.85</td>
<td>4.605</td>
<td>9.456</td>
<td>9.738</td>
<td>2.949</td>
<td>6.205</td>
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</tr>
<tr>
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<td>5.588</td>
<td>9.514</td>
<td>9.761</td>
<td>3.963</td>
<td>7.219</td>
<td>72.19</td>
</tr>
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<td>0.95</td>
<td>7.280</td>
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<td>9.782</td>
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</tr>
<tr>
<td>19.9</td>
<td>0.995</td>
<td>12.860</td>
<td>9.662</td>
<td>9.798</td>
<td>11.325</td>
<td>14.581</td>
<td>145.81</td>
</tr>
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<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

NOTE:–

\[
(2.427)\log_e A = \frac{1}{1-Z} \\
(0.376)\log_e B = (0.376)\log_e (Z^2 - 0.674Z + 0.086) \\
(0.395)\log_e C = \frac{2Z - 1.014}{2Z - 0.344}
\]
Fig. 11. Computed Profile of Translation wave
ARTICLE 10. DISCHARGE OF TRANSLATION WAVE

Similarly to Art. 8, by taking the river one foot wide the quantity of water in a short length $dL$ passing a given section is,

$$dQ = DdL = D_1ZdL, \quad Z = D/D_1$$

Since,

$$dL = \frac{\frac{dP}{dL}}{S} \left[ \frac{Z^{m+2}}{(KZ - (K-1))^{m+2}} \right] dZ$$

Therefore,

$$dQ = \frac{\frac{dP}{dL}}{S} \frac{Z^{m+3}}{(KZ - (K-1))^{m+3}} dZ$$

(a) When $m = 1/2$ equation (43) may be written,

$$dQ = \frac{\frac{dP}{dL}}{S} \frac{Z^4}{(KZ - (K-1))^{3/2}} dZ$$

Decomposing,

$$dQ = \frac{\frac{dP}{dL}}{S} \left\{ -K^2 dZ - Z dZ + \frac{2-K^2}{3-2K} \frac{dZ}{1-Z} \right\}$$

Integrating,

$$Q = \frac{\frac{dP}{dL}}{S} \left\{ -K^2 Z - \frac{Z^2}{2} + \frac{2-K^2}{3-2K} \log_e \frac{1}{1-Z} \right\}$$

$$- \frac{(K-1)^3}{3-2K} \log_e \frac{Z^2 - (K^2-1)Z - (K-1)^2}{3-2K}$$
\[
\frac{K(K-1)^3}{(3-2K)(K+1)^2-4} \log_e \frac{2Z - (K^2-1) + (K-1)\sqrt{(K+1)^2-4}}{2Z - (K^2-1) - (K-1)\sqrt{(K+1)^2-4}} + c
\]

(44)

(b) When \( m = 2/3 \) equation (43) becomes a more complicated form than equation (38-B), p 49. It is also simpler to use the step method to determine the quantity \( Q \), expressed as follows:

\[
dQ = D(L_1 - L_2) = D_1 Z(L_1 - L_2)
\]

By

\[
L_1 - L_2 = \frac{4D_1}{S} \frac{(z_1 - z_2)}{\left[ \frac{K - (K-1)}{2} \right]^2 1 - \left( \frac{z_1 + z_2}{2} \right) \left( \frac{z_1 + z_2}{2} \right)^{4/3} - 1}
\]

(41)

and

\[
Z = \frac{z_1 + z_2}{2}
\]

\[
dQ = \frac{4D_1^2}{S} \frac{(z_1 + z_2)}{\left[ \frac{K - (K-1)}{2} \right]^2 1 - \left( \frac{z_1 + z_2}{2} \right) \left( \frac{z_1 + z_2}{2} \right)^{4/3} - 1}
\]

(45)
(c) When \( m = 1 \)

\[
dQ = \frac{4D_1^2}{s} \frac{Z^5}{s} \left[ KZ - (K-1) \right]^2 - Z^4 \, dZ \tag{43-c}
\]

Separating,

\[
dQ = \frac{4D_1^2}{s} \left\{ \left( \frac{K^2 - K + 1}{2[KZ - (K-1) - Z^2]} \right) \right\} \frac{Z}{dZ}
\]

Integrating,

\[
Q = \frac{4D_1^2}{s} \left\{ \frac{Z}{2} + \frac{K^2 - K + 1}{4} \right\} \log_e \left[ (K-1) - KZ + Z^2 \right]
\]

\[
- \frac{K[K^2 - 3(K-1)]}{4 \sqrt{K^2 - 4(K-1)}} \frac{2Z - K + \sqrt{K^2 - 4(K-1)}}{2Z - K + \sqrt{K^2 - 4(K-1)}}
\]

\[
- \frac{K^2 + K - 4(K-1)}{4} \log_e \left[ (K-1) - KZ - Z^2 \right]
\]

Simplifying,

\[
Q = \frac{4D_1^2}{s} \left\{ \frac{Z}{2} + \frac{K^2 - K + 1}{4} \right\} \log_e \left[ (K-1) - KZ + Z^2 \right]
\]

\[
+ \frac{K^2 + K - 4(K-1)}{4} \frac{2Z - K + \sqrt{K^2 - 4(K-1)}}{2Z - K + \sqrt{K^2 - 4(K-1) + 2Z - K}}
\]

\[
+ c \tag{46}
\]
CHAPTER III

VELOCITY OF FLOOD WAVES

ARTICLE 11. EXAMPLES OF VELOCITY OF FLOOD WAVES

Theorically, the rate of propagation of a flood wave varies directly with the difference in the rates of discharge and inversely with the difference in the stages of the channel, however, it is more or less modified by other conditions in the river.

