Bibliography and Discussion of Flood-Routing Methods and Unsteady Flow in Channels

GEOLOGICAL SURVEY WATER-SUPPLY PAPER 1690

Prepared in cooperation with the Soil Conservation Service of the U.S. Department of Agriculture

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UNITED STATES GOVERNMENT PRINTING OFFICE, WASHINGTON : 1964

UNITED STATES DEPARTMENT OF THE INTERIOR

STEWART L. UDALL, *Secretary*

GEOLOGICAL SURVEY

Thomas B. Nolan, *Director*

The U.S. Geological Survey Library has cataloged this publication as follows:

Yevdjevich, Vujica M 1913–
Bibliography and discussion of flood-routing methods and unsteady flow in channels. Washington, U.S. Govt. Print. Off., 1964.
iii, 235 p. 24 cm. (U.S. Geological Survey. Water-supply paper 1690)
Prepared in cooperation with the Soil Conservation Service of the U.S. Dept. of Agriculture.
1. Flood control—Bibl. 2. Flood control. 3. Floods. I.U.S. Soil Conservation Service. II. Title. (Series)

For sale by the Superintendent of Documents, U.S. Government Printing Office
Washington, D.C., 20402

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BIBLIOGRAPHY AND DISCUSSION OF FLOOD-ROUTING METHODS AND UNSTEADY FLOW IN CHANNELS

By Vujica M. Yevdjevich

INTRODUCTION

Several methods of channel flood routing are in use; for example, about half a dozen methods are used by the Soil Conservation Service of the U.S. Department of Agriculture. Considerable disagreement on flood-routing methods exists between agencies, and much time is being spent in an attempt to improve these methods or to develop new ones. A survey of published literature on the subject is needed to eliminate loss of time and to increase efficiency of studies now in progress. To meet this need, a cooperative project was initiated between the Soil Conservation Service and the Geological Survey, U.S. Department of the Interior; funds were provided by the Soil Conservation Service, and the work was done by the Geological Survey.

The objective of this project was the preparation of a bibliography of literature on channel routing and a commentary on the various methods. No attempt was made to single out one particular method for universal use; however, the compilations and comments indicate the inutility of some methods and establishes the limits of application of those that can be used.

This report consists of a general discussion of unsteady flow in channels and the present status of flood-routing methods, plus a bibliography of domestic and foreign literature. The bibliography is arranged chronologically and contains abstracts of all papers that were obtainable (about 90 percent of them) within the time limits of the project; items are indexed by authors and by subjects.

Helpful background material was provided by a study of unsteady flow sponsored by the Corps of Engineers, U.S. Army, through the National Bureau of Standards and the Geological Survey. Coordination with the Soil Conservation Service was maintained through Harold O. Ogrosky, Chief, Hydrology Branch, Engineering Division. Special acknowledgment is due the staff of the Geological Survey library for their cooperation and extra effort in obtaining not readily available and foreign publications.

UNSTEADY FLOW IN CHANNELS AND RESERVOIRS

In analytical studies, water flow in natural or artificial channels is generally classified as steady or unsteady. Flow of water in river channels, canals, reservoirs, lakes, pools and free surface flow in conduits or tunnels, where the velocity changes with time, is defined as unsteady flow (nonpermanent, nonstationary, or with time variable flow). Water flow in channels is almost always unsteady flow, but when the change of discharge with time is very gradual, unsteady flow approximates steady flow. The discharge hydrographs of flow in natural water courses are largely comprised of flood waves followed by their recession curves. Only those flows occurring during a prolonged drought and those occurring for short time intervals at the highest and lowest points of the hydrograph can be considered to be steady flow. In hydraulic problems it is important to know when an unsteady flow may properly be treated as a steady flow. For practical purposes, the answer is directed by judgment rather than by criteria that have been developed mathematically or experimentally.

The mathematical treatment of unsteady flow in any channel is considered to be among the most difficult problems in fluid mechanics. Basically, this difficulty exists because too many variables enter into the functional relationship and because developed differential equations cannot be integrated in closed forms except in very simple situations. Apart from these difficulties, it may be assumed that the velocity-distribution and friction-resistance factors in unsteady flow are generally approximated by those valid for steady flow, which are, in turn, approximations of the true relationships in steady flow. All formulas developed for unsteady flow, whether based on mathematics or experimentation, must be considered to be only approximations, and uses of the formulas should be cognizant of the basic assumptions and simplifications made in their derivation.

Because the velocity of flow in large reservoirs and lakes is much smaller than the velocity in channels, the approximations of unsteady flow usually differ. The influence of factors neglected in the approximations is much less in analyses pertaining to large bodies of water than for those pertaining to narrow sloping channels. Many cases fall between these two extremes, and it is often difficult to distinguish between a reservoir and a channel. The differentiation is usually based on judgment rather than on an application of objective criteria.

Waves are classified according to type to facilitate mathematical or experimental treatment. The meaning of "wave" must be understood in a broad sense, thus: a surface wave is any change with time in the surface shape of a body of water with the change being propagated along the body of water. A standing wave is the limit of propagation waves.

The water wave in a river channel or reservoir is generally conceived as being any change (disturbance, intumescence) of discharge and stage with time or any unsteady movement in or along the body of water. Unsteady water movement at sea, in the form of sea waves, is treated as surface-wave motion. Unsteady flow along channels is also analyzed as movement of surface waves, and the general theory of surface water waves applies to both phenomena. In physics, water waves are placed in the same category as the waves of other media, and the principles applicable to gas waves, capillarity waves, elastic waves, and others can be applied with moderate success to water waves in channels and reservoirs. However, the diversity of variables, especially those defining the boundary conditions, such as shape and roughness of channels, and the contact of two fluids, air and water, makes the treatment of channel water waves very specific. In practical application, the analogy of water waves with waves of other media is still further decreased in its importance.

Hydraulic research on waves in channels has been carried on by three basic methods: by theoretical (or analytical) studies, by experiments using small scale models, and by observations in nature. Combinations of these methods have given the most useful results.

Studies of unsteady flow in channels were started more than 150 years ago with the work of the French mathematicians Laplace and Lagrange. The statement of the Lagrange celerity formula for small waves in shallow water provided the impetus for these studies. The chronological annotated bibliography, which is the last part of this report, shows the amount of knowledge that has been amassed since the beginning of research on unsteady flow. The bibliography also shows when the different methods of unsteady-flow analysis were popular and how these methods (theoretical, experimental, and observational) have been combined by individual workers.

Knowledge concerning unsteady flow in channels is summarized in numerous text books that present both general and specific treatment of the topic. This body of literature covers topics from the point of view of fluid mechanics or of general or applied hydraulics, and in terms of hydrology. The most important books in many languages are abstracted in the chronological bibliography.

CLASSIFICATION OF WATER WAVES

There is no unique classification of waves, but use of identifying criteria permits classification for study purposes.

The waves are generally classified as being: (1) orbital waves (nontranslatory), waves whose water particles describe closed trajectories during wave movement, such as sea waves; and (2) translatory waves, whose water particles constantly progress during the wave movement. Unsteady flow, as applied to flood routing, is usually that of translatory wave movement, but may also be that of waves of small length moving in channels in which the water is at rest.

By comparing the direction of wave movement with the channel axis, waves may be classified as being either longitudinal or transverse. Although the transverse waves have hydraulic implications in some problems, unsteady free surface flow or flood routing is concerned only by longitudinal waves.

Waves are further classified as being deep-water, when only the surface layers of water are disturbed by the wave moment, or shallow-water waves, when the entire cross section of the body of water is disturbed by wave movement. The flood-routing problems are all of the shallow-wave type, except that the movement of floods along the lakes, reservoirs, and pools is influenced highly by density currents, which are caused either by differences in sediment load or differences in temperature.

Only two basic types of unsteady flow are generally considered important in hydraulics of surface flow in channels: discontinuous unsteady flow (progressing surges, bores, and depressions), and very gradually varying unsteady flow (progressing long waves in reservoirs, or long flood waves in rivers). These two basic types are generally differentiated in mathematical analysis and formulas involved, each being treated with somewhat different assumptions. Because all transitions between these two extremes exist in nature, the usual approach in analysis is to apply one of the two extremes to the case at hand, being mindful that the resulting evaluation of wave, as computed by corresponding formulas, is subject to error.

Waves are further classified according to the main forces that control wave movement and evaluation. In this respect they are classified as waves subject wholly or mainly to gravitational or momentum control (where friction losses are small and generally neglected, as for progressing steep surges, bores, and depressions), and waves subject largely to friction control as well as to the gravitational or momentum control (where friction losses are approximately of the same or greater influence as the gravity force, as is generally true for long waves, or current flood waves).

The waves in channels may be classified as being solitary waves or a train of waves. The solitary wave is considered to be a gradually varied wave followed theoretically on both extremes in channel by steady flow, with one rising limb, one peak, and one recession limb. For the most part, individual flood waves are treated as solitary translatory waves. The close sequence of several waves creates a wave train. The characteristics of succeeding waves may be quite different. This difference occurs when several flood waves follow in close sequence, and the second flood wave is superimposed on the recession of the first, the third is then superimposed on the recession limb of both previous waves, and so forth. Conversely, the waves may be nearly of the same shape, amplitude, and period, such as sea waves or tidal waves that penetrate estuaries and other channels, regular release waves that emit from powerplants and lock operations, and a group of incident and reflection waves. The close proximity of succeeding waves determines whether the individual wave of a wave train shall be treated as the solitary wave, or if the complete wave train should be treated as a whole. Sometimes the waves are considered as simple (solitary), multiple (several, but a restricted number of waves, or superimposed flood waves), and train wave (large number of individual waves, such as roll waves).

Direction of wave movement is a further criterion for wave classification. When the waves move along a channel that has a horizontal bottom and in which the water is at rest, the direction of the wave's movement is not important. If the direction of wave movement is the same as the direction of the slope of a channel, the wave direction is considered to be downstream. Wave movement that is opposite to the slope of channel has an upstream wave direction. Although the water usually moves downstream in river channels, wave movement may also create upstream water movement, such as a flood wave that moves upstream from the junction on a tributary or joining river and the tidal movement along estuaries. The upstream and downstream waves, therefore, correspond to the direction of slope of channel, and not to the direction of water flow. Generally, in flood routing in rivers, the direction of channel slope and of water flow coincide, but there are important exceptions.

When a wave moves along a channel that already contains steady flow (uniform or nonuniform) or a very gradual unsteady flow, the wave surface can be higher or lower, or occasionally both, than the level of the underlying regimen of flow. If the wave surface is higher, the wave is referred to as being positive; if the wave surface is lower, the wave is considered to be negative; and if the wave surface is both higher and lower than the level of flow, it is a combined positive-negative wave.

Waves are further classified as being single-faced (more commonly called "monoclinal") and two-faced (normal). A surge wave (positive-bore or negative-depression wave), which starts with abrupt discontinuity in water level and discharge, passes either to a monoclinal wave (a negative wave always passes to monoclinal), or proceeds as an abrupt wave (bore).

Normal waves also conceived as being composed of two monoclinal parts are classed as being symmetrical and asymmetrical (skew) waves.

The foregoing discussion is summarized in the following table:

Descriptive criteria and classification of translatory, longitudinal, shallow waves

Criterion	*Classification*
Rate of variation	Surges
	Intermediate waves
	Long waves
Controlling force	Gravity
	Gravity and friction
Frequency of occurrence	Simple solitary
	Multiple
	Wave train
Direction of movement	Relative to channel bed slope
	Downstream
	Upstream
	Relative to underlying flow
	Downstream
	Upstream
Wave surface elevation	Relative to level of underlying flow
	Positive (higher)
	Negative (lower)
	Positive-negative
Form	Single-faced
	Two-faced
	Symmetrical
	Asymmetrical

By this classification, it is implied that only translatory, longitudinal and shallow-water waves are applicable to those under study as flood waves in channels and conduits having free surface flow. Orbital, transverse, and deep-water waves are not applicable.

Treatment of the different aspects of wave movement in channels is complex, due to the diversity of wave forms and the multiplicity of underlying factors which influence their evaluation.

For a sound approach to computing the transformations of the flood wave progressing along a body of water it is important that a clear definition first be made of the type of wave involved, its physical characteristics, and all possible forms to which it can change by wave translation along a channel subject to changing characteristics. The second step is the selection of the tools best adapted for treating that particular wave and channel complex. The third step is an evaluation of reliability of the results to be obtained; this evaluation takes into account the underlying assumptions in the selection of tools and in the method of interpretation and application of the tools, apart from their dependence on the accuracy of background data.

MATHEMATICAL TOOLS

LONG WAVES

The basic partial differential equations for the gradually varied unsteady flow developed by De Saint-Venant in 1871, are the continuity (or mass-conservation) equation

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}+q=0 \tag{1}$$

and the momentum (or dynamic) equation

$$\beta\frac{\partial V}{\partial t}+g\frac{\partial H_e}{\partial x}+gS_f-\beta\frac{Vq}{gA}=0 \tag{2}$$

where A=area of cross section, Q=discharge, x=channel length, t=time, q=lateral outflow (lateral inflow negative) per unit channel length; V=mean water velocity (Q/A), g=acceleration of gravity, S_f=friction slope (determined by an appropriate formula, as Manning's or Chezy's), and H_e=distance of the energy line to a reference level, which is

$$H_e=H_z+\alpha\frac{V^2}{2g}=H+z+\alpha\frac{V^2}{2g} \tag{3}$$

where H_z=distance of water level to the reference level, H=water depth in channel, z=distance of channel bottom to the reference level, and α and β are velocity-distribution coefficients of velocity distribution in the cross section. Putting $Q=AV$, and

$$\partial A/\partial t=B\partial H/\partial t,$$

in which B=width at the water surface, and using equation 3, the two equations 1 and 2 become

$$B\frac{\partial H}{\partial t}+BV\frac{\partial H}{\partial x}+A\frac{\partial V}{\partial x}+q=0 \tag{4}$$

and

$$\frac{\partial H}{\partial x}+\frac{\beta}{g}\frac{\partial V}{\partial t}+\alpha\frac{V}{g}\frac{\partial V}{\partial x}=S_0-S_f+\frac{\beta Vq}{gA} \tag{5}$$

in which $S_0=-\partial z/\partial x$.

The last term in equations 2 and 5 is usually neglected, assuming the momentum of lateral flows to be small.

Four terms in equation 4 when multiplied by $(dxdt)$ give dimension of volume. In the order of sequence, they have the following physical meanings: (1) storage caused by the rate of rise of level with time; (2) wedge storage, owing to the difference of depths at the beginning and the end of the elementary reach dx; (3) prism storage; and (4) storage (positive or negative) due to lateral inflow or outflow. Six terms in equation 5, in the order of sequence, have the following physical meanings: (1) rate of change of depth (depth-taper or depth-change term), or the slope created by the change of depth along the channel: (2) acceleration term (ratio of accelerations, or ratio of the change of velocity with time and the acceleration of gravity, also called acceleration-head term, velocity-hydrograph inclination, localized acceleration gradient), or the slope created by the acceleration; (3) rate of change of velocity head (called also dynamic head, velocity-head term, energy grade line inclination, instantaneous energy gradient), or the slope created by the change of velocity head along the channel; (4) bottom-slope or bed-slope term; (5) friction slope, hydraulic-friction term, or hydraulic gradient; (6) part of the gradient on the energy line created by lateral outflow or inflow.

The equation for gradually varied unsteady flow is variously expressed in different papers: dimensionless as in equation 5, or with dimension of head, acceleration, momentum, energy, or other term.

The basic and general assumptions underlying the development and the applicability of equations 4 and 5 are:

1. The vertical acceleration can be neglected in comparison with the horizontal acceleration, because of the gradual change of depth and discharge with time and with distance.
2. The flow is gradually varied, or the vertical velocities are considered small in comparison with the longitudinal velocities.
3. The flow patterns are the same in vertical planes parallel to the longitudinal axis of the channel (in curvilinear channels the vertical cylindrical surfaces parallel to the longitudinal axis have the same flow pattern), or the influence of the channel sides and of its curvature on the flow patterns can be neglected.
4. The velocity distribution along a vertical, in unsteady flow, is the same as in steady flow, or the velocity-distribution coefficients α and β in equation 5 are constants for given values of discharge, depth, and velocity, or the unsteady flow does not influence these coefficients (but because this assumption depends on the rate of change of velocities with time and distance, it is justified only when there is a small rate of change).
5. The friction resistance in unsteady flow is the same as in steady flow. This assumption is justified only if the rate of change of velocities with respect to time and distance is small.

Because no data exist in the literature to show the numerical effect of these factors, either individually or as a group, on the computed or observed waves along the river channel, evidence is lacking for justification of these five assumptions in terms of the specific characteristics of a wave and of channel and lateral inflow or outflow.

The continuity equation involves the cross-section area, and the momentum equation is based on the rate of change of energy line, or of water surface position, plus the dynamic head. For irregular channels, having changing bottom slope and irregular cross-section shape and area, the bridge between these two partial differential equations introduces the first complexity in the mathematical analysis. The rate of change of cross-section characteristics, as related to the bottom position, and the rate of change of bottom slope with distance, when expressed in functional form, generally provide the bridge between the two equations. Some assumptions and simplifications for cross section and for bottom position are necessary to enable analytical treatment of equations 4 and 5.

The practical problems in following wave progress along a channel are those of wave deformation (attenuation and amplification), and the celerity with which either of the wave elements moves along the channel.

When the rates of change of acceleration and velocity along the channel are small in comparison with the rate of change of depth along the channel, or when the second and third terms in equation 5 are negligible in comparison with the first term, then only equation 4 or the continuity equation may be used. The volume of storage produced by a change of wave height then acts as the main factor, causing reduction of the peak of the wave as it progresses along the channel. This equation, generally called the water-storage (differential) equation, starts from the known-balance equation and implies that for a limited reach of channel the inflow minus the outflow for a given time interval is equal to the change of water storage in that reach. The two partial differential equations are thus reduced to a simple differential equation.

The few existant studies on this subject do not give sufficient evidence either of numerical and analytical criteria when a long wave may be treated by the simple water-storage differential equation, or of the errors caused by the neglect of momentum partial differential equation.

The celerity of propagation of long waves depends on the definition of magnitude of the property involved, such as the discharge, the depth, the center of gravity of the wave, the peak, or the same water volume before the propagating depth. Because the waves deform in travelling along the channel, the celerities of the individual discharges and depths differ, although they are related. Because the integration of equations 4 and 5, for V and H the dependent and x and t the independent variables, gives two functions: $V=F_1\ (x, t)$ and $H=F_2$

(x, t), it can be seen that the celerities of any discharge and the depth at which this discharge passes are interrelated.

Studies of the motion of waves in a uniform rectangular channel have revealed that the wave propagates with the celerity

$$C=\sqrt{gH}=\sqrt{g(H_0+h)} \tag{6}$$

in which g=the acceleration of gravity, where the celerity is referred to undisturbed water. This equation is valid when the wave length is great in comparison with the original water depth (H_0), or the wave is long, and when the wave height (h) is small in comparison to the initial depth (H_0).

For both positive and negative waves, occurring in water moving along the channel with mean velocity V, the celerity is

$$C=V\pm\sqrt{g(H_0\pm h)} \tag{7}$$

where the first plus sign is for the wave moving downstream and the first minus sign is for the wave moving upstream, and where the second plus sign is for the positive wave of small height and the corresponding minus sign is for the negative wave of small height.

The literature has supplied many corrections of equations 6 and 7 for the waves which are not small. Some of these corrections involve use of other second- or third-order terms under the radical sign, or introduce a coefficient μ, which takes into account both the friction resistance and the higher order terms under the radical.

For long waves, where the friction force is of greater or same magnitude as the inertia and gravity forces, the general expression for the wave celerity (Graef, Kleitz, Seddon, Forchheimer) is

$$C=\frac{dQ}{dA}=\frac{1}{B}\frac{dQ}{dH} \tag{8}$$

For a monoclinal wave of two depths, H_1 and H_2 (with area A_1 and A_2), and the corresponding discharges Q_1 and Q_2, the celerity is

$$C=\frac{Q_1-Q_2}{A_1-A_2}=\frac{Q_1-Q_2}{B(H_1-H_2)}=\frac{\Delta Q}{B\Delta H} \tag{9}$$

If Chezy's formula for friction resistance is used for large rectangular channels, equation 8 gives $C=3V/2$; for Manning's formula, $C=5V/3$. Equation 8 states that the slope of tangent at the rating curve, with respect to the Q axis, divided by the channel width, is the celerity of wave for the corresponding discharge and height.

SURGES

If the surges are small, positive or negative, equation 7 is generally used for celerity. If the surge height (h) is not small, then the expression for surge celerity is

$$C=\sqrt{gh\left(\frac{A}{A_1}-1\right)+gf} \tag{10}$$

where h=surge height, A=cross-section area before, and A_1=cross-section area after the surge passes, and where f=distance from the center of gravity of area $\Delta A=A-A_1$, or area $\Delta A=A_1-A$, to the lower level during the passage of the surge.

The expression for the mean velocity of accelerated water is

$$V=V_0\pm U=V_0\pm \frac{A_1}{A-A_1}\sqrt{gh\left(\frac{A}{A_1}-1\right)+gf} \tag{11}$$

where U is the velocity created by acceleration, plus or minus depending on which direction the surge of height (h) is moving in comparison with the direction of velocity (V_0) of undisturbed fluid.

Many theoretical and experimental formulas relate the surge height to discharge change and channel characteristics and express celerity and velocity of water in surge movement.

