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REGULATION OF STREAM FLOW FOR MILITARY PURPOSES

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MILITARY HYDROLOGY BULLETIN 11
JUNE 1957

A
CORPS OF ENGINEERS
RESEARCH AND DEVELOPMENT REPORT

PREPARED UNDER DIRECTION OF
CHIEF OF ENGINEERS

BY

MILITARY HYDROLOGY R & D BRANCH
U. S. ARMY ENGINEER DISTRICT, WASHINGTON

202469

Declassified in Part - Sanitized Copy Approved for Release @ 50-Yr 2013/10/25 : CIA-RDP81-01043R002300060003-3

Military Hydrology R&D Branch, U.S. Army Engineer District, Washington, D. C.
REGULATION OF STREAM FLOW FOR MILITARY PURPOSES
June 1957, 31 pp. illus. 20 tables & plates
(Military Hydrology Bulletin 11)
DA R&D Project 8-97-10-003

Unclassified Report

This bulletin presents methods of computation of operation of dam outlets to produce controlled variation in river depth, width, and velocity at downstream points, including methods of evaluating the proper magnitude, duration, and timing of reservoir releases and the downstream hydraulic effects.

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RESEARCH AND DEVELOPMENT PROJECT NO. 8-97-10-003

FOR
ENGINEER RESEARCH & DEVELOPMENT DIVISION
OFFICE, CHIEF OF ENGINEERS

BY
MILITARY HYDROLOGY R&D BRANCH
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JUNE 1957

PRINTED BY ARMY MAP SERVICE, CORPS OF ENGINEERS, 3-58

Declassified in Part - Sanitized Copy Approved for Release @ 50-Yr 2013/10/25 : CIA-RDP81-01043R002300060003-3

PREFACE

This bulletin is the eleventh of a series dealing with the various aspects of hydrology involved in military operations and with the hydrologic techniques and methods of analysis which are considered most suitable for Army use. A number of these techniques were developed in the course of Research and Development Project No. 8-97-10-003, assigned to the Army Engineer District, Washington, on 14 March 1951 by the Office, Chief of Engineers. Printing of the Bulletin was authorized by the Office, Chief of Engineers, on 9 May 1957.

Mr. A. L. Cochran of the Office, Chief of Engineers, formulated the objectives and scope of this Bulletin. Mr. B. G. Baker developed the original procedures and Mr. W. B. Craig prepared the final method and text under the supervision of Mr. R. L. Irwin and Mr. F. B. Barkalow, Military Hydrology R&D Branch, Washington District.

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SUMMARY

Military crossings of major rivers in the face of the enemy represent engineering achievements of a high order, even when normal river conditions prevail. When natural river flows and stages are augmented by carefully-timed flood-wave releases of predetermined magnitude from upstream reservoirs, the difficulties encountered in the crossing operation may well become almost insurmountable. This manual describes the hydraulic factors involved in the release of a flood wave through the outlet structures of a dam to produce sudden changes in depth, width, and current velocity at the downstream crossing site. It then presents formulas and short-cut methods of evaluating these effects, with examples. Lastly, the manual includes a discussion of cyclical flood waves, produced by a series of reservoir releases.

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CHAPTER I
INTRODUCTION

1. Purpose. The purpose of this bulletin is to discuss the sudden changes in discharges, depths, velocities, and widths of streams created by manipulating the regulating outlets of a dam; and to provide methods for obtaining rapid estimates of such effects, which are suitable for hydraulic studies pertaining to military operations.

2. Scope. The bulletin presents an analysis of the factors involved in estimating reservoir releases to effect desired hydraulic conditions at selected points downstream. While the analysis presented herein is oriented toward establishing the timing and manner of operation at dam outlets to produce a desired hydrograph at a selected point downstream, essentially the same procedures can also be used where it is necessary to estimate the downstream results from given outlet operations. The bulletin outlines the data required to establish the outflow hydrograph at a dam outlet or outlets, and the factors which modify the outflow hydrograph as it travels downstream. A semigraphical method of routing the reservoir outflow hydrograph from the dam to the downstream point is presented. The construction of solution diagrams is facilitated by utilizing a coefficient expressing the relationship between discharge and storage. The methods for deriving a gate operation schedule to produce a block-type reservoir outflow hydrograph meeting the downstream requirements are outlined. A procedure is developed to determine the effect of cyclical flood waves, consisting of a series of block releases alternated by periods of low flow. Examples are then given to illustrate the application of the derived equations and diagrams to specific problems.

3. Discussion of Problem. a. Many high dams impounding large volumes of water have been constructed throughout the world's potential theaters of operation, and have important military capabilities for artificial flooding. The term "artificial flooding" is used in this manual to denote an artificially-created increase in stage over the existing natural-flow conditions in a stream or area. It is generally accomplished by increasing the streamflow by regulation or breaching of dams, or by creating drainage obstacles. The manipulation of the regulating outlets of a dam to produce sudden changes in stream discharge, thereby affecting the depth, width, and velocity throughout the stream channel, is one example of the techniques of artificial flooding.

b. A given hydraulic system, consisting of one or more reservoirs in a river basin, may have certain capabilities for offensive or defensive operations that are of major importance to a field commander. Artificial flooding operations have proven to be a definite obstacle in the assault of a river line, as encountered by our forces during the Rapido and Ruhr River crossing operations in World War II. Experience has shown that it is extremely difficult to assault a fortified river line if there are major fluctuations in the stream stages, surface widths, or velocities.

Under such conditions assault boats are carried far downstream in the high-velocity current and tactical control may be lost. The second phase of the crossing operation, raft and bridge construction, is even more critical, since its completion is required for the build-up of the heavy weapons necessary in securing the bridgehead against an enemy counterattack. Cyclical floodwaves, created by block reservoir releases above the bridgehead spaced at appropriate intervals during this phase of the crossing, would make almost impossible the construction of rafts and bridges and the build-up of fire support. Without an adequate and rapid build-up of the fire-support echelon of the attacking forces, a river-crossing operation is normally doomed to failure. From the above considerations it is apparent that the field commander should be informed of the capabilities of hydraulic structures in his vicinity to disrupt or aid his military operations, and that he should be furnished data as accurate and as far in advance as possible on the depths, widths, and velocities of artificial flood waves resulting from the use of these structures as weapons of warfare.

4. Arrangement. The remaining chapters of the bulletin are arranged to provide a logical development of the techniques of solution of artificial flood wave problems. Chapter II presents the basic hydraulic principles and theory directly involved in the solution. Chapter III describes a semi-graphical method, convenient for military use, of determining the extent of modification of the reservoir outflow hydrograph as it travels downstream to the target point. Chapter IV outlines methods of determining gate operation schedules of reservoir releases. Lastly, Chapter V presents examples of solution of specific flood-wave problems, using the methods described in the preceding chapters. Plates providing graphic illustrations and detailed explanation of the text, are presented at the end of the bulletin.

5. Related References. A bibliography of related references appears in the rear of the manual following the text. The following bulletins, also prepared by the Military Hydrology R&D Branch, are of special significance as they are frequently referred to in this bulletin:

Military Hydrology Bulletin 10, "Artificial Flood Waves".

Military Hydrology Bulletin 12, "Handbook of Hydraulics".

Dept. of the Army Technical Bulletin, TB 5-550-3 "Flood Prediction Techniques".

CHAPTER II
PRINCIPLES AND BASIC THEORY

6. Pertinent Factors. The determination of possible streamflow variation as a result of operating dam outlets requires a knowledge of several pertinent factors. These include the physical and hydraulic characteristics of the reservoir, the dam and appurtenant works, and the reaches of the stream from the dam to the stream location where artificially regulated patterns of discharge and stage are to be determined. In the case where a certain magnitude of stage and frequency of variation cycle is desired at a given stream location, there is required, in addition to the aforementioned data, the amount of water available for regulation, and the characteristics of the outlet structures including the rate of opening and closing the outlets. The shape of the outflow hydrograph during its passage downstream is a function of the physical and hydraulic characteristics of the stream which include the slope of the stream, configuration and dimensions of the stream boundaries, meander pattern, changes in cross section, vegetative cover, sediment characteristics (sediment size, sand bars, etc.), length of water course and other factors. Depth and velocities of flow at selected locations depend upon the discharge and upon the shape and dimensions of the cross section at that location. Having determined the discharge through routing procedures and knowing the cross-sectional characteristics, depths of flow and velocities at problem locations may be readily determined. The way in which the techniques of streamflow regulation are affected by the physical and hydraulic characteristics of the stream channel are described in the following paragraphs.

7. Reservoir Effective Storage. a. Streamflow regulation, to be effective as a military weapon, requires an adequate supply of water for a specific period of time. The effective reservoir storage is defined as the water stored above the spillway crest or above the gate seats of the outlet conduits. In any river hydraulic system, the effective storage is the most important single factor in determining the capability of the system for artificial flooding. Normally over one-half the storage of a reservoir is impounded in the upper quarter of its depth. This portion of the storage, impounded at a high head, creates high discharge rates through the outlet structures and thus constitutes a favorable condition for artificial flooding. Heavy rainfalls may occur over the drainage basin during certain periods of the year, and the resulting inflows would augment the capabilities of the reservoir for artificial flooding.

b. The effective storage of a reservoir at a given pool elevation is obtained by subtracting from the total storage at that elevation the storage at the elevation of the lowest outlet structure. These values are taken from the elevation-capacity curve, which is usually obtained by the following method:

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(1) Planimeter the area within each contour of elevation on a topographic map of the reservoir site, and by subtraction compute the areas within adjacent contours;

(2) Multiply the area within adjacent contours by their difference in elevation, and accumulate these products;

(3) Plot the accumulated products against the corresponding upper-contour elevations.

c. A method of estimating the reservoir capacity curve is given in "Engineering Construction - Flood Control" by the Engineer School, Ft. Belvoir, Va., and in MH Bulletin 10, "Artificial Flood Waves", by the U. S. Army Engineer District, Washington. In this method, the storage curve is computed by a simple exponential equation in which the exponent is selected from a table based on a general classification of the terrain characteristics of the reservoir site.

8. Reservoir Regulated Outflow. The regulated outflow from a dam, as considered in this bulletin, consists of either a single block hydrograph or a series of cyclical block hydrographs released through the outlet structures. The peak discharge, duration, and timing of these hydrographs must be selected in such a way as to satisfy critical downstream requirements pertaining to depth, velocity, etc. This selection must take into consideration the modification of the outflow hydrograph by channel conditions as it proceeds downstream. An equally important consideration in making this selection, however, is the discharge capacity of the outlet structures in relation to the amount of effective reservoir storage. A block release hydrograph with a high peak and of short duration may accomplish with greater economy of water volume the same result at the target location as a release hydrograph with a lower peak and of longer duration. Of course, the capacity of the reservoir outlets must be sufficient to release a discharge that will give the required flood hydrograph at the problem location. When the gates on the structure can be rapidly opened, the resulting flood wave may advance as a bore, causing a rapid change in stage and thus producing damage not obtainable by high velocities alone. The maximum discharge that can be released at any given reservoir elevation is determined from the combined discharge rating curve of all the outlet structures, computed by methods described in MH Bulletin 12 "Handbook of Hydraulics", from the physical description of the spillway and outlet works. When the various components of the problem (including the target objectives, downstream hydrograph modification, storage available, and outlet discharge capacity) have been determined, the peak discharge, duration, and timing of the block outflow hydrograph may be suitably selected.

9. Physical Characteristics. a. From the moment the block hydrograph leaves the outlet structures of the dam, it is subject to continual modification in the stream channel. The physical characteristics which produce this modification, and which must be evaluated before the eventual hydrograph at the target location may be determined, are the following:

(1) The length of the stream from the dam to the downstream target location.

(2) The variation in the cross-sectional area and shape of the channel and overbank for each reach.

(3) The variation of the water-surface width with change in stage and discharge.

(4) The bottom profile of the stream.

(5) The roughness coefficients of the channel and overbank.

(6) The storage capacities of the channel and overbank.

(7) The locations and physical characteristics of channel bends, constrictions, and rapids.

b. For the purposes of this bulletin, additional inflow between the dam and the selected point downstream was not considered. However, tributary inflow could be added to the base flow with no major difficulty. Rises resulting from natural causes such as precipitation could also be added to base flow.

10. Hydraulic Elements. a. The procedure whereby the hydrograph at the lower end of a reach is determined from the known or assumed inflow hydrograph at the upper end of the reach is known as flood routing. The method of flood routing used in this manual is based on the "Muskingum Method" of storage routing, and involves the determination of certain hydraulic elements from the physical characteristics listed above. These elements include discharge and velocity rating curves at a sufficient number of cross sections to represent the entire reach, and other hydraulic data.

b. The most direct method of determining the hydraulic elements pertaining to a given cross section is to conduct a series of discharge measurements covering the expected range of flows, using stream-gaging procedures described in standard hydraulic handbooks and in MH Bulletin 3. When circumstances do not permit making discharge measurements, a stage-discharge curve may be computed by the use of Manning's equation, as described in MH Bulletin 12, "Handbook of Hydraulics", or in standard textbooks on open-channel hydraulics.

The data collected in the discharge measurements may also be utilized in evaluating other hydraulic elements.

11. Average Wave Velocity. The flood routing method used in this bulletin is a modification of the Muskingum Method of flood routing and involves the determination of the average wave velocity "U" for each routing reach. In a given reach, the value of the average wave velocity "U" corresponding to a given discharge is obtained by averaging the wave velocities for that discharge at representative cross sections in the reach. The wave velocity at a single cross section is determined by plotting discharge as ordinate against the corresponding cross-sectional area as abscissa, and drawing a smooth curve through the plotted points. The slope of this discharge-area curve at any point is then the wave velocity for that discharge. If a typical cross section for the reach can be selected, the wave velocity for each discharge at that cross section may be considered the average wave velocity at that discharge for the entire reach.

12. Determination of ΔK. a. Both the standard Muskingum Method of flood routing and the Semi-Graphical modification used in this bulletin require the determination of a storage factor "K" for each reach through which the flood hydrograph is to be routed. In this bulletin, the storage factor for a single reach is designated "ΔK". If, as is usually the case, the length of channel from the dam to the target point must be divided into a series of subreaches in order to satisfy the requirements for routing by the Muskingum Method, the storage factor for a subreach is designated "ΔK", and that for the entire series of subreaches is designated "ΣK".

b. The value of "ΔK" for a given single reach or subreach may be computed in a number of ways, depending on the types of basic data available. "ΔK" is defined as the change in storage per unit change in discharge, and has the dimension of time. The following methods of computing "ΔK" are frequently used:

(1) A loop storage curve, derived from observed flood hydrographs at the upper and lower ends of the reach, is determined by plotting accumulated storages at the end of successive time intervals against the corresponding outflows, with storage as ordinate and outflow as abscissa, and drawing a mean line through those points. The value of "ΔK" for the incremental time period is equal to the slope of this line.

(2) ΔK is the travel time of the center of mass, or other characteristic point, of the hydrograph through the subreach.

(3) ΔK is the quotient obtained by dividing the length of the subreach by the average wave velocity "U", described in the previous paragraph.

(4) ΔK is the slope of the curve representing the volume within the reach under the computed profiles for steady flows versus the corresponding discharges.

(5) Studies described in MH Bulletin 10 indicate that ΔK varies not only with reach length but also with discharge in the reach, and provide a method whereby the "ideal reach length" for routing a given discharge, and the corresponding value of ΔK, may be determined from the physical characteristics of the reach. The value of ΔK corresponding to a given discharge in a subreach of ideal length is given by the equation

$$\Delta K = \frac{0.000365\ Q}{U^2\ b_w\ S_o}$$

where:

Q = the discharge in cfs
U = the average wave velocity at discharge Q in ft/sec
b_w = the water-surface width in ft at discharge Q
S_o = the average bottom slope of the reach

The coefficient in this equation corresponds to an "X"-value of zero in the Muskingum Method of flood routing.

13. Determination of ΣK. The value of ΣK, the storage factor for the entire series of subreaches, may be obtained by the methods shown above, or by summating the values of ΔK for the various subreaches.

14. Determination of Δt. Flood routing is ordinarily accomplished by a numerical procedure involving selection of discharge hydrograph ordinates at fixed time intervals "Δt". The following considerations are involved in the selection of Δt:

(1) Discharge ordinates taken at intervals of Δt units must adequately define the hydrograph.

(2) The water-surface profile in the subreach during the interval Δt should be relatively straight.

(3) The travel time of the flood wave through the subreach must be equal to or greater than Δt.