From old experience the velocity of a flood wave is somewhat greater than that of the flood flow, when the maximum stage of the river is moderate or just bank-full, the acceleration is certainly due to the action of wave translation. While it is much slower when the flood overflows river's banks, the retardation is presumably due to temporarily valley storage.

Furthermore, the wave motion is subject to changing its velocity at different portion of the river. When such a condition is happened the shape of the flood wave becomes very complicated and its rate of propagation can not be determined with direct application to the mathematical treatment.

The following table shows several examples of velocity of flood waves in various rivers. They vary greatly owing to the different conditions, which they occurred.
### TABLE 5

**EXAMPLES OF VELOCITY OF FLOOD WAVES**

<table>
<thead>
<tr>
<th></th>
<th></th>
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<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Mississippi</td>
<td>Cairo</td>
<td>Mumphis</td>
<td>230</td>
<td>60</td>
<td>3.80</td>
<td>(1)</td>
</tr>
<tr>
<td></td>
<td>Mumphis</td>
<td>Vicksburg</td>
<td>370</td>
<td>110</td>
<td>3.40</td>
<td>(2)</td>
</tr>
<tr>
<td></td>
<td>Cairo</td>
<td>Mumphis</td>
<td>230</td>
<td>120</td>
<td>1.90</td>
<td>(3)</td>
</tr>
<tr>
<td></td>
<td>Mumphis</td>
<td>Vicksburg</td>
<td>370</td>
<td>220</td>
<td>1.70</td>
<td>(4)</td>
</tr>
<tr>
<td>Black</td>
<td>Cathage Dam</td>
<td>Lyon's Fall</td>
<td>32</td>
<td>2</td>
<td>16</td>
<td>(5)</td>
</tr>
<tr>
<td>Conemaugh</td>
<td>South Fork</td>
<td>Johnstown</td>
<td>13</td>
<td>.68</td>
<td>19</td>
<td>(6)</td>
</tr>
<tr>
<td>Tennessee</td>
<td>Kingston</td>
<td>Chattanooga</td>
<td>95</td>
<td>76.3</td>
<td>1.25</td>
<td>(7)</td>
</tr>
<tr>
<td>South Platte</td>
<td>Kersey</td>
<td>Julesburg</td>
<td>143</td>
<td>165</td>
<td>.87</td>
<td>(8)</td>
</tr>
<tr>
<td>Loire</td>
<td>-----</td>
<td>-----</td>
<td>---</td>
<td>---</td>
<td>2.5 to 4</td>
<td>(9)</td>
</tr>
<tr>
<td>Marne</td>
<td>-----</td>
<td>-----</td>
<td>---</td>
<td>---</td>
<td>1.6 to 2</td>
<td>(10)</td>
</tr>
<tr>
<td>HaiHo, China</td>
<td>North Fort</td>
<td>Red Bridge</td>
<td>40</td>
<td>4.0</td>
<td>10.0</td>
<td>(11)</td>
</tr>
<tr>
<td></td>
<td>Red Bridge</td>
<td>North Fort</td>
<td>40</td>
<td>5.3</td>
<td>7.5</td>
<td>(12)</td>
</tr>
</tbody>
</table>

**Remarks:**

- Flood stage within banks, (1) and (2).
- Flood stage overflowing banks, (3) and (4), and (7).
- Flood caused by failure of reservoir, (5) and (6).
Remarks:-

Flood discharge decreases as it is propagating down the stream, due to diversions, (8).

Average values for high and for low floods, (9) and (10).

Rising tidal wave, (11).

Falling tidal wave, (12).

References of above examples:-

(1), (2), (3), and (4) Thomas and Watt, The improvement of Rivers, part I, p. 34.


(9) and (10) Thomas and Watt, The Improvement of Rivers, part I, p. 34, Original Reference "Rivieres a courant libre, p. 65."

(11) and (12) Harbor Engineering by Professor H. A. Petterson, Peiyang University, Tientsin, China, part IV, Plate II, Data obtained from HaiHo Conservancy Commission Report, 1914.
ARTICLE 12. OBSERVATIONS ON VELOCITY OF FLOOD WAVES IN THE IOWA RIVER

During the summer in 1924, there were occurred several floods in the Iowa River, Iowa. These floods are of good examples of simple waves, propagating down the river without interference from other bodies of waters, as the Iowa River has no large branches in the portion between Marengo and Iowa City. The two big floods shown in Figs. 12 to 15, were chiefly caused by the excessive rainfall over the valley of Bear Creek, which enters the main river just above the city of Marengo. The overflow of the first flood from this creek swept the whole town of Marengo and the neighboring countries in a tract of more than 3 miles. The Iowa River was of course overtopped in the reach below Marengo, however the flood came down still confined in banks in the lower reaches due to increased slope and larger capacity of the channel. The time of maximum flooding recorded at Marengo was 3:00 P. M., June 28, 1924 (see Marengo Republican News, July 3, 1924). The time of the peak at other stations is shown in Figs. 12 to 15.