INTERMEDIATE WAVES

Between the very long wave and a vertical surge are many intermediate conditions. One extreme is the infinitely long wave, with $h/L=0$, or close to zero, where the flow is almost completely governed by the channel friction. Equations 4 and 5 give an adequate mathematical description of the physics of this wave. The other extreme is the vertical surge, with h/L large, as in tidal bores, sudden water release waves, dam-breach waves, and so forth, where the flow is entirely influenced by gravity (momentum or dynamic effect). Equations 10 and 11 give the celerity and the velocity of this wave movement.

All channel waves occur in the range between these two extremes. Therefore $\lambda=h/L$ and $\sigma=h/H_0$ can be the basic characteristics of transient-wave shapes. Very few theoretical studies treat this intermediate condition of wave flow, and there are no general equations having λ and σ as parameters and equations 4 and 5, and 10 and 11 as the limit conditions.

Whenever a wave departs from the extreme conditions and when the equations for these intermediate conditions are used, an approximation is always involved; a discrepancy exists between the analytical treatment of wave and the true physics of the wave. This situation, among others, explains why there are so many approximate hydraulic procedures for computing the behavior of translation wave in channels.

FLOOD ROUTING

DEFINITIONS AND DISTINCTIONS

Flood routing is a term given to the group of methods and procedures, based either on the theory of unsteady flow (long wave or surges) or on the simple water storage equation by which a flood hydrograph at a point of a channel is determined from the known hydrograph at some point upstream or downstream by using the known channel characteristics and the characteristics of lateral inflow or outflow between the two points.

Three groups of parameters or characteristics define a flood-wave movement:

1. The wave characteristics, which are determined by initial, intermediate, or final conditions, by means of stage hydrographs, discharge hydrographs, and wave profiles (the profiles are generally longitudinal surface profiles for given times; however, under specific circumstances, wave profiles may be defined by mathematical functions with appropriate parameters);
2. The characteristics of channel, such as its cross section, bottom slopes, channel storage, roughness coefficients, junctions, or singularities;
3. The characteristics of lateral inflow or outflow, either surface or underground, or both.

For short reaches of channel, flood routing may be used also for determining the channel characteristics for known flood-wave characteristics and known lateral inflow or outflow, or it may be used for obtaining lateral inflow or outflow for known wave and channel characteristics.

As it is commonly employed, flood routing is a way of determining the shape and timing of a flood wave as it progresses along a channel.

It is necessary to distinguish between flood routing and the theory of wave movement in channels, generally called the unsteady flow. The two subjects are based on the same physical principles and relationships; they differ only in the way they are used in practical determination of wave movement, or in the degree of approximation used in each to solve the wave-movement problems. In some countries, there is no difference in terminology or any distinction made between physics of channel waves, which is part of hydrodynamics and hydraulics, and flood waves in natural channels, which is part of hydrology. The distinction that exists between the two may be compared to the distinction made with reference to the underground flow of water from the point of view of fluid dynamics or hydrology. Flood routing may be conceived, therefore, as being the hydraulics of water waves in channels, treated in terms of the complex conditions that generally exist in hydrology.

Differential equations for wave movement along channels are solved analytically by the introduction of many assumptions (prismatic channel, or constant channel cross section with constant bottom slope, with confined water flow, with uniform roughness across the contour and along the channel) which can be applied strictly only to man-made conduits such as canals, tunnels, pipes, and galleries. The celerity and deformation of the wave as it progresses along a conduit are usually referred to as "unsteady free-surface flow."

The closer the approach made by theoretical treatment is to actual channel conditions, the more complex becomes the relationship between variables, and the greater is the necessity for more data describing the channel, in order to arrive at a good approximation of the true hydrograph at a point downstream or upstream from the point where a hydrograph is initially known. Inasmuch as the obtainable data concerning channel conditions are always limited by economics and time, the complex procedure in studies of flood waves based on the complete theoretical coverage of the problem becomes less justified with a decrease of basic data coverage.

On one hand, approximations of theoretical tools are required to reproduce actual wave movement in natural channels and reservoirs, in those instances where all characteristics (such as bottom slope, cross-section shape and area, hydraulic radius, roughness coefficient, channel storage, width for given depth, as well as lateral inflow or outflow) vary significantly from place to place along the river. On the other hand, practical methods of computing flood-wave movement along the channel must be derived. Together, these approximations of tools and their application comprise the concept of flood routing.

The theory of unsteady water flow in channels has lost its unity during the century and a half of its development, but surely it has gained in scope by broader application.

Inasmuch as wave theory imposes a logical framework of relationship, which requires more or less simplification of assumptions, the various theoretical results or results of applied theory do not agree with observations, and relationships between the two groups of results are not always clearly indicated. Therefore, some justification exists for maintaining for awhile a distinction between the concepts of unsteady flow in conduits and flood routing in channels and reservoirs. However, the two fields will tend more and more to become one, as further research, experience, and the generalizations derived therefrom widen the area of common application.

The interrelationship of unsteady water flow and flood routing should be considered with the aforementioned definitions and distinctions kept in mind.

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PROBLEMS SOLVED BY FLOOD ROUTING

Flood-routing procedures have been developed chiefly to provide a broad scope for the study of floods traveling through reservoirs, lakes, and river channels. The steady increase in number of pools and reservoirs being built and operated for water-resources development has heightened the need for knowledge concerning the unsteady movement of water in reservoirs, rivers, and artificial channels.

Waves that occur in rivers and reservoirs may be created by natural or artificial forces, and the wave shapes that may result are so varied that it is possible to find, in practice, all types of waves that are described in the literature. Therefore, acquaintance with the results of many studies of many different types of waves is necessary for practical application to problems of unsteady flow in channels.

Flood routing was first used for determination of the time and magnitude of flood peaks occurring at points along a river as a flood traveled downstream. This information was needed for forecasting purposes and for the design of structures that would confine the flood at its crest. Further developments in water use have revealed the need for information concerning the shape of the flood wave (stage profile, discharge or stage hydrograph) as the flood moves downstream. Knowledge of peak timing and of the changes of peak magnitude is no longer sufficient to accomplish all the purposes of flood routing.

Currently, flood routing is employed for solution of a wide variety of problems associated with water use. Some of these include: (1) evaluating of past floods, for which records are incomplete; (2) determining hydrographs of channel flow from hypothetical design floods on tributaries and upstream reaches of the main channels; (3) forecasting floods along the main stem of a river, by use of observed or predicted hydrographs at key inlet points in the channel net; (4) determining hydrographs modified by reservoir storage; and (5) studying the effects on the downstream flow conditions of any operation of storage reservoirs or pools.

GENERAL APPROACH

The type of problem involved, the economy of data survey that is imposed, and the accuracy of results required are the factors determining the amount of departure from the basic theory that will be necessary for derivation of a feasible flood-routing procedure. The selection of the best procedure, among those available, is often difficult, but it may be aided by consideration of the following questions, with regard to the particular situation at hand:

1. In view of the known factors influencing wave movement of a given flood wave complex, what parts of the basic theory, or what differ-

ential equation or equations, would provide the most suitable basis for approach?

2. What method of integration is feasible for solution of the differential equation or equations?
3. What degree of accuracy of flood wave prediction can be expected by use of the selected equations and integration method?

Having answered these questions, the next steps are operational: data collection and arrangement for routing, flood routing, and analysis of results obtained.

METHODS OF FLOOD ROUTING

CRITERIA FOR CLASSIFICATION

Among many possible criteria for classifying flood-routing methods, the following may be convenient as guides for selection of method to to be used in a systematic study: (1) physical principles and equations used as the theoretical basis for flood routing; (2) the procedures or methods used for integration of basic differential equations or equation; (3) some specific assumptions and approximations used in treating the flood-wave movement; and (4) problems to be solved. The first two criteria for classification of flood-routing methods seem the most important and will be used here.

CHARACTERISTICS OF METHODS

Individual flood-routing methods will not be specifically described here. Many books and papers, referred to in the bibliography and particularly in the index by subjects, give detailed analysis of one or several flood-routing methods. This discussion will consider such generalities of method as: characteristics of principal technique, data necessary, type of work involved, speed, flexibility, expenses necessary to carry out the routing, and reliability. The purpose, here, is to give a general description of different groups of flood-routing methods.

ROUTING OF FLOOD WAVES BY TWO PARTIAL DIFFERENTIAL EQUATIONS

Those flood-routing methods that are based on the two partial differential equations 4 and 5, usually expressed in different forms according to the two dependent variables selected and the simplications introduced, generally give the closest approximations of the actual flood movement through channels, if the basic conditions for applying the two equations are approximately satisfied. The most important condition is that of gradual variability of the flood; this condition is fulfilled in the majority of natural floods. Many integration procedures are used, and this group of flood-routing methods is best classified according to the integration procedures or methods used.

ANALYTICAL INTEGRATION

The partial differential equations 4 and 5, with friction slope S_f proportional to square of velocity or to square of discharge, are nonlinear differential equations of hyperbolic type with changing coefficients. Due to inherent mathematical difficulties, there is no way to carry out the integration in closed form, unless many simplifications are introduced.

The classical approach, made first by De Saint-Venant, neglects the friction resistance and assumes the channel to be horizontal with rectangular cross sections. These assumptions deviate so much from reality for flood-wave movement in channels that the wave characteristics resulting from analytical integration are generally not comparable with the true wave characteristics.

This classical approach by means of analytical integration is an extreme; it may be considered to be a rough approximation, and, in accuracy, can be compared with some of the very simple integration procedures of routing that are based on the water storage differential equation.

Use of the method of anlytical integration makes it necessary to approximate both the initial wave conditions (hydrograph, wave profile) and the boundary conditions (channel cross sections, stage-discharge relationships, slope, and lateral inflow or outflow) by analytical expressions, which are to be used in equations 4 and 5.

The hydrographs and wave profiles of long waves progressing in a channel may be approximated by considering them to be either symmetrical or asymmetrical waves, with functions of bell-shaped curves (binomial, Poisson, Pearson-functions, and others). The channel conditions are represented by the cross-section area or width as functions of depth and distance along channel, with roughness coefficient usually being used as a constant, and bottom slope used either as constant or as a function of distance along channel. The lateral inflow and outflow are taken as constant or approximated as simple functions of channel and lateral flow characteristics and of time.

The great diversity in shape and roughness of natural channels and the complexity of pattern of the lateral inflow and outflows tend to complicate the analytical expressions that approximate these conditions to the extent that the analytical integration of two partial differential equations becomes impossible.

In summary, the two partial differential equations for unsteady flow can be integrated analytically, with expressions for wave evolution, by rather restrictive and very simplifying conditions, which generally are not acceptable for solution of current practical problems.

There are different mathematical approaches for the analytical integration of simplified partial differential equations. One approach introduces a new variable, ϕ, which is used in equations $Q=AV=\partial\phi/\partial t$, and $A=\partial\phi/\partial x$, where ϕ is the volume of water traveling with the current between two cross-sectional planes.

Analytical integration has practical application in those cases when wave height (h) of the long wave is relatively very small. Neglect of high-order terms, in expressions developed in power series form, can then be justified.

INTEGRATION BY FINITE-DIFFERENCES METHOD (OR METHOD OF INSTANTANEOUS GRAPHS)

The two partial differential equations 4 and 5 are usually approximated by the two finite-differences equations, replacing the increments dx, dt, dQ, dV, dA, dB, dH, by the finite differences Δx, Δt, ΔQ, ΔV, ΔA, ΔB, ΔH. At the same time the partial derivatives are replaced by ratios of finite differences:

$$\partial H/\partial x \text{ by } \Delta H/\Delta x,\ \partial V/\partial t \text{ by } \Delta V/\Delta t,$$

for example, equations 4 and 5 become:

$$B\frac{\Delta H}{\Delta t}+BV\frac{\Delta H}{\Delta x}+A\frac{\Delta V}{\Delta x}\,q=0 \tag{12}$$

and

$$\frac{\Delta H}{\Delta x}+\frac{\beta}{g}\frac{\Delta V}{\Delta t}+\frac{\alpha V}{g}\frac{\Delta V}{\Delta x}=S_0-S_f+\frac{\beta Vq}{gA} \tag{13}$$

Where Δx and Δt are selected in some way, ΔV and ΔH are changes for these finite differences, and B, V, A, q, S_0, and S_f are the mean values for both Δt and Δx. For given values, Δx and Δt, the mean values of all variables, including assumed V and H at the end of Δx and Δt, are computed, and ΔV and ΔH are then determined from equations 12 and 13. If the assumed V and H at the end of Δx and Δt are good, the values $V+\Delta V$, and $H+\Delta H$ should give the assumed values. The channel is divided into reaches Δx_1, Δx_2, Δx_3, and so forth (equal or unequal), and the corresponding time-intervals Δt_1, Δt_2, Δt_3, and so forth, are determined. The trend is now to use the relationship

$$\Delta x \leq C\,\Delta t \tag{14}$$

where C is an approximation of celerity along the reach, for the time interval. As C is a function of wave and channel characteristics, it changes continually during flood routing, and the selection of minimum C is a practical problem that must be solved before applying the finite-differences method of integration.

The basic characteristics of this approximation are: (1) All variables during time Δt, and along channel reach Δx, change linearly (or stay constant); (2) the accuracy of the method depends on the selection and relationship of finite differences Δt and Δx; (3) the smaller are the values of Δx and Δt, the greater is the computational work, and also the greater is the accuracy; (4) the values Δx and Δt depend also on the amount and accuracy of background data, and on initial and boundary conditions; for a given status in terms of available data there are the minimum values of Δx and Δt which can be economically justified by the amount of computational work required; (5) the values of variables computed for the end of Δt_1 and for the downstream end of Δx_1, become the initial values for Δt_2 and the values for the upstream end of Δx_2; (6) an iterative procedure is commonly used in determining the dependent variables, V and H, or Q and H, or Q and A, or others, at the end of time Δt_1 and channel section Δx_1.

Use of this time-consuming method of trial-and-error procedures, which are normally performed, is justified only when results with the high degree of accuracy are required.

However, with development of the digital computer, which, in time, will provide fast and inexpensive computations, the present drawbacks in economy of performing the operations of the finite-differences method of integration will be largely eliminated. The method can be expected, then, to be highly favored, inasmuch as it is probably the most accurate of all practical methods of flood routing in channels. The fast advent of new numerical methods helps this trend.

INTEGRATION BY METHOD OF CHARACTERISTICS

The results of integration are given for two dependent variables as functions $V=F_1(x,t)$, or $H=F_2(x,t)$. These two functions represent surfaces in the space (V, x, t) or (H, x, t). If there is any discontinuity in the four partial derivatives of equations 4 and 5, $\partial V/\partial t$, $\partial V/\partial x$, $\partial H/\partial t$, and $\partial H/\partial x$, these discontinuities propagate along the channel, and the projection of the position of discontinuities at surfaces F_1 and F_2 in (x, t) plane produces curves that are called "characteristics," or "characteristic lines."

These lines are usually curves, but in application may be replaced by straight lines.

The characteristic lines are usually very simplified and given in the form

$$dx=(V\pm\sqrt{gH})dt=Cdt \tag{15}$$

and

$$d(V\pm2\sqrt{gH})=g(S_0-S_f)dt \tag{16}$$

which equations are applicable for rectangular prismatic channels. It can be proved that equations 15 and 16 are equivalent to equations 4 and 5, and therefore can replace them.

For other channel cross-sectional shapes, that take into consideration velocity distribution coefficients α and β, equations 15 and 16 become more complex and show characteristics that generally are curved lines.

Equations 15 and 16 are usually integrated by replacing dx and dt with Δx and Δt, and $d(V+2\sqrt{gH})$ with $\Delta(V+2\sqrt{gH})$. Many graphical procedures have been developed for this purpose that use approximations in the form of straight-line characteristics, in the finite-differences form.

Certain features of the method of integration by characteristcs should be pointed out for consideration of the method applicability for particular cases in flood routing: (1) its use of finite differences Δx and Δt; (2) its consideration of the long wave as being composed of many elementary waves in the form of small surges, so that for the time Δt and the reach Δx, the velocity change ΔV and height change ΔH are considered as discontinuities traveling with celerity $V \pm \sqrt{gH}$ (providing only a rough approximation in the case of long flood waves, where the friction forces are not negligible); (3) its use of straight-line characteristics as approximations instead of curve-line characteristics; (4) its use of graphical integration of equations; (5) the length and tediousness of the integration; (6) the use of both planes (x, t) and (V, H); and (7) the complexity of its procedure when friction resistance, channel slope, sudden changes of cross section, bifurcations, junctions, and similar changes, are to be taken into consideration.

Few actual flood-routing cases in natural channels can justify use of the method of integration by characteristics in view of the amount of work required and also in view of the limitations imposed by properties of the method itself.

With the advent of digital computers and new numerical methods, the labor of integration of finite differences equations 15 and 16 and of partial differential equations 4 and 5 was eliminated. The comparative advantage of these two methods will bear scrutiny. From the standpoint of accuracy and labor required for the graphical or semigraphical part of the two methods there is a better choice. But considering work relief afforded by the digital computer, the finite-differences method as applied to the two partial-differential equations is favored sometimes over the finite-differences method as applied to the four-characteristic equations.

CONCLUSIONS

All three methods—analytical, numerical of finite differences applied to partial differential equations, and numerical of finite differences applied to characteristics (ordinary) differential equations—when applicable give sufficiently accurate results, if enough data are available and the methods are pushed to their limits of accuracy.

These methods can be successfully applied to the analysis of particular waves which have been observed. The prediction of wave shapes and movement, however, requires a considerable amount of trial-and-error-work, especially when the net of channels is complex (tides, flood through river branches).

The mathematical difficulties of analytical integration of the two partial differential equations, the need for a large amount of data, and the accompanying drawbacks of time-consuming procedures and cost in applying the other approximate methods of integrating, have occasioned development of other simpler, but generally less accurate, flood-routing methods.

FLOOD ROUTING BY STORAGE DIFFERENTIAL EQUATION

The methods generally called "storage-routing methods" are those that are based only on the water-storage differential equation, or on the continuity or mass-conservation differential equation, equation 4, based on the principle that for any reservoir or channel, defined as total reservoir, or reaches of channels or reservoirs, the inflow minus the outflow is equal to the stored or depleted water in a given time interval.

When the space is defined for which the inflow, outflow, and change in water storage are considered (total reservoir, a reach of channel), equation 4 can be expressed as

$$Pdt-Qdt=dW=AdH$$

where P=inflow discharge, Q=outflow discharge, and W=stored volume of water, with $dW=AdH$, where A=area of reservoir or channel water surface, and H=depth or elevation of that area. Inflow is given as $P=f(t)$, the storage is generally $W=f(H)$, or $W=f(H_1$, H_2 and so forth) for some depths, and if $Q=f(H)$, then, by elimination, the function $W=f(Q)$ can be determined. In that case, equation 17 has only two variables Q and t, in form of differential equation, whose integration gives $Q=f(t)$.

Equation 17 serves generally for the computation of relations between five functions: (1) inflow hydrograph, $P=f_1(t)$; (2) outflow hydrograph, $Q=f_2(t)$; (3) stage hydrograph, $H=f_3(t)$; (4) outflow rating curve, $Q=f_4(H)$; and (5) storage function, $W=f_5(H)$, or area function $A=f_6(H)$, with five variables: Q, P, H, W, and t.

When three of five functions with boundary conditions are given (and three variables can be excluded), equation 17 enables the computation of the relation between two remaining variables.

In the application of equation 17, the following curves are usually known quantities: (1) storage or area function (obtained by land survey); (2) outflow rating curve (obtained by gaging, hydraulic computation, model study, or other means); and (3) inflow or outflow hydrograph. Two other functions must be computed: (1) outflow or inflow hydrograph, and (2) stage hydrograph.

The basic conditions of applying equation 17 for flood routing are: (1) the storage space (reservoir, channel reach) responds in less time to any unsteady inflow or outflow, than the time unit generally used for integrating equation 17 by finite differences; (2) the wave is long, so that the change of discharge is very gradual; (3) the accuracy of basic data and the required accuracy of results do not justify any method that takes into consideration dynamic effects in unsteady flow; (4) the velocity and velocity changes along the reservoir or channel reach are relatively small, so that the dynamic effect is negligible in comparison with the storage effect during wave movements.

This approach has been the basis for nearly all flood-routing studies made in reservoirs, and for the majority of reservoirs built the storage capacity has been determined by this storage differential equation. The equation has then been applied to flood-routing studies in channels, with some adaptations for the more complex relationship of discharge storage or stage storage in a channel reach.

Many integration methods have been developed. They will be only briefly discussed here.

ANALYTICAL INTEGRATION

By fitting mathematical expressions for $P=f(t)$, $Q=f(H)$ and $W=f(H)$, or $W=f(Q)$, and using these relationships in equation 17, a differential equation with variables Q and t is obtained. The type of the functions fitted to inflow hydrograph, outflow rating curve, and storage curve determines the possibility of carrying out integration in closed form.

This integration method was used successfully with schematic inflow hydrographs, and simple linear relationships of storage and outflow discharge. The formulas are obtained on this basis for computing decrease of flash-flood peaks in a reservoir with free spillway and relatively small water-surface fluctuations.

The difficulties in fitting natural inflow hydrographs by tractable mathematical expressions, and the difficulties of analytical integration when these expressions become complex, limit this integration method to specific problems.