CHAPTER III
FLOOD ROUTING BY THE SEMI-GRAPHICAL METHOD

15. Semi-Graphical Method. a. Many different methods and procedures of flood routing have been described in engineering literature. In general, the methods that attempt a strict mathematical treatment of the many complex factors affecting flood-wave movement are not practical for routing floods through long reaches of natural river channels. The flood routing methods normally used in practice either ignore many complex factors, or make simplifying assumptions that reduce computation difficulties to an acceptable level. The two basic methods of flood routing commonly used in practice are:

(1) Storage methods, in which energy factors are largely neglected and only the effects of storage in the reach and local inflows are considered.

(2) Average inflow lag methods, in which a time-displacement method of averaging the inflows is used to determine the shape of the flood wave at the lower end of the reach.

b. The "Muskingum Method" of storage routing is used in this bulletin to derive a less laborious semi-graphical method of routing artificial flood releases. The method represents an adaptation of the procedures presented in MH Bulletin 10, "Artificial Flood Waves". The procedures used in deriving the basic curves and computational techniques and in applying the results are described in the following paragraphs.

16. Basis of Derivation of Semi-Graphical Method. A rectangular block hydrograph representing a reservoir release was successively routed through a series of subreaches by the Muskingum Method, with ΔK equal to Δt in each subreach and with a Muskingum X-value of zero. The peak discharge of the initial block hydrograph was set at 100 units of flow, and its duration 100 units of time. Several sets of routings were performed, using values of ΔK equal to 1, 2, 5, 10, 20, and 40 units of time. It was assumed that all subreaches have equal values of ΔK. Thus the value of ΣK is obtained by multiplying ΔK by the total number of subreaches through which the hydrograph was routed. Values of ΣK ranging as high as 520 were attained by the routing procedure. Selected representative hydrographs resulting from these routings are shown on Plates No. 1-a and 1-b. The downstream hydrograph ordinates are expressed in percent of the initial hydrograph peak, and the time abscissa are shown as percentages of the duration of the initial hydrograph. These routings constitute the basis upon which the curves and procedures described in the following paragraphs were derived.

17. Peak Modification Curves. As a flood hydrograph moves down a river channel the peak discharge is reduced. A study of the routings

described above showed that the reduction of the downstream peak is a function of the product $\Delta K \cdot \Sigma K$. For convenience, ΔK and ΣK were expressed as ratios of the duration of the initial hydrograph L_i, and denoted as ΔK_r and ΣK_r, respectively. Values of the downstream hydrograph peaks, computed as shown above for various values of ΔK_r and ΣK_r, were expressed as percentages of the initial hydrograph peak and plotted against corresponding values of the product $\Delta K_r \cdot \Sigma K_r$, and smooth curves were drawn through these points. These curves are designated "Peak Modification Curves" and are shown on Plate No. 2. The peak modification curve determines the peak discharge of the downstream hydrograph as a functiom of $\Delta K_r \cdot \Sigma K_r$. It will be noted from the curve that the peak of the downstream hydrograph does not fall below the peak of the initial block hydrograph until a $\Delta K_r \cdot \Sigma K_r$ value of about 0.022 is reached.

18. Weighted Discharge. a. The use of the semi-graphical method of flood routing through a series of subreaches involves the determination of the routing flow for the entire reach. An inspection of the peak modification curve on Plate No. 2 shows that the peak does not decrease uniformly as the hydrograph moves downstream, so that an average of the peaks of the reservoir outflow hydrograph and a downstream hydrograph is not a true mean of the peak through the reach. Neither is a peak discharge computed by averaging the intermediate routing peaks a true index of the proper routing flow through the reach as the shape of the hydrograph also has an influence in selection of the routing flow.

b. A "Weighted Discharge Curve", shown on Plate No. 3, was developed that gives a proper routing flow for use in routing a flood hydrograph from dam release through a series of uniform subreaches in one computation. The procedure used in the development of the curve is described as follows: Hydrographs at downstream points with peak discharges of 100, 95, 90, 75, 50 and 24.6 percent of the initial peak reservoir release were arbitrarily selected from those obtained in the basic routings, and the value of $\Delta K_r \Sigma K_r$ corresponding to each of these peaks was determined from the peak modification curve. Six sets of computations, one for each of the above peaks, were then made to determine the weighted mean flow for the successive downstream hydrographs through a series of subreaches corresponding to a given value of $\Delta K_r \Sigma K_r$ at the lower end. Each set of computations was determined in the same manner as that for the 50 - percent peak, which is described in the following steps:

(1) $\Delta K_r \Sigma K_r$ has the value 0.566 for the downstream peak corresponding to 50-percent of the initial peak, as determined from Plate No. 2.

(2) Successive hydrographs below the dam were selected from the previously-routed hydrographs that had $\Delta K_r \Sigma K_r$ values increasing from zero to 0.566 by varying increments.

(3) The average discharge of each hydrograph during the period from the beginning of rise to the point on the recession side where the discharge equals 50 percent of the initial peak release was determined.

(4) The mean discharge of each successive hydrograph, as computed in step (3), was weighted by the incremental value of $\Delta K_r \Sigma K_r$ in the subreach above it.

(5) The weighted discharge for 50-percent peak modification was computed by accumulating the weighted discharges of step (4), and dividing the total by 0.566, the value of $\Delta K_r \Sigma K_r$ determined in step (1).

(6) The value of the weighted discharge (34 percent) was plotted against the value of $\Delta K_r \Sigma K_r$ and formed one point on the "Weighted Discharge Curve".

c. The five other sets of computations were obtained and plotted in the same manner, and a smooth curve, as shown on Plate No. 3, drawn through these points.

19. Hydrograph Ordinates. a. The hydrographs of Plate No. 1 were cross plotted to form a series of curves used in determining the ordinates of the downstream hydrograph. The series of curves shown on Plate No. 4 are expressed as time from beginning of rise (in percent of the duration of the initial hydrograph) as ordinate and the product $\Delta K_r \Sigma K_r$ as abscissa; the parameter represents the hydrograph ordinates (in percent of the downstream peak) for both the rising and recession legs of the hydrograph.

b. The hydrographs of Plate No. 1 show a fillet shape at the beginning of rise which would probably not actually occur with a block-type release. To give an abruptly rising hydrograph at the beginning of rise, it was arbitrarily assumed that the time of the beginning of rise of the downstream hydrograph was the time of occurrence of the 5-percent ordinate. The percentage values of the parameter curves shown on Plate No. 4 were arbitrarily selected and are adequate to define the shape of the downstream hydrograph in most cases.

c. Table 1 shows the method of computing one point on each parameter curve. The example hydrograph selected was for $\Delta K = 5$ and $\Sigma K = 250$ with a peak discharge of 84.8 percent of the initial block hydrograph peak (See Plate No. 1a).

TABLE I

Hydrograph Ordinates		Time from release at dam	Time from beginning of rise
Percent of downstream peak Col. 1	Percent of initial peak Col. 2	Col. 3	Col. 4
Rise			
5	4.2	193	0
25	21.2	223	30
50	42.4	243	50
75	63.6	263	70
90	76.3	277	84
100 = Peak	84.8	300	107
Recession			
90	76.3	322	129
75	63.6	337	144
50	42.4	356	163
25	21.2	379	186
5	4.2	415	222

Explanation

(1) The products of the percentages of Col. 1 and the peak discharge in percent of the initial block hydrograph (84.8) were entered in Col. 2.

(2) The time of travel from the dam in percent of the duration of the initial hydrograph was determined from the hydrograph on Plate 1a for each ordinate of Col. 2, and entered in Col. 3.

(3) The time from beginning of rise was computed as the difference of the times of Col. 3 and the 5 percent ordinate (193), and entered in Col. 4.

(4) The time from beginning of rise of Col. 4 was plotted on Plate 4 as ordinate for the value of the product $\Delta K_r \Sigma K_r = 0.125$. Each value of the time from beginning of rise formed one point on the parameter curves corresponding to the value of percent of the downstream peak in Col. 1. Ordinates for other hydrographs were computed and plotted in the same manner as described above and smooth curves drawn between each set of points and shown on Plate No. 4.

20. Downstream Hydrograph Peak (P_o). a. The required peak discharge (P_o) of the downstream hydrograph depends on the tactical situation and the purpose for which it is to be used in the flooding operation. The required peak is established by the conditions that are desired to be produced - such as increased river velocity, depth, or width. The relation of these to the required peak (P_o) depend upon the physical and hydraulic characteristics of the cross section at the problem location.

b. The average velocity curve of the channel and the area, surface width, and discharge curves of the total cross section are determined at the downstream problem location by the methods described in Military Hydrology Bulletin 12 "Handbook of Hydraulics". For any desired velocity, stage, or surface width the corresponding discharge can then be determined. This discharge is then the required downstream hydrograph peak (P_o).

21. Release Hydrograph Peak (P_i). The procedure considered in this bulletin is based on the assumption that a choice of dam operation exists and that it is possible within wide limits to release any desired discharge for any selected length of time in order to create desired conditions at the target site. Under these assumptions, an initial hydrograph is selected and then routed downstream to ascertain if the desired P_o occurs at the selected downstream point. If the initial hydrograph is fixed by conditions, the procedure illustrated in this bulletin could nevertheless be used. The peak discharge of the initial block hydrograph released at the dam is arbitrarily assumed as somewhat greater than the required downstream hydrograph peak (P_o). Its value is selected after considering the discharge capacity of the outlet structures and the effective storage in the reservoir. The various factors affecting the selection of the release hydrograph peak are discussed in Paragraphs 7 and 8.

22. Procedure for Solution. The procedure for routing a block hydrograph down an open channel by the semigraphical method of flood routing is based on the application of the dimensionless graphs shown on Plates 2 to 4, inclusive. Knowing the required downstream hydrograph peak (P_o) and assuming the initial hydrograph peak (P_i), the procedure for determining the duration of the initial release hydrograph (L_i), the downstream hydrograph ordinates and the times from beginning of release is described in the following paragraphs.

23. Average Hydraulic Characteristics of Reach. The average wave velocity, ΔK, and ΣK are determined for each reach as explained in Paragraphs 11, 12 and 13. The average values of ΔK and ΣK for each reach are plotted against the corresponding discharge and smooth curves drawn between the points. The average values of ΔK and ΣK

are read from the curves and the product $\Delta K\Sigma K$ computed and plotted. The three average hydraulic characteristic curves (ΔK, ΣK, and $\Delta K\Sigma K$) plotted for each reach are used in determining the duration of the initial block hydrograph.

24. Duration of Initial Block Hydrograph (L_i). The value of the downstream peak discharge (P_o) in percent of the peak discharge of the initial block hydrograph (P_i) is computed from the known data. $\Delta K_r \Sigma K_r$ is then determined from Plate 2 corresponding to this value. The routing flow is then determined from Plate 3 for the above value of $\Delta K_r \Sigma K_r$. The product $\Delta K\Sigma K$ is determined from the average hydraulic characteristic curves (derived in paragraph 23) for the value of the routing flow. From this, the duration of the initial block hydrograph is computed as:

$$L_i = (\Delta K\Sigma K/\Delta K_r \Sigma K_r)^{0.5}$$

Maintaining release from the dam at the initial peak (P_i) for this duration (L_i) will produce the desired downstream peak discharge (P_o), after which the downstream hydrograph will recede back to normal.

25. Time of Downstream Peak. The time of the peak discharge (P_o) of the downstream hydrograph as measured from the time of beginning of release of the initial block hydrograph at the dam is computed as:

$$\Sigma K + (L_i/2)$$

The values of ΣK and L_i used are those determined in preceding paragraphs 23, 24 respectively.

26. Time of Beginning of Rise of Downstream Hydrograph. As previously explained in paragraph 19, the point of beginning of rise of a downstream hydrograph has been arbitrarily assumed as the time of occurrence of the 5-percent ordinate. By entering Plate 4 with the same value of $\Delta K_r \Sigma K_r$ used in paragraph 24, the time interval (in percent of L_i) between the 5-percent and 100 percent parameter curves can be determined. This (converted into actual units of time by multiplying by the value of L_i) and subtracted from the time of peak as determined in paragraph 25, will then establish the time of beginning of rise of the downstream hydrograph.

27. Downstream Hydrograph Shape. The shape of the downstream hydrograph can be determined from Plate 4, by entering with the same value of $\Delta K_r \Sigma K_r$ used in paragraphs 24 and 26, and reading the times of occurrence from the beginning of rise (in percent of L_i) for each parameter discharge percentage curve. These can be converted into actual time and discharge units by multiplying by L_i and P_o, respectively. By adding these time units to the time of beginning of rise as determined in paragraph 26, the time of each ordinate of the downstream hydrograph as measured from the beginning of release at the dam can be determined. Reference may be made to Plate 11b for an example of these computations together with more detailed explanation.

28. Verification of Results. The hydrographs computed by the semigraphical method were checked by the storage-indication method of flood routing. This method, although more laborious than the semigraphical method, is particularly suitable as a check, since it provides for the use of a variable storage factor while maintaining an equal volume under the inflow and outflow hydrographs. The details of the method of computing the storage indication routing are given in Bulletin TB-5-550-3, "Flood Prediction Techniques for Military Purposes", or in standard text books on hydrology. The storage-indication routings checked the semigraphical method of routing within 5 percent of the peak discharges of the downstream hydrographs. The hydrographs exhibited a good degree of consistency in shape and time from beginning of release at the dam.

29. Discussion of Adopted Flood Routing Method. a. The adopted semigraphical method of flood routing is based on curves obtained by routing a dimensionless block hydrograph using a uniform "K" value corresponding to a uniform wave velocity. The degree of accuracy to be attained by the semigraphical method depends on how closely the following conditions are satisfied:

(1) The reach is fairly uniform in cross section and slope, and capable of representation by an average cross section.

(2) The average wave velocity does not vary greatly through the range of discharge.

b. The limits of permissible variation from these conditions which will still give satisfactory results have not been established at this time. Further studies would have to be made to determine the effects of valley cross sections exhibiting large variations in average wave velocities through a reach.

30. Cyclic Discharges. Repeated streamflow variation may be possible under some cases where a sufficient volume of water is available for regulation and where the enemy lacks the power to prevent further streamflow variation after the first cycle. While it may be possible to cause any desired downstream discharge by releasing an equal discharge at the dam, the length of time and thus the volume of water required may be greater than desirable. By increasing the discharge at the dam (P_i) the desired downstream peak (P_o) can be caused with a much smaller volume of water. The

duration of the flood wave however, will also be smaller. Where it is desirable to maintain the discharge above a selected minimum at all times at the downstream point, the selection of the most efficient release may become involved. In the following paragraphs, there has been developed a procedure for determining the timing of cyclical releases to produce waves at the downstream location with desired peak and trough discharges.

31. Efficiency Ratio. a. The "Efficiency Ratio" is a working tool used in computing the duration of each cycle, or the time interval from the start of one block hydrograph to the start of the next. It is based on the following considerations: Suppose that a given peak discharge (P_o) is desired at a certain point downstream from the dam, P_o may be achieved at the downstream point by releasing an equal discharge at the dam for a period of sufficient length, say L_h hours. P_o may be achieved at the downstream point by releasing a higher discharge (P_i) for a shorter period (L_i). The latter release will produce a hydrograph at the downstream point rising to P_o and then receding. Assume that the hydrograph at the downstream point recedes to a flow of P_t at L_t hours after the time of beginning of rise at that point. Then, if flows lower than P_t, called the trough discharge, are not to be permitted, it follows that a new cycle must begin at the dam L_t hours after the start of the first cycle. The efficiency ratio, used in computing the timing of successive cycles, is equivalent to the fraction:

$$\frac{L_t \, P_o}{L_i \, P_i} = \text{Efficiency Ratio}$$

where L_t = time at downstream point from beginning of rise to trough
P_o = downstream peak discharge
L_i = length of dam release
P_i = peak discharge at dam

b. The "Efficiency Ratio Curves" presented on Plate No. 5 show the efficiency ratio as ordinate and the peak discharge (P_o) of the downstream hydrograph, (in percent of the initial hydrograph peak P_i) as abscissa. The parameter curves represent trough discharges (P_t) corresponding to 5, 10, 15, 20, and 25 percent of the downstream peak. Six sets of computations were made to determine the curves of Plate No. 5 - - one for each downstream peak discharge of 100, 95, 90, 80, 70, and 50 percent of the initial peak. As a preliminary step, supplementary working curves of the "discharge hydrograph ordinates" for the recession ordinates equal to 10, 15, and 20 percent of the downstream peak, similar to those shown on Plate No. 4 and discussed in paragraph 19 were computed and drawn. Each set of computations was then determined in the same manner as that for the 90-percent peak discharge, which is described in the following steps:

(1) The value of $\Delta K_r \Sigma K_r$ of 0.092 was determined from the peak modification curve (Plate 2) for a downstream peak of 90 percent of the initial peak.