The flood waves at Amana was secured from Mr. F. A. Koch, who has been interested in recording floods since 1912. The datum he used in recording floods is the top of a railroad tie on the bridge across the river near Middle Amana. He assumed this datum to be an elevation of 10 feet.
Fig. 12-B. Flood Waves of Iowa River At S. U. I. Gage Station Iowa City, Iowa
Fig. 12-C Flood Waves of Iowa River At S.U. I. Gage Station Iowa City, Iowa
Fig. 13A: Flood Waves of Iowa River At S.U.I. Dam Iowa City, Iowa
Fig. 13-B. Flood Waves of Iowa River at S.U.I. Dam, Iowa City, Iowa.
Fig. 14-C. Flood Waves of Iowa River at United Power Co. Dam Coralville, Iowa
Fig. 15. Flood Waves in the Iowa River at Amana, Iowa.
In order to determine the velocity of these two flood waves, we observed. The time of peaks and distance between stations are then tabulated in the following table.

**TABLE 5**

**DATA FOR COMPUTING THE VELOCITY OF FLOOD WAVES IN THE IOWA RIVER 1924**

<table>
<thead>
<tr>
<th>No. of Flood</th>
<th>Station</th>
<th>Time of Peak</th>
<th>Interval Hrs</th>
<th>Dist. Miles</th>
<th>Ave. Vel. Mi/Hr</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st.</td>
<td>Marengo</td>
<td>June 28, 3:00 P. M.</td>
<td>14.7</td>
<td>10</td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>Amana</td>
<td>June 29, 5:40 A. M.</td>
<td>48.8</td>
<td>53.5</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td>Iowa City</td>
<td>July 1, 6:30 A. M.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2nd.</td>
<td>Amana</td>
<td>July 25, 4:00 P. M.</td>
<td>50.0</td>
<td>53.5</td>
<td>1.07</td>
</tr>
<tr>
<td></td>
<td>Iowa City</td>
<td>July 27, 6:00 P. M.</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

From above table we see that the velocity of the flood waves in the Iowa River is rather slow in comparison with that shown in Table 4. This is due to the change of flood area which spreads to a wide tract in the upper portion and is confined to the normal channel section in the lower portion. Hence in determining the velocity of flood waves, the direct
application of equation (3) will not bring out the exact values as those shown in Table 5.

From Fig. 12-A the maximum stage attained by the first flood at Iowa City is 54.4, Iowa City datum, for which Fig. 2-B gives the maximum discharge \( q_1 = 21600 \) c.f.s. and the cross-section area \( A_1 = 4580 \) sq. ft. The low water stage before the flood is 48.7 for which the corresponding discharge and area are \( q_2 = 8000 \) c.f.s. and \( A_2 = 2800 \) sq. ft. respectively.

The maximum discharge and the low water area may be assumed to have the same values of those at Iowa City. But the cross-section occupied by the peak stage is much greater than that at Iowa City. From Mr. Koch's information the typical cross section of the Iowa River at Amana is plotted with careful judgement, as shown in Fig. 16. The maximum area at peak stage is then computed to be \( A_1 = 17200 \) sq. ft.

Now if we use \( A_1 = 17200 \), \( q_1 = 21600 \), and \( A_2 = 2800 \), \( q_2 = 8000 \), substituting in equation (3), it gives,

\[
V_0 = \frac{q_1 - q_2}{A_1 - A_2} = \frac{21600 - 8000}{17200 - 2800} = 0.912 \text{ ft./sec.}
\]

\[
= 0.66 \text{ mi./hr.}
\]

which is low in comparison to the observed average velocity,

\[
V_0 = 1.10 \text{ mi./hr.}
\]

If we use \( A_1 = 4580 \) instead of \( A_1 = 17200 \) and substitute in equation (3), we obtain,
Fig. 16. Typical Cross-Section of the Iowa River at Amana
\[
\frac{21600 - 8000}{4580 - 3800} \times \frac{4}{2800} = 7.64 \text{ ft./sec.}
\]

\[
= 5.20 \text{ mi./hr.}
\]

which is too high than that of the average, even the mean of these two values is still not close enough to the actual observed velocity.

Suppose we take 3 more sections at an equal distance between Amana and Iowa City. Assuming the area at each section to vary in a straight line ratio, we compute each area as placed in Table 6. Then the velocity \( V_0 \) between each two sections is determined in two ways by equation (3), the values of \( q_1, q_2 \) and \( A_2 \) being the same as before. In the first method \( V_0 \) is calculated separately for each section and mean \( V_0 \) is then determined for the two adjacent sections. While in the second method the average area \( A_1 \) is first computed for each two sections and \( V_0 \) is determined next. In both methods the time of travel from section to section is computed by dividing one-fourth of the total distance by \( V_0 \):

\[
\frac{53.5}{4} \div \frac{13.4}{V_0} = \frac{53.5}{4V_0} = \frac{13.4}{V_0}
\]

From Table 6 we then determine the average of \( V_0 \) for the whole reach:

(1) by first method

\[
\frac{53.5}{45.5} = \frac{53.5}{45.5} = 1.18 \text{ mi./hr.}
\]
(2) by second method

\[
\frac{53.5}{46.8} = 1.14
\]

The latter is closer to the observed value.