NUMERICAL INTEGRATION BY FINITE DIFFERENCES

Equation (17), as written in finite-differences form, is

$$(P-Q)\ \Delta t=A\ \Delta H=\Delta W \tag{18}$$

where P, Q and A are mean values during the time interval Δt, and corresponding level difference ΔH. Taking P_1 and P_2, Q_1 and Q_2, and A_1 and A_2, the values at the beginning and the end of Δt, with a linear change during a sufficiently small Δt, then

$$\left[\frac{P_1+P_2}{2}-\frac{Q_1+Q_2}{2}\right]\Delta t=\frac{A_1+A_2}{2}\Delta H=\Delta W \tag{19}$$

For known P_1, P_2, Q_1, A_1 and Δt, and known relationship of W and Q, or A and H, and Q and H, it is possible by trial-and-error method to determine the value Q_2. Expressing equation 19 in different ways, especially by using storage factors, $(W+Q\Delta t/2)$ and $(W-Q\Delta t/2)$, as functions of Q, a trial-and-error method may be replaced by a direct numerical integration procedure.

Personal differences in arranging equation 18 or equation 19 for the step-by-step computations may account for differences of the tabular numerical procedures. Procedures vary according to which difference factor (Δt, ΔH, ΔW, ΔQ) is given at the beginning, and which must be determined.

Accuracy of method depends on the accuracy of selection of the basic difference factor (generally Δt), apart from the general accuracy involved in the use of only one differential equation.

The main problems to be solved in using equations 18 and 19 for flood routing along a river channel are: 1) relationship of discharge and storage in a reach (or stage and storage), which may involve other variables (slope, level difference of the reach, celerity of wave, and so forth); 2) selection of reaches Δx, of their limits and length; and 3) selection of time interval Δt for integration. Both Δx and Δt may be constant, or may change to facilitate handling of changing factors (channel or wave shape).

This numerical integration is the most common method used in flood routing through reservoirs.

GRAPHICAL INTEGRATION BY FINITE DIFFERENCES

As the functions represented in equations 17, 18, and 19, and also the finite differences, may be arranged in many combinations to enable graphical integration of a simple differential equation, there are numerous graphical procedures, with different shortcuts, for computing an outflow hydrograph or solving other problems. However, evaluation of the advantages and disadvantages of each procedure and its shortcuts would entail lengthy comparisons made by application of each method to a group of selected routing examples. A simple narrative description of the various procedures and short cuts would not properly illuminate them for comparative purposes.

There are two general approaches in graphical integration: (1) a mass-curve procedure, that represents the given hydrograph in form of its summation or mass curve and obtains the routed hydrograph in the form of its mass curve; and (2) a procedure that uses the inflow or outflow directly for integration. Flash floods are not suitable for routing by the mass-curve procedure, because the error in the determined outflow hydrograph increases.

There are many semigraphical methods, which combine a numerical tabular procedure with partial graphical integration. Generally, Δt is selected as a constant, although ΔH, ΔW, or ΔQ may be also selected as constants. The accuracy of results, neglecting errors due to use of only one differential equation, is dependent upon the length of basic difference (usually Δt), the graphical scale and graphical procedure used. If the graphical procedure is well applied to a body of water having low velocities and a relatively large storage capacity, the accuracy of results corresponds to the accuracy of basic data for flood wave, for body of water, and for lateral inflows and outflows.

This method is also commonly used for flood routing. It is based on the mean or total inflow and outflow during time Δt, and during changes of ΔH or ΔW. The accuracy of results depends on Δt, with better results to be expected for smaller values of Δt. As the amount of work increases with a decrease of Δt (an increase of computational steps, and thus of total time for flood routing), Δt must be a reasonable value, when there are many flood routing applications to be made. The graphical methods are generally restricted to simple problems, because for complex problems and many routing computations, the time involved becomes economically prohibitive. In flood forecasting, usefulness of this method may be impaired by the great time lag between the data supply and the occurred results.

COEFFICIENT METHODS

The coefficient methods comprise a group of procedures that approximate, by simple relationships, the complex relationships existing between the volume of water stored in a channel reach and the hydraulic magnitudes (inflow and outflow discharge, stages, slope, or others). The simple relationships, in the form of coefficients, give weight in a specific way to each variable involved, the simplest being the weight coefficients for inflow and outflow discharge and for time of travel of a wave moving through a reach.

Although the coefficients are mostly empirical, being determined from flood wave movements that have actually occurred, they are useful in the sense that they have broadened the application of the storage differential equation to include channel flows having rather large mean velocities.

The empirical coefficients take care of the relatively great effect of changing inflow and outflow on the water stored in a reach. There are still many possibilities for theoretical development of coefficients on the basis of wave and channel parameters.

This group of procedures may be considered an attempt to bridge in a simple way two methods, by using two partial differential equations, and by using the storage differential equation as the basic tool, but by taking care of dynamic conditions in a reach by use of empirical coefficients.

Improvements of these methods may be expected in the future.

The principal disadvantage of the coefficient methods is their dependence upon empirical coefficients, for which a large body of data is necessary to provide reliable determinations.

A practical rule for selecting the reach length Δx is that $\Delta x \leq C\Delta t$, with C determined either as a small disturbance movement, or better, as the celerity of a long wave controlled by friction forces.

Good reach division of a channel having singularities (such as junctions, slope drops-rapids, controls, beginning and end of converging and diverging channel sections, and bridges and places with reliable stage-discharge relationships), may enhance the overall accuracy of the method. This is especially true in the case of flood-routing procedures in channels, which are based on a single storage differential equation.

The storage-discharge (or storage-stage) relationship is determined by one of the following methods or combinations thereof: (1) by observed inflows and outflows, when topographic data are lacking; (2) by using stage-discharge relations; (3) by valley topography and observed cross sections; (4) by channel dimensions in the case of regular channel shapes.

When bank inflows and outflows occur in a reach, routing and unrouting (reverse routing) can reveal the lateral inflows and outflows in the reach, if they are not a function of certain hydraulic magnitudes (stage, discharge, boundary area, channel length, or others). This procedure may detect on a hydrograph the time and the effect of a levee break, as well as a later occurrence of a reverse flow.

DEVICES AND SPEEDUPS

There is a group of devices and speedups apart from electrical machines and analogs, that is designed to make flood routing faster and cheaper, with procedures that are less laborious and tedious than are the previous methods of integration based on the simple storage differential equation.

Such devices and speedups are: nomographs, templates and transparent scales, slide rules (straight-line, and circular), polar diagrams, and mechanical integrators. (Practically speaking, all devices and speedups that are generally available for solving first-order differential equations are being applied for flood routing.)

Very little empirical data exist that provide comparative analysis of the different devices and speedups, based on same typical examples, to enable making a reliable estimate of their limitations and advantages.

The inventors or zealous users of some of these devices or speedups are inclined to make biased judgments on basis of rather limited comparisons with other devices and speedups, especially as concerns their individual advantages and disadvantages under specific conditions of flood waves, channel characteristics, and lateral flows.

HYDRAULIC MODELS

Hydraulic models, used for studies of unsteady flow, are nothing more than integrating machines or devices. Achievement of close similitude between the prototype and the model is a difficult problem when a natural channel is to be reproduced. The model needs to be distorted, with two or three scales for geometrical dimensions (one for cross section, another for length; or one for depths, another for widths, and a third for lengths), so that calibration of the model, to assure exactness between model and prototype, especially in roughness, and in hydrographs of a series of floods reproduced at various points, involves work which sometimes becomes prohibitive economically.

It is generally agreed that the technique of flood routing by hydraulic models should be resorted to only under special conditions, such as when: (1) the model is built for other purposes, and flood routing is only an additional study that involves supplemental but not excessive costs; (2) the model has some educational and worthwhile purposes apart from hydraulic studies and flood routing; and (3) the channel net is so complex that the hydraulic model can compete economically with other methods of integration.

The model generally is adapted to past floods of a given shape. If the flood shapes are changed, some adjustments in roughness, geometrical elements, or lateral inflows may be necessary. The same hydrographs of past floods at different places along the rivers may be produced by combination of distorted channel shapes, of roughness and of local inflows or outflows.

As the model scales for long rivers with tributaries must be limited for economical reasons, accuracy of model results is generally of the same order of magnitude as that obtained by the methods of integration by finite differences or by characteristics, or even storage flood routings. The models are reliable for the range of discharges tested during the calibration of models. When there are many flood-routing studies to be made, use of the model is very time consuming. Hydraulic models are relatively expensive computers.

As long as the graphical or numerical processes of integration could be performed by manual procedures only, the hydraulic models, as a method of integration, were considered by some specialists to be competitive and even feasible under specific conditions. The use of digital computers and electrical analogs seems to be highly competitive with hydraulic models, if the models are built mainly for the purpose of flood routing. The maintenance of hydraulic models involves expenses regardless of the needs for flood routing.

The hydraulic models are included here under the flood routing by storage differential equation, because the calibration of model, model distortion, and other approximations give to this integration procedure the same accuracy, in general, as the accuracy of other methods based on storage equation.

ELECTRICAL MACHINES AND ANALOGS

There are three types of electrical machines used in flood routing: (1) high speed, general purpose electric analog computers, which are flexible, speedy and have variable computing speeds, but any change in flow conditions requires alteration of programming (a delaying operation, performed manually); (2) differential analysers, which have been found very suitable, especially when the flood-routing method is based on the simple storage differential equation (the change of flow conditions can be handled readily, and the accuracy is very high); and (3) digital computers, which are the best choice for justifiably high accuracy, and where the change in flow conditions can be programmed automatically (the computational speed is greater than for an analog, but the storage capacity of the machine restricts its use).

It is evident that availability of electrical machines, both digital computers and analogs, will tend to make the integration of differential equations in flood routing a much faster and more accurate operation. But integrating machines require expensive equipment, special recording graph papers or printing tables, and trained personnel.

The accuracy of results is not generally dependent on integrating procedures, but rather upon the accuracy of basic data. Inasmuch as the use of digital computers and of analogs is relatively expensive, their use should always be justified from the point of view of speed, economy, or of accuracy that is commensurate with that of the basic data and with the needs of the problem.

The electrical machines have a promising future in flood routings.

CONCLUSIONS

Although flood-routing methods based on simple storage differential equations have been used ever since routings of inflow hydrographs through reservoirs and flood prediction along the rivers were started, there are only a few studies which compare the flood-routing methods based on two partial differential equations to those based on the storage differential equation.

The literature contains only a small number of studies which compare accuracy, cost, advantages and disadvantages of various methods, devices, or speedups under different conditions of waves, channel and lateral flow.

FLOOD ROUTING BASED ON SIMPLIFIED PARTIAL DIFFERENTIAL EQUATIONS

This is a group of methods which are transitions between those methods which use both the continuity and the momentum equation and those methods which use the storage (continuity) equation only. Some terms in the equations, mostly in the momentum equation, are neglected, or some other assumptions are made (that is, constant bottom slope, linear change of channel characteristics between two cross sections used, separate routings of storage component and translation component of a flood wave, and others). Examples of these intermediate cases are: neglect of acceleration term, $\frac{\partial V}{\partial t}$, which is the second term in equation 5; neglect of velocity head term, $V\frac{\partial V}{\partial x}$, which is the third term in equation 5, or neglect of both acceleration and velocity terms; division of the total discharge in two parts, as discharge of steady flow plus a changing discharge caused by the unsteady flow, with some simplifications in the momentum equation.

The basic characteristic of most of these transition methods is the use of wave celerity, so that both the wave translation as well as the channel storage are taken into account.

A second characteristic is the lack of comparative studies between the transition methods and other methods. For certain conditions, it could be demonstrated that the transition methods would give more accurate results than methods based on the storage differential equation, at little increase in cost. Another comparative study might show under what conditions the transition method would give less accurate results at substantial savings in work time, compared with methods based on the two partial differential equations. There has been more effort in the past to invent or derive new methods of integrating the different types of equations in flood routing than to analyze the limitations and advantages of each method in particular and to compare all of them in general.

EMPIRICAL METHODS

The empirical method of flood routing is mostly based on the relationships developed from past floods. Graphs or equations are developed, which relate different wave characteristics at adjacent places of a river system. The most common method applied is the stage-relationship, covering two or many stations, on the main channel and on tributaries.

This empirical procedure is generally used as the first step in developing a forecasting service on a river, with a general trend to replace it by more accurate and reliable procedure, as soon as data are sufficiently supplied.

The accuracy of these empirical methods is less than the accuracy of other flood-routing methods.

ROUTING OF SURGES AND STEEP WAVES

The literature on surges is very abundant (see Bibliography with abstracts and indexes) concerning their height, celerity, and evolution as they progress along the channel.

The formulas for celerity, surge height, and velocity of accelerated water by surge movement, as equations 6 and 7, or others, are the basis for the computation of surge movement.

As the surges are discontinuities of partial derivatives, which propagate in the (x, t) plane along the characteristics lines, the most logical method of surge computation is the method of characteristics, whenever there are many surges mutually interfering or reflecting during their progress along the channel.

Nearly all surges (except some bores) change their shape by progressing to an intermediate form between a surge and long wave. Negative surges (depressions) usually take the shape of monoclinal waves.

When a wave is still steep, in a position between surge and long wave, either some adaptations of the two partial differential equations or the water-storage equation, are currently used, or the wave is approximated by a surge and treated as such.

This approximation of computation of rather steep waves ($\partial H/\partial x$, $\partial H/\partial t$, $\partial V/\partial x$, and $\partial V/\partial t$ are not small for some parts of the wave) by flood-routing procedures for long waves, or by formulas for surges, is always a source of errors, apart from errors involved in the methods applied. With existing theories of unsteady flow, it can be stated that for waves between surges and long waves, the routing methods will not produce results as accurately as can be obtained for pure surges or long waves.

ADDITIONAL REMARKS ON UNSTEADY FLOW AND FLOOD ROUTING

After surveying the literature on unsteady flow in channels and reservoirs and on flood-routing methods, during the work on abstracting references given in the second part of this study, the following general remarks and derivations can be made:

1. The literature on unsteady channel flow and flood routing increases by an exponential law; it was nearly doubled in the 18-year period 1941–59. Taking into consideration war years, with the reduced research activities in this field, the rate of increase may be now considered as a growth doubling the number of papers, reports, and chapters in books in about 15 years. By 1975 it might be expected that the actual literature, if not by substantial contributions, will be doubled at least by the amount of references.
2. The screening of literature for the current use in research and practice becomes important, especially as new methods of computing wave movement and flood routing are added every year to a rather large amount of existing methods and procedures.
3. Nearly all physical aspects of channel waves are studied, either theoretically, or experimentally and by observations in nature, and many specific problems were attacked together with the basic problems.
4. The references in chronological order show a great deal of repetitive research, where the authors of new methods or new development were unaware of and unacquainted with previous studies which produced similar or even the same results.
5. The studies and developments of new flood-routing methods are often undertaken with approximately the same approach as previously used.
6. The trend in the last three decades has been towards development of new methods of integrating the basic differential equations, rather than to attack the basic equations with new approaches.
7. The increase of the number of methods for flood routing has probably come to the point where there is more need to investigate the feasibility of available methods than to develop new methods. The new methods can be developed as well from the analysis of existing methods as from new ideas, independent of the properties of existing methods.
8. There is a rather limited exchange of ideas between different countries in this field; it is not unusual that a large study in this field carries only the references from one country, with rather limited or unimportant references from other countries.

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9. Studies of errors resulting from approximations used in the different mathematical tools for the specific wave-, channel-, and lateral-flow conditions are lacking.
10. Studies of errors resulting from insufficient basic data and information are also lacking.
11. A great deal of comparisons of flood routing results, computed by a given method, with the experiments or observations under natural conditions are done by the authors for their method, usually under the specific conditions for waves and channels.
12. There is a lack of studies comparing the feasibility of many flood-routing methods under different conditions, taking into account the type of data available, errors in basic tools used, accuracy of results, and the economical aspects of the methods used.
13. The techniques which exist for treating flood wave evolution along channels are manifold, but there is a clear deficiency in the appraisal of the characteristics, errors, and economic feasibility encountered by using these techniques.

PROBLEMS FOR STUDY

There are many problems in the physics of unsteady water flow, and especially when applied to flood routings, which need a new scientific approach and treatment.

The complexity of fluid mechanics of waves in channels, especially when boundary characteristics change along channels, with new variables involved, is the source of many potential studies and research in the future. The actual approximations can be pushed much nearer to the real physical aspects of the wave movement.

A solitary wave propagates without changing the shape in a frictionless horizontal channel, at least as proved by some mathematical analyses. If the friction force is added, and the channel has a slope which approximately compensates for the mean head loss during wave propagation, the question is, can a solitary wave be without change of shape, since the constant slope does not compensate equally for the friction loss in all parts of the wave?

What are the criteria when a wave starts to steepen, or what are conditions under which a wave is stable and in attenuation, or is unstable and in amplification, under different conditions of natural or artificial channels?

What are the friction losses in an unsteady flow, as function of rate of change of velocity along the channel, taking into account the changes in boundary layer, and when can they be assumed to be equal to losses in steady flow, with the given tolerance error?

What is the real influence of vertical acceleration of steep waves on the wave movement?

How does unsteady flow influence the velocity distribution coefficients?

There are many problems open for further study in the physics of channel unsteady flow, which, when solved, may throw light on the order of approximation of mathematical tools now used and also how to improve the tools.

When flood-routing methods are concerned, it is probably as important to appraise the methods now in existence, as it is to develop new methods. From the practical point of view, the appraisal of existing methods may seem to be much more useful.

The opposition sometimes encountered in advancing new research projects in flood routing may be proof that the sound appraisal of existing methods is necessary to find the best existing tools for practical use, and as a preliminary work for the advancement of new ideas and research projects which are justified by new approaches.

A possible approach to appraise the feasibility of existing or newly developed flood-routing methods may be designed in the following steps:

1. Several reaches on rivers and on artificial channels must be selected, which would cover a large variety of conditions (small river, medium river, large river, very large river, one or two artificial channels), and which should have the following characteristics:
   (a) the boundary conditions (cross section, slopes, roughness, singularities) are known;
   (b) lateral inflows and outflows are adjustable at will, or if they can not be adjusted, that they can be easily determined;
   (c) a pool or reservoir upstream from the reach, with possibility to create any type of flood wave, and if this operation would cause downstream difficulties, that there is a pool or reservoir downstream from the reach to dampen the created waves; and
   (d) facilities for obtaining observations, and conducting research work should be available.
2. The individual characteristics and advantages and disadvantages of existing flood-routing methods must be appraised (bibliographical part of this study can be considered as preliminary work for this step).
3. A research program must be designed on the basis of appraisal of flood-routing methods, and the available facilities of selected channel reaches.

4. Flood waves of different forms must be created and their progress observed at selected points along the reach.
5. After sufficient experience has been obtained in applying each method, the above-mentioned floods should be routed through the reach with different routing methods, different amounts of background data, and varied finite differences. The observed hydrographs for selected sections could then be compared with the flood hydrographs obtained with step-by-step routing. The cost of routing by different methods should be tabulated for study.
6. By using statistical methods, the observed and the routed flood hydrographs, or flood profiles for the reach, should be analyzed to obtain the errors in each specific case.
7. It is necessary to derive the relationships of accuracy of results in flood routing to the cost of routing, taking into account the basic cost of equipment and all other necessary tools, for applied flood-routing methods.
8. The feasibility of all selected flood-routing methods must be appraised from the point of view of accuracy, conditions under which a method supplies a given accuracy-cost relationship, and the conditions under which the basic investments start to be insignificant in the unit cost of routing.
9. Recommendations for practical use must be established.
10. New approaches must be suggested or new problems to be attacked by new research projects in the field of unsteady channel flow and flood routing.

For given funds, time, and problem at hand, with the devices available, there is always the most accurate flood-routing method. Comparative studies of this kind do not exist, and the differences and controversies about the selection and use of flood-routing methods come from the lack of this comparison of methods. Currently, the the intuitive judgment and general experience are used more than the exact criteria in selecting a flood-routing method to be applied.

It seems that developments in the water-resources field have reached the point where routing of water waves along the natural and artificial channels has become, especially in discharge and stage forecasting, a widely prevalent need. By using the theory of errors, it would be possible to classify flood-routing method by objective criteria, and to restrict further the "sound engineering judgment," following thus the general trend in the contemporaneous scientific endeavor.

The development of methods of treating water waves along channels has now reached great breadth. The immediate need for an objective appraisal of methods is strongly stressed. With sufficient comparative studies and results, objective criteria could be established to enable selection of optimum methods for practical use under given conditions.

BIBLIOGRAPHY

The explanations and remarks contained in this introduction are made to facilitate use of the bibliography and also, it is hoped, to increase its worth to the reader.

Subject of bibliography.—Basically, the subject of this bibliography is encompassed by one body of theory that is contained within one homogeneous area of research concerning unsteady flow of water in channels. However, practical application makes it advisable to treat unsteady flow in channels and reservoirs as being applicable to the field of hydraulics or fluid mechanics, and the derivative subject, flood routing, as being applicable to the field of hydrology. Both the hydraulic and hydrologic aspects of the reference studies are given, in this bibliography, in order to present abstracts of the literature according to applicability.

Many types of bodies of water are considered to be included in the category of reservoirs: artificial reservoirs; natural and artificially-controlled lakes; pools; and large channels with very small slopes, large cross-sectional areas, and small velocities. Similarly, the category of channels includes many types of conduits: river channels; torrent channels; estuaries; artificial canals; also, tunnels, pipes, culverts, and any elongated conduit with free water surface flow.

References on subjects devoted to the general theory of wave motion in channels as well as references describing practical methods and procedures for flood routing are included.

The range of method coverage is large, extending from simple empirical methods of problem solution to purely theoretical and mathematical treatment of subjects. Many references treat experimental work done in laboratories; others deal with observations made in nature or experiments conducted in actual river channels or canals. The characteristics of waves and channels and their interrelationships are subjects of many references and abstracts. References pertaining to the practical problems of flood-wave transformations and of predictions occupy a substantial part of the bibliography. The subject of tidal-wave movement in estuaries and channels is included, insofar as the references are concerned entirely or partly with unsteady flow in channels.