(2) The time from beginning of rise of the downstream hydrograph for values of recession ordinates of 25, 20, 15, 10, and 5 percent of the downstream peak were determined from the discharge hydrograph ordinate curves corresponding to a $\Delta K_r \Sigma K_r$ value of 0.092.

(3) The values obtained in Step 2 were multiplied by the factor 0.90, the ratio of the downstream hydrograph peak to the initial peak, to compute the efficiency ratio for each recession ordinate.

(4) The five efficiency ratios of Step 3 were plotted against a downstream peak discharge of 90 percent of the initial hydrograph peak. Each ratio has as a parameter the corresponding value of recession ordinate, or trough discharge in percent of downstream hydrograph peak.

c. The steps described above were repeated for downstream peak discharges of 100, 95, 80, 70, and 50 percent of the initial peak, and smooth curves as shown on Plate No. 5 were drawn through the sets of points thus computed.

32. Duration of Cycle. The "Efficiency Ratio Curves" of Plate No. 5 are used in determining the timing of cyclic block releases to produce a downstream hydrograph of desired peak and trough discharges. The duration of a block release at rate equal to the downstream peak is obtained by multiplying the initial block hydrograph duration (L_i) by the ratio of peaks (P_i/P_o). The product of this value and the efficiency ratio (as determined from Plate No. 5 for the desired trough discharge) is then the time interval between beginning of successive cyclic releases. Reference may be made to Table II of Plate 11a for an example of this computation.

CHAPTER IV
GATE-OPERATION SCHEDULE

33. Problem. After the initial block hydrograph is determined by the methods described in Chapter III, the problem remains of determining a gate-operation schedule to release the hydrograph. The release hydrograph, instead of being a true rectangular block hydrograph with a constant discharge, in actual operations will be a saw-tooth hydrograph with the discharge varying with time. As the reservoir water surface drops with release of water through the gate opening, the discharge will also drop, unless the gate opening is increased. If the average discharge for each gate setting is to be the required peak discharge of the initial block hydrograph, the initial discharge of each gate setting must be above and the final discharge below that required discharge. It is obvious that the more gate settings, the more nearly will the release hydrograph approximate a true block hydrograph. If the gates are continuously changed during the release it is theoretically possible to release a constant discharge. For military purposes it is not necessary to have such a close approximation. The initial data required to solve the operation problem and the method of computing a reservoir water surface and gate-operating schedule are described in the following paragraphs.

34. Initial Data. The initial data required to compute a gate-operation schedule include the following:

(1) The volume of water to be released (computed as the product of the required peak discharge (P_i) and the time duration (L_i) of the block hydrograph as described in Chapter III.

(2) The elevation-capacity curve of the reservoir (computed by the methods described in paragraph 7.)

(3) The discharge rating curves for the outlet structures of the dam with partial and full gate openings. The method of determining the rating curves is described in Military Hydrology Bulletin 12 "Handbook of Hydraulics".

(4) The initial reservoir elevation or reservoir storage (usually given or assumed in the basic data of the problem).

35. Schedule of Reservoir Water Surface. A schedule is first computed of the elevation of the reservoir water surface after the block release. The schedule is computed by tabulating the difference between the initial reservoir storage and the volume of water released. The reservoir water-surface elevation at the end of the release is then determined from the elevation-capacity curve. The drop in the reservoir water surface is computed and tabulated in the schedule.

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36. Gate Opening and Discharge. a. The opening necessary to release the required peak discharge for each gate setting is based upon the discharge rating curve. The discharge is determined for a head equal to the water surface elevation at the midpoint elevation of the drop in water surface. If the drop in the reservoir water surface is large, the gate schedule should include several gate settings for that release.

b. The method of determining the required number of gate settings for each release is described as follows: Determine the discharge at the beginning and end of the gate setting from the discharge rating curve. If the difference in discharge is large or if the discharge rating curve is not a straight line over the range of discharges, the gate operation schedule should be divided into several gate settings. The duration of the initial block hydrograph is arbitrarily divided into smaller increments of time, and the volume of release computed for each time increment at the required initial peak discharge rate. The water surface elevation at the end of each time increment is determined from the elevation-capacity curves. The change in discharge from the beginning to the end of each time increment is determined from the discharge rating curve. If the change in discharge is still large, the hydrograph duration is broken up into shorter time increments, and the above procedure repeated.

37. Principles of Selecting the Gate Opening. The actual gate setting for a particular water-surface elevation is not specific, but is an arbitrary opening selected from several choices. The principles which should govern the ultimate choice are as follows:

(1) Regulating valves, normally found in dams for irrigation projects, give the closest regulation for the various types of outlet structures.

(2) Partially-opened gates on a spillway crest give better control over the discharge than partially-opened high pressure gates in the outlet conduits.

(3) The head on a partially-opened spillway gate should have a ratio of head-to-gate opening greater than 2 to give a reliable discharge rating.

(4) Generally, gates should be fully opened if possible to give the required discharges, leaving only one gate partially opened to give the final regulation required in the gate operation schedule.

(5) The stilling basin of an overfall or chute spillway may require a relatively even distribution of flow over the spillway to prevent excessive scour at the foundation of the dam. If such is the case, several or all spillway gates should be operated to give an even distribution of flow at partial gate opening rather than a single gate fully opened to give the required discharge.

(6) The time required to open the gates may be a controlling factor. The gate-operating machinery of a dam may not be workable because of the lack of power or enemy damage. In such cases, the gates should be operated by hand or by hoists that are powered by organizational equipment. Operation of as few gates as

possible to give the required discharges in this case may then be the controlling factor of the gate-operation schedule.

38. Gate-Operation Schedule. The gate-operation schedule is determined from the basic data and rating curves, reservoir water surface schedule, and gate-opening principles as described in the preceding paragraphs. The operation schedule is determined as follows:

(1) From the reservoir water surface schedule and discharge rating curves the required number of gate settings is determined, as described in Par. 36.

(2) The volume of water remaining in the reservoir after each gate setting is computed and the reservoir water-surface elevation determined from the elevation-capacity curve.

(3) From the tabulation of reservoir elevations, compute the elevation of the midpoint of the drop in the reservoir water surface for each gate setting.

(4) Enter the discharge rating curves of each outlet structure at the elevation of the midpoint of the drop in the reservoir water surface for the first gate setting. Determine the discharge for each type gate at various gate openings for this elevation.

(5) From the discharge of each gate and the number of gates involved, select the gate schedule that will give the required discharge, using the principles of operation stated in Par. 37. This determines the gate setting for the first increment of the initial hydrograph release.

(6) If the initial hydrograph release is divided into several time increments, the gate setting must be changed at the end of each time increment. To determine the gate opening for each setting the same procedure is carried out as described above. The gates are closed at the end of the last time increment. The method of computing an operation schedule is described in more detail in Chapter V for problem I.

39. Adjustment of Outflow Hydrograph Peak. As the reservoir water surface lowers, a point is usually reached in which the outlet structures will not release the required initial peak discharge of the block hydrograph. At this time the block hydrograph is adjusted by reducing the required peak discharge and increasing the duration of release. The procedure to accomplish this is to select a substantially lower peak discharge for the release hydrograph, and recompute the required duration to obtain the same downstream effects at the problem location. The method of computation is the same as described in Par. 21 to 27. When the new initial hydrograph peak discharge and duration have been determined, the computation of the gate-operation schedule is continued in the same manner as described in Par. 38.

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40. Cyclical Releases. When cyclical releases from the reservoir are involved, the time between beginning of releases of each cyclical block discharge must be determined as described i- Paragraph 22 in addition to the other initial data as outlined in Paragraph 34. The procedures outlined in this chapter are repeated for each successive release hydrograph to complete the gate-operation schedule for the series of cyclical releases.

CHAPTER V
EXAMPLES

41. Outline of Examples. Three example problems are presented in this section and are shown on Plates Nos. 6 to 20, inclusive. The problems are representative of the basic data and conditions that probably would exist under field conditions. The first two examples assume a uniform river channel with a varying bottom slope, and the third has a varying channel cross section as well as a changing bottom slope. The examples illustrate the methods of determining the required initial release block hydrograph that would cause flooding in a downstream reach, and the reverse procedure of determining the downstream flooding from a given initial release hydrograph.

42. Problem I. a. SITUATION. A large reservoir located on a tributary stream within the army combat zone appears to be capable of augmenting the defense of the river line to our immediate front. It is desired to evaluate the effects of artificial flooding by cyclical regulation to prevent an enemy assault crossing of the river.

b. GIVEN DATA. Construction drawings and other intelligence data furnished the following information:

(1)	Dam	
	Type of construction	Concrete gravity
(2)	Spillway	
	2 spillway gates	Tainter gates 50' long 40' high
	Spillway crest profile	See Plate No. 6
	Design Head	40'
	Coefficient of discharge at design head	3.96
(3)	Outlet Conduits	
	10 outlet conduits	Sq. 6' x 6' concrete
	Length	120 ft.
	Entrance condition	Bell mouth without trash racks
	Gates & location	Vertical-lift slide & guard gate 100 & 90 ft. from conduit entrance, respectively
	Gate seat elevation	1000 ft. msl
	Manning's "n"	0.013
	Tailwater effects on discharge	Negligible
(4)	Reservoir	
	Elevation-storage capacity curve shown on Plate No. 7	

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(5) Physical Characteristics of the River. The channel cross section is trapezoidal in shape, with a 40 ft. bottom width and 1 to 1 side slopes to a depth of 10 ft. The overbank side slopes are 1 on 10. The channel roughness coefficient is assumed to be 0.040 and the overbank roughness 0.080. The bottom slope varies in accordance with the equation:

$$S_m = 750/(150 + x)$$

Where: S_m = bottom slope in feet per mile
x = the distance in miles below the dam.

c. REQUIRED. The hydrographs and gate-operation schedule causing maximum surface velocities of 11 ft/sec and minimum trough discharges of 1700 cfs at a point 40 miles below the dam.

d. SOLUTION.

(1) Rating Curves of Outlet Structures. The discharge rating curves for a partially-opened tainter gate and single-outlet conduit gate (computed by the methods described on Plates 614, 805, 905, 908 of Military Hydrology Bulletin 12, "Handbook of Hydraulics") are shown on Plate No. 6.

(2) Hydraulic Elements of a Cross Section. The hydraulic elements of the 40-mile river reach are based upon the physical data of the river. The hydraulic elements of area, water surface width, average channel velocity, and conveyance are computed for the average cross section of each reach. The channel and overbank cross sections are assumed to be uniform for the entire river reach; therefore the hydraulic elements are computed for a single cross section, as shown on Plate No. 8a.

(3) Velocity and Discharge at Mile 40. The channel discharge at mile 40 is computed as the product of the channel conveyance and the square root of the bottom slope at mile 40. The average channel velocity is computed as the channel discharge divided by the channel area. The total discharge of the channel and overbank at mile 40 is computed as the product of the total conveyance and the square root of the bottom slope at mile 40. The average channel velocity and total discharge at mile 40 are computed and plotted on Plates No. 8a and 8b, respectively.

(4) Weighted Average Bottom Slope. The bottom slope of the channel changes for each mile of river reach in accordance with the equation given in the basic data. To determine the average hydraulic characteristics of the 40-mile reach of river it is necessary to compute a weighted bottom slope. The point bottom slope is computed every four miles from the dam through mile 40. The average slope is computed for each of the four-mile subreaches and is multiplied the length of the subreach. The above products are accumulated and divided by the sum of the subreaches to give the weighted bottom slope for any reach between the dam and a downstream point, as shown on Plate No. 9.

(5) Discharge Rating Curve for the 40-Mile Reach. The average discharge rating curve for the 40-mile river reach is computed as the product of the conveyance for the total cross section and the square root of the weighted average bottom slope for the 40-mile reach. The computation is shown in Col. 14 of Table I, Plate No. 8a, and the resulting curve plotted on Plate No. 8b.

(6) Average Hydraulic Characteristics for the 40-mile Reach. The average wave velocity, ΔK, and ΣK are determined from the hydraulic elements and the discharge rating curves of Plate 8b. For convenience, area and surface width curves may be plotted against discharge, as shown on Plate 8c. The values of ΔK are computed by Escoffier's equation as given in par. 12b (5). The computation of the average wave velocity, ΔK, ΣK, and $\Delta K \Sigma K$ is shown in Tables Nos. I and II of Plate 10a and these values are plotted on Plate 10b.

(7) Determination of the Downstream Peak Discharge (P_o). From the requirements of the problem it is desired to create a channel surface velocity of about 11 ft/sec at mile 40. Data from Military Hydrology Bulletin 2, "River Characteristics and Flow Analysis for Military Purposes", indicates that the channel surface velocity in a stream is about 1.5 times the average channel velocity. Therefore, an average channel velocity of about 7.5 ft/sec would be equivalent to a surface velocity of 11 ft/sec at mile 40. From Plate 8b with an average channel velocity of 7.5 ft/sec, the peak discharge at mile 40 is determined to be 14,000 cfs.

(8) Peak Discharge of Initial Block Hydrograph (P_i). The peak discharge of the release hydrograph at the dam is arbitrarily assumed. Its value is selected after considering the discharge capacity of the outlet structures and the effective storage in the reservoir as described in paragraphs 7 and 8. For this problem the peak discharge (P_i) is selected to be 28,000 cfs.

(9) Duration of Initial Block Hydrograph (L_i). The duration of the initial block hydrograph (L_i) for a peak discharge of 28,000 cfs is computed in Table I, Plate 11a. The general method of determining the initial hydrograph duration is as follows: A routing flow is first computed for an initial peak of 28,000 cfs from the peak modification and weighted discharge curves. The value of the product $\Delta K \Sigma K$ is then determined from the average hydraulic characteristic curves for the weighted discharge. From this, the initial hydrograph duration is computed as the square root of the ratio $\Delta K \Sigma K / \Delta K_r \Sigma K_r$. The time from beginning of release to the time of closure for each cyclical flood hydrograph is thus computed to be 4.55 hours.

(10) Duration of Cycle. The time from the beginning of release of the first cycle to the time of beginning of release for the second cycle is computed to be 16.4 hours, as explained in Table II, Plate 11a.

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(11) Downstream Hydrograph. The discharge ordinates and the time from the beginning of release from the dam are computed and explained in Table III, Plate 11b. The resulting hydrographs for the first two cycles are plotted on Plate 11c.

(12) Gate Operation Schedule. The gate operation schedule for ten cyclical release hydrographs is determined from the elevation-capacity curve (Plate No. 7), the discharge rating curves (Plate No. 6), and the hydraulic characteristic curves (Plate 10b). The reservoir water surface schedule is computed and explained in Table I, Plate 12a. Each release cycle is divided into two time increments of 2.55 hours and 2.0 hours. The time sequence of each gate setting and the gate schedule are computed and explained in Table II, Plate 12b.

43. <u>Problem II</u>. a. SITUATION. Assumed identical to Example Problem I, Par. 42a.

b. GIVEN DATA. The physical and hydrological data are the same as Example Problem I, Par. 42b.

c. REQUIRED. The downstream hydrograph at mile 21.07 when a block hydrograph is released with a peak discharge of 28,000 cfs and duration of 4.55 hours at the dam. The trough discharge between cyclical hydrographs at mile 21.07 is to be about 1700 cfs.

d. SOLUTION. (1) Average Hydraulic Characteristics of the Reach. The average wave velocity, ΔK and ΣK are determined in the same manner as described on Plate 10 for Example Problem I, but with a weighted bottom slope of 8.86×10^{-4}, as read from Plate 9. The average hydraulic characteristics for the 21.07 mile reach are shown on Plate 13.

(2) Determination of the Downstream Peak Discharge. A weighted discharge (P_w) is assumed and $\Delta K_r \Sigma K_r$ is determined from Plate 3 for the assumed value of P_w in terms of P_w % P_i. The value of $\Delta K \Sigma K$ is determined from Plate No. 13 for the assumed value of P_w. A check value of $\Delta K_r \Sigma K_r$, is computed by dividing the value of $\Delta K \Sigma K$ by the square of the hydrograph duration (L_i). If the values of $\Delta K_r \Sigma K_r$ check, the assumed value of the weighted discharge (P_w) was correct; conversely, if the values of $\Delta K_r \Sigma K_r$ do not check out, another trial is made with the new $\Delta K_r \Sigma K_r$. The weighted flow in percent of the initial hydrograph peak is determined from Plate No. 3 for the new value of $\Delta K_r \Sigma K_r$ and P_w computed from the initial peak discharge. The same procedure is followed as in the first trial in computing the check $\Delta K_r \Sigma K_r$. The process is continued until the two values of $\Delta K_r \Sigma K_r$ check. After the weighted discharge has been determined as described above, the downstream peak discharge is computed from Plate No. 2 for the $\Delta K_r \Sigma K_r$ determined from the final trial computation. The ordinates of the downstream hydrograph and the time of release from the dam is computed by the method described on Plate 11b. The downstream hydrograph for the first cycle at mile 21.07 is computed and explained on Plate 14a and plotted on Plate 14b.