The computations for the velocity of the second flood wave are similarly arranged in Table 7. The average \( V_0 = 1.12 \text{ mi./hr.} \), and \( 1.11 \text{ mi./hr.} \) by the first and second method respectively.
TABLE 6

COMPUTATIONS FOR VELOCITY AND TIME OF TRAVEL
OF FLOOD WAVE IN THE IOWA RIVER
1924 FIRST FLOOD

<table>
<thead>
<tr>
<th>Section</th>
<th>A(_1) Sq. Ft.</th>
<th>(V_0) Mi./Hr.</th>
<th>Ave. (V_0) Mi./Hr.</th>
<th>Time Hrs.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Amana</td>
<td>17200</td>
<td>0.66</td>
<td>0.74</td>
<td>18.1</td>
</tr>
<tr>
<td>1</td>
<td>14045</td>
<td>0.82</td>
<td>0.98</td>
<td>13.7</td>
</tr>
<tr>
<td>2</td>
<td>10890</td>
<td>1.14</td>
<td>1.51</td>
<td>9.9</td>
</tr>
<tr>
<td>3</td>
<td>7735</td>
<td>1.88</td>
<td>3.54</td>
<td>3.8</td>
</tr>
<tr>
<td>Iowa City</td>
<td>4580</td>
<td>5.20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td>45.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Section</th>
<th>(A_1) Sq. Ft.</th>
<th>Ave. (A_1) Sq. Ft.</th>
<th>(V_0) Mi./Hr.</th>
<th>Time Hrs.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Amana</td>
<td>17200</td>
<td>15622</td>
<td>0.72</td>
<td>18.6</td>
</tr>
<tr>
<td>1</td>
<td>14045</td>
<td>12465</td>
<td>0.96</td>
<td>14.0</td>
</tr>
<tr>
<td>2</td>
<td>10980</td>
<td>9312</td>
<td>1.42</td>
<td>9.4</td>
</tr>
<tr>
<td>3</td>
<td>7735</td>
<td>6158</td>
<td>2.76</td>
<td>4.8</td>
</tr>
<tr>
<td>Iowa City</td>
<td>4580</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td>46.8</td>
</tr>
</tbody>
</table>

\[ V_0 = \frac{q_1 - q_2}{\frac{A_1 - A_2}{A_1}} = \frac{21600 - 8000}{A_1 - 2800}, \quad t = \frac{13.4}{V_0} \]
TABLE 7

COMPUTATIONS FOR VELOCITY AND TIME OF TRAVEL
OF FLOOD WAVE IN THE IOWA RIVER

1924 SECOND FLOOD

<table>
<thead>
<tr>
<th>Section</th>
<th>A1 Sq. Ft.</th>
<th>V0 Mi./Hr.</th>
<th>Ave. V0 Mi./Hr.</th>
<th>Time Hrs.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Amana</td>
<td>6200</td>
<td>0.67</td>
<td>0.70</td>
<td>19.1</td>
</tr>
<tr>
<td>1</td>
<td>5415</td>
<td>0.83</td>
<td>0.97</td>
<td>13.8</td>
</tr>
<tr>
<td>2</td>
<td>4630</td>
<td>1.11</td>
<td>1.39</td>
<td>9.6</td>
</tr>
<tr>
<td>3</td>
<td>3845</td>
<td>1.68</td>
<td>2.49</td>
<td>5.4</td>
</tr>
<tr>
<td>Iowa City</td>
<td>3060</td>
<td>3.32</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- Total 47.9

<table>
<thead>
<tr>
<th>Section</th>
<th>A1 Sq. Ft.</th>
<th>Ave. A1 Sq. Ft.</th>
<th>V0 Mi./Hr.</th>
<th>Time Hrs.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Amana</td>
<td>6200</td>
<td>5808</td>
<td>0.74</td>
<td>18.1</td>
</tr>
<tr>
<td>1</td>
<td>5415</td>
<td>5022</td>
<td>0.95</td>
<td>14.1</td>
</tr>
<tr>
<td>2</td>
<td>4630</td>
<td>4238</td>
<td>1.33</td>
<td>10.1</td>
</tr>
<tr>
<td>3</td>
<td>3845</td>
<td>3452</td>
<td>2.22</td>
<td>6.0</td>
</tr>
<tr>
<td>Iowa City</td>
<td>3060</td>
<td>3480</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- Total 48.3

\[ V_0 = \frac{q_1 - q_2}{A_1 - A_2} = \frac{9450 - 5600}{A_1 - 2270} \]

\[ t = \frac{13.4}{V_0} \]
CHAPTER IV
FLOOD PROBLEM

ARTICLE 13. GENERAL CONSIDERATIONS

The most difficult problem an engineer can be called upon to solve is to predict the probable intensity, magnitude and frequency of maximum floods which may take place in future times. The security of engineering projects and especially hydraulic works affected more or less by stream flow and floods is often dependent, for their proper designs, upon a knowledge of the intensity, magnitude and frequency of floods which must be provided for. Engineers planning those projects lack of such knowledge will be constantly encountered with difficulty standing in the way of obtaining satisfactory results. The consequences of under estimating of flood intensities and magnitudes, or of improper designs become more serious and involve great damages of property and losses of life on the contingency of calamitous floods. On the other hand, an imagination of exceptionally great flood will of course require works too expensive which may not be warranted by local conditions.

In planning works necessary to mitigate or prevent inundation or floods, the engineer must constantly keep in
mind the economic conditions. No undertaking will be warranted unless, first the benefits from the improvement exceed in value the cost of expenditure involved, second the benefited communities are able to raise the funds to meet the expense, and third the probable flood loss in the future is great.
ARTICLE 14. FACTORS AFFECTING FLOODS

It is obvious that all floods are produced by storms, and their magnitudes are more or less affected by the combination of other circumstances, the principals are intensity and amount of precipitation; character and conditions of watershed; and meteorology of the region. Each of these primary causes may be subdivided into several minor causes which will be discussed separately.

PRECIPITATION. - Precipitation is the principal cause, which produces floods, as we cannot have floods without precipitation over a part or whole drainage area of a given stream. The important factors of precipitation to be considered in the study of floods are as follows:

(1) Amount, intensity and duration of rainfall.
(2) Distribution of rainfall.
(3) Direction of storm.