The bibliography comprises references to papers that are original reports of studies made, and also references to papers that are restatements, summarizing the results of previous studies.

The bibliography also includes reports on the physics of the reference subject. The resulting mathematical formulas are often given in the abstract or clarifications are made for the approximations that have been introduced to describe the phenomena of wave movements. For their instructional value, abstracts of a few studies that reached erroneous conclusions are also included in the bibliography, and the errors are pointed out.

Purposes of bibliography and abstracts.—In addition to the obvious purposes served by a comprehensive bibliography, this bibliography attempts to be a step toward systematization of the body of knowledge that concerns the hydraulic laws of unsteady flow in channels and reservoirs. It further attempts to provide the basis for a functional digest of flood-routing methods.

It is hoped that this bibliography, as an incidental chronicle by covering the literature in many languages, will also have some historical value or value for a future historical survey of the contributions to the sciences of hydraulics and hydrology made, over the years, by numerous scientists working in different countries, at different times, and frequently being unaware of an identical or closely related study performed elsewhere.

Format used for references and their abstracts.—Each reference in the bibliography is identified by: number, author, year of publication, language (if other than English), title, journal or other organ of publication, volume, page or pages, and (for books) publisher's name and location. For those references concerning papers published in a language other than English, the title is given in the original language, followed by its English translation. Occasionally, particularly in regard to references taken from the older Proceedings of the French Academy of Sciences and other French journals, only the last name of the author is available, but fortunately, there is only one author by this surname so that no confusion of authorship results.

Abstracts, included herein, that are not restatements, are generally informative rather than being merely descriptive.

As far as possible, the text of each abstract has been kept within 100 to 150 words. Some abstracts are necessarily longer but an effort has been made not to exceed a length of 500 words.

Sources of abstracts.—The abstracts contained in this bibliography are from four principal sources:

1. Abstracts written by the compiler of this bibliography, comprising a large majority of the total number of abstracts. These abstracts do not show authorship and were compiled during the last half of 1959 or early in 1960, with some revisions, corrections, and additions in February 1962.
2. Author's own abstract (or summary, synopsis, introduction, or conclusions), used partly or in its entirety in this report, with author credit shown at the end of the abstract.

3. Abstracts taken from published Transactions of American Geophysical Union, commencing with Part 4, 1938, p. 362. The name of the author of the abstract is shown at the end of the abstract.
4. Abstracts taken from "Bibliography on tidal hydraulics," compiled by the Committee on Tidal Hydraulics, Corps of Engineers, U.S. Army, as published in several reports and supplements. At the end of each abstract used from this source, the source is shown, together with the year of its publication.

Symbols used.—For convenience, unified symbols have been used for magnitudes and terms commonly employed. All formulas in the original articles have been transliterated by using the unique set of symbols shown in "List of Symbols" (p. 36). Specific symbols in the original articles have been retained and their use explained in the abstracts. If a symbol used in the abstract is not clear to the reader, he will find its explanation in the list of symbols.

Chronological bibliography with abstracts.—The references, together with their abstracts, are presented in chronological sequence, by year, according to date of first publication (and not according to republication date). Within a particular year, the sequence of references is by author's name, in alphabetical order. Thus arranged, references have been numbered consecutively. Each reference can therefore be identified by its number, author or authors, and year of publication. This chronological sequence provides a unique chronicle of theory development, showing how different aspects and problems of unsteady flow and of flood routing through reservoirs and channels were treated and solved during the last century and a half of active study.

Index by authors.—The index by authors is useful in showing the whole array of an author's contribution to the subjects of this bibliography and indicates the finding number for each reference. This index is compiled in alphabetical order according to the author's name, and each of his reports (article, paper, book) is identified by two numbers, the first being the number of that reference in the bibliography's chronological listing, and the second being the year of publication.

Index by subjects.—An index by subjects is also given, compiled in alphabetical order of subject, as determined from content of abstracts. Following each subject name are numbers referring to the references treating the subject, with these identifying numbers being taken from the bibliography's chronological list of references. The larger the number, the more recent is the reference. It is therefore possible to follow references on a subject in sequential order. The subject index does not attempt to be complete in covering all the ramifications of subject that might be inferred in the content of an abstract, but rather attempts to orient the reader toward the major subjects treated by the references in the bibliography.

Last years. The coverage of years, 1959–61 is not as complete as it is for the previous years.

LIST OF SYMBOLS

These are the symbols most frequently used in the abstracts, which are not explained in the text. When these symbols are used differently than given here, they are explained in the corresponding abstract. All other symbols used are described in the text.

A	Cross-sectional area
B	Width of channel or reservoir
C	Wave celerity (velocity of wave propagation)
c	Resistance coefficient in Chezy's formula
F_1, F_2	Storage factors
f	Sign for a function, in general, as in $f(x)$
g	Acceleration of gravity
H_0	Depth in channel or reservoir for steady flow
H	Depth in channel or reservoir for unsteady flow
h	Wave height $(H-H_0)$, or wave amplitude
K	Coefficient in the routing formula
l	Wavelength
L	Length of reach or reservoir
n	Resistance coefficient in Manning's formula
P	Inflow discharge
Q	Outflow discharge
q	Outflow discharge per unit of width
R	Hydraulic radius
S	Longitudinal slope of water surface
S_0	Bottom slope of channel
S_f	Friction slope
t, Δt	Time and time interval
U	Mean velocity or mean velocity increment, created by acceleration due to the wave movement
V	Mean velocity
W	Storage volume
X	A dimensionless constant in the coefficient method for flood routing
x, Δx	Any length along the channel or reservoir
y	Any wave height ($h=y$ max)
z	Elevation of water surface above the reference level.

CHRONOLOGICAL BIBLIOGRAPHY AND ABSTRACTS

1. Newton, Sir, I., 1685–86, Principia: Royal Soc. [London], Book 2, Propositions 44–46 [1687].

It is assumed that the waves can be compared to the oscillations of water in an inverted syphon. This leads to the conclusion that the propagation celerity of waves is proportional to the square root of the wavelength (from crest to crest), and that each wave travels that length in a time equal to the oscillation of a pendulum with the length of the double wavelength. This was not confirmed by further studies.

2. Laplace, P. S., 1775–76, Recherches sur quelques points du système du monde [Researches on some points of world system]: Acad. Sci. [Paris] Mém., (complete works, v. 9).

The differential equations of the water oscillations are given in the first approximation, starting from a fluid at rest. It is assumed that fluid disturbed from the state of equilibrium acquires surface shape that is trochoidal.

3. Lagrange, I. L., 1781, Mémoire sur la theorie du mouvement des fluides [Memoir on the theory of movement of fluids]: Acad. Royal [Berlin] Mem., p. 151–198, [1783]; and p. 192–198 [1788].

This is the first development of the theory that the celerity of wave propagation along canals is $C=\sqrt{gH}$.

4. Lagrange, I. L., 1788, Mécanique analytique [Analytical Mechanics]: Paris, Bertrand's ed., pt. 2, sec. 2, act. 2, p. 192; or Oeuvres [Works] sec. 11, act. 36.

First in the Memoirs of Berlin, then in the Analytical Mechanics, Lagrange treats the theory of waves in water of shallow constant depth. He stresses the analogy of wave and sound propagation celerities, which are independent of the original disturbance. The celerity is proportional to the square root of the fluid depth in a canal of constant width. A disturbance of the surface in any depth of fluid is transmitted a very small distance from the surface, and the celerity is proportional to the square root of that distance. This last conclusion is weakened by the uncertainty of distance determination due to the progressively diminishing velocity of particles.

5. Gerstner, Franz von, 1802, Theorie der Wallen [Theory of Waves]: Koen. Boehmischen Gesell. [Prague], Wiss. Abh.; and Gilbert's Annalen d. Phys., v. 32, 1809.

The concept of the relative movement of particles in circular oscillation in deep water sea waves is introduced by mathematical treatment. The particles form circles having fixed centers and decreasing radius in a geometric progression from the surface downward.

6. Young, Cf. T., 1813, A theory of tides: Nicholson's Jour., v. 35, 1813, 1823; and Misc. Works, London, p. 262, 291, 1854.

Simple methods are shown for expressing several of the principal parts of the canal theory of tides.

7. Poisson, S. D., 1816, Mémoire sur la théorie des ondes [Memoir on the theory of waves]: Acad. sci. [Paris] Mém., v. 1, p. 71–186.

Two differential equations of wave movement are given and integrated, assuming neglect of one horizontal dimension of fluid. An analysis is made of: (1) the propagation of surface waves in a vertical canal of constant width and great depth; (2) the propagation of fluid motion in the vertical direction, with the abstraction of one horizontal dimension. By employing all three dimensions of fluid, differential equations of the first order are integrated. By considering two horizontal dimensions, wave propagation is shown. By assuming that all three dimensions are known, analysis is made of the propagation of movement inside the fluid.

8. Bidone, George, 1824, Expériences sur la propagation du remous [Experiments on the wave propagation]: Reale Acad. sci. [Torino, Italy] Mem., v. 30, p. 195–292.

This is one of the earliest experimental studies on wave propagation. The experiments employ a small rectangular canal, 2 feet wide by 40 feet long. Study is made of the propagation of a negative wave produced when steady flow, having uniform depth H and velocity V, is suddenly stopped at the downstream end of a canal. By using the continuity equation, the propagation celerity C is derived as $Ch = VH$, where h=wave height. The wave height is derived from the equation $h = (2H+h)\, V^2/2gh$ (shown by later studies to be incorrect). Other aspects of wave propagation are studied, including the propagation of tidal bores in estuaries. The propagation celerity found for h is approximately the same as that found in later experiments by Russell and Bazin.

9. Weber, Ernest, and Weber, William, 1825, Wellenlehre auf Experimente gegruendet, oder ueber die Wellen tropfbaren Fluessigkeiten mit Anwendung auf die Schall- und Licht-Wellen [The theory of waves based on the experiments, or the theory of sound and light waves applied to theory of liquid waves]: Leipzig, Gerhard Fleischer.

A compilation of studies made in the field of wave theory, from the time of Newton to 1825, that includes the works of Laplace, Poisson, Cauchy, and others, and constitutes a valuable history of wave research. Included also are the authors' experiments, principally on oscillatory waves in quiet water.

10. Cauchy, A. L., 1827, Théorie de la propagation des ondes à la surface d'un fluide pésant d'une propondeur indéfinie [Theory of wave propagation on the surface of a heavy fluid with infinite depth]: Acad. sci. [Paris] Mém., v. 1, p. 3–312.

With surface shape of wave and wave producing forces being known, equations giving the initial conditions of fluid motion are developed. Then equations are developed for fluid motion and surface configuration during any time of motion. The general laws of wave movement are presented and constants of the development equations are determined. Use of the analytical formula is demonstrated in the concluding notes. This study constitutes an extensive analytical treatise on the general theories of waves. (Manuscript was submitted Sept. 1815.)

11. Green, G., 1837, On the motion of waves in a variable canal of small depth and width: Philos. Soc. [Cambridge] Trans., v. 6, pt. 3, p. 457–462 [1938]. Reprinted *in* Mathematical Papers of George Green, London, MacMillan and Co., p. 223–230, 1871.

By integration, using Bernouilli's studies and continuity equation, small wave propagation in a canal is shown to have the propagation celerity $dx/dt=\sqrt{gH}$. Wave height, velocity of the fluid particles, and wavelength are expressed as functions of the variable width and of the depth of the canal. (Read May 15, 1837.)

12. Russell, J. S. (Scott), 1837, Experimental researches into the laws of certain hydro-dynamic phenomena that accompany the motion of floating bodies, and have not previously been reduced into conformity with the known laws of the resistance of fluids: Royal Soc. [Edinburgh] Philos. Trans., v. 14, p. 47–109 [1840]. Published in French *in* Ponts et Chaussées Annales, sem. 2, p. 143–234, 1837.

In addition to treatment of title subjects, the paper also includes results of experiments made in 1834 and 1835 on wave-propagation celerities in channels and canals, which later produced the formula $C=\sqrt{g(H_0+h)}$. (Read Apr. 3, 1837.)

13. Green, G., 1839, Note on the motion of waves in canals: Philos. Soc. [Cambridge] Trans. Reprinted *in* Mathematical Papers of George Green, London, MacMillan & Co., p. 273–280, 1871.

This study (a continuation of the author's theoretical studies of one of the subjects published in 1837) based on Russell's experiments and derivations, analyses the circular orbit of solitary wave movement as it occurs in still water in a channel. The wave celerity is analyzed. It is revealed that the propagation celerity of a long wave occurring in a triangular canal having one vertical side is the same as that occurring in a rectangular canal of half the depth. (Read Feb. 18, 1839.)

14. Kelland, P., 1839, On the theory of waves, Part I: Royal Soc. [Edinburgh] Trans., v. 14, p. 497–545 [1840].

The celerity of a solitary wave is developed analytically in a complex, exponential form, involving canal depth, elevation of wave, and wavelength. An attempt was made to substantiate the formula by use of the experimental data of J. S. Russell. However, a discrepency was found in the experimental data. The author's analysis found the propagation celerity to be $\sqrt{gA/B}$, where A=area of the section of the canal and B=width of the canal at the water surface. (Read Apr. 1, 1839.)

15. Russell, J. S. (Scott), 1842–43, Report on waves: British Assoc. Adv. Sci. Proc. for 1844, p. 311–390 [1845].

The wave is analyzed as a vehicle of energy. Water waves are classified as (1) waves of translation (solitary), (2) oscillating waves, (3) capillary waves, (4) corpuscular waves (solitary or sound waves). First-order waves (waves of translation), which are (1) positive or negative, (2) free or forced, are studied experimentally, the conclusions presented concern damping, celerity (empirical equation $C=\sqrt{gH}$ or $C=\sqrt{g(H_0+h)}$, wavelength, wave form, and velocities of water particles. Also, studies are made of oscillatory, capillary, and corpuscular waves. The wave-celerity formula developed is compared with the theoretical formulas of Kelland and Airy. (Reported to British Association for Advancement of Science at meetings in 1842–43.)

16. Airy, Sir George Biddle, 1845, Tides and waves: Encyclopedia Metropolitana, London, p. 241–396.

A study of long waves in canals with variable cross sections. Kelland's formula of wave celerity is redeveloped on a simplified basis. The theories of oscillatory waves and standing oscillations are treated fully. A thorough analysis of tidal wave is made. On the assumption that the horizontal motion of water is effectively the same, from top to bottom, tidal-wave calculation is presented in simplified form. It is shown that the front of the tidal wave becomes shorter and steeper as the wave progresses. Expressed for the first time is the theory that solitary wave without friction resistance is unstable and deforms as it progresses. (Doubt has been cast upon this theory by the subsequent investigations of Boussinesq, Lord Rayleigh, and De Saint-Venant.)

17. Stokes, G. G., 1846, Report on recent researches in hydrodynamics (chapter on waves): British Assoc. Adv. Sci., 16th Mtg., Southhampton, 1846. Proc.; also Report on the State of Science, [London], p. 1–20, [1847] Reprinted *in* Stokes Math. and Phys. Papers [Cambridge], p. 157–187, 1880.

An appraisal of the progress of science literature on wave motion up to the mid-19th century. The author wrongly supports Earnshaw's thesis that the change of wave shape is not chiefly due to friction but is an inherent characteristic of its motion.

18. Earnshaw, S., 1847, The mathematical theory of the two great solitary waves of the first order: Philos. Soc. [Cambridge] Trans., v. 8, pt. 3, p. 326–341.

A discussion of problems involved in the integration of differential equations for fluid motion in wave movement. The need for combined experimental and analytical research is stressed. Russell's experimental data and conclusions are discussed, together with a mathematical analysis of positive and negative solitary waves. A complex expression of celerity of small waves is derived that permits neglect of some terms in obtaining the approximate celerity $\sqrt{gH}$. It is the author's conclusion (later proved to be wrong) that wave degradation is due not only to friction but also to the consequence of initial motion of fluid particles. (Read Dec. 8, 1845.)

19. Stokes, G. G., 1847, On the theory of oscillatory waves: Philos. Soc. [Cambridge] Trans., v. 8, pt. 4, p. 411–455. Reprinted in Stokes Math. and Phys. Papers [Cambridge], p. 195–219, 1880.

The author contends that, except for an infinite succession of oscillatory waves (discussed with an approximation), there can be no wave propagation without change of form or velocity. Second-order approximations are presented to indicate that, for small waves of finite height, wave celerity is independent of wave height. Expressions are derived for wave celerity, velocities of particles, and other characteristics of oscillatory waves.

20. Baumgarten, 1848, Jaugeages sur la Garonne [Flow measurements on Garonne River]: Ponts et Chaussées Annales [France], July-Aug., p. 45.

Observations on the Garonne River are presented indicating that during a flood rise the velocities and discharges are greater for a given stage than they are during a falling flood.

21. Caligny, Anatole de, 1848, Expériences sur une nouvelle espéce d'ondes liquides à double mouvement oscillatoire et orbitaire [Experiments on a new type of liquid waves with double motion, oscillatory and orbital]: Jour. Math. Pures et Appl. de M. Liouville [France], ser. 1, v. 13, p. 91–110.

Experiments with the waves of great height, and especially with solitary waves, are described. It is concluded that they can not be produced experimentally. The differences and similarities of flowing and solitary waves are discussed.

22. Partiot, H. L., 1858, Mémoire sur mascaret (Extrait par l'auter) [Author's abstract: Memoir on the tidal bore]: Acad. sci. [Paris] Comptes rendus, v. 47, p. 651–654.

This paper describes the tidal bore, with particular reference to two bores of the Seine (measuring 2.18 and 1.68 meters), which provide basis for the conclusions reached. Based on Virla's theory (Annales Ponts et Chaussées, 1833). The memoir treats the analysis of the tidal bore.

23. Caligny, Anatole de, 1861, Expériences sur la génération des ondes liquides dites courantes [Experiments on the creation of liquid waves called flowing waves]: Acad. sci. [Paris] Comptes rendus, v. 52, p. 1309–1311.

This is a brief discussion of the paper "Cenni sul moto ondoso del mare e sulle correnti di esso" [Remarks on the wave motion of the sea and on the currents produced by it] by Cialdi, Academy of Sciences, no. 4, Rome, 1856.

24. Hagen, G., 1861, Ueber die Wellen auf Gewaessern von glechmaessiger Tiefe [On the wave in channels with uniform depth]: Abh. Kgl. Akad. der Wiss. [Berlin], Math. Abh. p. 1–79, [1862].

A study that is chiefly applicable to deep-water waves. The celerity of waves in the form $C=\sqrt{g\lambda/\pi}$ is given (Gerstner's studies), and the velocity of particles in a wave movement is studied.

25. Partiot, H. L., 1861, Mémoire sur le mascaret [Memoir on the tidal bore]: Ponts et Chaussées Annales [French], sem. 1, no. 2, p. 17–48.

The tidal bore on the Seine River is observed, and the results and explanations are given. The cosine line is applied to the shape of a tide, and the properties of tidal waves are discussed.

26. Partiot, H. L., 1861, Étude sur les mouvements des marées dans la partie maritime des fleuves [Study on the movements of tides in the maritime portion of rivers]: Paris.

27. Bazin, H., 1862, Expériences sur les ondes et la propagation des remous [Experiments with waves and the propagation of negative waves]: Acad. sci. [Paris] Comptes rendus, v. 55, p. 353–357, [1862], and v. 57, p. 302–312, [1863].

This study demonstrates that a positive wave, having a wave height h that is maintained unchanged for a long distance, travels with celerity $C=\sqrt{g(H_0+h)}$. The negative wave, which cannot be maintained unchanged for as long a distance and which is always followed by a group of oscillatory waves, travels with celerity $C=\sqrt{g(H_0-h)}$. In flowing water, with velocity V, the celerity is $C=V\pm\sqrt{g(H_0\pm h)}$, for both positive and negative waves. Considering the case of sudden stoppage of water movement (from V to 0), the celerity of the backwater is given both for the positive and the negative bore. Experiments reported in this paper are on a much greater scale than previous similar experiments made by Russell.

28. Rankine, W. J. M., 1863, On the exact form of waves near surface of deep water: Philos. Mag. [London] (Rankine's Paper), p. 481.

This report contains the complete solution for the trochoidal waves, as obtained by Gerstner in 1801 and by Rankine (independently) in 1862.

29. Caland, N., 1864, Untersuchungen ueber die Wirkung der Ebbe and Flut in dem maritimen Teile der Fluesse [Investigations on the tidal effect in the maritime portion of rivers]: Allg. Bauzeitung.

30. Bazin, H., 1865, Recherches expérimentales sur la propagation des ondes [Experimental research on wave propagation]: Acad. sci. [Paris] Mém., v. 19, p. 495–644. Also, Dunod, Paris, 1865, in second part of hydraulic researches of Darcy and Bazin.

Experiments constitute a confirmation of Russell's formula $C=\sqrt{g(H_0+h)}$. The relative propagation celerity in moving water is shown to be $C=\sqrt{g(H_0+h)}\pm V$, where $C=\sqrt{g(H_0+h)}+V$ for the positive wave moving in the current, and $C=\sqrt{g(\mathrm{H}_0+h)}-V$ for the positive moving against the current, with the change of the sign of h for the negative wave. The positive and negative translation waves are studies by experiments which prove the above formulas. Experiments of the propagation celerity of steep waves (surges and depressions) are made. These indicate that formulas approximately the same as the above can be used for rapidly changing unsteady flow. Study of the tidal bore (mascaret) is made.

31. Caligny, Anatole de, 1866, Expériences diverses sur les ondes en mer et dans les canaux, etc. [Different experiments on the sea and canal waves, etc.]: Jour. Math. Pures et Appl. de M. Liouville [France], s. 2, v. 11, p. 255–265.