44. <u>Problem III</u>. a. SITUATION. The situation is the same as in Example Problem I, Par. 42a.

b. GIVEN DATA. The information given on construction drawings and other intelligence data is the same as Example Problem I, Par. 42b, except that the river cross section and bottom slope have been changed as shown in the following table.

PHYSICAL AND HYDRAULIC DATA OF RIVER

Reach River Mile Below Reservoir	Channel Cross Section: Bottom Width Feet	Channel Cross Section: Side Slope	Stream Slope: Feet Per Mile	Stream Slope: S_o	Base Flow c.f.s.
0-10	40	1 on 1	10	0.00189	0
10-25	60	1 on 1-1/2	4	0.000757	0
25-45	80	1 on 2	2-1/2	0.000473	1,000

c. REQUIRED. The initial block hydrograph to cause a maximum surface velocity of 11 ft/sec at a point 45 miles below the dam with a minimum trough discharge of 1700 cfs.

d. SOLUTION. The discharge rating curves for the spillway gates and outlet conduits, and the reservoir storage curve are the same as Example Problem I, and are shown on Plates No. 6 and 7, respectively.

(1) Average Hydraulic Characteristics. The hydraulic elements of area water surface width, conveyance, and discharge are computed in the same manner as Example I for the three reaches (Mile 0-10, 10-25 and 25-45) and are shown on Plates No. 15 to 17. Values of ΔK, ΣK, and the product $\Delta K \Sigma K$ are computed for each reach and plotted on Plates No. 18 and 19. The total values of ΣK and $\Delta K \Sigma K$ are determined for the 45 miles of river and also plotted on Plates No. 18 and 19. The above hydraulic characteristics were computed in the same manner as described on Plate 10a.

(2) Determination of the Downstream Peak Discharge at Mile 45. The channel discharge at mile 45 and the average channel velocity are computed as described in Example I. For an average channel velocity of about 7.5 ft/sec (equivalent to a surface velocity of 11 ft/sec), the discharge of the total cross section is determined to be 32,500 cfs.

(3) Determination of the Initial Block Hydrograph. The duration of the initial block hydrograph for an assumed peak discharge of 60,000 cfs is computed on Plate No. 20a in the same manner as given for Example Problem I.

(4) Determination of the Downstream Hydrograph. The discharge ordinates for a single release hydrograph and their time from beginning of release are computed and explained on Plate No. 20a. The hydrographs for a single cycle is plotted on Plate No. 20b. Successive cyclical hydrographs can be computed in the same manner as for Example I as described on Plate 11b.

REFERENCES

Military Hydrology Bulletins

1. MHB 1: Applications of Hydrology in Military Planning and Operations
2. MHB 2: River Characteristics and Flow Analyses for Military Purposes
3. MHB 3: Stream-Gaging Methods and Equipment for Military Purposes
4. MHB 4: Transmission of Hydrologic Data for Military Purposes
5. MHB 5: Card-Indexing and Filing of Information Pertinent to Military Hydrology
6. MHB 6: Directory to European Sources of Information on Military Hydrology
7. MHB 7: Glossary of Terms Pertinent to Military Hydrology
8. MHB 8: Selected References on Military Hydrology
9. MHB 9: Flow Through a Breached Dam
10. MHB 10: Artificial Flood Waves
11. MHB 11: Regulation of Stream Flow for Military Purposes
12. MHB 12: Handbook of Hydraulics

Department of the Army Technical Bulletins.

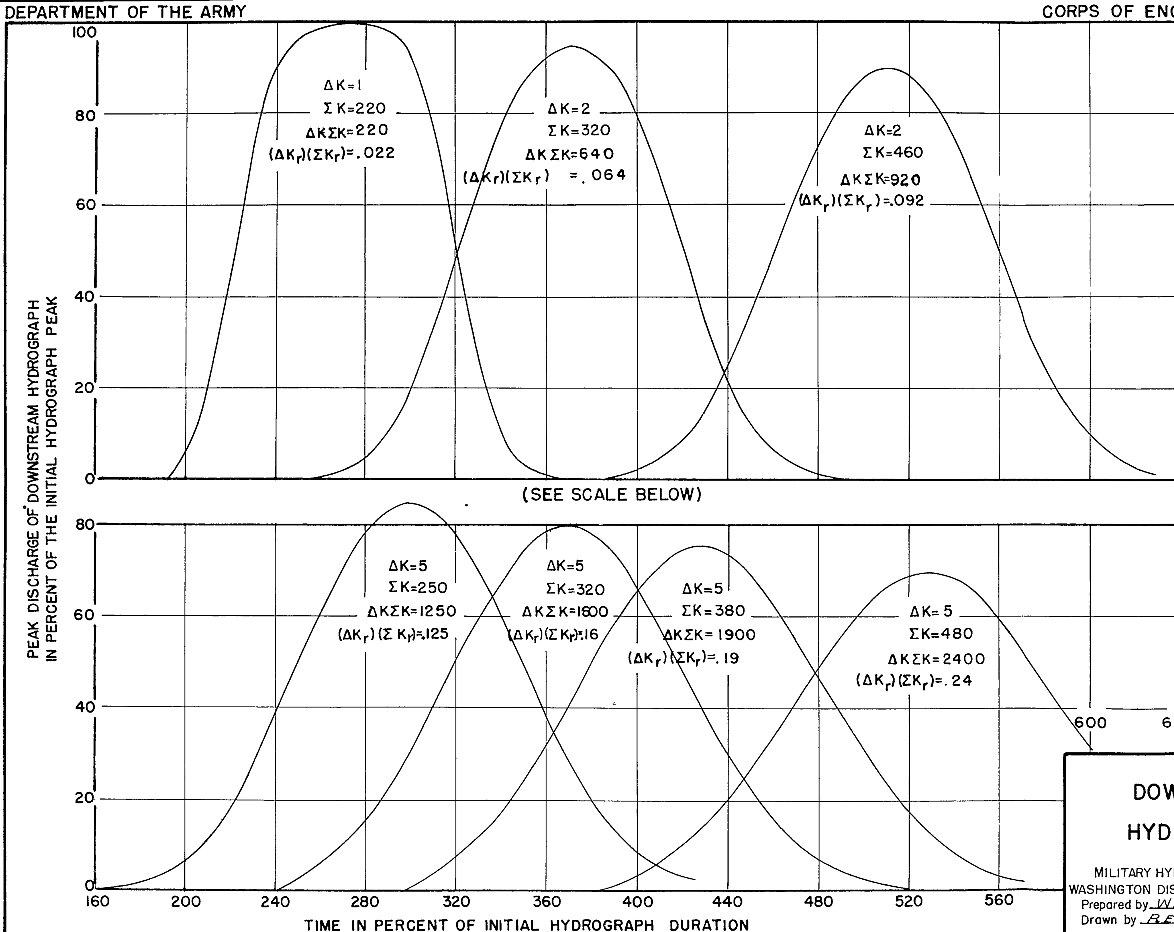
13. TB 5-550-1: Flood Prediction Services
14. TB 5-550-2: Compilation of Intelligence on Military Hydrology
15. TB 5-550-3: Flood Prediction Techniques

Other Publications.

16. Rouse, H. (Editor). Engineering Hydraulics, New York: John Wiley and Sons, 1950.
17. Part CXIV, Hydrology and Hydraulic Analysis, Chapter 8, "Routing of Floods Through River Channels". Engineering Manual for Civil Works, Office, Chief of Engineers, Corps of Engineers, Dept. of the Army, Sept. 1953.
18. "Flood Control," Engineering Construction Text X-156. The Engineer School, Fort Belvoir, Virginia, 1946.
19. Linsley, Kohler & Paulhus. Applied Hydrology. New York: McGraw-Hill, 1949.

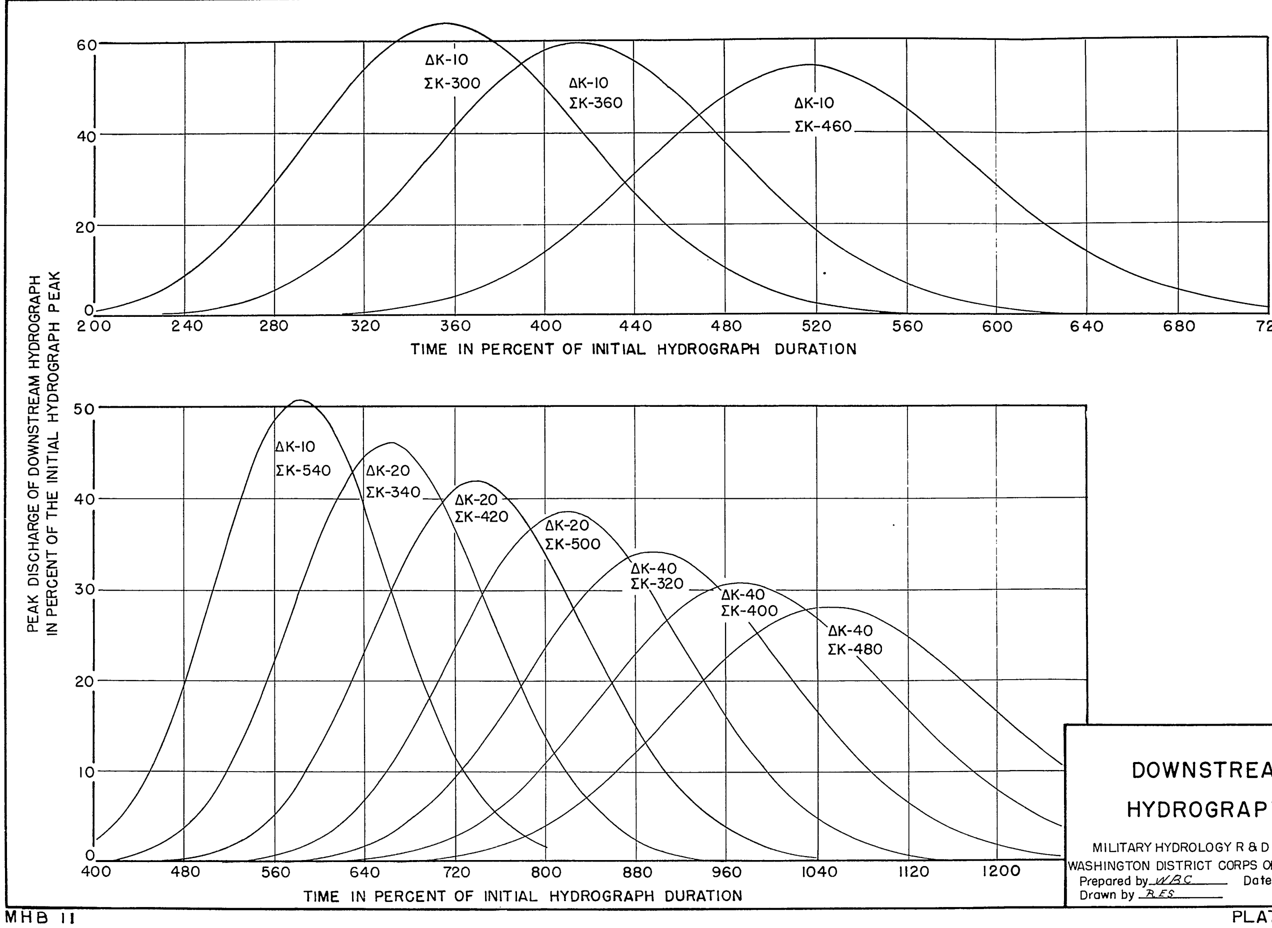
LIST OF PLATES

Declassified in Part - Sanitized Copy Approved for Release @ 50-Yr 2013/10/25 : CIA-RDP81-01043R002300060003-3



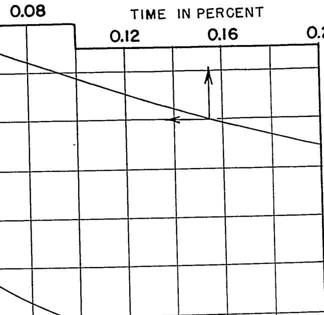
DOWNSTREAM HYDROGRAPHS

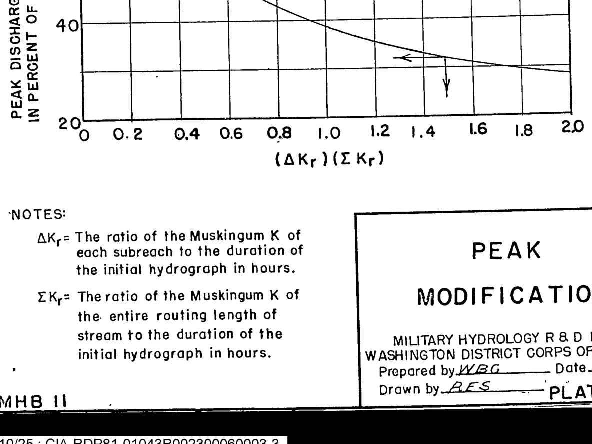
MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBC Date Aug 1955
Drawn by RFS



DOWNSTREAM HYDROGRAPHS

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBC Date Aug 1955
Drawn by RES





NOTES:

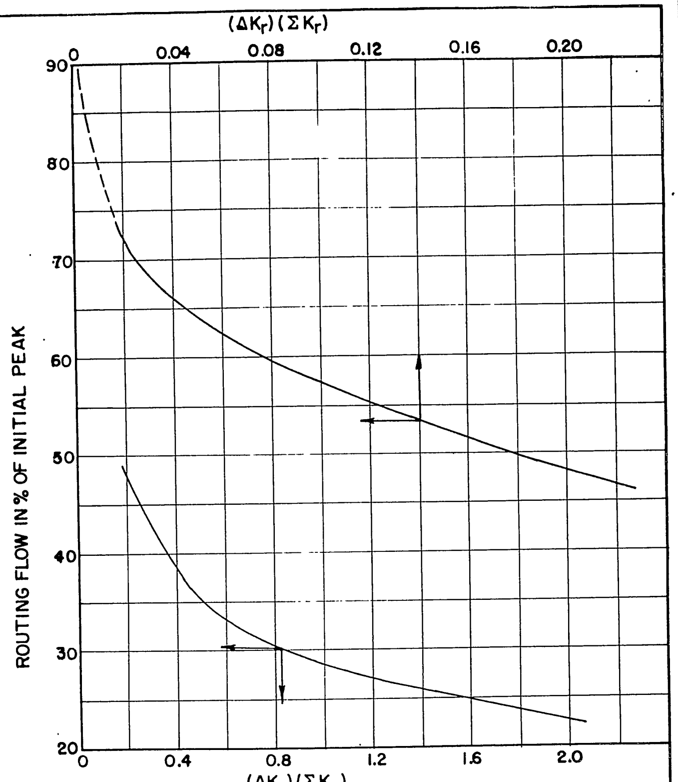
ΔK_r = The ratio of the Muskingum K of each subreach to the duration of the initial hydrograph in hours.

ΣK_r = The ratio of the Muskingum K of the entire routing length of stream to the duration of the initial hydrograph in hours.

PEAK MODIFICATION

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBG Date July 1955
Drawn by RES

PLATE NO. 2



NOTES

ΔK_r = The ratio of the Muskingum K of each subreach to the duration of the initial hydrograph in hours.