(1) The amount of precipitation depends upon both the intensity and duration of either rainfall or snow. Experience shows that the intensity varies inversely as duration. However, the total amount of precipitation increases with the increase of duration; while the maximum rate of flood run-off usually depends greatly upon the intensity of rainfall.

(2) All maximum floods of large rivers are almost produced by heavy storms of wide distribution, as every
portion of the main river will be raised by the excessive runoff from tributaries, especially if the storms occur at such times and places that the flood waves travelling down coincide in the main stream. On the other hand, a highly concentrated cloudburst is the only cause which produces intense floods in small streams.

(3) The direction of storm plays another effect upon the magnitude of floods. When a storm moves in the direction of the flow of a long stream and at a velocity equal to the rate of propagation of the flood wave created by a preceding storm or by the present one, the peak of the flood at certain point in the lower course would be several times greater than if the storm progresses in the reversed direction. If it moves across the axis of the drainage area, it commonly extends over a small portion of the area and passes in a short duration, and consequently the amount of precipitation over the area is very small and no freshet will be felt in the down stream, except in the vicinity in the path of the storm.

WATERSHED.- The factors related to the type and condition of watershed, which have a marked effect on the magnitude of floods are:

(1) Size.

(2) Shape, arrangement and density of tributaries.

(3) Topographic features.

(4) Surface and under ground conditions.
(5) Vegetations and forests.
(6) Storage conditions.
(7) Artificial works.

(1) The size of watershed, therefore has an important effect upon the magnitude of flood. A small area which becomes the center of a heavy storm will receive a greater intensity of rainfall, and consequently furnish a relatively greater rate of flood runoff. In a large drainage area the occurrence of being entirely covered with a intense precipitation is very rare, hence the larger the drainage area the lesser will be the relative magnitude of the flood.

(2) The shape of the watershed and the arrangement of tributaries also effect the magnitude of flood. In the fan arrangement of tributaries, when the flood waves from these tributaries come down simultaneously to the main stream, they often cause serious congestion and extreme flood conditions at places below their common junction. While in a fern leaf arrangement of tributaries, the flood waters come from tributaries in the lower course will be carried away early before those come from upper ones, thus the flood wave may have several peaks with comparatively smaller heights, provided that the other conditions are the same. The network arrangement of small streams and artificial ditches brings the surface flow more directly and quickly into the main stream and thus tends to increase the flood height, especially when it happens
in the first case.

(3) Mountainous districts favor conditions of cloud-burst, because the temperature at high elevations is lower than that at low elevations. When currents of moist air striking the slopes are deflected up become cooled, and the water vapor is condensed and becomes downpour on the windward side. This process, if the hills are not lofty, may not produce its full effect till the air currents have passed over the hills, and thus the rainfall on the leeward slope may produce a flood greater than elsewhere, but in inner and more lofty ranges the rainfall and floods are generally greatest on the windward side.

Topography of drainage area has another effect on the quantity and rate of runoff. Flat areas are slowly drained during storm, thus each field becomes a temporary detention basin resulting great damages to agricultures. On the other hand, the steep hilly slopes deliver rain water so rapidly to the creeks and valleys which in turn give up their waters to the main stream. A mountainous flood may do some damages as well as a big one destroying bridges and other structures that it encountered in its way.

(4) Planted areas with pervious soil on flat slopes retard a great amount of rainfall on the surface, which then finds its way into the permeable stratum and becomes ground water or subterraneous flow. Hence they have a high degree
of reducing both the rate and quantity of flood flow. While they lose their effect when they are saturated by moisture due to long continuation of rainfall. Cultivation loosens the compactness of soil and tends to increase the absorbent power of soil, thus it is frequently favorable to flood regulation.

(5) The presence of vegetation and especially of forests prevents rapid removal of rain water on hilly slopes during small storm. The leaves of trees and humus formed from leaves, etc., retain moisture acting like a reservoir, so that the runoff takes place slowly and the denudation and erosion of soil is checked. On the other hand, the temperature in forests is much cooler than that in the open air, thus, when mountains and hills are covered with deep forests, they are more favorable to the formation of orographic storms described under the third factor of watershed. Forests tend to prevent early melting of accumulated snow. During the spring when warm rains fall on snow, the condition may become more dangerous, as not only the rain water but also the melting snow will find its way to the swollen channel below. Therefore, forests can not be depended as a regulator of flood flow. However, they have an important effect on preventing denudation and checking erosion on steep slopes of certain character, but it is indirect and limited to some particular places.
Natural storage in lakes, ponds, and swamps has a marked retarding effect upon floods. When streams, in flood time, flowing into a lake produce only a slight rise over the whole area of the lake, thus preventing rapid discharge in the river below.

Ground storage produces same effect as surface storage. Drainage areas possessing extensive pervious deposits, absorb rapidly the rainfall which becomes a part of the ground water and flows slowly into the stream, thus they reduce the excessive runoff from the surface.

Artificial storage constructed for various purposes, particularly for flood control, has greatest effect on reducing maximum rate of flood runoff. Since it is artificially and purposely constructed its value and degree of security of preventing floods are, of course, far excess than those of natural factors. However, when a storage is introduced on a drainage area and used purely for flood control purpose its effort may be more or less apparent due to proper or improper location and design of the dam. A storage reservoir or a retarding basin ceases to be any protection if its capacity is not great enough and the flood continuous too long to fill it up to such a height that the discharge at the outlet is equal to the entire influent. A reservoir in good service may be generally considered to have its influence in controlling floods at places below its site to a distance covering an un-
protected area equal to twice of controlled area above the site, assuming a general similarity of watershed, as a reduction of flood flow by one third of its peak discharge will decrease its destruction. In the case of a flood caused by failure of reservoir dams, it is not due to the unreliability of the action of the reservoir, but it is due to the falseness in its designs.