The experiments with waves in a canal in Versaille (near Paris) are described. In the discussion of "flowing waves" and solitary waves, it is revealed that the author could not produce the solitary wave (although he could compute its celerity) but that he could produce flowing (current) waves by experiment. Studies made of the curves described by the course of a water particle during wave movement are discussed. The experiments on the formation of solitary waves are discussed extensively. At the end, remarks concerning waves produced by the emptying of navigation locks are given.

32. De Saint-Venant, Barré, 1870, Demonstration élémentaire de la formule de propagation d'une onde ou d'une intumescence dans un canal prismatique; et remarques sur les propagations du son et de la lumière, sur les ressauts, aussi que sur la distinction des rivières et des torrents [Elementary demonstration of the propagation formula for a wave or a translatory wave in a prismatic channel; remarks on the propagation of sound and light, on hydraulic jumps; remarks on the distinction between rivers and torrents]: Acad. sci. [Paris] Comptes rendus, v. 71, p. 186–195.

Starting with Babinet's development of sound propagation in fluids, the celerity $C=\sqrt{gH}$ is derived theoretically, thus supporting Lagrange's mathematical developments and Russell's and Bazin's experiments, as well as Partiot's observations of tidal-wave propagations.

33. Boussinesq, Joseph, 1871, Théorie de l'intumescence liquide appelée onde) solitaire ou de translation, se propageant dans un canal rectangulaire [Theory of the liquid intumescence, called a solitary wave or a wave of translation, propagated in a channel of rectangular cross section]: Acad. sci. [Paris] Comptes rendus, v. 72, p. 755–759.

The author presents a theoretical analysis of the celerity of a solitary wave in an indefinitely long canal of constant depth containing still water.

On the basis of the De Saint-Venant studies, a formula for celerity is derived as $C=\sqrt{g(H_0+3h/2)}$, where H_0=depth before wave arrives, and H= wave height or the crest height above undisturbed water. The author supports the two Lagrange formulas for wave movement: $C=\sqrt{gH}$ and $UH=CH$, where U=velocity of accelerated water. An equation is derived in exponential form for the profile of symmetrical waves.

34. De Saint-Venant, Barré, 1871, Théorie du movement non-permanent des eaux avec application aux crues des rivières et à l'introduction des marées dans leur lit [Theory of unsteady water flow, with application to river floods and to propagation of tides in river channels]: Acad. sci. [Paris] Comptes rendus, v. 73, p. 148–154, 237–240. (Translated into English by U.S. Corps of Engineers, no. 49–g, Waterways Experiment Station, Vicksburg, Miss., 1949.)

The first part of this study, inspired by Partiot's studies, demonstrates the celerity and profile of the tidal wave in an estuary. An increase of discharge, and of depth, from H to y increases the water velocity, as shown by the continuity equation: $U=2\sqrt{gy}-2\sqrt{gH}$, if $U=0$ for $y=H$, then the wave celerity $C=3\sqrt{gy}-2\sqrt{gH}$. The application is discussed. The second part is the derivation of partial differential equations, now called "De Saint-Venant equations" (continuity and momentum equations), for gradually varied, unsteady flow:

$$\partial A/\partial t+\partial (AV)/\partial x=0 \text{ (continuity)}$$

$$\partial z/\partial x=(1/g)\partial V/\partial t+(V/g)\partial V/\partial x+S_f \text{ (momentum)}$$

where A=area, V=mean velocity, z=position of water surface above reference level, S_f=friction slope, x=length along the rectangular prismatic canal, and t=time. S_f is given in a general form. A comparison is made with Bazin's equations and some applications are shown.

35. Partiot, H. L., 1871, Mémoire sur les marées fluviales [Memoir on the tides in rivers]: Acad. sci. [Paris] Comptes rendus, v. 73, p. 91.

On the basis of observation for formula for tidal wave, celerity C is given as $C=m\sqrt{gH_0}+V$, with m=numerical coefficient representing the friction and other factors, and V =the mean velocity at previous water depth. The longitudinal-wave profile is given, starting with the sine-wave profile, at the entrance of the river estuary. The profile or river bed is represented by steps or by many horizontal reaches, and the velocity V takes the sign of C.

36. Boussinesq, Joseph, 1872, Théories des ondes et des remous qui se propagent le long d'un canal rectangulaire horizontal, en communuiquant au liquide continu dans ce canal des vitesses sesivlement paralleles de la surface au fond [The theory of waves and backwaters which propagate along a rectangular horizontal canal, subject to continuous flow and having velocities in the canal that are approximately parallel to the bottom]: Jour. Math. Pures et Appl. de M. Liouville [France], s. 2, v. 17, p. 55–108; additions, v. 18, 1873, p. 47–52.

Starting from the fundamental partial differential equations for unsteady flow, developed by De Saint-Venant (1871), the formula for velocities due to the acceleration by wave movement is developed, and wave celerity is shown to be

$$C^2=g[H_0+3h/2+(H_0^3/3h)d^2h/dx^2]$$

From this the formula for velocity is produced. The movement of the center of gravity of a gradually varied wave is analyzed analytically. The shape of solitary wave is derived. The moment of instability of waves is introduced and defined. Nonsolitary waves are also studied briefly.

37. Caligny, Anatole de, 1872, Rescherches sur les oscillations [Researches on oscillations]: Paris, p. 1–258.

38. Boussinesq, Joseph, 1873, Théorie des ondes liquides periodiques [Theory of liquid periodic waves]: Acad. sci. [Paris] Mémoires présentés par divers savants, v. 20, p. 509–615.

The paper discusses equations of continuous movement of any medium; the case of a heavy liquid; periodic waves of liquid, with fundamental relations; plane waves; waves propagated from any disturbance region, with first and second approximations; the special cases of the waves propagating in a canal, and of waves propagating in a infinite reservoir, starting from the same center of disturbance; the diffraction phenomena in liquid waves; the case (Note 1) when the terms of square and product orders are kept in the differential equations, with theory of sea waves; the case (Note 2) of the effort of internal resistance of fluids on wave phenomena; and cases (Notes 3 and 4) that treat wave energy questions.

39. Graeff, 1875, Mémoire sur le movement des eaux dans les réservoirs à alimentation variable [Memoir on water movement in reservoirs with variable inflow]: Acad. sci. [Paris] Mém., v. 21, p. 393–538.

The theoretical study of the modification of flows by routing through reservoirs, based on the equation: inflow minus outflow equals reservoir storage. Various functional relationships are studied: area of volume of reservoirs as functions of the depth, use of the outflow hydrographs to produce simple curves of storage volume versus depth and to produce outflow rating curves (representing use of valves or gates). Flood routing through a reservoir and the reservoir effects on floods is developed. The time required for partial emptying of reservoir storage is derived for simple cases. Some examples are given. This is probably the first published paper on flood routing through reservoirs.

40. Graeff, 1875, Mémoire sur l'action de la digue de Pinay sur les crues de la Loire à Roanne [Memoir on the effect of Pinay Dam on the floods of the Loire at Roanne]: Acad. sci. [Paris] Mém., v. 21, p. 539–626.

The computation of the effect of a storage reservoir on flood waves is given. It is based on the general storage equation: inflow minus outflow for a given time unit is equal to the change of volume of stored water in the same time unit. The inflow and outflow hydrograph, the computational procedure, and other subjects (gaging of water, equations for uniform flow of water) are discussed. The propagation celerity of a flood wave is given as $C=(Q_1-Q_2)/(A_1-A_2)$, where Q_1 and Q_2 are the discharges in two nearby sections, and A_1 and A_2 are the corresponding cross-sectional areas.

41. Graeff, 1875, Sur l'application des courbes de débits à l'étude du regime des riviéres et au calcul de d'effet produit par un systeme multiple de réservoirs [On the application of the hydrograph to the river regime and to the computation of the effect produced by a system of multiple reservoirs]: Acad. sci. [Paris] Mém., v. 21, p. 627–674.

The propagation celerity C of a wave with $C=(Q_1-Q_2)/(A_1-A_2)$ is discussed. The influence of a reservoir on the downstream flood is analyzed, showing that reservoir influence decreases with distance from the reservoir. The effects of many reservoirs on the downstream flow is discussed.

42. Herschel, Clemens, 1875, On waves of translation: Am. Soc. Civil Engineers Trans., v. 4, p. 185–200.

For a given water elevation in an upstream body of water, and a given area of the bottom gate opening, the wave thus created in the downstream channel is discussed. Based on Bazin-Darcy experiments and formulas, together with author's experiments, the surge and celerity in the channel are discussed, and an expression is developed as the function of the water surface elevation in a body of water and for an area of opening.

43. Rayleigh, Lord, 1876, On waves: Philos. Mag. [London], ser. 5, v. 1, no. 4, p. 257–279.

Another method of approximation for solitary wave shape and propagation celerity is presented. By comparison with Boussinesq's paper (1871), this analysis produces an equation for surface shape that is similar to Boussinesq's and for velocity of propagation that is the same as Boussinesq's.

44. Russell, J. S. [Scott], 1876, On waves: Philos. Mag. and Jour. Sci. [London], sec. 5, v. 1, p. 257–279.

The celerity of long waves with small heights in any cross section with area A and width B at the surface is redeveloped as $C=\sqrt{gA/B}$. (Use of mean depth is introduced). The celerity of solitary wave is $C=\sqrt{gy}$, where y is the distance from crest of wave to bottom of channel. The equation for the shape of solitary wave is derived. Periodic waves in deep water are analyzed. The concept of the moving coordinate system with constant celerity of solitary wave is introduced.

45. Boussinesq, Joseph, 1877, Essai sur la théorie des eaux courantes [Treatise on the theory of flowing water]: Acad. sci. [Paris] Mém., v. 23, p. 261–529.

The third part (268 pages) constitues an extensive mathematical treatment of gradually varied unsteady flow. References are made to the experiments and observations of others. This treatise on fundamental theories of flowing water is of permanent value in this area of hydraulic investigations. Selected examples of the numerous analyses are given here; the second approximation formula for small-wave celerity in a frictionless horizontal rectangular canal is shown to be

$$C=V\pm\sqrt{gH_0}[1+3h/4H_0+(H_0^2/6h)d^2h/dx^2]$$

Center of gravity of a wave travels with celerity $C=\sqrt{g(H_0+3y)}$, where y is the distance from the center of gravity to the initial water level in channel; the wave energy $E=2Wy$ (where W=volume of water) does not change in a wave, when the friction is neglected; long-wave celerity is expressed as $C=\sqrt{gH_0}+(1+3h/2H_0)$; differential equations of wave with great curvature of water filaments are developed, with the celerity

$$C=1.032V+\sqrt{0.966g(H_0+h)+0.017V^2},$$

which approximates $C=V\pm\sqrt{g(H_0+h)}$; waves decrease in height by pro-

gressing, except in supercritical flow, where they can increase; the greater the velocity V of the permanent flow, the greater the decrease of maximum wave height by wave progression; increase of friction and canal slope decrease the celerity of all wave parts; V_{max}, Q_{max}, and H_{max} are rederived; by neglecting friction in a horizontal channel with changing cross section, the wave celerity is the same as in a rectangular section of the mean depth of changing section; the celerity of a given discharge is $C=V \pm dQ/dA$, which can be greater or lower than V. Some of the general problems treated are: (1) waves of small height in a channel that result in flow that differs little from the basic permanent flow; (2) the celerity of propagation of intumescence in waves of great curvature of surface; (3) the form, energy, moment of instability; and other aspects of the celerity of propagation of intumescence and on the mean velocity in a section; (4) the quasi-permanent regimen of floods in large rivers.

46. Boussinesq, Joseph, 1877, Additions et éclaircissements au mémoire intitulé: Essai sur la théorie des eaux courantes [Additions and clarifications for the memoir with the title: Treatise on the theory of flowing water]: Acad. Sci. [Paris] Mém., v. 24, s. 2, p. 51–58.

Some clarifications of the analyses of unsteady flow as published by Acad. Sci. (Paris) Mémoires, v. 23, p. 261–529 are presented.

47. Kleitz, ch., 1877, Sur la théorie du mouvement non-permanent des liquides et sur son application à la propagation des crues des rivières [On the theory of unsteady flow of liquids, and on its application to the propagation of river floods]: Ponts et des Chaussées Annales (France), sem. 2, no. 48, p. 133–196.

Unsteady flow is studied analytically with two basic differential equations developed. It is found that the maximum discharge propagates with celerity $C=(1/B)(dQ/dH)$, and any height of wave propagates with the celerity $C=V \pm \sqrt{g(H_0+y)(1-\psi)}$, where ψ depends on the resistance and is positive, so that, with the increase of friction, the celerity C decreases. Other forms of celerity formulas are derived and discussed, including the furmula for the celerity of small disturbance which is given as $C=V \pm [(H_0+h)\ \sqrt{g(H_0+h)}]/H_0$.

48. Raynolds, Osborne, 1877, On the rate of progression of groups of waves and the rate at which energy is transmitted by waves: Nature, [London and New York], v. 16, p. 343–344. Reprinted in Osborne Reynolds Papers on Mechanics and Phys. Subjects, v. 1, 1869–1882, p. 198–203, 1900.

In waves of deep water the rate at which the energy is carried forward is half the energy of disturbance per unit of length multiplied by the rate of propagation. If this is true, the author concludes, then the celerity of a group of waves will be half the propagation celerity of the individual waves. The results are modified in shallow water, but are strictly in accordance with the theory. The particles have elliptic paths in shallow water and wave energy is transmitted by wave propagation. When paths approach straight lines, all energy is transmitted, and a group of waves propagates with the same celerity as the solitary wave. Therefore, Russell's solitary wave theory is possible.

49. Stokes, G. G., 1880, On the theory of oscillatory waves (Appendix): Stokes's Math. and Philos. Papers [Cambridge], p. 219–229.

In this supplement to his paper of 1847, the author carries his analysis one step farther in developing an approximation of an infinite train of oscillatory waves.

50. Comoy, M., 1881, Étude pratique sur les marées fluviales et le mascaret [Practical study on the river tidal movement and the bore]: Paris.

This paper studies the different aspects of tides in rivers, their movement and the bores created.

51. Graeff, 1883, Traité d'hydraulique [Hydraulics, chapter on flood forecast]: Paris, p. 438–442.

The flood hydrograph from one station is routed to a downstream station (neglecting lateral inflows and friction resistance) by the use of the rating curve and the flood celerities, expressed as a function of stages H, or as duration curves of propagation celerity for a given reach.

52. Rippl, W., 1883, The capacity of storage-reservoirs for water-supply: Inst. Civil Eng. [London] Proc., v. 71, no. 1864, p. 270–278.

Mass curves of the discharge hydrograph are used in this study to determine the necessary storage capacity of reservoirs. This marks the introductory use of the mass curve for flood routing through reservoirs. The properties of mass curves are discussed.

53. Havestadt, A., 1884, Wasserbewegung im Flutgebiet [The water flow in tidal region]: Festschrift der Tech. Hochschule, Berlin [Germany].

54. Lechalas, M. C., 1884, Hydraulique fluviale [River hydraulics]: Annex, Chap. I, Théorie générale des réservoirs d'emmagasinement des crues [General theory of flood storage reservoirs]: Paris, Librairie Polytech., Annex, p. 434–460 (The Dayton-Morgan Engineering Company has a translation of this article, prepared by K. C. Grant; see Am. Soc. Civil Engineers Trans., v. 93, p. 741, 1929).

Comparison of reservoir inflow and outflow hydrographs is made, and flood modification by reservoirs is discussed. The conditions under which a reservoir is useful for flood control are studied. The question of the best distribution of flood control reservoirs in a river basin is approached. The author contends that there are no general rules that pertain to this matter. The flood having only one peak is considered. Noncontrolled and controlled outflows are discussed. The storage necessary for a given amount of flood reduction is analyzed. The Kleitz (1877) method and ideas are discussed.

55. Bazin, H., 1885, Expériences sur la propagation des ondes le long d'un cours d'eau torrentueux, et confirmation par ces expériences des formules données par Boussinesq, dans sa théorie du movement graduellement varié des fluides [Experiments on wave propagation in torrential flow and the confirmation of Boussinesq's equation for gradually varied flow of fluids]: Acad. sci. [Paris] Comptes rendus, v. 100, p. 1492–1494.

The celerity of the solitary wave, as derived analytically by Boussinesq, is

$$C=\frac{1+a_2}{2}V+\sqrt{\frac{gh_0}{1+2b}+\left[\left(\frac{1+a_2}{2}\right)^2-\frac{a_1}{1+2b}\right]V^2},$$

where a, a_2 and b are coefficients which depend on the nonuniformity of water velocities. Bazin gives the values a_1, a_2, b (different symbols are used by Boussinesq and Bazin) for three experiments. He writes $\lambda^2=V^2/gH_0$.

Bazin finds that $C=V\pm k\sqrt{gH_0}$, where k depends on b and λ. When $\lambda^2>0.202$, k is greater than unity.

56. De Saint-Venant, Barré, 1885, Mouvement des molécules de l'onde dite solitaire, propagée à la surface de l'eau d'un canal [The movement of molecules of a solitary wave propagated on the water surface of a canal]: Acad. sci. [Paris] Comptes rendus, v. 101, p. 1101–1105.

For a large rectangular channel with horizontal bottom, the vertical velocity in a wave movement is introduced, and the momentum equation takes the form

$$\frac{\partial V}{\partial t}+V\frac{\partial V}{\partial x}+g\frac{\partial z}{\partial x}+\frac{H_0}{3}\,\frac{\partial^3 x}{\partial t^2 \partial x}=0$$

in which z=wave depth (H_0+h), h=wave height. With the above equation the celerity of a solitary wave is

$$C^2=gH_0\left(1+\frac{3}{2}\,\frac{h}{H_0}+\frac{H_0^2}{3h}\,\frac{\partial^2 z}{\partial x^2}\right).$$

57. Kelvin, Lord (Thomson, William), 1886, On stationary waves in flowing water: Philos. Mag., v. 22, p. 353–357, 445–452, 517–530 and v. 23, p. 52–58.

Two dimensional problems of stationary waves in flowing water are treated. The permanent steadiness of the motion in this case is analyzed, with velocity $\sqrt{gH}$ discussed as the criterion. Part 4 treats the stationary waves on the surface that are produced by equidistant ridges on the bottom.

58. Klunzinger, P., 1886, Ueber die Beziehungen der Flussregulirungssysteme zu dem Verlaufe der Hochwaesser [On the effects of river regulation on the flood flow]: Zeitschr. Oesterreich Ingenieuren u. Architekten-Ver. [Austria], v. 38, p. 10–19.

This is an analytical treatment of the attenuation of flood waves by storage reduction of peak discharges. The schematic waves are introduced and the water volume at peak of hydrograph is determined by integration.

59. Kelvin, Lord (Thomson, William), 1887, On the waves produced by a single impulse in water of any depth, or in a dispersive medium: Royal Soc. [London] Proc., sec. A., v. 42, p. 80–85.

The relation between the wave celerity and the wave length of an endless procession of periodic waves is studied. The formulas for the celerities in deep water, in water with depth H, and for various types of waves are derived.

60. Kelvin, Lord (Thomson, William), 1887, Stationary waves on the surface produced by equidistant ridges on the bottom: Philos. Mag., v. 28, p. 52–87 or Math. and Phys. Papers, Cambridge Univ. Press, 1910, v. 84, p. 296–302.

A first-approximation study of waves due to ridges on the bottom that takes into account the effect of gravity is presented.

61. Basset, A B., 1888, A treatise on hydrodynamics, volume II, Chapter XVII, On liquid waves: Cambridge, Deighton Bell and Co., p. 144–187.

A general review of the theory of waves up to 1888 is given in a condensed form. Assumptions and derivations are discussed. Theories of the propagation celerity and wave shape for the long wave in shallow water, for the solitary wave, and for other types of waves are reviewed.

62. Ekdahl, O. Z., 1888, Om beräkningsmetoderna vid uppgörande af förslag till sjöars sänkning och reglering [About the methods of computation in a suggestion for lowering and regulating a lake]: Lund [Sweden].

The study uses the storage equation for hydrograph routing through reservoirs. The storage factors $(W/\Delta t)+(Q/2)$ and $(W/\Delta t)-(Q/2)$ are used in graphical and tabular form to facilitate the routing.

63. Flamant, A., 1889, Des ondes liquides non périodiques et, en particulier, de l'onde solitaire [About liquid nonperiodic waves, and particularly, about the solitary wave]: Ponts et Chaussées Annales [France], s. 6, v. 18, sec. 2, p. 5–48.

This is a simplified treatment of problems of solitary wave, discussed more extensively by Boussinesq in his treatise on the theory of flowing water (1877). Wave celerity is defined as the velocity of a vertical cross section having all the time the same volume of the wave before it. The Boussinesq formula for wave celerity is rederived on the same assumption. The center of gravity of the wave is derived, and its celerity is given in the second approximation as $\sqrt{g(H_0+3h/2)}$. When friction is neglected, wave energy stays constant during its propagation, inasmuch as the acceleration of gravity is constant. The solitary wave is the most stable wave, because the moment of instability (introduced by Boussinesq) is the smallest in the solitary wave.

64. Francis, J. B., 1889, The effect of rapidly increasing supply of water to a stream on the flow below the point of supply: Am. Soc. Civil Engineers Trans., v. 21, p. 558.

The conditions in which gradually varied waves will become a bore are discussed. The explanation is given that in a torrent river and in an estuary the waves often build up to become a bore, and gradually varied waves become progressively steeper up to the breaking over into a tidal bore. Dam-breach waves are mentioned. T. P. Frizell discusses all three phemomena: bores in estuaries, bores in mountain rivers, and bores due to dam-breaches.