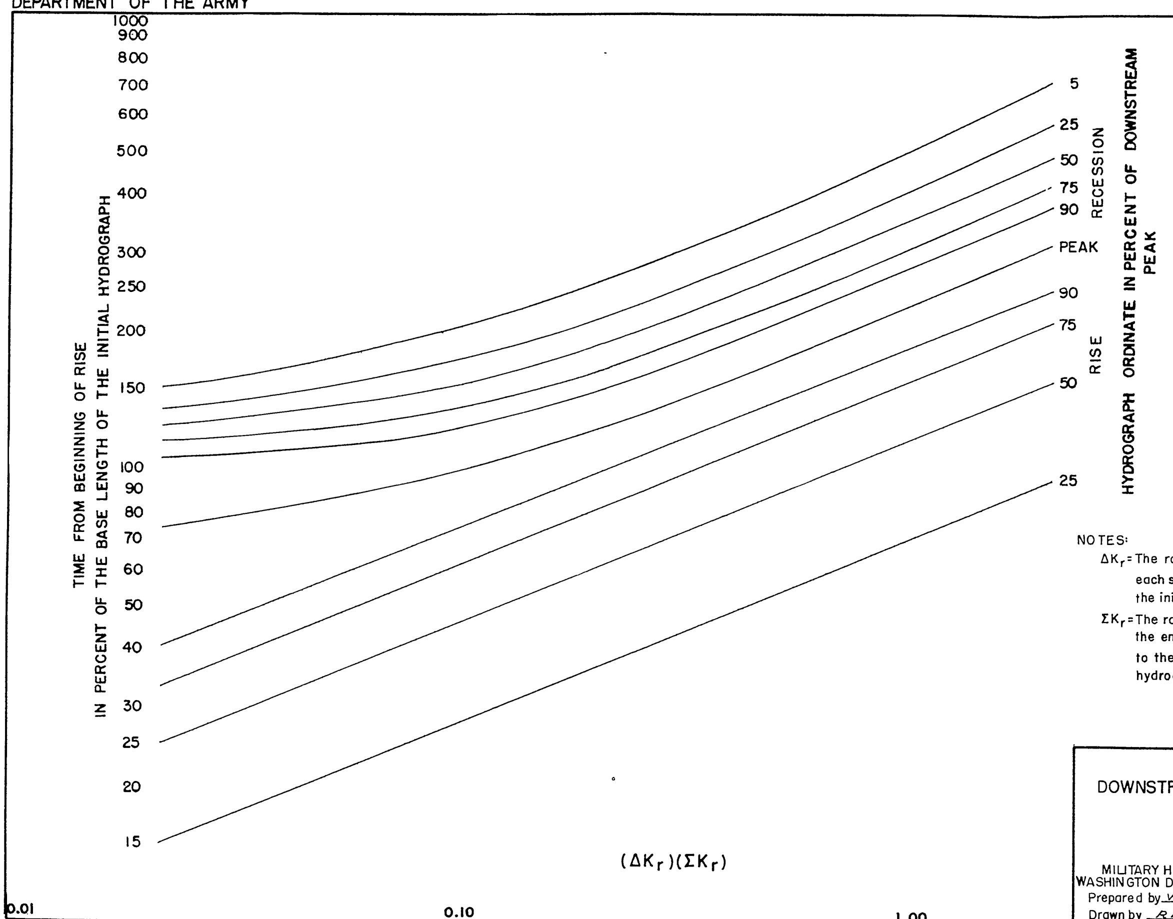
ΣK_r = The ratio of the Muskingum K of the entire routing length of stream to the duration of the initial hydrograph in hours.

WEIGHTED DISCHARGE

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBC Date July 1955
Drawn by RES

PLATE NO. 3

MHB 11



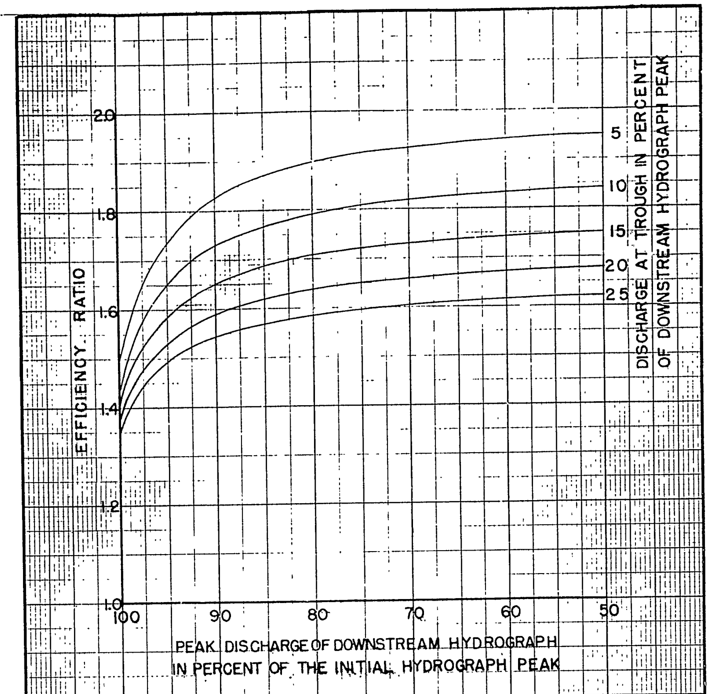
NOTES:

ΔK_r = The ratio of the Muskingum K of each subreach to the duration of the initial hydrograph in hours.

ΣK_r = The ratio of the Muskingum K of the entire routing length of stream to the duration of the initial hydrograph in hours.

DOWNSTREAM HYDROGRAPH ORDINATES

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBC Date July 1955
Drawn by RES



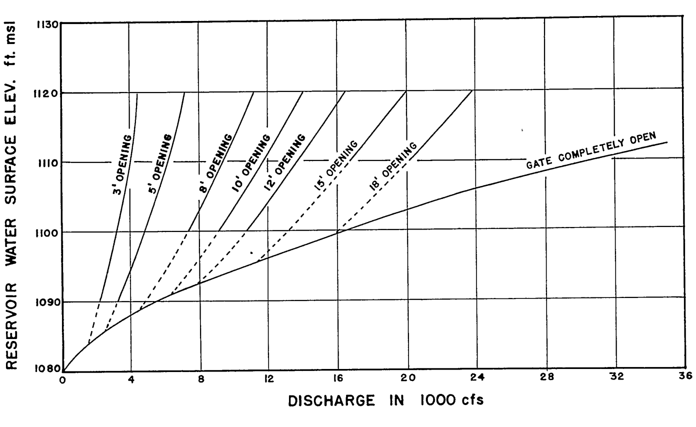
EFFICIENCY RATIO OF CYCLICAL OPERATION

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBC Date JULY 1955
Drawn by RES

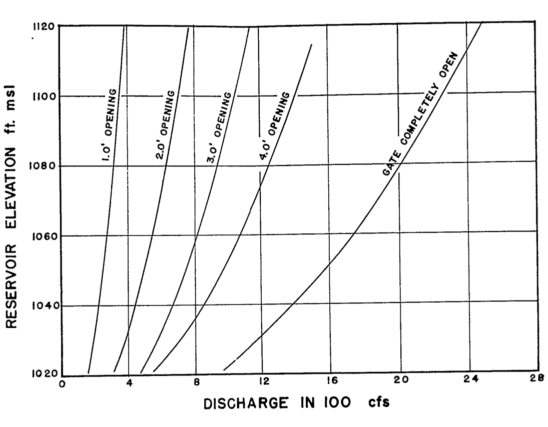
MHB 11

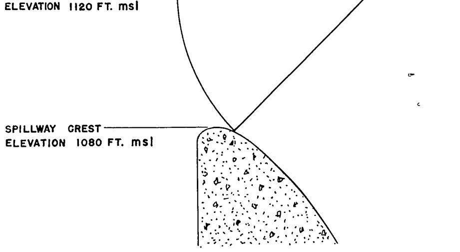
PLATE NO. 5

SINGLE TAINTER GATE 50' x 40'



SINGLE OUTLET CONDUIT 6' x 6'





SPILLWAY CREST & TAINTER GATE DETAIL

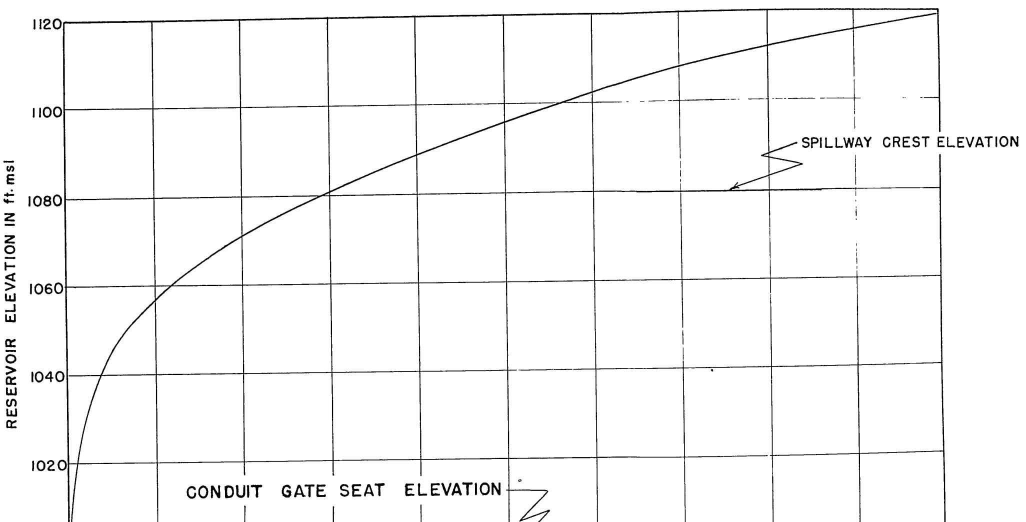
NOTE:
DESIGN HEAD 40'
$C_d = 3.96$

EXAMPLE PROBLEM I

DISCHARGE RATING

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by W.B.C. Date 4 August, 1955
Drawn by J.A.M.

PLATE NO. 6



EXAMPLE PROBLEM I

RESERVOIR STORAGE

MILITARY HYDROLOGY R&D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by W.B.C. Date Aug 1955
Drawn by RES/JAM

PLATE NO. 7

DETERMINATION OF THE AREA SURFACE WIDTH, CONVEYANCE, AVERAGE CHANNEL VELOCITY AND DISCHARGE CURVE AT MILE (40) FORTY

INITIAL DATA

Item

(1) Trapezoidal channel 40 feet bottom width and 1:1 side slopes for a depth of 10 feet
(2) Overbank side slope 1:10
(3) Channel n = 0.040, overbank n = 0.080
(4) Bottom slope:

$S_m = 750/(150 + x)$

where S_m = Bottom slope in feet/mile = 750/190 = 3.947
x = Distance in miles below dam = 40
S_o = Bottom slope in ft./ft. = 0.000748 at Mile 40

TABLE I
SUMMARY OF COMPUTATIONS

CHANNEL						OVERBANK			TOTAL SECTION				REACH
y	A	b_w	$k' = \frac{1.486}{n} AR^{2/3}$	$Q@S_o = 7.48 \times 10^{-4}$	V	A	b_w	$k' = \frac{1.486}{n} AR^{2/3}$	A	b_w	k' = (Cols. 4+9)	$Q@S_o = 7.48 \times 10^{-4}$	$Q@S_o = 8.4 \times 10^{-4}$
feet	feet^2	feet	Units x 10^4	1000 cfs	ft./sec	feet^2	feet	Units x 10^4	feet^2	feet	Units x 10^4	1000 cfs	1000 cfs
Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9	Col. 10	Col. 11	Col. 12	Col. 13	Col. 14
2	84	44	0.469	0.128	1.52				84	44	0.47	0.128	0.135
4	176	48	1.49	0.408	2.32				176	48	1.49	0.407	0.43
6	276	52	2.94	0.804	2.91				276	52	2.94	0.83	0.85
8	384	56	4.78	1.31	3.41				384	56	4.78	1.31	1.39
10	500	60	7.00	1.91	3.82	0	0	0.00	500	60	7.00	1.92	2.03
12	620		10.02	2.74	4.42	40	40	0.074	660	100	10.1	2.76	2.92
14	740		13.47	3.68	4.97	160	80	0.470	900	140	13.9	3.80	4.02
16	860		17.3	4.73	5.50	360	120	1.39	1220	180	18.7	5.11	5.40
18	980		21.5	5.88	6.00	640	160	3.0	1620	220	24.5	6.70	7.08
20	1100		26.1	7.13	6.48	1000	200	5.4	2100	260	31.5	8.61	9.10
25	1400		39.0	10.6	7.61	2250	300	16.0	3650	360	55.0	15.0	15.9
30	1700	60	53.9	14.7	8.66	4000	400	34.4	5700	460	88.3	24.1	25.5

EXPLANATION OF COMPUTATIONS

Item

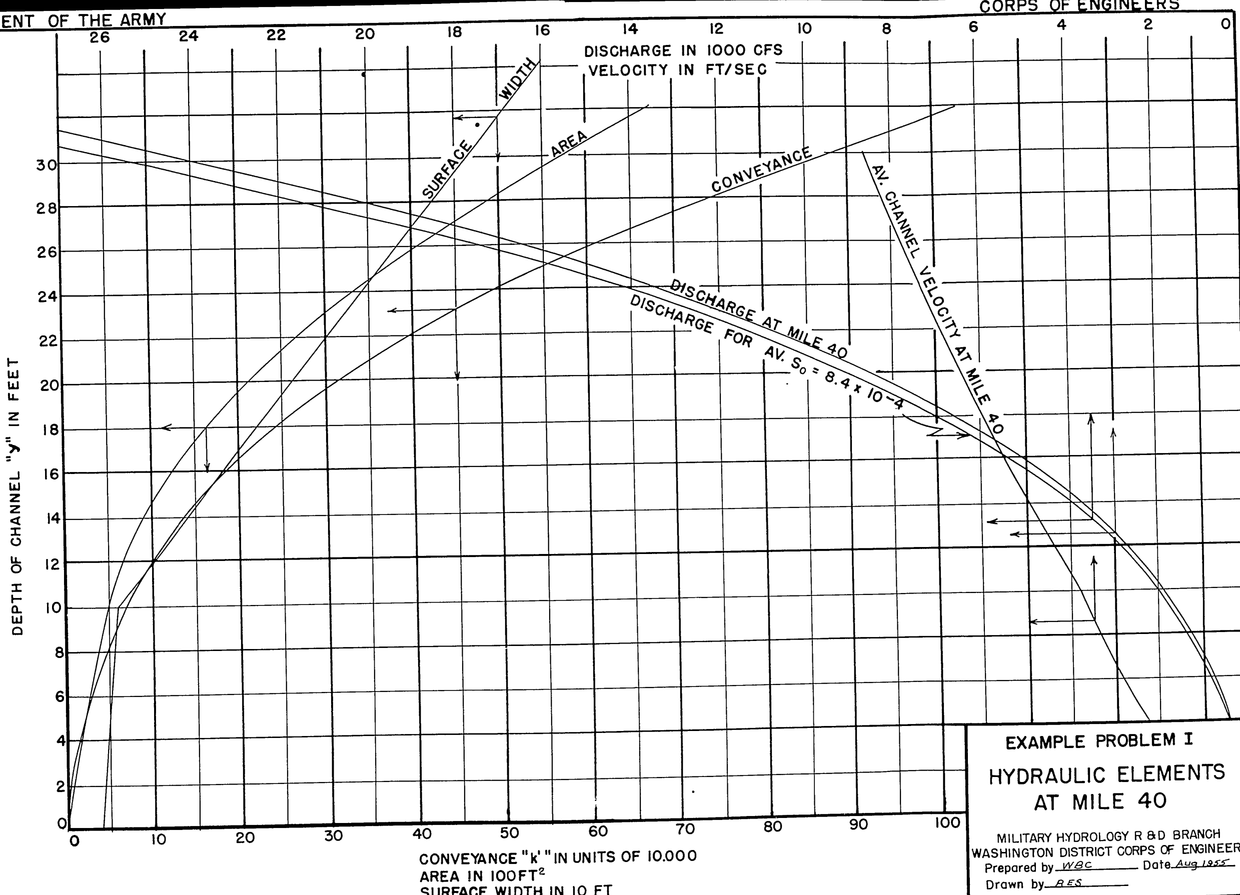
(1) The hydraulic elements given in Table I are computed by the methods described on Plate 507 M H Bulletin 12, "Handbook of Hydraulics".

(2) The channel discharge in Column 5 is computed by Mannings equation for the channel cross section and bottom slope at mile 40.

(3) The average channel velocity (Column 6) is computed by dividing the discharge of Column 5 by the area of Column 2.

(4) The discharge at mile 40 (Column 13) for the total cross section is computed as the product of the total conveyance of Column 12 and the square root of the bottom slope at mile 40.

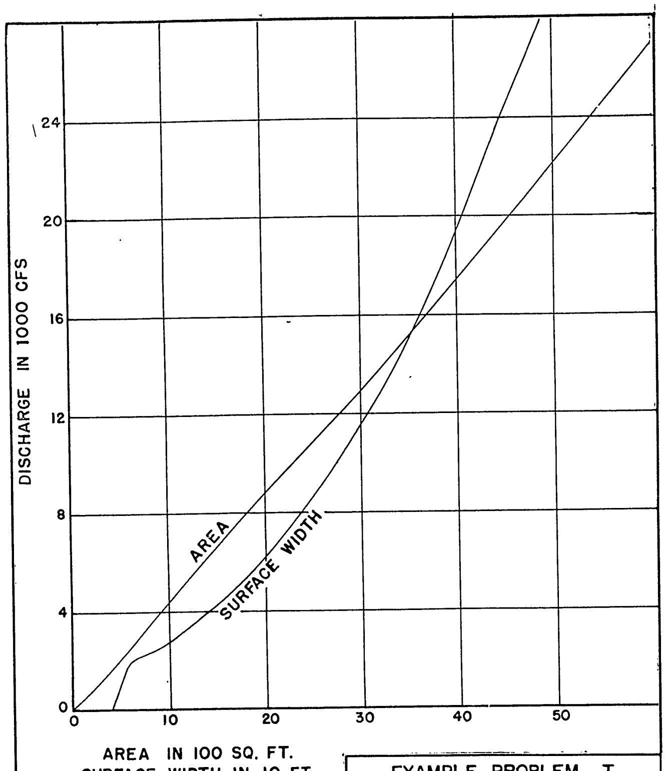
(5) The average discharge in the forty mile reach is computed as the product of the total conveyance of Column 12 and the square root of the weighted average bottom slope of the 40 mile reach (8.4×10^{-4} taken from Plate 9) and entered in Column 14.