(7) Aside from storage reservoirs there are many artificial works which have influences upon flood heights. By the channel improvement works such as lowering the bed, enlarging the cross-section, or straightening the channel, the discharge capacity is increased and thereby flood stages are reduced. In the diversion works flood heights are decreased in one place and increased in the other provided when no harm is done by such an increment of head. Levees, dikes and other protection works do increase the flood heights to some extent, but when sufficient head is provided no damage will be caused. The construction of dams, bridge piers and abutment, filling of low lands and encroachments on banks, while not intended to interfere with floods, often tend to produce higher flood levels.
METEOROLOGICAL CONDITIONS.- The important factors include:

(1) Temperature.
(2) Atmospheric Movement.
(3) Evaporation.

(1) Temperature has a marked effect upon winter floods. Snow and ice formed on the drainage area are stored up during winter season, especially in high altitude and low temperatures. When spring comes with its consequent increase in temperature which causes sudden melting of such storage of the accumulated snow and ice, and produce floods without accompanying adequate precipitation. The most important effect is that of frost which destroys much of vegetation and covers the ground with an impenetrable coating. A relative small amount of precipitation will result an excessive flood flow due to the frozen condition of the ground. If the cold is very excessive, the frozen weather has a further effect in temporarily arresting all flow. On the other hand, high temperatures, especially accompanied by wind movements, favor evaporation and consequently reduce flood flow.

Atmospheric movements and cyclonic storms are favorable to produce precipitation and floods. The relation of their path to the position and size of watershed under consideration has an important influence and modification of flood
flow, as mentioned in foregoing factors.

Evaporation varies with the locality, the season of the year, the state of the water, the condition of the air and the character of the soil on which the rain falls. Evaporation plays a great effect upon surface water after a sudden shower in the summer day. The rainfall is then returned rapidly to the atmosphere by evaporation produced by reshining of the sun and carried away by blowing winds to the other place. And thus it reduces the surface runoff.
ARTICLE 15. FLOOD AND RAINFALL RECORDS

In predicting the probable maximum future floods which may be expected from a given district, it is necessary that all the flood records and stream data should be collected and investigated. Especially, the particular ones should be thoroughly studied, providing that they covered a sufficiently long period and included all the unfortunate events that are likely to be happened in the future.

The long continuous stream records are always the best data to be used as a basis for predicting future floods. Private notes describing the height and time of particular flood or floods, kept by people who was interested in floods are the another useful source of informations. Sometime the height of the greatest past flood can often be traced with considerable accuracy from old people's memory or from the remaining water-marks on bridge piers or structures in neighborhood of the river course. Old newspapers often give the time of occurrence of threatening floods in certain places, but they usually do not define their exact heights. It is also of great help to make an examination on flood records obtained elsewhere, especially those covering a long period of time and under similar climatic conditions as a guide or reference for estimating floods of the stream in question.

It is usually required not only to determine the
heights, but also the discharges of the past floods. This may be calculated without considerable error by equation (15)

\[ Q = AV = A(CD^mS^n) \]

providing that the channel conditions have not greatly changed. Thus, having determined the probable flood height \( D \) it is an easy matter to ascertain the probable flood discharge \( Q \).

Rainfall records of most streams are more complete and cover longer periods than river records. A thorough analysis of all available rainfall and runoff data will give more accurate judgement of the probable maximum flood rate than if collected flood data, alone, are relied upon, for the greatest rainfall in a given time do not always produce the greatest flood due to the fact that other circumstances may not coincide with occurrence of the storm. The intensity, duration and distribution of rainfall producing great floods should be plotted and studied in connection with runoffs of the resulting floods. The relations between rainfalls and runoffs should be determined as closely as possible, so that comparison of the recorded storms can be made with the most intense storms of the neighboring streams, which may be expected to take place in the given stream in question, and will then afford as a basis for estimating probable extreme future flood.

In streams, where no flood records available, the rainfall statistics becomes the only source of data which can be
used to accomplish in securing a significant idea as to what are likely to the probable maximum floods'.
ARTICLE 16. FLOOD FREQUENCY

It is also necessary important to study the frequency of the particular floods and their damages. In some situations expensive construction of unimportant works provided for occasional maximum floods is usually unwarranted by economic conditions. The damages caused by unfrequent floods upon such works may not seriously affect public benefit and common wealth. While, the extra cost of more extensive and permanent construction of a capacity suitable for extreme conditions, if set aside at interest would raise a sinking fund which will be sufficient to pay the damages and cost of their reconstructions. Commonly the flood which will probably occur once in ten, fifteen, twenty-five or fifty years is used as a basis of designing these works according to their importance of condition and costs of construction.

In the protection of large districts where the failure of protective works may involve great sacrifice of human life with enormous property losses, greater safety must be provided, the frequency of great floods becomes more or less unimportant. However, a knowledge of the probabilities of their occurrences would also bring an idea as to what extent or interval of years the particular flood may be expected.

In general small floods occur more frequently than large ones, however not with regular intervals but as an aver-
Since hydraulic data of most streams in the world are not sufficiently long enough, there are floods still greater than past, that will probably occur in unknown periods. It will not be sufficient to provide for the greatest known past flood, but the extent of the increasing tendency of the future floods must be determined.