65. Massau, Junius, 1889, L'intégration graphique [Graphical integration]: Assoc. Ingénieurs Sortis des Écoles Spéciales de Gand [Belgium] Annales, v. 12, p. 435.

The principles of a new method for graphical integration of partial differential equations of unsteady flow are given. For the arbitrary curves, along which the derivatives exist, the elements are determined one after another. The discontinuities propagate according to the characteristics of Monge, and for unsteady flow the characteristics represent the movement of two infinitely small waves, the wave from upstream and the wave from downstream. The wave becomes finite for a bore.

66. Massau, Junius, 1889, Appendice au Mémoire sur l'intégration graphique [Appendix to Memoir on graphical integration]: Assoc. des ingénieurs Sortis des Écoles Spéciales de Gand [Belgium] Annales, v. 12, p. 185–444.

Explanations are made and procedures given for the graphical method of integrating partial differential equations for unsteady flow by methods of characteristics.

67. Auria, L. d'., 1890, A new theory of the propagation of waves in liquids: Franklin Inst. Jour., v. 130, no. 6, p. 456–465.

Formulas derived for computing the delay between high and low waters. Discusses use of the formulas as applied to tidal rivers. Includes illustration based on the Delaware River.
[Abstract from Corps of Engineers, Bibliography on Tidal Hydraulics, Corps of Engineers, June, 1955.]

68. Mullins, J., 1890, Irrigation manual (section on reservoirs: The influence of reservoirs as flood regulators): Madras Gov. [India], p. 215–223.

For flood routing purposes, the finite-difference method is shown. The time interval, Δt, is used as an independent variable in the storage difference equation, $P-Q=AdH/dt$, where P=inflow, Q=outflow, A=area of reservoir surface, and H=height of reservoir surface. In computing the outflow, Q, occuring in a given time, Δt, values are assumed for ΔH, and by trial and error method the assumptions are corrected.

69. McCowan, J., 1891, On the solitary wave: Philos. Mag. [London], sec. 5, v. 32, p. 45–58.

The solitary wave can be included in the general theory of long waves only as a rough approximation, because its celerity does not agree closely with that of the long wave, nor does it gradually increase in steepness in front as the long wave does. The change of a solitary wave is simply a decrease in height and consequent increase in length, which are caused by a dissipation of energy due to friction. It is derived theoretically, to the degree of approximation involved, that the wave consists solely of an elevation and that there cannot be a wave of depression capable of propagating itself unchanged with constant velocity (Russell's experimental results). The celerity is obtained as $C^2=g(H_0+h_0)$, where h_0 is the function of wave heights. The theoretical approximate values of wavelength (where height is smaller than prescribed), volume of wave (is approximately $2\ aH_0$, where a=coefficient), energy of wave, limiting height of wave and breaking of wave (where $H_0=0.8\ \sqrt{W}$, where W=volume of the wave), are given. It is concluded that a solitary positive wave can propagate with constant celerity without change of shape.

70. Partiot, H. L., 1892, Étude sur les rivières à marée et sur les estuaires [Study on tidal rivers and estuaries]: Paris.

71. Ritter, A., 1892, Die Fortpflanzung der Wasserwellen [The propagation of water waves]: Ver. Deutsch Ingenieure Zeitschr. [Berlin], v. 36, pt. 2, no. 33, p. 947–954.

This study of wave propagation is based on the assumption of a horizontal prismatic canal, with the friction loss neglected and with uniform distribution of velocity in any cross section. If the water is suddenly accelerated to the velocity U (by forcing the water of by opening a gate), the celerity

$$C=\sqrt{gH_0}\ \sqrt{(1+h/H_0)(1+h/2H_0)}\ \text{ with }\ U=(h/H_0)\ \sqrt{gH_0}\ \sqrt{1-h/2(H_0+h)}.$$

Considering small h to be negligible, $C=\sqrt{gH_0}$ and $U=h\ \sqrt{g/H_0}$, or $C:U=H_0:h$. If the water has already the velocity V, it must be added to C or U. A continuous wavefront is analyzed as many finite discontinuous steps. The sudden removal of a gate in the canal shows the outflow velocity in that section to be $U=(2/3)\sqrt{gH_0}$, the depth, $y_0=(4/9)H_0$ and the constant discharge, $Q=8/27H_0\sqrt{gH_0}$. The wave profile has equation $y=H[(x+2\lambda)/$

$3\lambda]^2$ where x=distance from the section, y=wave depth, and $\lambda=t\sqrt{gH_0}$ (t=time). The application is made for dam breaches, with the upper curve $y=f(H_0, x, \lambda)$ as the limiting curve. At the beginning of outflow the point $y_0=4H_0/9$ is a turning point where the first surface lines of the outflow wave appear. From a shape of concave upward it passes to a shape of convex upward, and the turning point shifts upstream.

72. Wheeler, W. H., 1893, Tidal rivers; Their hydraulics, improvement, navigation: London and New York, Longmans, Green & Co., p. 1–467.

Chapter 1: The development of hydraulic science history, theories, formulas; tides, waves, and currents. Chapter 3: The motion of water in tidal rivers: details of flow, friction, disturbance, turbulence; directions and mixing of fresh and salt water; formulas. Chapter 5: The tides: terms, theories, tidal waves; Southern Ocean wave, ocean tidal wave, the coast tide, estuary tide, river tide, bores. Chapter 7: Bars at the mouths of tidal rivers, and littoral drift: theories; examples of bars in different locations and in different materials.

73. McCowan, J., 1894, On the highest wave of permanent type: Philos. Mag. [London], sec. 5, v. 38, p. 351–358.

This paper, a supplement of the author's paper in 1891 on the solitary wave, investigates an approximation better adapted to the discussion of the extreme case of the wave at the breaking height.

74. Karteweg, D. J., and DeVries, G., 1895, On the change of form of long waves advancing in a rectangular canal, and on a new type of long stationary waves: Philos. Mag. [London], sec. 5, v. 39, p. 422–443.

Lamb and Basset state that long waves in a rectangular canal necessarily change their shape as they advance, becoming steeper in front and less steep behind, even when friction is neglected. Investigations on solitary waves by Boussinesq, Lord Rayleigh, and De Saint-Venant have cast doubt on this theory. The authors define the cnoidal waves (so called in in analogy with the sinusoidal waves), under conditions that produce the solitary wave as a special case. In a frictionless liquid there may exist absolutely stationary waves. The form of their surface and the motion of the liquid below these waves may be expressed by a rapidly convergent wave series. The positive solitary wave, occurring as a stationary wave, has the known shape of sechyperbolic function. The stationary periodic waves (cnoidal waves) are analyzed. The deformation of nonstationary waves is discussed. It is concluded that a solitary wave that is steeper than a stationary one, but has the same wave height, becomes less steep in front and steeper behind; the opposite occurs when the solitary wave is less steep than the stationary wave.

75. Polak, Ignatz, 1895, Die See-Retention. Graphische Darstellung derselben nach Prof. A. R. Harlacher [Lake-retentions. Their graphical representation by the method of Prof. A. R. Harlacker]: Zeitschr. der Oesterreichischen Ingenieur u. Architekien-Ver. [Austria], v. 47, no. 50, p. 593–597.

Inflow and outflow hydrographs are used for the solution of different reservoir problems involving the use of storage differential equation. Graphical shortcuts are used for integration of the hydrographs for given time intervals.

76. Klunzinger, P., 1896, Weitere Studien ueber den Verlauf der Hochwasser [Further studies on flood-wave flow]: Zeitschr. der Oesterreichischen Ingenieur u. Architekten-Ver. [Austria], v. 48, no. 4, p. 33–39; no. 5, p. 49–56.

The flood wave is approximated by a mathematical function. Using this function, the influence of storage on the flood transformation is derived analytically.

77. Boussinesq, Joseph, 1897, Théorie de l'écoulement tourbillonnant et tumultueux des liquides dans les lits rectilignes à grande section [Theory of turbulent motion of liquids in rectangular channels with large section]: Paris, Gautheir-Villards et Fils, p. 22–27.

The momentum partial differential equation is given in somewhat different form, taking into consideration the velocity distributions in cross sections. The use of this equation for wave propagation is demonstrated. The celerity of waves is given in a more exact form, as

$$C=V\pm\sqrt{gH_0}+\frac{\alpha-1}{2}V\left(3\pm\frac{V}{\sqrt{gH_0}}\right)-\eta V\left(\frac{5}{2}\pm\frac{V}{\sqrt{gH_0}}\pm\frac{\sqrt{gH_0}}{V}\right),$$

where

$$1+\eta=\int\left(\frac{v}{V}\right)^2\frac{dA}{A},\text{ and }\alpha\int\left(\frac{v}{V}\right)^3\frac{dA}{A},$$

where v is the velocity at any point, and V the mean velocity.

78. Fantoli, G., 1897, Sul regime idraulico dei laghi [On the hydraulic regime of lakes]: Ulrico Hoepli, Milano [Italy], p. 1–349.

An analytical study of lake inflow and outflow for the case when lake-level fluctuation is a sine function and outflow is a linear function of lake levels.

79. Bobylev, D., 1898, Ocherk teorii vodyanikh techeniy, vyrabo tannyy Bussineskom [Sketch of the theory on water movements, developed by Boussinesq]: SPC. [U.S.S.R.].

A summary of Boussinesq's works.

80. Bourdelles, 1898, Étude du régime de la marée dans le Canal de Suez [Study of tidal regime in the Suez Canal]: Ponts et Chaussées Annales [France] trim. 3.

The propagation of tidal waves in the Suez Canal is studied.

81. Lévy, Maurice, 1898, Leçons sur la théorie des marées [Lessons on the theory of tides]: Paris, Gauthier-Villars.

The theory of unsteady movement given by De Saint-Venant is verified. The tide in canals of both constant and variable width is treated, but with the additional consideration of the friction exerted upon the tidal wave. The height of wave decreases during its propagation with coefficient of extinction. The propagation celerity is the function of heights of tide and wave depth at half tide. The maximum current of water occurs before the highest and lowest levels of tide wave and decreases with an increase of the distance from the canal entrance. Taking the friction into account, this theory was closer to observations than Airy's or De Saint-venant's. In an attempt to apply his derived equations to the Suez Canal, the author found that the computed celerities were far less than the actual, and the attempt to use the equations for second approximation still did not give results that agreed completely with observations.

82. Roudzki, 1898, Ueber eine Classe hydrodynamischer Probleme mit besonderen Grenzbedingungen [On a class of hydrodynamic problems with special boundary conditions]: Math. Annalen [Germany], v. 50, p. 269.

The author considers that it is not known whether pressure is equally distributed on the water surface for any given surface shape.

83. Seddon, J. A., 1898, A mathematical analysis of the influence of reservoirs upon streamflow: Am. Soc. Civil Engineers Trans., v. 50, p. 401–427

In order to study the influence of storage on the hydrograph, the storage differential equation (inflow minus outflow is the change of storage in a given time increment) is integrated for a cycle inflow function (a sine hydrograph) and for an outflow rating curve, which is linear in respect to reservoir level. The function $H=f(t)$, or reservoir level hydrograph is determined, where the area of the reservoir is either constant or changing with the reservoir level, and where H is the reservoir level and t, the time. The effect of storage on the amplitude of the crest flow and on phase shifting (the postponement of maximum Q) is derived analytically for one or more reservoirs.

84. Bourdelles, 1899, Étude du régime de la marée de La Manche [Study of the tidal regime in the English Channel]: Ponts et Chaussées Annales [France], trim. 3.

The propagation of tidal waves in the English Channel is studied.

85. Haerens, E., 1899, Mouvement varié des eaux; propagation d'une crue et d'une marée fluviale [Unsteady flow; propagation of a flood and of a river tide]: Travaux Publiques de Belgique [Bruxelles], Annales s. 2, v. 4, p. 1–19.

The momentum and continuity equations for unsteady flow are derived and shown in finite difference form in the equation of unsteady flow, for cross sections 0 and 1, that are Δx distance apart, and for time difference of Δt:

$$\Delta z=\frac{V_1^2-V_0^2}{2g}+\frac{1}{g}\left(\frac{\Delta V_0}{\Delta t}+\frac{\Delta V_1}{\Delta t}\right)\frac{\Delta x}{2}+b\left(\frac{P_0}{A_0}V_0^2+\frac{P_1}{A_1}V_1^2\right)\frac{\Delta x}{2},$$

and

$$(\Delta A_0+\Delta A_1)\ \Delta x=[2(Q_0-Q_1)+(\Delta Q_0-\Delta Q_1)]\ \Delta t$$

where Δz is the drop of surface level for Δx and Δt, b is a coefficient; V_0 and V_1, Q_0 and Q_1, A_0 and A_1, P_0 and P_1, are velocities, discharges, areas and wetted perimeters of two cross sections and of two times respectively. This is a trial-and-error method with finite differences. Two examples are given: (1) for a given discharge hydrograph, the hydrographs and wave profiles are computed downstream; (2) for a given tide shape at river confluence the waves upstream are determined.

86. Bourdelles, 1900, Étude sur le regime de la marée dans les estuaires et dans les fleuves [Study of tidal régime in estuaries and in rivers]: Ponts et Chaussées Annales [France], trim. 2.

A dissertation on the theory of the propagation of translation waves in

canals, with the purpose of studying how closely the observed facts confirm the theory, is given.

87. Flamant, A., 1900, Hydraulique, Chap. 8, Mouvements ondulatoires [Hydraulics, Chap. 8, Undular movements]: Paris, p. 413–508.

This extensive treatise on waves includes: translation waves (definition, celerity, celerity in function of wave shape, energy, and application); solitary waves (definition and equation, energy and stability, long intumescences, and wave propagation on moving liquid); sea waves; and tides. It is a comprehensive summary of the existent knowledge of waves at the turn of the century.

88. Gwyther, R. F., 1900, The classes of progressive long waves: Philos. Mag. [London, Edinburgh, Dublin], ser. 5, v. 1, p. 213–216, 308–312.

Starting from Lord Rayleigh's paper (1900) "On waves," the first approximation of the solution for long waves is obtained, and special cases are discussed, including the case of the solitary wave, which is emphasized in the discussion contained in the "Appendix". The study is analytical.

89. Gwyther, R. F., 1900, The general motion of long waves, with an examination of the direct reflexion of the solitary wave: Philos. Mag. [London, Edinburgh, Dublin], ser. 5, v. 1, p. 349–352.

The long waves are studied analytically, giving a first approximation in the most general manner. The steady motion of long waves is dealt with by finding the differential equation which the velocity potential must approximately satisfy.

90. Massau, Junius, 1900, Mémoire sur l'intégration graphique des équations aux derivées parcielles [Graphical integration of partial differential equations]: Assoc. des Ingénieurs Sortis des Écoles Spéciales de Gand [Belgium] Annales, v. 23, p.95–214.

This work is a fundamental treatise on the method of characteristics, used as a graphical method for integrating the two partial differential equations for unsteady flow in channels, through four ordinary differential equations, called "characteristic equations," and equivalent to the two partial differential equations of unsteady flow. The basic approach is first explained, then such problems are treated as: integration by elements following a variable; graphical integration by characteristics, covering the linear equations of first order, linear equations of second order, any equation of second order, and simultaneous linear equations; unsteady flow with the hydraulic jump; celerity of wave, with characteristics and their use; the propagation of finite wave; and the formation of finite wave in the continuous wave (tidal bore). (Translated into English by Putman, Rocky Mountain Hydraulic Laboratory.)

91. Seddon, A. J., 1900, River hydraulics: Am. Soc. Civil Engineers Trans., v. 43, p. 179–243.

The celerity of a flood wave on a river is developed as: $C=(1/B)$ dQ/dH, where C=celerity, B=width of the river, and dQ/dH slope of the rating curve at the place where Q, H, B and C are related.

92. Maillet, Edmond, 1901, Sur les graphiques et les formules d'annonces des crues [On the graphs and formulas for flood forecast]: École Polytech. [Paris] Jour., ser. 2, no. 6, p. 148.

Various procedures and formulas are given for purposes of flood forecasting of stage-discharge relationships at two or three stations in the river system.

93. Partiot, H. L., 1901, Recherches sur les rivières à marée [Research on tidal rivers]: Paris, 33 p.

94. Forchheimer, Ph., 1903, Wasserbewegung in Wanderwellen [The motion of water in travelling waves]: Akad. Wiss. [Vienna] Minutes Proc., sec. 2–a, p. 1697–1720.

In this study of wave trains on a steep sloping canal the hypothesis is advanced that the discharge is uniformly constant, as shown by the equation

$$H(C-V)=H_1(C-V_1)=H_2(C-V_2).$$

The celerity is computed as

$$C(H_1-H_2)=cS^{1/2}+(H_1^{3/2}-H_2^{3/2})$$

where c=coefficient in Chezy formula, and S=slope of the canal. It is developed that $C=3Q/2H=3V/2$. For other formulas of steady flow the celerity C is also derived. Wave trains are briefly discussed and analyzed.

95. Maillet, Edmond, 1903, Sur les lois des montées de Belgrand et les formules du debit d'un cours d'eau [On Belgrand's laws of increase and the discharge formulas of a water course]: École Polytech. [Paris] Jour., ser. 2, no. 8, p. 1.

For stage-forecast purposes, stage relationship (based on the equation $Q_1=Q_2+Q_3$) is developed between three stations, two on tributaries upstream from their confluence and the third below the confluence. The relationship is developed in the form of three rating curves: $f_1(H_1)=f_2(H_2)+f_3(H_3)$, with H_1, H_2, H_3 as the corresponding stages. The law of stage increases is developed according to Belgrand's analysis. (See abstract of Maillet's paper of 1922.)

96. Chrystal, 1904, Some results in the mathematical theory of seiches: Royal Soc. [Edinburgh] Proc., v. 25, p. 328.

Progressive waves in a canal with varying sections are studied, and solvable cases for different cross sections are investigated.

97. Forchheimer, Philipp, 1904, Wasserbewegung und Wanderwellen [Water movement and translatory waves]: Gewaesserkunde Zeitschr., v. 6, 6, p. 321–339.

The propagation celerity of a monoclinal wave with peak height H_1 (and V_1, or Q_1) and with lower height H_2 (and V_2, or Q_2) is $C=(Q_1-Q_2)/B(H_1-H_2)$, or $C=(1/B)\,dQ/dH$, the same celerity as that derived by Graef or Seddon. If H_2 is a maximum, then $B(H_1-H_2)$ depends on H_2. If V_2/V_1 is between zero and unity, then it is found theoretically that C/V_1 is between unity and 1.5. However, by the experiments C/V_1 is shown to be between 1.7 and 2.0.

98. Massau, Junius, 1905, L'intégration graphique des équations aux dérivées parcielles [Graphical integration of partial differential equations]: Assoc. Ingénieurs Sortis des Écoles Spéciales de Gand [Belgium] Annales, v. 4, p. 65.

This is a complete development of the theory of the method of characteristics, which method is suitable for graphical integration of partial differential equations employed as the equations for unsteady flow when the boundary conditions have discontinuity.

99. Cornish, Vaughan, 1907, Progressive and stationary waves in rivers: Engineer [London], v. 84, p. 118, 284, 347.

The paper describes roll waves (also called freshets and spates) on the rivers Tees and Ure (England), the Nile (Egypt), and the Nikko (Japan). It is stated that uniformity of depth in the cross section is essential for the persistence of such a wave. If the channel consists of a deep central groove with shelving banks, the waves will change direction and lose energy by breaking on the shores. A river consisting of alternate pools and shallows is favorable to the formation of roll waves. Small wedge-shaped solitary waves flowing down a precipitous cliff at Grindelwald are described; also waves in a 14-inch wide conduit having a slope of 1:2 at Territet (Lake of Geneva). Here the roll waves existed for low flows but not for large flows. Roll waves in steep open channels at Gruenbach and at Guntenbach are are also described. Other types of waves in rivers and estuaries are discussed.

[Abstract by Herbert N. Eaton, Am. geophys. Union Trans., pt. 4, 1938.]

100. Levi-Cività, Tullio, 1907, Sulle onde progressive di tipo permanente [On the progressive wave of the permanent type]: Reale Accad. Lincei [Rome] Atti., v. 16, p. 777–790.

The irrotational wave motion of an incompressible liquid in a prismatic canal with horizontal bottom is called permanent, if the surface seems to be progressing without change and with a constant celerity, or if the motion seems to be stationary, as observed by an observer moving at the rate of wave celerity. The motion will be permanent if the discharge observed by the moving observer is not affected by the time of observation.

101. Grunsky, C. E., 1908, Discussion on "The flood of March 1907 in the Sacramento and San Joaquin River Basins, California" (by W. B. Clapp): Am. Soc. Civil Engineers Trans., v. 61, p. 331–345.

This is a general discussion of wave propagation and other flood phenomena. Mass curves are used as a graphical device for determining the maximum discharge and the flood-wave shape at the downstream point, using the upstream hydrograph. The continuity or the storage equation is used.

102. Maillet, Edmond, 1908, Sur les services hydrométriques et d'annonces des crues en France [On the services of hydrometry and flood forecast in France]: Navigation Con., 11th Saint Petersburg [Leningrad], 1908.

A general description of forecasting methods, which this paper classified as being: methods of partial or total increase of stages or of discharges; methods of comparison of stages or discharges; or a combination of these methods.

103. Flamant, 1909, Hydraulique [Hydraulics]: 3d Ed., Paris, p. 386.

Waves are studied theoretically. Expressions are given for waves produced by moving a vertical plate horizontally in a rectangular channel.