EXAMPLE PROBLEM I

HYDRAULIC ELEMENTS AT MILE 40

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBC Date Aug 1955
Drawn by RES

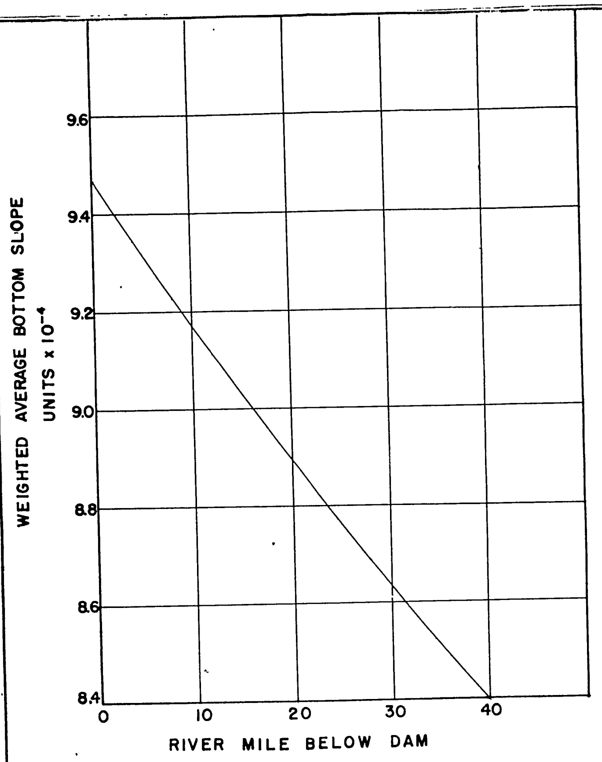


EXAMPLE PROBLEM I
AVERAGE DISCHARGE – AREA AND SURFACE WIDTH CURVES
40 MILE REACH

MILITARY HYDROLOGY R&D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by CLW Date DEC. 2, 1955
Drawn by JAM

PLATE NO. 8c

MHB 11



EXAMPLE PROBLEM I

WEIGHTED AVERAGE

BOTTOM SLOPE

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBC Date Aug 1955
Drawn by RES

PLATE NO.9

MHB 11

DETERMINATION OF THE AVERAGE HYDRAULIC CHARACTERISTICS OF MILE FORTY REACH

TABLE I
SUMMARY OF COMPUTATIONS OF ΔK AND ΣK

Q 1000 cfs	A feet^2	$U = \Delta Q/\Delta A$ feet/sec	U^2	b_w feet	$\Delta K = \frac{0.435Q}{U^2 b_w}$ hours	$\Sigma K = \frac{211,200}{3600 U}$ hours
Col. 1	Col. 2	Col. 3	Col. 4	Col.5	Col. 6	Col. 7
1.0	300	3.33	11.1	53	0.74	17.6
2.0	500	5.00	25.0	60	0.58	11.7
3.0	690	5.26	27.6	107	0.44	11.1
4.0	930	4.17	17.4	140	0.71	14.1
6.0	1380	4.44	19.7	196	0.68	13.2
8.0	1840	4.35	18.9	238	0.77	13.5
10.0	2310	4.26	18.1	275	0.87	13.8
12.0	2780	4.26	18.1	308	0.94	13.8
16.0	3690	4.40	19.3	362	1.00	13.3
20.0	4520	4.82	23.2	405	0.92	12.2
24.0	5370	4.71	22.1	445	1.06	12.4
28.0	6190	4.88	23.8	484	1.06	12.0

TABLE II
AVERAGE HYDRAULIC CHARACTERISTICS OF FORTY MILE REACH

Q 1000 cfs	Av. ΔK from curve	Av. ΣK from curve	$\Delta K \cdot \Sigma K$ hours^2
Col.1	Col.2	Col.3	Col. 4
6.0	0.74	13.4	9.9
8.0	0.82	13.4	11.0
10.0	0.88	13.4	11.8
12.0	0.93	13.4	12.45
14.0	0.96	13.3	12.75
16.0	0.97	13.2	12.8
18.0	0.98	12.85	12.6
20.0	0.99	12.6	12.47
22.0	0.99	12.4	12.27
24.0	1.00	12.2	12.2
26.0	1.00	12.2	12.2
28.0	1.00	12.1	12.1

TABLE I
EXPLANATION OF COMPUTATIONS

Col.

(1) Discharges are selected to cover the range of flows expected in the problem and entered in Column 1.

(2) The total channel and overbank areas in Column 2 are determined for the discharges of Column 1. The cross sectional area is determined from the hydraulic element curves for an average bottom slope of 8.4×10^{-4} on Plate 8b or directly from Plate 8c.

(3) The average wave velocity in Column 3 is computed as the ratio of the increment in discharge to the increment in cross sectional area for each discharge.

(4) The average wave velocities of Column 3 are squared and entered in Column 4.

(5) The water-surface widths at each discharge of Column 1 are determined from Plate 8c and entered in Column 5.

(6) The value of ΔK is computed by means of the Escoffier Equation, described in Paragraph 12, and entered in Column 6.

(7) The ΣK is computed in Column 7 for each discharge as the ratio of the forty mile reach length in feet to the average wave velocity in feet/sec.. The ratio is divided by 3600 to convert ΣK from seconds to hours.

Note The values of ΔK and ΣK in Columns 6 & 7 respectively are plotted against the corresponding values of the discharge in Column 1 and a smooth curve drawn through the points, as shown on Plate 10b.

TABLE II
EXPLANATION OF COMPUTATIONS

Col.

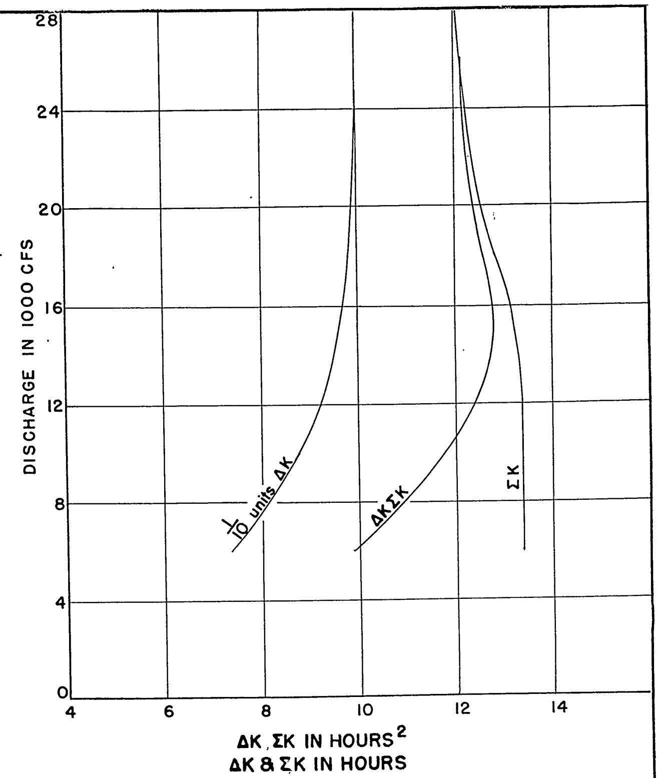
(1) Representative discharges.

(2) For each discharge listed in Column 1, the average value of ΔK is determined from curve on Plate 10b and entered in Column 2.

(3) For each discharge listed in Column 1, the average value of ΣK is determined from curve on Plate 10b and entered in Column 3.

(4) The products of the values ΔK and ΣK are determined and entered in Column 4.

Note The values of ΔK ΣK of Column 4 are plotted against the corresponding discharges of Column 1 and a smooth curve drawn between the points as shown on Plate 10b.



EXAMPLE PROBLEM I

AVERAGE HYDRAULIC CHARACTERISTICS

40 MILE REACH

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBC Date Aug 1955
Drawn by RES PLATE NO. 10b

MHB 11

DETERMINATION OF CYCLICAL HYDROGRAPHS AT MILE FORTY

TABLE I
DURATION OF INITIAL BLOCK HYDROGRAPH

Line	Item	Unit
1	P_o	14,000 cfs
2	P_i	28,000 cfs
3	P_o % P_i	50%
4	$\Delta K_r \ \Sigma K_r$	0.566
5	P_w % P_i	33.8%
6	P_w	9,460 cfs
7	ΣK	13.4 hours
8	$\Delta K \, \Sigma K$	11.65 hours^2
9	$(L_i)^2$	20.6 hours^2
10	L_i	4.55 hours

EXPLANATION OF COMPUTATIONS
TABLE I

Line

(1) The downstream peak discharge (P_o) is determined from the discharge curve at mile forty, (Plate 8b) for an average channel velocity of 7.5 feet / second.

(2) The initial block hydrograph peak is assumed after considering the factors described in Paragraphs 7 and 8.

(3) The downstream peak is determined as a percent of the initial peak (14,000/28,000) 100.

(4) The ratio $\Delta K_r \Sigma K_r$ is determined from the "Peak Modification Curve" (Plate 2) for P_o = 50% P_i

(5) The routing flow P_w in percent of the initial peak is determined from the Weighted Discharge Curve (Plate 3) for a $\Delta K_r \Sigma K_r = 0.566$ (Line 4).

(6) The routing flow (P_w) is computed as the product of 33.8% and 28,000 cfs (Lines 5 & 2).

(7) The value of ΣK is determined from Plate 10b for a routing flow of 9,460 cfs (Line 6).

(8) The value of $\Delta K \Sigma K$ is also determined from Plate 10b for a P_w of 9,460 cfs.

(9) The square of the initial hydrograph duration is computed as the ratio of $\Delta K \Sigma K$ on Line 8 to the value of $\Delta K_r \Sigma K_r$ on Line 4. $11.65/0.566 = 20.6$ hours^2.

(10) The square root of Line 9 is the initial hydrograph duration in hours.

TABLE II
DURATION OF ONE CYCLE

Line	Item	Unit
1	Duration of P_o for equivalent volume hydrograph	9.1 hours
2	P_t % P_o	12.1%
3	Efficiency ratio	1.80
4	Duration of one cycle	16.4 hours

EXPLANATION OF COMPUTATIONS
TABLE II

Line

(1) The duration of an equivalent block release having a peak discharge equal to the downstream hydrograph peak is computed as $L_i P_i / P_o = 4.55 \times 28{,}000/14{,}000 = 9.1$ hours. These values are taken from Lines 10, 2, and 1, respectively, Table I.

(2) The trough discharge (P_t in basic data) is computed in percent of P_o. $(1700/14{,}000) \cdot 100 = 12.1\%$.

(3) The efficiency ratio is determined from Plate 5 for the trough discharge 12.1% P_o (Line 3) and the downstream peak discharge of 50% P_i (Line 3, Table I).

(4) The duration of one cycle is computed as the product of the efficiency ratio (Line 4) and the duration of the equivalent block hydrograph (Line 1), $1.80 \times 9.1 = 16.4$ hours.

DETERMINATION OF CYCLICAL HYDROGRAPHS AT MILE FORTY

TABLE III
DOWNSTREAM CYCLICAL HYDROGRAPHS

Line	FIRST CYCLE					SECOND CYCLE	HYDROGRAPH TROUGH			
	percent of downstream Peak (P_o) %	Time % L_i %	Time beginning of rise hours	Discharge cfs	Time from beginning of release at dam hours	Time from beginning of release at dam hours	Time hours	Discharge 1st cycle cfs	Discharge 2nd cycle cfs	Discharge total cfs
	Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9	Col. 10
1	0	0	0	0	7.4	23.8	23.8	1800	0	1800
2	25	57	2.6	3500	10.0	26.4	25.0	900	1500	2400
3	50	94	4.3	7000	11.7	28.1	26.0	300	3000	3300
4	75	126	5.7	10500	13.1	29.5	27.0	0	4800	4800
5	90	150	6.8	12600	14.2	30.6				
6	100	183	8.3	14000	15.7	32.1				
7	90	224	10.2	12600	17.6	34.0				
8	75	246	11.2	10500	18.6	35.0				
9	50	282	12.8	7000	20.2	36.6				
10	25	323	14.7	3500	22.1	38.5				
11	5	391	17.8	700	25.2	41.6				

EXPLANATION OF COMPUTATIONS

FIRST HYDROGRAPH

Col

(1) The values of the parameters on Plate 4 (Hydrograph Ordinates in Percent of the Downstream Peak) are entered in Column 1.

(2) The time from beginning of rise of the downstream hydrograph (expressed in percent of the initial hydrograph duration) is determined from Plate 4 for each ordinate of Column 1. The time of each ordinate is taken for the value of $\Delta K_r \Sigma K_r$ of 0.566 (determined on Line 4, Table I Plate 11a) and entered in Column 2.

(3) The time from the beginning of rise of Column 3 is computed as the product of the values of L_i and Column 2. The duration of the initial hydrograph (L_i) is taken from Line 10, Table I, Plate 11a, to be 4.55 hours.

(4) The downstream hydrograph ordinates are computed as the product of Column 1 and the required downstream peak discharge of 14,000 cfs and entered in Column 4.

(5) The time from beginning of release at the dam to the time of peak of the downstream hydrograph is computed as the sum of ΣK and $L_i/2$ which equals 15.7 hrs.. ΣK and L_i are taken from Lines 7 & 10 respectively in Table I, Plate 11a.

Note: The time of beginning of rise of the downstream hydrograph is computed as the difference of Line 6, Col. 5 and the time of peak on Line 6, Col. 3, Table III. The time of beginning of rise equals 7.4 hours.

Column 5 lists time from beginning of release to each downstream hydrograph ordinate and is computed as the sum of Line 1, Col. 5 and Col. 3. The first cyclical hydrograph is plotted on Plate 11c from Col. 4 and Col. 5.

SECOND HYDROGRAPH

Col

(6) The time in hours from the beginning of release at the dam for the first cycle to each downstream hydrograph ordinate of the second cycle is listed in Column 6. The duration of one cycle (16.4 hours from Line 5, Table II) is added to the values of Column 5. The values of Column 4 & Column 6 are plotted on Plate 11c and determined the second cyclical hydrograph.

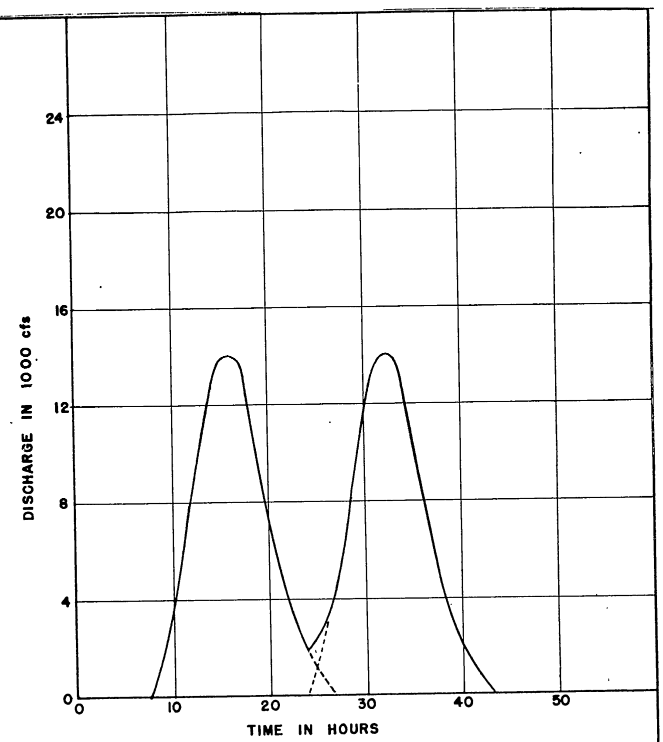
HYDROGRAPH TROUGH

(7) Time abscissas are selected to define the hydrograph trough and entered in Column 7.