Mr. Fuller has collected available flood records of all rivers in United States, and reduced their various magnitudes in ratios to the average annual floods on the respective streams. These records have then been considered and added together as a continuous flood record on one stream covered a period of about 17 centuries. From this record he has derived a formula for the probable frequency of various flood ratios as in the following:

\[ \frac{Q_{\text{max}}}{Q_{\text{ave}}} = 1 + (0.8 \log T) \]

in which
- \( Q_{\text{max}} \) = maximum flow at flood peak;
- \( Q_{\text{ave}} \) = average yearly flood;
- \( T \) = number of years in the period considered.

This formula is applicable to any stream under ordinary conditions, and it is of especially useful for determining
the flood ratios of a river, where flood record is not long enough, however, the values of the estimated probable floods are dependent upon the accuracy of determining the average flood. Hence, the number of years available for the average may considerably affect the result. The longer the record the more accurate the average.

In a river, where a record of more than 50 years is available it is better to study its own data and to derive a particular formula for the probable frequency of various floods.
ARTICLE 17. FLOOD FORMULAS

There are a large number of flood formulas been prepared for the purpose of estimating the probable flood flow from drainage area of various size. They have usually been developed for certain streams or for certain places possessing particular characteristics. However, they have been also applied frequently for other streams and places under conditions similar to those for which they were derived. In the places, where no hydrological and meteorological data are available so that the cause of flood can not be studied, the use of such formulas may be more justifiable because they serve as rough known guide. The two well-known formulas were deduced by Kuichling, in the Report on New York Barge Canal, 1901. The first formula is for the great floods that are likely to occur occasionally, and the second is for the greatest floods that sometimes occur but rarely.

\[
(1) \ Q = \frac{44000}{M} + 20 \ M \\
(2) \ Q = \frac{127000}{M} + 7.4 \ M
\]

in which

\[Q = \text{maximum (24-hour average) flood in cubic feet per second},\]

\[M = \text{drainage area of the stream in square miles}.\]
There are also many formulas devised for estimating runoff from rainfall. Their application is limited to the small areas from 1 to 1000 acres, particularly for solving sewer problems, but is not so extensively used for determining flood rates in large streams. They have usually a common form of expression, such as:

\[ Q = CIA^mS^n \]

in which,
- \( Q \) = discharge in cu. ft. per sec.,
- \( C \) = Coefficient (from 0.7 to 2),
- \( I \) = maximum intensity of rainfall in inches per hour,
- \( A \) = area drained in acres,
- \( S \) = average slope in feet per 1000,
- \( m \) = 0.75 to 1.0,
- \( n \) = 1/6 to 1/4.

From the relation between rainfall and runoff Kui-chling derived another formula which is expressed by,\(^{16}\)

\[ Q = fAIt \]

where,
- \( Q \) = discharge in cu. ft. per sec.,
- \( f \) = rate of runoff to precipitation,
- \( A \) = area in acres,
- \( I_t \) = average intensity of rainfall in time required for concentration in inches per hour.
During heavy storm, when a flood is threatening, the use of these formulas is more desirable for it will give an early information as to what extent the immediate inundation will be. The difficulties attending to this method are the uncertainties existing in the distribution of rainfall, the relation between runoff and precipitation, and other minor conditions. If, however, observations are made to some extent they will bring out some better idea of selecting proper coefficient and suitable powers to be used in these formulas.
ARTICLE 18. HYDROGRAPH OF FLOOD WAVE

The use of a flood hydrograph is important in both channel improvement and retarding basin design. In the former it is necessary to determine the maximum height and the corresponding rate of flow; while in the latter it is not only to know the maximum flood rate but also the body of flood waters. The hydrograph of a flood wave is generally a diagram showing the height or discharge of river at different hours of the days during the flood. This may be well illustrated by Figs. 12 to 15. In streams, where daily height or discharge is recorded from the data of an automatic gage, flood hydrographs may be directly obtained with reference to the original records. It is necessary, however, to construct a hydrograph of a flood wave when no continuous record is available.

Suppose in a stream the maximum and normal depths of a passed flood wave have been determined from some sources to be $D_1 = 20'$ and $D_2 = 10'$ respectively. The average slope of the river bed has also found to be $S = 2$ ft. per mile or $0.00038$, and the coefficient of roughness $n = 0.035$.

From these conditions we find the coefficient of velocity from Kutter's formula, approximately $C = 70$, for the average value corresponding to $D_1$ and $D_2$. Next we find the maximum current velocity by Chezy formula from Fig. 3,

$$V_1 = 6.10 \text{ ft. per sec.}$$
Since \( y = \frac{P_2}{D_1} = \frac{10}{20} = 0.50 \), Fig. 5 gives the function of flood wave velocity \( K = 1.294 \). Therefore, by equation (11)

\[
V_0 = KV_1 = (1.294)(6.10)
\]

\[
= 7.9 \text{ ft. per sec.}
\]

\[
= 5.38 \text{ miles per hour}
\]

The profile of this flood wave is exactly the same as that computed in Table 4, from which it is rearranged in Table 8, and time of travel of the flood wave is then determined by the expression, \( t = \frac{L}{V_0} \). Considering the falling flood wave to have the same shape of profile of the rising wave the complete flood hydrograph is finally plotted in Fig. 17.

With the data of gage height obtained from actual readings, or determined from the method above described, a hydrograph of flow can also be constructed if it is required.