104. Kabelač, Karl, 1909, Ueber ein neues Verfahren zur graphischen Loesung der See-Retentions-Aufgabe [On a new procedure for graphical solution of storage-retention problem]: Zeitschr. der Oesterreichischen Ingenieur u. Architekten-Ver. [Austria], v. 61, no. 22, p. 353–355.

Starting with the simple storage equation, a graphical procedure is derived for the step-by-step integration. The use of the slope for $tg\alpha = dH/dt = (P-Q)/A$ as the development procedure is the characteristic of this method.

105. Cornish, Vaughan, 1910, Waves of the sea and other water waves: Chicago, Open Court Pub. Co. [or T. Fisher Onwin, London, 1910], p. 367.

This is a descriptive book on waves; it contains no analytical derivations. Its three parts consider: (1) the size and celerity of deep-sea waves; (2) the action of sea waves to transport shingle, sand, and mud; and (3) stationary and progressive waves in rivers.

106. Cisotti, Umberto, 1910–11, Integrale generale dei piccoli moti ondosi di tipo permanente in canali molto profondi [The general integral solution for small wave movements of permanent type in very deep channels]: Atti. del Reale Ist. Veneto [Italy], v. 70, p. 33–47.

Using the velocity potential, the function of flow, and the complex variables, the unsteady flow differential equations are integrated, with a general solution in the approximate form. The equations for wave heights and for wave celerity are developed.

107. Rayleigh, Lord, 1911, Hydrodynamical notes: Philos. Mag. [London], ser. 6, v. 21, p. 177–195.

Brief mathematical notes are presented on the following topics: potential and kinetic energies of wave-motion; waves moving into shallower water; concentrated initial disturbance with inclusion of capillarity; periodic waves in deep water advancing without change of type; tide races; rotational fluid motion in a corner; steady motion in a corner of viscous fluid. In each analysis the fluid is regarded as incompressible and the motion is assumed to take place in two dimensions. When there is no dispersion, the energy of a progressive wave of any form is half potential and half kinetic. Thus, in the case of a long wave in shallow water "if we suppose that initially the surface is displaced such that the particles have no velocity, we shall evidently obtain (as in the case of sound) two equal waves travelling in opposite directions whose total energies are equal, and together make up the potential energy of the original displacement. Now the elevation of the derived waves on behalf of that of the original displacement and, accordingly, the potential energy, is less than the ratio of 4:1. Since, therefore, the potential energy of each derived wave is one-quarter and the total energy one-half that of the original displacement, it follows that in the derived wave, the potential and kinetic energies are equal." [Abstract by F. T. Mavis, Am. Geophys. Union Trans. pt. 4, 1938.]

108. Bakhmetev, B. A., 1912, O neravnomernom dvizhenii zhidkosti v otkrytom rusle [On the unsteady movement of liquids in open channels): Chap. 5, [Russia].

The tabular integration of the storage equation, given as $dt/dH = A/(P-Q)$, for routing floods through reservoirs and channels.

109. Cisotti, Umberto, 1912, Sull'intumescenza del pelo libero nei canali a fondo accidentato [On the intumescence of free water surface in canals with an uneven bottom (a sill on the bottom)]: Rend. della Reale Accad. dei Lincei [Italy], v. 21, no. 8, p. 588–593.

A sill of height p in a horizontal canal is a resistance that creates an intumescence. The height of wave thus created as $\pi V^2/8gH^2$, and the

profile of a wave is given as $y=H+(\pi V^2/8gH)$ sech $(\pi x/2H)$ which is the profile of the solitary wave. The flow remains steady, but the standing wave that is created has the shape of a solitary wave. The parametric equations of the free surface, the heights of wave in exact form, the height of the sill (great or small) are developed or discussed.

110. Cisotti, Umberto, 1912, Sulle onde superficiali dovute a particolare conformazione del fondo [On the surface wave due to a particular configuration of the bottom]: Rend. della Reale Accad. dei Lincei [Italy], v. 21, p. 704–708.

Wave motion in a canal with transversal sills (ridges) of equal height and distance is studied. The problem is analyzed rigorously (referring to the approximate approach by Lord Kelvin, 1887), for the conditions in which it is legitimate to neglect the effect of gravity.

111. Cisotti, Umberto, 1912, Onde brevi causate da accidentallità periodiche del fondo [The short waves caused by equidistant ridges on the bottom]: Rend. della Reale Accad. dei Lincei [Italy], v. 21, p. 760–764.

This is a continuation study (author's paper 1912, p. 704–708) of the problem of surface standing waves in a steady flow over a bottom with equidistant ridges (vertical sills). Gravity can be neglected if V is of the order of fall velocity of height $H/20$. A new approximation is introduced, with results showing that the free surface has the profile of Stokes's waves of second approximation.

112. Ekdahl, O. Z., 1912, Ueber die Bewegung des Wassers in Kanaelen und natuerlichen Wasserlaeufen, und ueber die Wasserverhaeltnisse in Seen [On the movement of water in canals and channels, and on the water relationships in lakes]: Leipzig [Germany], Wilhelm Engelmann, p. 1–195.

The storage equation is used for flood routing through storage reservoirs. Two storage factors, $(W/\Delta t+Q/2)$ and $(W/\Delta t-Q/2)$, introduced in tabular and graphical forms, are used for flood routing and for the determination of necessary reservoir storage. This author is the first to use the storage factors in the above form.

113. Jacob, C. C., 1912, Computing the size of a reservoir spillway: Eng. News-Rec., v. 67, p. 1134.

The mass-curve method of flood routing is discussed in contrast to the numerical-integration method using the inflow hydrograph, as discussed by Palmer (1912).

114. Levi-Cività, Tullio, 1912, Sulle onde di canale [On the canal waves]: Rend. della Reale Accad. dei Lincei [Italy], v. 21, p. 3–14.

A theoretical study of wave motion that develops the concept of absolute and relative discharge. Complex variables are introduced. The total flow is integrated for a given time interval. The Stokes-Rayleigh's theorem is generalized, and a general equation between the global elements is developed.

115. Palmer, W. S., 1912, Methods of computing the size of spillway of a dam: Eng. News-Rec., v. 67, p. 524–525.

The numerical integration of the differential equation for obtaining the linear increase of the area of reservoir surface, with level $A=a+bH$, and for a spillway with the rating curve $Q=c\mathrm{LH}^{3/2}$, where H=height

above spillway crest, L=spillway length and c=coefficient is shown. The spillway effect upon the hydrograph and the storage-spillway discharges are discussed.

116. Cisotti, Umberto, 1913, Intumescenze e depressioni che dislivelli del letto determinano in un canale scoverto [The intumescences and depressions which the drops in channel levels determine in a free surface channel]: Rend. della Reale Accad. dei Lincei [Italy], v. 22, p. 417–422.

The shape of a stationary wave in steady flow over a sill in channel bottom is studied analytically on the same basis as in the author's previous paper (1912, p. 588–593). The sill is not vertical, in this study, but is a ridge of triangular form, either positive (above the bottom) or negative below the bottom). The equations for shape of positive or negative standing waves are developed.

117. Cisotti, Umberto, 1913, Sulle onde semplici di tipo permanente e rotazionale [On the simple waves of permanent and rotational type]: Rend. del Reale Inst. [Italy], v. 46, p. 917–925. Reprinted in "Nuovo Cemento," [Italy], v. 7, p. 251–259, 1914.

The simple progressive waves having the sinusoidal profile of m re generalized form than Airy's waves are studied analytically.

118. Mueller, R., 1913, Ermittelung der Wirkung eines Ueberfalles bei einem Stauweiher [The determination of the spillway effect in a reservoir]: Allg. Bauzeitung [Austria], v. 78, p. 59.

Mass curves are used as the flood-routing procedure for determination of spillway effect on flood waves in a reservoir.

119. Fuller, W. E., 1914, Flood flows: Am. Soc. Civil Engineers Trans., v. 77, p. 564–694.

In his analysis of flood frequency and development of a formula for computing the frequency interval (return period) the author demonstrates, for some simple hydrographs, that the influence of river basin storage upon flood attenuation is a function of the percentage of storage to total flood volume.

120. Karlson, 1914, Svallninger i oeppna kanaler [Waves in open channels]: Tekniske Tidskraft [Sweden], p. 137.

121. Koženy, J., 1914, Ueber den Hochwasserverlauf in Fluessen und das Retentionsproblem [On the passage of flood wave in rivers, and the problem of storage retention]: Oestereich, Ingenieur u. Architeklen Zeitschr., [Vienna], v. 61, p. 261.

122. Massau, Junius, 1914, Mémoire sur l'intégration graphique des équations aux dérivées partielles [Memoir on graphical integration of partial differential equations]: Ed. du Centenaire par les soins du Comité Nat. de Mécanique, Mons [Belgium].

123. Mises, R. von, 1914, Elemente der technischen Hydromechanik [The elements of technical hydromechanics]: Leipzig.

General outlines of a method for the study of gradual damping of translation waves under the effect of roughness are given in this book.

124. Rayleigh, Lord, 1914, On the theory of long waves and bores: Royal Soc. [London] Proc., ser. A, v. 90, p. 324–328.

A long wave of finite height can propagate in still water only with change of shape inasmuch as it is impossible to satisfy the requirement of a free surface for a stationary long wave, if h is the height of wave above the original level, unless h^2 can be neglected. The force acting upon a free surface is determined, and conditions that must exist in order for a wave to become stationary are analyzed.

125. Running, T. R., 1914, Filling and emptying of reservoirs: Eng. News-Rec., v. 69, no. 3, p. 67–68.

Filling and emptying of nonprismatic reservoirs are studied by using the storage equation and the graphical method of its integration [$P-Q=AdH/dt$]. Since $P=f(t)$, $Q=f(H)$, $A=f(H)$, then $dH/dt=f(H, t)$. The family of curves of $dH/dt=f(t)$ for constant H can be plotted, when P, Q and A are known. Using the starting point B_0 (H_0, t_0), and selecting the point B_1 on the line H_1, so that the area under B_0B_1 is (H_1-H_0), the point t_1 is obtained, and so on. The function $H=f(t)$ is thus obtained.

126. Wedderburn, J., 1914, On long waves: Am. Jour. Math., p. 211.

Progressive waves in a canal with slowly varying cross section are studied and solvable cases are given, in which the elevation of water is inversely proportional to the square root of width and to the fourth root of the average depth of cross section.

127. Cisotti, Umberto, 1914–15, Nuovi tipi de onde periodiche permanenti e rotazionali; Note Ia and IIa [The new types of periodical permanent and rotational waves, Notes Ia and IIa]: Rend. della Reale Accad. dei Lincei [Italy], v. 23, p. 556–561 and v. 24, p. 129–133.

This paper is an analytical treatment of special waves which become periodic waves.

128. Bakhmetev, B. A., 1915, Vvedenie v izuchenie neustanovivshegosya dvizheniya [Introduction to the study of unsteady flow]: Leningrad.

129. Palatini, Attilio, 1915, Sulla influenza del fondo nella propagazione delle onde dovute a perturbazioni locali [On the influence of bottom on the propagation of wave caused by local disturbances]: Rend. del Circ. Matematico di Palermo [Italy], v. 39, sem. 1, p. 362–384.

The case when channel depth is not small, but still has influence on the wave propagation, is studied analytically. Taking into account the boundary conditions of the water-free surface and of the bottom, the integration of wave equations is reduced to a unique partial differential equation of fourth order. The equation of free surface of water is expressed for each time of motion for a relative depth by a converging series. By further analytical treatment the influence of bottom is represented.

130. Palatini, Attilio, 1915, Sulla influenza del fondo nella propagazione delle onde dovute a perturbazioni locali; Studio asintotico del pelo libero [On the influence of bottom on the propagation of wave caused by local disturbances; the asymptotic study of free water surface]: Rend. del Circ. Matematico di Palermo [Italy], v. 40, sem. 2, p. 169–184.

The Rieman method is used for integrating the influence of depth on wave movement, in order to determine the asymptotic value of an integral. The asymptotic study of the free-water surface is developed.

131. Fry, S. A., 1916, A graphical solution of the problem of storm flow through a reservoir: Eng. and Contracting, v. 45, p. 370–372.

On the basis of the storage equation (inflow minus outflow equals the stored water) and using the records of the change of stage at the upstream gage to determine relationships of inflow, stage and time, graphs are developed to show time, in hours, required to fill the reservoir to any given elevation.

132. Jones, B. E., 1916, A method for correcting river discharge for a changing stage: U.S. Geol. Survey, Water-Supply Paper 375–E, p. 117–130.

For unsteady flow in a river, the ratio of two discharges Q_1 and Q_2 at a river gaging station for the slopes S_1 and S_2 is

$$Q_1/Q_2=\sqrt{1/[1+\Delta H/S_1 V\Delta t]}$$

where $\Delta H/\Delta t$ is the rate of change of stage and V is the surface velocity.

133. Alibrandi, M. P., 1917, Sur la théorie des ondes de crue [On the theory of flood waves]: Annali d'Ingegneria e d'Architettura (Italy), v. 32, no. 10 and 12, and a large summary by Goupil, Ponts et Chaussées Annales [France], 1917, t. 40, v. 5, no. 29, p. 200–210.

The analytical treatment of wave propagation gives $C=V+3\sqrt{g(H_0+y)}-2\sqrt{gH_0}$, and for $y=o$ (beginning and end of wave), $C_0=V+\sqrt{gH_0}$, while for inside values, $y=h$, the celerity is $C_h-C_0=\pm 3\sqrt{gH_0}(\sqrt{1+h/H_0}-1)$. The author compares this formula with Boussinesq's. By integration of partial differential equations for flood waves, it is concluded that the deformation of flood waves in rivers is more complex than that of the wave in prismatic channels without lateral inflow. The method followed in derivation is a combination for each variable of the value for steady flow and a changing value for unsteady flow neglecting the second- and third-order magnitudes of changing values.

134. Johnson, R. D., 1917, Surges in an open channel: Am. Soc. Civil Engineers Trans., v. 81, p. 112–115.

The surge in a resistance-free rectangular channel is compared to the hydraulic jump, and two corresponding equations are found. Discussion of the author's conclusions and of earlier studies is provided by Church.

135. The Miami Valley Flood-Protection Work, 1917, Study of retarding-basin operation (the fourth of a series of five staff articles): Eng. News-Rec., v. 77, p. 186–188.

A short description of the flood-routing method using the storage equation is described. A demonstration is given of the trial-and-error method, which employs numerical integration by tabulation of factors (inflow, outflow, storage, water height, and so forth), that are enumerated, with reference to time, and are integrated for successive time intervals.

136. Schoklitsch, Armin, 1917, Ueber Dammbruchwellen [On waves created by dam breaches]: Akad. Wiss. [Vienna] Proc., v. 126, pt. 2A, p. 1489–1514.

The works of Ritter (1892) and Forchheimer (1914) are used as the starting basis for these experiments. Darcy's and Bazin's experimental data (1865) are used, as well as Zeitlinger's (1914) in small wooden channels for positive surges, with $C=\sqrt{gH}$ verified (for $H=3.7$ cm, C measured was 91 percent of computed). The author's tests in a small laboratory channel and in an open ditch have shown: (1) the celerity is modified as the formula $C=\sqrt{g(H+kQ/V}$, where Q is the discharge creating the surge, V=velocity

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due to Q and $k=1.5$ for smooth wavefront, and $k=1.0$ for a breaking wavefront (many secondary waves); (2) the turning (rotation) point of wave surface at a sudden gate removal is $H/2$ and not $4h/9$, as Ritter found by an analytical treatment; (3) the emptying of a reservoir (L-long, H-deep) is (6 to 10)$+L/\sqrt{gH}$ seconds, if a side is suddenly removed. In the case of a sudden opening in the profile being smaller than the cross section, the height is given for the wave depth.

137. Thomas, H. A., 1917, Flood-retarding reservoir problem directly solved: Eng. News-Rec., v. 79, no. 5, p. 226.

The storage equation $P-Q=dW/dt$ is solved by mass curves. Given: Mass-curve of P, storage $W=f(Q)$ computed from $W=f(H)$ and $Q=f(H)$. An interval Δt is selected as fixed, inside of which all changes are considered linear. On the storage W for each outflow Q, $\frac{1}{2}Q\Delta t$ is added and a new curve $W+\frac{1}{2}Q\Delta t=f(Q)$ is plotted. If the accumulated outflow is known for t_1, and Δt added, in the middle of Δt the value of $\Sigma P_i\Delta t$ is determined;

$$\Sigma \mathrm{P}_i\Delta t-\Sigma Q_i\Delta t=W+\tfrac{1}{2}Q\Delta t,$$

and with this value from the curve $W+\frac{1}{2}Q\Delta t=f(Q)$ the mean value of Q in the interval Δt is obtained. The outflow during the interval is $Q\Delta t$, and the new point of $\Sigma Q\Delta t$ becomes $\Sigma Q_i\Delta t+Q\Delta t$. In this manner, the integration is carried out.

138. Cisotti, Umbero, 1918, Equazione caratteristica dei piccoli moti ondosi in un canale di qualunque profondità; Note Ia e IIa [The characteristic equations for small wave movements in a channel of any depth; Notes Ia and IIa]: Rend. della Reale Accad. dei Lincei [Italy], v. 27, p. 255–257, 312–316.

It is shown that the motion of small waves in a canal of finite depth can be represented by a unique function which is the characteristic equation of the wave motion. The second note shows how use of this characteristic equation produces a more feasible expression that eliminates the use of finite differences.

139. Cisotti, Umberto, 1918, Sulle onde superficiali progressive di tipo permanente [Progressive surface waves of permanent type]: Rend. del Reale Inst. Lombardo [Italy], v. 51, p. 85–94.

This is a study of the wave that propagates without altering its form and thereby appears to the observer, moving with the celerity C, to be steady flow.

140. Feifel, Eugen, 1918, Ueber die veraenderliche nicht-stationaere Stroemung in offenen Gerinnen, insbesondere ueber Schwingungen in Turbinen-Triebskanaelen [On unsteady flow in open channels, expecially on the fluctuations in the canals of powerplants]: Forschungsarbeiten, Ver. Deutsch Ingenieure [Berlin], no. 205, p. 1–100.

The celerity formula $C=\sqrt{gH}$ is rederived. The height of a positive surge in upstream direction is given as $\Delta H=C\sqrt{H/g}$ is the final approximate form, but a better approximation is given as $\Delta H=H_0+\sqrt{H_0^2+2H\,H_0}$, where $H_0=V^2/2g$. The energy of the surge is determined. The water-surface profiles are given for constant celerity and for changing celerity along the canal. The positive surge due to increase of canal discharge is analyzed, showing results that are correspondingly similar to those of the previous case. The model experiments are described and the celerity

formula is found to be a good approximation. The influence of the turbines' closing time on the surges created is studied experimentally, with practical formulas derived for maximum heights of surges. Experiments made in actual power-plant canals supplement the model experiments.

141. Horton, R. E. 1918, Determining the regulating effect of a storage reservoir (Differential equation for inflow, outflow and storage relations solved by using time interval as independent variable): Eng. News-Rec., v. 81, no. 10, p. 455–458.

The storage differential equation in the form $P\Delta t = A\ \Delta H + [(Q_1+Q_2)/2]\Delta t$ is discussed for integration (Q_1 and Q_2 at the beginning and at the end of Δt, A and H area and height of reservoir surface respectively, Δt=time interval). For given Δt, P_1, A and Q_1, and assumed ΔH, Q_2 can be determined, so thatΔH and Q_2 can be obtained by successive approximations. In order to avoid this procedure, two functions are developed: $F_1 = AH_1/\Delta t - Q_1/2$ and $F_2 = Q_2/\Delta t + AH_2/\Delta t$. As Q_1 or Q_2 are functions of H_1 or H_2, $F_1 = f(H_1)$ and $F_2 = f(H_2)$, and two curves can be computed that may be easily plotted. $F_2 = P - F_1$, and for given F_1 and P_1, the value F_2 and H_2 (Q_2) can be obtained, and so on.

142. Maillet, Edmond, 1918, Sur la propagation des crues [On the propagation of floods]: Ponts et Chaussées Annales [France], year 88, ser. 9, tom. 46, v. 5, no. 20, p. 197–204.

The author derives analytically that the maximum discharge of a flood is a decreasing function with an increase of channel distance traveled, and that the minimum discharge of a recession curve following a rise becomes increasingly greater with the distance traveled.

143. Parsons, W. B., 1918, The Cape Cod Canal: Am. Soc. Civil Engineers Trans., v. 78, p. 1–157.

The application of Lévy's equations to Cape Cod Canal is given. Recognizing that the friction coefficient is not a constant, but varies slightly in an inverse ratio with the velocity, a new proposal is made that a friction coefficient, changing as a linear function of the mean velocity, will absorb the same total energy in the interval during which the velocity changes from zero to maximum.

144. Cisotti, Umberto, 1919, Sulle onde progressive di tipo permanente oscillatorie [On progressive waves of the permanent oscillatory type]: Rend. della Reale Accad. dei Lincei [Italy], v. 28, p. 174–178.

The equation of Levi-Civita for the progressive waves of permanent type is further developed (in second approximation) and discussed.

145. Maillet, Edmond, 1919, Sur le mouvement graduellement varié nonpermanent et la propagation des crues [On unsteady, gradually-varied flow and the propagation of floods]: Ponts et Chaussées Annales (France), Partie Tech., (3) May-June, p. 289–330.

The author starts from a general momentum partial differential equation in Flament's Hydraulics (1909) and makes approximations by neglecting some of the five terms. He derives the formula for unsteady flow $bV^2/R = S_0 - \lambda\partial H/\partial x$ with the analysis of coefficinet λ and b under different conditions. The relation of hydraulic parameters of steady and unsteady flow is developed from the above expression. The orders of magnitude of the neglected terms in comparison with the term $(1/S_0)\partial H/\partial x$ are determined for many rivers. From the above formula it is concluded that all flood

peaks attenuate and that sharp peaks attenuate most. The same conclusions are derived for discharges. The components of multipeak hydrographs are discussed, as are other aspects of unsteady flow, all derived from the above formula.