(8) The discharges on the recession leg of the first hydrograph are determined from Plate 11c at the times of Column 7 and entered in Column 8.

(9) The discharges on the rising leg of the second hydrograph are read at the times of Column 7 and entered in Column 9.

(10) The hydrograph trough discharge is equal to the sum of Columns 8 & 9 and entered in Column 10. The trough hydrograph is plotted on Plate 11c at the times of Column 7 and the discharges of Column 10.



EXAMPLE PROBLEM I

HYDROGRAPHS

AT MILE 40

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by W.B.C. Date 9 August, 1955
Drawn by J.A.M. PLATE NO. 11c

MHB 11

DETERMINATION OF THE GATE OPERATION SCHEDULE

TABLE I
RESERVOIR WATER SURFACE SCHEDULE

Cycle No.	Reservoir Inflow ac.ft.	Reservoir storage End of cycle ac.ft.	Reservoir elevation End of cycle ft.msl	Water surface drop per cycle ft.
Col. 1	Col. 2	Col. 3	Col. 4	Col. 5
Initial	0.0	200,000	1120.0	0.0
1	0.0	189,450	1118.0	2.0
2	0.0	178,900	1116.0	2.0
3	0.0	168,350	1114.0	2.0
4	0.0	157,800	1112.0	2.0
5	0.0	147,250	1110.0	2.0
6	0.0	136,700	1107.4	2.6
7	0.0	126,150	1104.5	2.9
8	0.0	115.600	1101.2	3.3
9	0.0	105,050	1097.8	3.4
10	0.0	94,500	1094.0	3.8

EXPLANATION OF COMPUTATIONS

Col.

(1) The initial reservoir conditions and the hydrograph cycle number are listed in Column 1.

(2) The inflow into the reservoir during the release period is listed in Column 2.

(3) The reservoir storage at the end of each cyclical release is listed in Column 3. The volume of water released in each hydrograph (10,550 acre feet taken from Line 1, Table II, Plate 11a) is subtracted from the reservoir storage remaining after the previous release and the inflow of Column 2 is added.

(4) The reservoir elevation at the end of each release cycle is determined from the reservoir storage curve (Plate 7) for each storage of Column 3 and entered in Column 4.

(5) The difference in water surface elevation for each release cycle is computed as the difference in elevations of Column 4 and entered in Column 5.

DETERMINATION OF THE GATE OPERATION SCHEDULE

TABLE II
GATE OPERATING SCHEDULE

Line	Hydrograph Cycle No.	Time Increment hours	Volume of Release ac.ft.	Reservoir Storage End of Per. ac.ft.	Res. W.S. Elevation End of Per. ft.msl	Time "H" hours Hrs. - Min.	Gate Operating Schedule	Alternate Setting
	Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8
1	1	0.00		200,000	1120.0	0 - 0	2 tainter gates at 10' opening	
2		2.55	5900	194,100	1119.0	2 - 33	2 tainter gates at 10' & 1 conduit at 1.7' opening	
3		2.00	4650	189,450	1118.0	4 - 33	Close spillway and conduit gates.	
4	2	0.00				16 - 24	2 tainter gates at 10.5' opening	
5		2.55	5900	183,550	1117.0	18 - 57	2 tainter gates at 10.5' & 1 conduit at 1.7' opening	
6		2.00	4650	178,900	1116.0	20 - 57	Close spillway and conduit gates.	
7	3	0.00				32 - 48	2 tainter gates at 11' opening	
8		2.55	5900	173,000	1115.0	35 - 21	2 tainter gates at 11' & 1 conduit at 1.1' opening	
9		2.00	4650	168,350	1114.0	37 - 21	Close spillway and conduit gates.	
10	4	0.00				49 - 12	2 tainter gates at 11' & 1 conduit at 2.2' opening	
11		2.55	5900	162,450	1113.0	51 - 45	2 tainter gates at 11' & 2 conduits open	
12		2.00	4650	157,800	1112.0	53 - 45	Close spillway and conduit gates.	
13	5	0.00				65 - 36	2 tainter gates at 12' opening (Near limit of gate opening for head on gate)	
14		2.55	5900	151,900	1111.0	68 - 09	2 tainter gates at 12' & 1 conduit at 1.1' opening	
15		2.00	4650	147,250	1110.0	70 - 09	Close spillway and conduit gates.	
16	6	0.00				82 - 00	2 tainter gates at 12.5' opening	1 tainter gate fully open for entire 4.55 hourperiod (Av. Q slightly high).Stilling action may be bad.
17		2.55	5900	141,350	1108.4	84 - 33	2 tainter gates at 12.5' & 1 conduit at 2.7' opening	
18		2.00	4650	136,700	1107.4	86 - 33	Close spillway and conduit gates.	
19	7	0.00				98 - 24	2 tainter gates at 13.5' opening	1 tainter gate fully open for entire 4.55 hour period Stilling action may be bad.
20		2.55	5900	130,800	1106.0	100 - 57	2 tainter gates at 14' opening	
21		2.00	4650	126,150	1104.5	102 - 57	Close spillway gates.	
22	8	0.00				114 - 48	1 tainter gate 7.0' opening & 1 tainter gate full open	
23		2.55	5900	120,250	1102.8	117 - 21	1 tainter gate 7.5' opening & 1 tainter gate full open	
24		2.00	4650	115,600	1101.2	119 - 21	Close spillway gates.	
25	9	0.00				131 - 12	1 tainter 10' opening, 1 tainter & 1 conduit full open, 1 conduit 0.7' open	
26		2.55	5900	109,700	1099.0	133 - 45	2 tainter gates full open	
27		2.00	4650	105,050	1097.8	135 - 45	Close spillway gates.	
28	10	0.00				147 - 36	2 tainter gates & 1 conduit full open, 1 conduit 0.7' open	
29		2.55	5900	99,150	1096.0	150 - 09	2 tainter gates & 3 conduits full open, 1 conduit 0.7' open	
30		2.00	4650	94,500	1094.0	152 - 09	Close spillway and conduit gates.	

EXPLANATIONS OF COMPUTATIONS

Item

(1) From an inspection of the amount of drop in the reservoir water surface for each release cycle as given in Column 5, Table I, and the slope of the discharge rating curves on Plate 6 the duration of each release cycle is divided into 2 time increments of 2.55 hours and 2.00 hours respectively.

(2) The volume of water released in each 2.55 hour and 2.00 hour increment period is determined as the product of the time increment in hours, the peak discharge in cfs, and the constant 0.0826.

(3) The hydrograph cycle number, the time increment from beginning of release for each hydrograph, and the volume of water released during each increment are entered in Columns 1, 2 & 3 respectively.

(4) The reservoir storage at the end of each increment is computed as the difference in reservoir storage at the end of the previous time increment and the volume released in each increment. The reservoir storage at the end of each increment period is entered in Column 4.

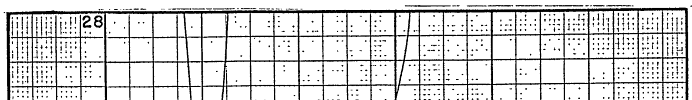
(5) The reservoir water surface elevation is determined from the storage curve (Plate 7) for each volume given in Column 4, and entered in Column 5.

(6) The time from the beginning of release of the first cyclical hydrograph to the beginning of release of eachsucceeding hydrograph is determined by adding the time increment of one cycle (16.4 hrs from Line 5, Table II, Plate 11-1) to the preceding hydrograph initial time. The lines giving the hydrograph cycle numbers in Column 1 also list the time of beginning of release of each hydrograph in Column 6.

(7) The time of the intermediate gate settings is computed as the sum of the time increments of Column 2 (2 hrs - 33 min. & 2 hrs) and the preceding time increment.

(8) The gate operating schedule is determined from the discharge rating curves at a head equal to the midpoint elevation of the reservoir for each gate setting (Column 5). At time zero (line 1, Column 6) the reservoir water surface is at elevation 1120.0 ft. msl (Line 1, Column 5). At the end of the time increment for the gate setting (2.55 hrs.) the water surface lowers one foot to elevation 1119.0 ft. msl. The average elevation for the gate setting is therefore 1119.5 ft. msl. A gate setting is selected from Plate 6 that will release 28,000 cfs at an average elevation of 1119.5 ft. msl. The gate setting selected is arbitrary as there are a large number of combinations of gate openings that will release the required discharge. From the considerations given in Section IV, "Gate Operating Schedule" the gate opening is selected to be 10' for the 2 tainter gates.

(9) The other gate settings of the operating schedule are determined in the same manner as in Item (8) above.



EXAMPLE PROBLEM II

AVERAGE HYDRAULIC CHARACTERISTICS

21.07 MILE REACH

MILITARY HYDROLOGY R&D BRANCH
WASHINGTON DISTRICT, CORPS OF ENGINEERS

Prepared by WBC Date Aug 1955
Drawn by RES

PLATE NO. 13

MHB 11

DETERMINATION OF DOWNSTREAM HYDROGRAPH AT MILE 21.07

TABLE I
DOWNSTREAM PEAK DISCHARGE

Line	Item	Unit
1	P_i	28,000 cfs
2	L_i	4.55 hrs
	FIRST TRIAL	
3	Assume P_w	12,000 cfs
4	P_w % P_i	42.9%
5	$\Delta K_r\ \Sigma K_r$	0.292
6	$\Delta K\ \Sigma K$	5.86 hrs^2
7	$(L_i)^2$	20.7
	SECOND TRIAL	
8	$\Delta K_r\ \Sigma K_r$	0.283
9	P_w % P_i	43.3%
10	P_w	12,100 cfs
11	$\Delta K\ \Sigma K$	5.87 hrs^2
12	$\Delta K_r\ \Sigma K_r$	0.284
13	P_o % P_i	65.6
14	P_o	18,400 cfs
15	ΣK	6.96 hrs.

EXPLANATION OF COMPUTATIONS
TABLE I

Line

(1)-(2) Given data

(3) As a first trial assume a weighted P_w discharge of 12,000 cfs.

(4) The weighted flow is computed as the percent of the initial peak discharge. (12,000/28,000)100.

(5) The product $\Delta K_r\ \Sigma K_r$ is determined from Plate 3 for a P_w % P_i of 42.9%.

(6) The values of $\Delta K\ \Sigma K$ in hours^2 is determined from Plate 13 for a weighted P_w discharge of 12,000 cfs.

(7) The value of the duration of the initial block hydrograph (L_i from Line 2) is squared and entered on this line.

(8) A check value of $\Delta K_r\ \Sigma K_r$ is computed as the ratio of $\Delta K\ \Sigma K$ (Line 6) and L_i^2 (Line 7). The assumed weighted peak discharge (Line 3) is not correct as the value of $\Delta K_r\ \Sigma K_r$ of Lines 5 & 8 are not equal.

(9) The weighted flow in percent of the initial peak is determined from Plate 3 at a $\Delta K_r\ \Sigma K_r$ of 0.283 (Line 8).

(10) The weighted peak discharge is computed as the product of Line 9 and the peak discharge (28,000 cfs).

(11) The value of $\Delta K\ \Sigma K$ is determined from Plate 13 for a weighted flow of 12,100 cfs.

(12) The check value of $\Delta K_r\ \Sigma K_r$ is computed from Lines 7 & 11 as for Line 8 and checks that value closely.

(13) The downstream peak discharge (P_o) in percent of P_i is determined from Plate 2 for a value of $\Delta K_r\ \Sigma K_r$ of 0.284.

(14) The downstream peak discharge at mile 21.07 is computed as the product of the initial peak discharge (28,000 cfs from Line 1) and the downstream peak discharge in percent of the initial peak discharge (65.6% from Line 13).

(15) The value of ΣK is determined from Plate 13 for a value of 12,100 cfs of the weighted P_w discharge.

TABLE II
DURATION OF ONE CYCLE

Line	Item	Unit
1	Duration of P_o for equivalent vol of hydrograph	6.90 hours
2	P_t % P_o	9.24%
3	Efficiency ratio	1.83
4	Duration of one cycle	12.6 hours

EXPLANATION OF COMPUTATIONS
TABLE II

Line

(1 -14) Compute the duration of one cycle in the same manner as described on Plate 11a.

TABLE III
DOWNSTREAM HYDROGRAPH

Line	Percent of downstream peak P_o %	Time % L_i %	Time from beginning of rise hours	Discharge cfs	Time from beginning of release at dam hours
	Col. 1	Col. 2	Col. 3	Col. 4	Col. 5
1	0	0	0.0	0	2.8
2	25	43	2.0	4,600	4.8
3	50	71	3.2	9,200	6.0
4	75	95	4.3	13,800	7.1
5	90	114	5.2	16,600	8.0
6	100	140	6.4	18,400	9.2
7	90	169	7.7	16,600	10.5
8	75	188	8.6	13,800	11.4
9	50	213	9.7	9,200	12.5
10	25	244	11.1	4,600	13.9
11	5	295	13.4	920	16.2

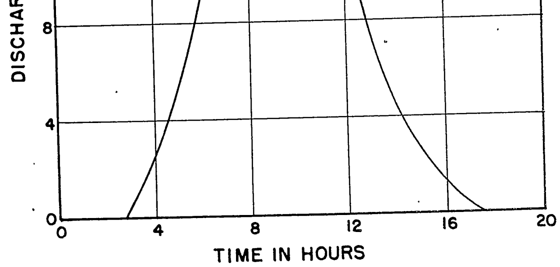
EXPLANATION OF COMPUTATIONS
TABLE III

Line

(1) The downstream hydrograph is computed in the same manner as described on Plate 11b.

PLATE NO. 14 a

MHB 11

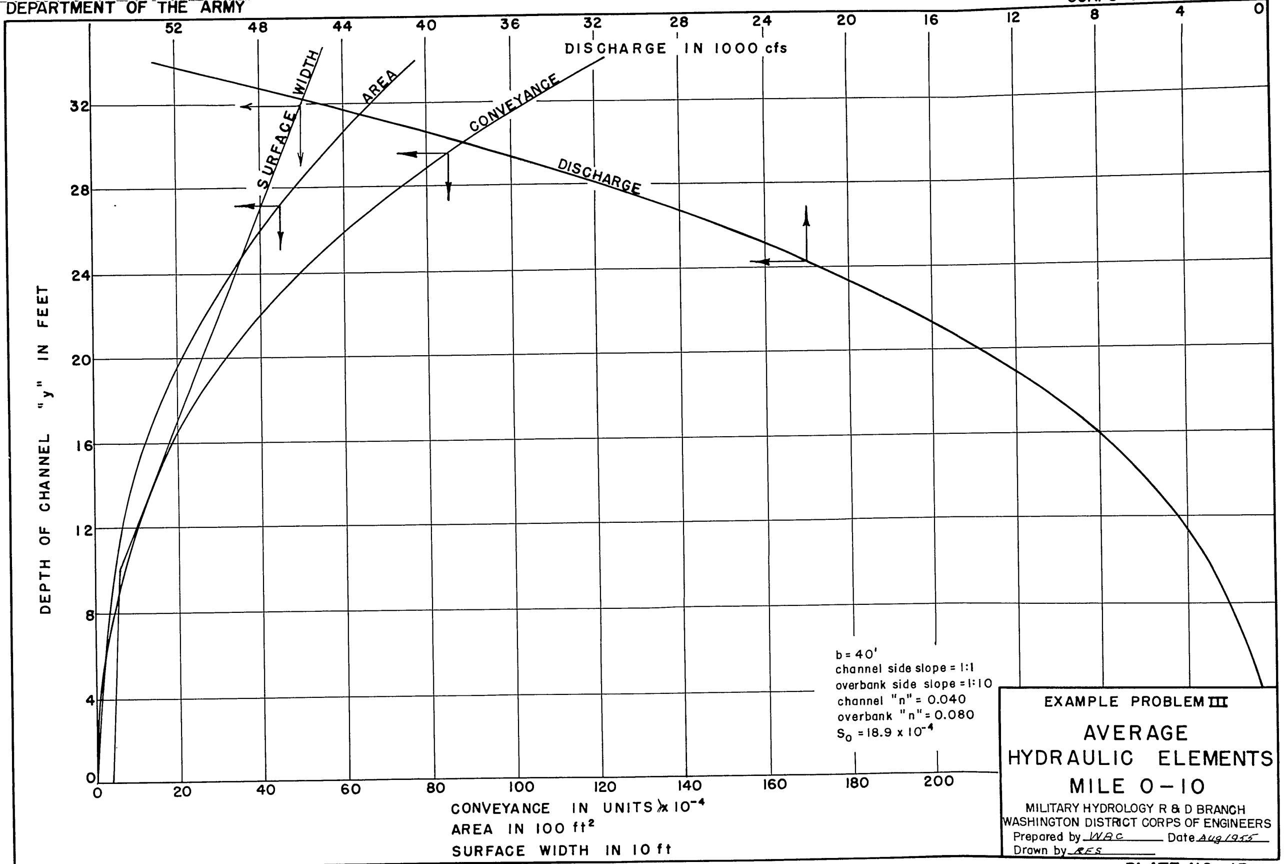


EXAMPLE PROBLEM II

HYDROGRAPH AT MILE 21.07

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBC Date July 1955
Drawn by RES PLATE NO. 14b