When a flood wave is recorded in terms of average 24-hour flow rates instead of the actual flood rates and a hydrograph is plotted based upon these readings it will be obviously composed of a group of stepped blocks instead of a smooth continuous curve. The bulk of the flood water computed from the former is, however, numerically the same as that estimated from the latter. But, their crest rates are quite different and the discrepancy should carefully distinguished.

In a large river, the flood usually requires several
TABLE 8

HYDROGRAPH OF FLOOD WAVE

\( D_1 = 20 \text{ ft}, \ D_2 = 10 \text{ ft}. \)

<table>
<thead>
<tr>
<th>Depth in Ft.</th>
<th>Time Before Crest of Wave Hrs.</th>
<th>Length from Head of Wave in Mi.</th>
<th>Length from Crest of Wave in Mi.</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>27.1</td>
<td>0.0</td>
<td>145.81</td>
</tr>
<tr>
<td>11</td>
<td>23.9</td>
<td>22.97</td>
<td>122.84</td>
</tr>
<tr>
<td>12</td>
<td>21.3</td>
<td>30.98</td>
<td>114.83</td>
</tr>
<tr>
<td>13</td>
<td>20.4</td>
<td>36.18</td>
<td>109.63</td>
</tr>
<tr>
<td>14</td>
<td>19.2</td>
<td>42.40</td>
<td>103.41</td>
</tr>
<tr>
<td>15</td>
<td>18.1</td>
<td>48.06</td>
<td>97.95</td>
</tr>
<tr>
<td>16</td>
<td>17.0</td>
<td>54.34</td>
<td>91.47</td>
</tr>
<tr>
<td>17</td>
<td>15.6</td>
<td>62.05</td>
<td>83.76</td>
</tr>
<tr>
<td>18</td>
<td>13.7</td>
<td>72.19</td>
<td>73.62</td>
</tr>
<tr>
<td>19</td>
<td>10.4</td>
<td>89.72</td>
<td>56.09</td>
</tr>
<tr>
<td>20</td>
<td>0.0</td>
<td>145.81</td>
<td>0.0</td>
</tr>
</tbody>
</table>
days or even weeks to reach its peak, and as slowly recedes, so the average height and flow of the day of maximum flood vary but little from those at the hour of maximum stage; in a small stream, however, it springs quickly and recedes as rapidly, and the period may be a few hours in duration, so there is a considerable difference between the heights and the corresponding discharges.

Mr. Fuller, previously referred to, has established an average relation between the maximum rate of flood and the average 24-hour flow for streams of various drainage areas. His expression is:

\[
\frac{Q_{\text{max.}}}{Q_{\text{ave.}}} = (1 + 2M^{-0.3})
\]

in which,

- \( Q_{\text{max.}} \) = maximum flood discharge in cu. ft. per sec.,
- \( Q_{\text{ave.}} \) = maximum average 24-hour discharge in cu. ft. per sec.,
- \( M \) = drainage area in sq. miles.

It is also possible to determine the actual crest height, by the following graphical method, when the maximum average 24-hour crest height is known.

Suppose the recorded average 24-hour maximum height \( D_a = 14.5 \) ft. and the constant normal stage \( D_2 = 10 \) ft. The average slope of the river bed \( S = 2 \) ft. per mile or 0.00038.
The coefficient of roughness \( n = 0.035 \).

Procedures:

1. Assume a trial peak height \( D_1 = 20 \) ft.
2. Calculate, by Chezy formula \( V_1 = 6.10 \) ft. per sec.
3. Find, from Fig. 3, \( K = 1.294 \).
4. Compute \( V = KV_1 = 7.90 \) ft. per sec. = 5.38 mi./Hr.
5. Compute the profile and plotted the hydrograph as shown in Fig. 17.
6. Assume the crest to be occurred at the end of the day of the peak, and find the depth at the beginning of the same day, see Fig.17.
7. From the portion of the hydrograph covered by the day in question determine the average height \( D_a' = 14.75 \) ft.

The average depth just found is close enough to the recorded value. So the assumed crest height \( D_1 = 20 \) ft. is satisfactory. If the average height above determined does agree with the recorded height, another \( D_1 \) should be assumed until the discrepancy between \( D_a' \) and \( D_a \) is very small.
FLOOD PICTURES
High Water Stage of 1924 Summer Flood In the Iowa River At Iowa City, Iowa.

Medium Stage of the Iowa River At Iowa City, Iowa.
High Water Stage of 1924 Summer Flood In the Iowa River at Coralville, Iowa.

Low Water Stage in the Iowa River at Coralville, Iowa.
Flood of the Summer 1934 Overflowing the Banks of the Iowa River above Iowa City, Iowa.

Low Water Stage in the Iowa River below Coralville, Iowa.
Fishing in the Iowa River During the Flood of the Summer 1924 at Iowa City, Iowa

Sounding in the Iowa River at S. U. I. Gaging Station, Iowa City, Iowa.
The Iowa River at Iowa City, Iowa.
Flood of the Summer 1924 Overflowing Marengo, Iowa.
Showing the Depth Attained by 1924 Summer Flood over the Sidewalk at Marengo, Iowa.
Damages to Structures Caused by 1924 Summer Flood in the Bear Creek at Marengo, Iowa.
Deposits Formed in the Farm Land near the Bear Creek at Marengo, Iowa, due to 1924 Summer Flood.

Deposits Formed in Banks of the Iowa River at Marengo, Iowa, due to 1924 Summer Flood.
The Iowa River at Marengo, Iowa.