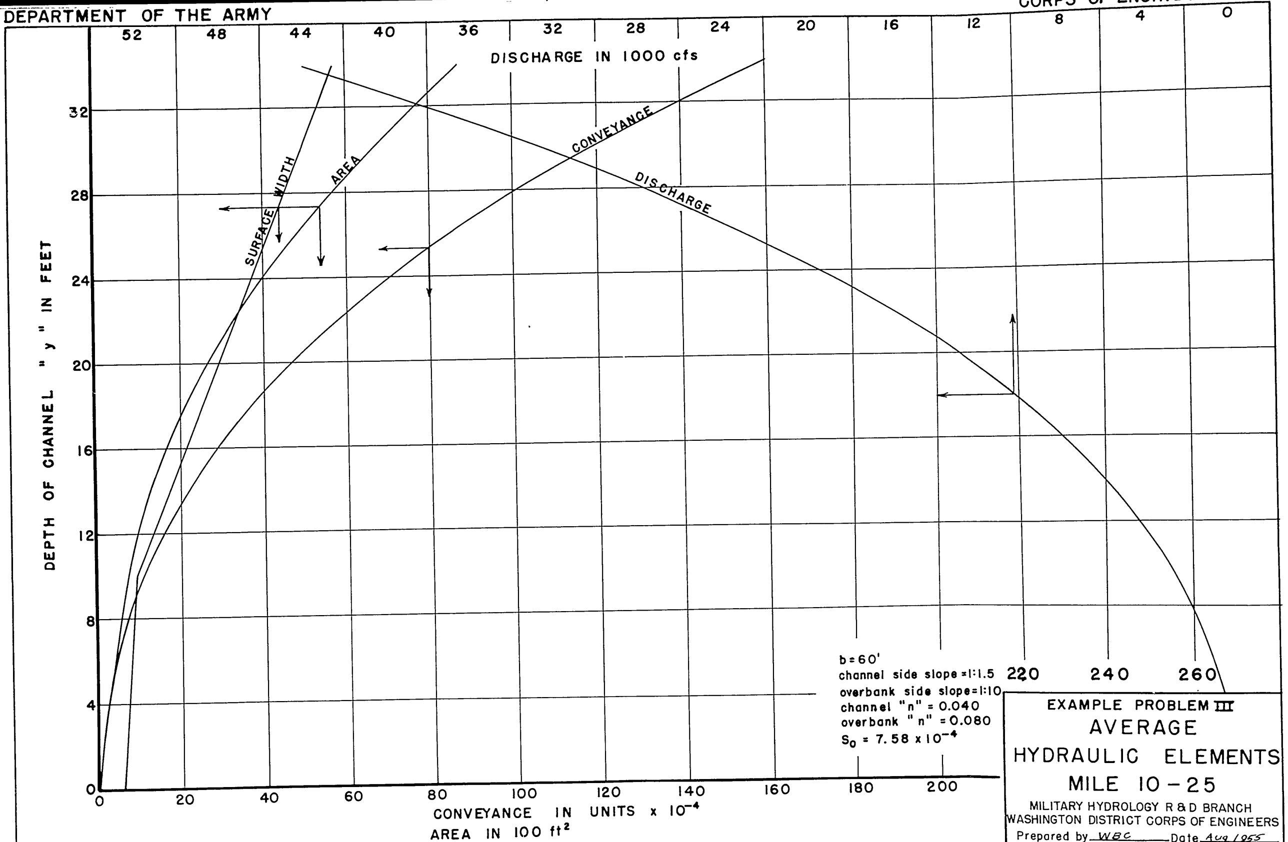
MHB 11



EXAMPLE PROBLEM III

AVERAGE HYDRAULIC ELEMENTS MILE 0–10

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBC Date Aug 1955
Drawn by RES



EXAMPLE PROBLEM III

AVERAGE HYDRAULIC ELEMENTS

MILE 10 – 25

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS

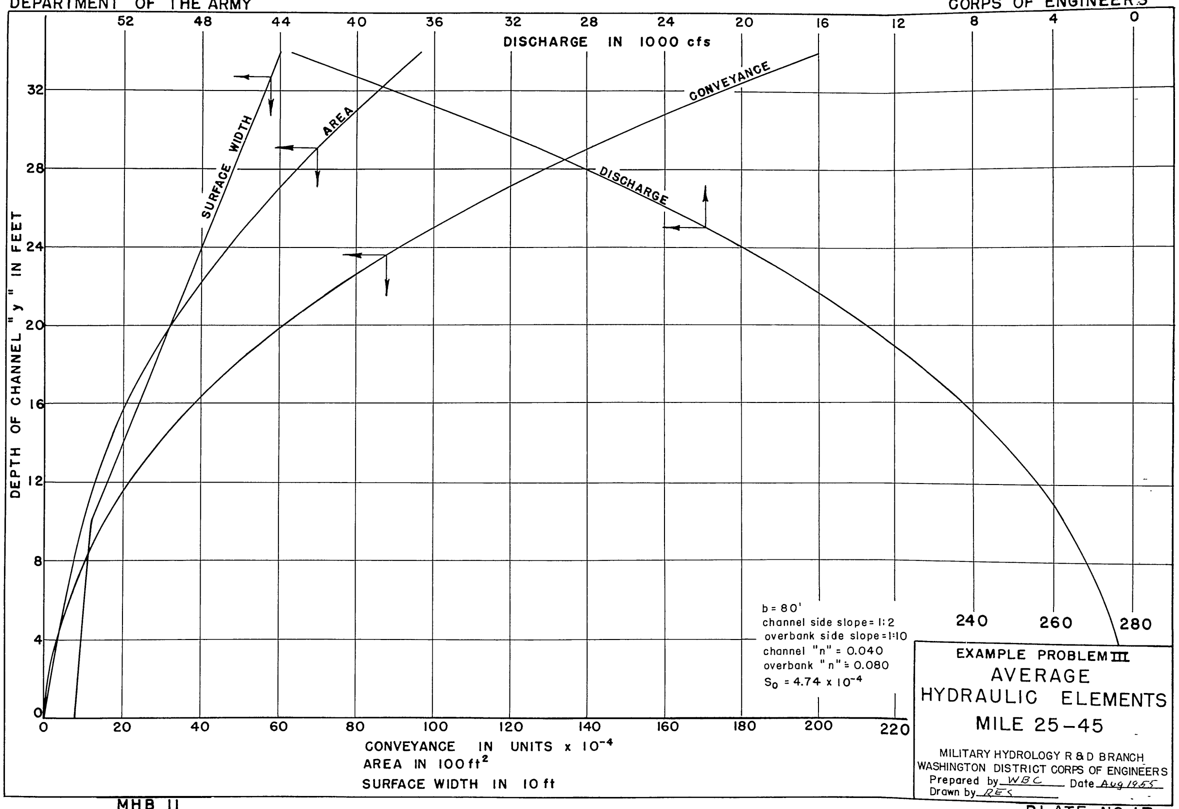
Prepared by WBC Date Aug 1955

Drawn by RES

PLATE NO. 16

DEPARTMENT OF THE ARMY

CORPS OF ENGINEERS



EXAMPLE PROBLEM III

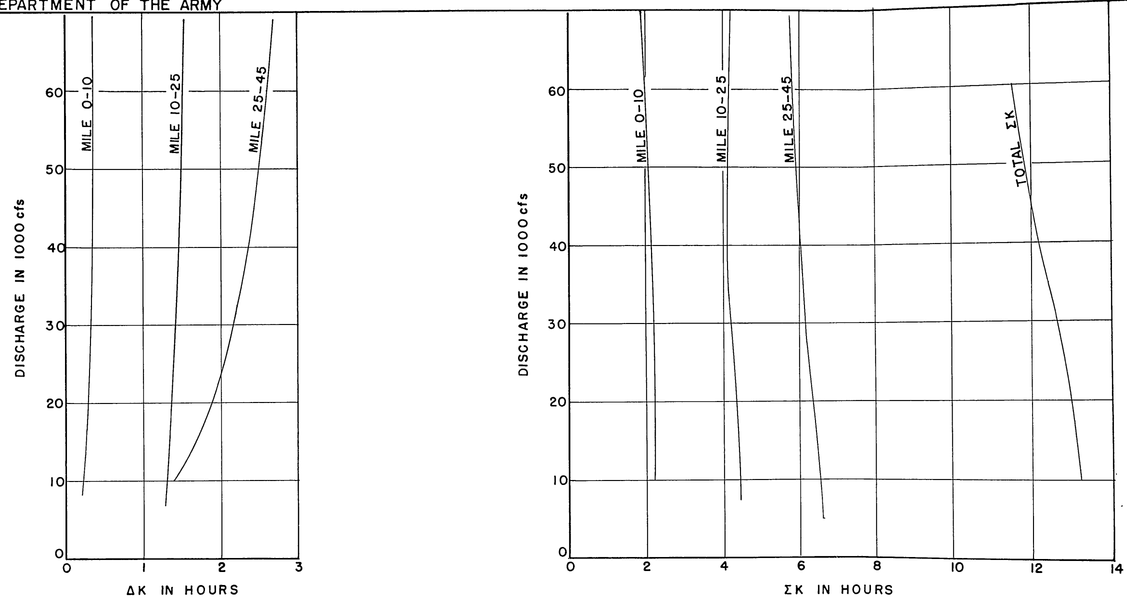
AVERAGE HYDRAULIC ELEMENTS

MILE 25–45

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBC Date Aug 1955
Drawn by RES

MHB 11

PLATE NO. 17

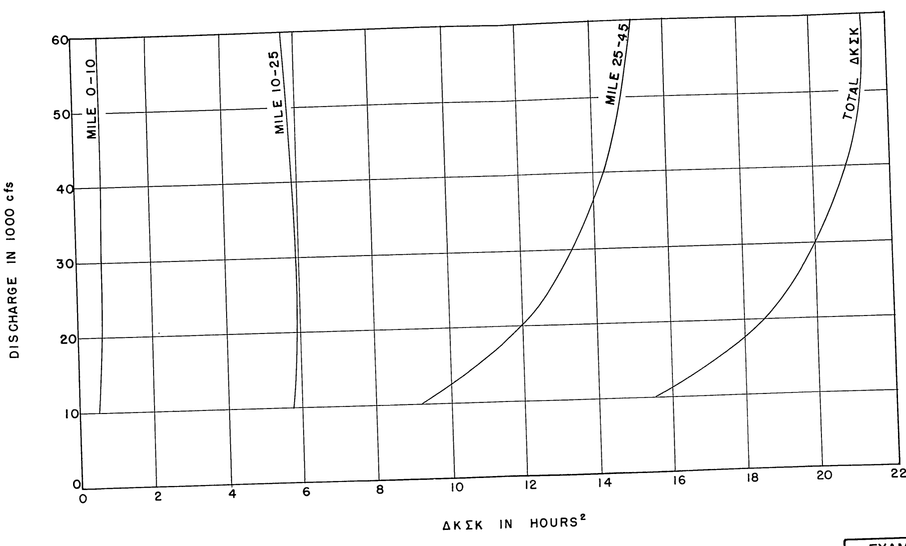


EXAMPLE PROBLEM III

ΔK & ΣK

45 MILE REACH

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBC Date Aug 1955
Drawn by RES



EXAMPLE PROBLEM III

$\Delta K \Sigma K$

45 MILE REACH

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by WBC Date Aug 1955
Drawn by RES

DETERMINATION OF CYCLICAL HYDROGRAPH AT MILE FORTY FIVE

TABLE I
DURATION OF INITIAL BLOCK HYDROGRAPH

Line	Item	Unit
1	Base flow	1,000 cfs
2	P_o total	32,500 cfs
3	P_o net	31,500 cfs
4	P_i	60,000 cfs
5	Net P_o % P_i	52.5%
6	ΔK_r ΣK_r	0.500
7	P_w % P_i	35.5%
8	P_w	21,300 cfs
9	ΣK	12.95 hours
10	ΔK ΣK	18.78 hours^2
11	$(L_i)^2$	37.6 hours^2
12	L_i	6.14 hours

EXPLANATION OF COMPUTATIONS
TABLE I

Line

(1) From the basic data of the problem a base flow of 100 cfs occurs in the reach from Mile 25 - 45.

(2) The total discharge for the cross section at mile 45 to give a surface velocity of 11 feet/second (Computed in the same manner as Problem I, Plate 11a.

(3) The net discharge at mile 45 computed as the total discharge minus the base flow.

(4)-(12) Detailed explanation the same as Lines 2-10, Table I, Plate 11a.

TABLE II
DURATION OF ONE CYCLE

Line	Item	Unit
1	Duration of P_o for equivalent volume hydrograph	11.7 hours
2	P_t % P_o	5.23%
3	Efficiency ratio	1.94
4	Duration	22.7 hours

EXPLANATION OF COMPUTATIONS
TABLE II

Line

(1)-(4) Computed in the same manner as described in Lines 1-4, Table II, Plate 11a.

TABLE III
DOWNSTREAM CYCLICAL HYDROGRAPH

Line	Percent of peak %	Time percent L_i %	Time hours	First Cycle downstream hydrograph: Discharge cfs	Total cfs	Time hours
	Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6
1	0	0.0	0.0	0	1,000	5.32
2	25	54.0	3.31	7,875	8,875	8.63
3	50	89.8	5.50	15,750	16,750	10.82
4	75	120	7.36	23,600	24,600	12.68
5	90	143	8.78	28,450	29,450	14.10
6	100	175	10.7	31,500	32,500	16.02
7	90	213	13.0	28,450	29,450	18.3
8	75	233	14.3	23,600	24,600	19.6
9	50	270	16.6	15,750	16,750	21.9
10	25	310	19.0	7,875	8,875	24.3
11	5	370	22.7	1,575	2,575	28.0

EXPLANATION OF COMPUTATIONS
TABLE III

Col.

(1)-(4),(6) Columns 1, 2, 3, 4 and 6 are computed in the same manner as explained on Plate 11b with the peak discharge of Line 6, Column 4 equal to the net peak of 31,500 cfs.

(5) The total downstream hydrograph is computed as the sum of Column 4 and the base flow (1000 cfs) and entered in Column 5. The values of the total discharge (Column 5) are plotted against the time (Column 6) and are shown on Plate 20b.

PLATE NO. 20a

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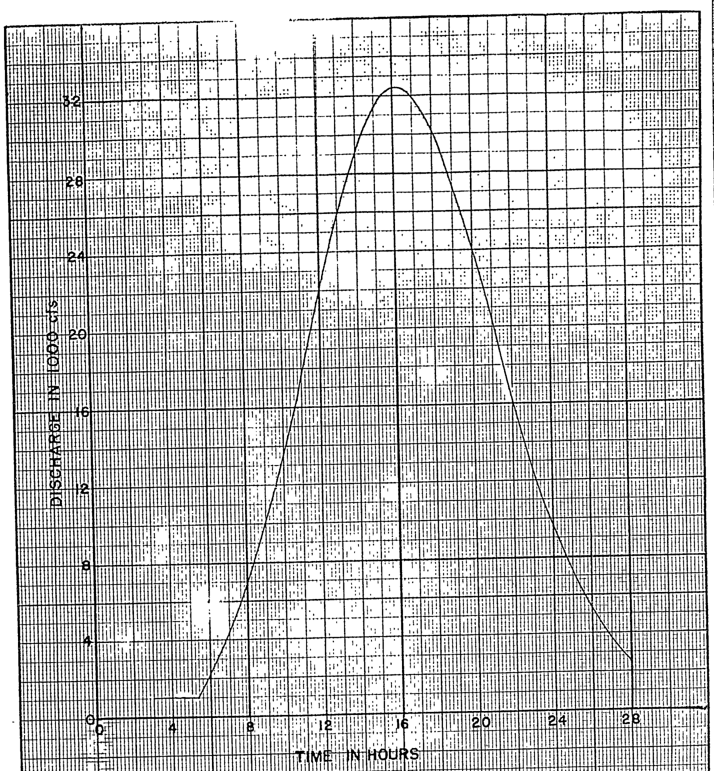

EXAMPLE PROBLEM III

HYDROGRAPH

AT MILE 45

MILITARY HYDROLOGY R&D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by W.B.S. Date 27 August, 1955
Drawn by R.E.S. PLATE NO.20b

MHB 11