146. Nekrasova, A., 1919, O volne Stoksa [On the wave of Stocks]: Izv. Ivanovo-Voznesenkogo Politekhnicheskogo Inst. [U.S.S.R.], Petrograd [Leningrad], no. 1, p. 81–89.

The Stocks type of wave is studied analytically, and the angle between the tangents at the wave peak is proved to be 120°. The formulas for shape of wave, wave height ($h=0.165\ L$), and celerity of wave (approximately $C=1.12+\sqrt{gL/2\pi}$ with L=length of wave)are given.

147. Oeltjen, J., 1919, Ueber die Berechnung von Flutwellenlinien in einem Tideflusse [On the computation of tidal wave profile in a tidal river]: Zentralbl. der Bauverwaltung [Germany], year 39, no. 27, p. 137–139.

Two differential equations for unsteady flow are rederived and applied to tidal-wave movement in a channel by the use of a simple finite-difference method.

148. Pigeaud, 1919, Note sur la propagation des crues [Note on flood propagation]: Ponts et Chaussées Annales [France], v. 48, no. 4, p. 29–57.

As the basis of this study, the author takes Graeff's flood-forecasting method (1883), based on the twofold thesis that stage-discharge relations and wave celerities in a reach are functions of wave height. Although Graeff's approximation is good, Pigeaud contests Graeff's first thesis on the basis of Baumgarten's law (so-called in France), that for a particular stage, the discharge is greater during the flood rise than during the recession. The celerity given as $C=dQ/dA$ is rederived, and Graeff's second thesis of wave translation celerity is discussed as giving higher peaks and lower troughs than actually occur in the rivers. Pigeaud replaces the celerity of Q_{max}, and a correction is applied to Graeff's method by using either d^2Q/dt^2 or d^2H/dt^2 from the flood hydrograph. These factors are approximately proportional to $(Q_1-2Q_2+Q_2)$ for the time of Q_2, and real $Q_r=Q+Kd^2Q/dt^2$, where Q is the discharge from rating curve, with K a coefficient.

149. Cisotti, Umberto, 1920, Sull'integrazione dell'equazione caratteristica dei piccoli moti ondosi in un canale di qualunque profondita [On the integration of the characteristic equation for small wave movements in a channel of any depth]: Rend. della Reale Accad. dei Lincei (Italy), v. 29, p. 131–133, 175–180, 261–264.

This report is a continuation and further development of the author's (1918) study (p. 255–259, 312–316).

150. Forchheimer, Ph. and Ruemelin, Th., 1920, Hydraulische Folgerungen aus Beobachtungen in Trostberg [Hydraulic consequences from the observations in Trostberg]: Schweiz. Bauzeitung [Switzerland], v. 75, p. 249; also v. 68, p. 21. 1916.

The experiments with surges (negative) in the canal Trostberg-Tacherling are described and analyzed. Starting from $Q=35.45$ m^3 per sec flow was suddenly stopped, and the celerity of surge was computed as 4.17 m per sec, but was measured 4.02 m per sec.

151. Maier, E. and Spaeth, K., 1920, Der Einfluss der Schiffschleusungen auf die Wasserkraftanlagen an dem zu kanalisierenden Neckar. Die Schleusungsversuche am Wasserkraftwerk Pappenweiler [The effect of manipulation of locks of canalized river on powerplants on the river Neckar. The experiments with lock manipulations at the water powerplant Pappenweiler]: Zentralbl. der Bauverwaltung, year 40, no. 77, p. 485–490.

Unsteady flow in a canal, caused by lock manipulations, is recorded. The records indicate clearly that the negative (depression) surges attenuate by progressing upstream. The attenuation of waves created by many manipulations is shown.

152. Woodward, S. M., 1920, Hydraulics of Miami flood control project (pt. 7, Routing floods through retarding basins): Miami Conservancy Dist. [Ohio] Tech. Repts. pt. 7, chap. 7, p. 156–200.

The word "routing" is used here, probably for the first time. According to the author, the expression "routing a flood through a retarding basin" refers to the operation of computing the probable effect which the retarding basin will have on the hydrograph of flood. For given $P=f(t)$, $W=f(H)$ and $Q=f(H)$, the $Q=f(t)$ and $H=f(t)$ are to be determined. Step methods and calculus methods are described. The step methods are: (1) trial-and-error methods (method with t assumed, method with t found by trial), and (2) direct graphical methods (using mass curve, and using inflow hydrograph). $W=f(H)$ and $Q=f(H)$ are usually combined in $W=f(Q)$ by eliminating H. For step method with t assumed, the trial is made by selecting Q_2 (end of Δt) and recomputing Q_2, and so on. Step method with t found by trial supposes a given H_2 (Q_2), and t is determined by trial-and error method. The direct graphical methods use: (1) curve $(W+\frac{1}{2}Q\Delta t)$, given by H. A. Thomas (1917), with mass curves, and (2) $W_1+\frac{1}{2}P\Delta t = W_2+\frac{1}{2}Q\Delta t$ together with inflow hydrograph (without mass curves). A five-sixth rule is developed as a rough approximation, showing that the average outflow discharge during a uniform inflow flood is approximately five-sixths of the maximum outflow discharge.

153. Kobelt, Karl, 1921, Ueber eine kuenstlich erzeugte Hochwasserwelle in der Aare [On an artificially created flood wave on the river Aar]: Mitt. des Eidg. Amtes fuer Wasserwirtschaft, [Switzerland], no. 14, p. 1–33.

The flood wave artificially created on the Aar River on February 6, 1920, is analyzed. The experiment is described, with description given of the waves as recorded at the origin and at many stations along the river. The celerity of wave is computed. The artificial wave is compared with a natural flood wave. Waves created by the operation of hydroelectric powerplants are discussed.

154. Krey, H., 1921, Die Wirkung der Schleusungen auf den Wasserstand und die Wasserbewegungen in den Haltungen [The effect of lock operations on the levels and water flow in the canals between locks]: Zeitschr. des Deutschen Wasserwirtschaft und Wasserkraft-Verbandes [Berlin], E. V., year 1, no. 5.

The wave movements created in canals by lock operations are studied.

155. Levi-Cività, Tullio, 1921, Les Ondes en los liquids, propagacion en los canals [The waves in liquids propagating in canals]: Instutut d'estudis

catalans [Barcelona, Spain], "Questiones de Mecanica clasica y relativista," lectures made in Jan. 1921.

156. Nekrasova, A., 1921, O volnakh ustanovivshegosiya vida [On steady waves]: Izv. Ivanovo-Voznesenskogo Politekhnicheskogo Inst. [U.S. S.R.] Petrograd [Leningrad], no. 3, p. 52–65.

This analytical study treats the movement of steady waves on a horizontal bed having infinite water depth. The wave height is obtained. It is found analytically that the formation of the symmetrical, steady wave with double symmetry, propagating on the surface of infinitely deep water, also does not occur.

157. Pardoe, W. S., 1921, Method of computing waste-weir capacity and reservoir rise: Eng. News-Rec., v. 86, no. 3, p. 114.

The graphical integration of the equation $dt=AdH/(P-Q)$ for given $Q=f(H)$ and $A=f(H)$, and for given $P=f(t)$ is shown.

158. Reineke, H., 1921, Die Berechung der Tidewelle im Tideflusse [Computation of tidal wave in a tidal river]: Jahrb. fuer die Gewaesserkunde Norddeutschlands [Berlin], Besondere Mitt., v. 3, no. 4, p. 1–22.

The movement of a tidal wave in a channel is studied by the use of two partial differential equations, with the momentum equation being in the form given by Boussinesq. An approximate method for the computation of tide movement in the channel is shown. The factor for change of slope is used in finite-difference form, either with reference to time, as dS/dt, or to channel reach, as dS/dx.

159. Stevens, J. C., 1921, Computing reservoir outflow and height from inflow and capacity: Eng. News-Rec., v. 87, no. 25, p. 1031–1032.

In the integration of storage differential equation $P-Q=dW/dt$, the function $W=f(Q)$ is replaced by $dW=f(Q)dQ$, where $f(Q)=m$ is the slope of storage function and may be considered practically constant in certain limits of Q. The trial-and-error method is based on different Δt and corresponding m. The general form of the storage differential equation for $Q=f(H)$ and $W=f(H)$ as power functions of H is developed, but not integrated because of the difficulty to fit the hydrograph $P=f(t)$ by a mathematical expression.

160. U.S. Bureau of Reclamation, 1921, Graphical determination of hydraulics for detention reservoirs: U.S. Bur. Reclamation, Denver, Colo., Office Design Data, no. 40–C–235, Aug. 8, 1 p.

A short description is given of a graphical and numerical method for determining the effect of a detention reservoir, by use of a storage-outflow discharge relation, and mass curve of inflow discharge. The trial method of integration of the storage equation starts from equal storage increments; the corresponding times, reservoir levels, and outflow discharges are determined.

161. Taylor, G. I., 1921–22, Tides in the Bristol Channel: Philos. Soc. [Cambridge], Proc., v. 20, pt. 3, p. 320.

The propagation of tides is studied in a channel in which both width and depth vary in proportion to the distance from the upper end. The Bristol Channel was used for the approximation of this case. The spring tidal ranges, observed in natural occurrence at several places inside the estuary, are compared with the theoretical solution, which involves the first order

of the Bessel function. The observed tidal ranges agree closely with the computed.

162. Bonneau, L., 1922, Étude sur les ondes stables dans les canaux et cours d'eau [Study of stable waves in canals and channels]: Ponts et Chaussées Annales [France], p. 273–279.

A study is made of fixed and traveling waves in canals where the wave length is generally not greater than the depth of water, except for the case of a solitary wave. The equation for a solitary wave is shown to be only a special case of the general equation for permanent waves. The assumption is made that the liquid portions of the waves are displaced parallel to themselves such that the particles trace homologous trajectories with regard to the bed. An expression is derived for permanent waves, with provision for the resistance of the bed and the excess of surface slope over bed slope.

$$\frac{H^2}{2}+\frac{H_1H_0^2}{3}\left[\frac{\partial^2 H}{\partial x^2}-\frac{1}{H}\left(\frac{\partial H}{\partial x}\right)^2\right]-\frac{P_0}{g}=H_1H_0-\frac{H_1H_0^2}{H}$$

where H=depth at any point, H_0=depth at upstream section, H_1=depth at downstream section, x=distance from upstream section, and P_0=total pressure against the upstream liquid cross section exerted in the downstream direction. For a solitary wave with a velocity of propagation $C=\sqrt{gH_1}+V_0$, the above equation reduces to $z=2f/(1+\cosh \mu x)$, where $\mu=\frac{1}{2}H_0\sqrt{3f/H_1}$, $z=(H-H_0)$ $f=(H_1-H_0)$, g=acceleration of gravity, and V_0=initial mean velocity. For a horizontal channel $V_0=O$ and $C=\sqrt{gH_1}$, as found by Bazin. The impossibility of the occurrence of negative solitary waves is proven. Solitary waves depend upon only one parameter (height), while oscillating waves depend on two (height and length). Roll waves on water courses are very similar to surf waves of the ocean. Roll waves take up differences of energy between successive sections in the same stream having widely different characteristics. The surges of this nature are found upon the Congo River. Wind produces roll waves on the surface of water. This is more pronounced for quiet water than flowing water.

[Abstract by C. A. Wright, Am. Geophys. Union Trans., pt. 4, 1938.]

163. Grover, N. C., 1922, Standing waves in rivers: Am. Soc. Civil Engineers Trans., v. 85, p. 1400–1404.

Explanation of standing flood waves moving in a stream. The bottom friction is considered to be the essential cause.

164. Johnson, R. D., 1922, The correlation of momentum and energy changes in steady flow with varying velocity and the application of the former to problems of unsteady flow or surges in open channels: Engineers and Eng., v. 39, p. 233–240.

The difference between the laws of conservation of energy and momentum is pointed out. Pressure waves and suction waves are discussed, the former as standing waves set in motion relative to the earth. The suction wave has no counterpart in a standing wave and moves with constantly changing profile, and the top of the wave moves more rapidly than the bottom with continual lengthening of the wave front. By dividing the canal into short frictionless sections with a tiny drop in water level at the end of each section, P. Wahlman showed that the wave behavior can be analyzed without significant error, it is stated. Wahlman's method is illustrated by an example.

[Abstract by F. T. Mavis, Am. Geophys. Union Trans., pt. 4, 1938.]

165. Levi-Civitá, Tullio, 1922, Risoluzione dell'equazione funzionale che caratterizza le onde periodiche in un canale molto profondo [The solution of functional equation which characterizes the periodic waves in a very deep canal]: Math. Annalen [Berlin], v. 85, p. 256–279.

The periodic waves in deep channels are studied analytically by using the function of a complex variable. The works of Airy, Lord Rayleigh, Stokes, and others are discussed. Airy's celerity formula for deep water, given as $C^2=g\lambda/2\pi$ is verified.

166. Maillet, Edmond, 1922, Sur les procedés d'annonces des crues fondés in en théorie [On the procedures of flood forecast, based on theory]: Ponts et Chaussées Annales [France], year 92, fasc. 1, v. 1, no. 8, p. 145–156.

This paper is a criticism of Pigeaud's paper (1919) on flood forecast by stages and discharges. It is assumed that the rating curves and celerity graphs produce the same results when used according to Graeff's system. For the forecast between two stations, the approximate equation is $H_1K_1=K(H_2+K_2)$, where K, K_1 and K_2 are constants. A similar relation comes from three stations, two on the main river and one on the tributary. For the general type $f(H)=aH^n$ there is a relation $\Delta H_1=(a_2/a_1)k_2^{n-1}\Delta H_2 + (a_3/a_1+(k_3/k_2)^{n-1}\Delta H_3$, with $H_1=k_2H_2$ and $H_3=k_3H_2$ and with k_2 and k_3 constants, which the author calls Belgrand's law of increases. It is found that the conical equation $a_1H_1^n=a_2H_2^n+a_3H_3^n$ can be replaced by some straight lines of type $\Delta H_1=c_2\Delta H_2+c_3\Delta H_3$, with two or three systems of c_2 and c_3 values. Different formulas are developed for stage relations. These methods are all considered as first approximations.

167. Winkel, R., 1922, Aenderungen des Wasserstandes in den Haltungen infolge Schiffsschleusungen [The changes of levels in canals between locks due to lock operations]: Zeitschr. des Deutschen Wasserwirtschafts u. Wasserkraft-Verbandes [Berlin], no. 13.

The wave movements created in canals by lock operations are studied.

168. Bonnet, L., 1922–23, Contribution à l'étude théorique des fleuves à marée [Contribution to the study of tidal rivers]: Trauaux Publiques Belgique [Brusselles] Annales, 1922, fasc. 3, 4, 5, 6, and 1923, fasc. 1, 2, 3, 5.

The propagation of a tidal wave is treated as a wave of translation continually modified by friction and by the shape of the bed. The author's thesis that movement of the tide along the river creates a countercurrent permits an explanation of all the characteristics of a river tide.

169. Forchheimer, Ph., 1923, Durchfluss des Wassers durch Roehren und Graeben grosser Abmessungen [Discharge through pipes and canals of great dimensions]: Berlin, Springer.

The water velocity of a positive wave due to the release of water, wit flow resistance taken into account and using Chezy's formula, is given as $V=\lambda R^{0.7}\sqrt{S-dy/dx}$, where S is the initial slope of water surface and dy/dx is the slope of the line connecting the points cut by the wavefront and the middle part of the wave. An expression for the height of steep wavefront of positive surge is given as function of x, h, C, Q, S, and other parameters.

170. Gibson, A. H., 1923, Hydraulics and its applications: 3d ed., Great Britain, 405 p.

The formula for the celerity of a long-wave crest, relative to the flow of the stream, C^1, is developed as $C^1=\sqrt{2g(H_0+h)^2/(2H_0+h)}$ with $H_0=$ depth of stream before the arrival of the wave and h=height of wave crest above surface of stream.

171. Krey, H., 1923, Einfluss von Querschnitteinengungen auf die Sturmfluthoehe [The influence of cross-sectional constrictions on the flood-wave height]: Zentralbl. der Bauverwaltung [Germany], no. 67, 68.

172. Macy, F. H., 1923, Reservoir spillway discharge: Eng. News-Rec., v. 90, no. 8, p. 361.

This is a presentation of the classical method of routing flood waves through reservoirs, using the storage equation and the mass curve, and is similar to the method presented by C. C. Jacob (1912).

173. Pacak, A., Smetana, J., and Till, J., 1923, Tests on the Vranany navigable canal at Horin: Permanent Internat. Assoc. Navigation Cong., 13th Cong., rept. no. 6.

Some experiments in a navigable channel are described. The paper is concerned principally with the utilization of navigable channels for power-plant purposes.

174. Sherman, L. K., 1923, Experiments on the effects of upper channel improvements upon the downstream flood heights: Western Soc. Engineers Jour., v. 28, no. 8, p. 293–318.

The effects of upstream channel improvements on downstream flood heights are studied experimentally in a flume 66 feet long. Use is made of of the celerity formula $C=(Q_2-Q_1)/(A_2-A_1)$, where Q^2 and A^2 (area) are the initial values and Q_1 and A_1 are the final values for time interval Δt. The celerity C computed by above formula is verified by experiments. The statement by the author and H. Thomas, that a monoclinal wave becomes stable after some time and for a given length travelled is questioned in discussion by S. M. Woodward. H. Thomas, in his discussion, derives the above formula for finite monoclinal wave with Q_1 and Q_2, A_1 and A_2 thus replacing by finite values, the Forchheimer small-difference concept of both Q and A. Thomas' proof starts from the assumption that a monoclinal wave becomes stationary after a given channel length, taking the flow resistance into account. For a small increase in shape he finds that $C=1.5\ V$.

175. Varet, 1923, Étude graphique des conditions d'exploitation d'un réservoir de régularisation [Graphical study of operation of a storage reservoir]: Ponts et Chaussées Annales [France], v. 93, no. 4, paper 21, p. 61–77.

The use of mass curves for routing hydrographs through a reservoir is shown, and the application is discussed.

176. Winkel, R., 1923, Hydromechanische Vorgaenge beim Schleusen eines Schiffes [Hydromechanical phenomena during the lock operation for a ship]: Die Bautechnik [Germany], no. 33.

177. Bonneau, 1924, Théorie de la propagation des crues [Theory of flood propagation]: Ponts et Chaussées Annales [France], v. 94, no. 2, paper 14, p. 282–325.

Study is made assuming negligible difference between bed slope and friction slope in the second partial differential equation for unsteady flow. Some actual data are given to justify the assumption. The integration gives solutions that are practically the same as the characteristics. The propagation of discontinuities in partial derivatives is derived as the classical solution. For very gradual waves without critical points, the velocity is

$V=c\sqrt{gH}\ [1-(\frac{1}{2}S)\partial H/\partial x]$ and celerity is

$C=(3V/2)[1-(H/3S)(\partial^2 H/\partial x^2)/(\partial H/\partial x).]$

For $y=H-H_0=2f/(1+\cosh\mu x)$, $C=3V/2$ (approximately)

Study is also made of the general case, without the above assumption, showing the corresponding integration of partial differential equations. The expressions for the coefficient of attenuation, β, and wavelength are given in function of other parameters, for the case $h_{max}\leq H_0/5$. The following problems are studied and discussed: the influence of river cuts on floods, the effect of the major river bed and of levees on floods, the effect of increase of bed level on floods, and so forth.

178. Bouasse, H., 1924, Houle, rides, seiches et marées [Waves, ripples, seiches, and tides]: Paris, p. 1–516.

An extensive treatise on the waves, which is a summary of wave-knowledge that existed after the First World War. In chapter 9 progressive nonperiodic waves and solitary waves (p. 271–317) are studied. In chapter 10 other progressive waves (bores, river floods, and so forth) are treated. In chapter 14 the propagation of tides in river channel is studied.

179. Egiazarov, I. V., 1924, Metod opredeleniya kolebaniy urovnya vody v dlinnykh byefakh pri regulirovanii [Method of determination of the water level fluctuations in the long canals due to pondage]: Nauchno-meliorativnyy Inst. [U.S.S.R.] Izv. (Trans.), no. 7, p. 1–75.

Many earlier methods for flood routing and wave progression are summarized. Special attention is given to the method which the author has developed for the Volkhov powerplant canals. This method deals with storage in a reach, and relates to water surface, in order to derive the stored volume. Largely, this paper is devoted to analysis of the work of other authors in the field of movement of waves and surges in channels.

180. Forchheimer, Ph., 1924, Wasserschwall und Wassersunk [Positive and negative surges]: Leipzig and Vienna, Franz Deutike, p. 1–42.

The equations for wave celerity, water velocity, length of travel, depth (H_0+y), and discharge for positive surge wave occurring in both downstream and upstream directions, and for negative surgewave (depression), occurring also in both directions, are derived analytically. Numerical examples for computation are given.

181. Levi-Civitã, Tullio, 1924, Fragen der klassischen und relativistischen Mechanik [The questions of classical and relativistic mechanics]: Published as four lectures, Berlin, p. 26–59.

Permanent wave motion, as defined in his previous paper (1907) is herein discussed in detail.

182. Levi-Civitã, Tullio, 1924, Ueber die Transportgeschwindigkeit in einer stationaeren Wellenbewegung [On the celerity in a stationary wave

movement]: Vortraege aus dem Gebiete der Hydro- und Aerodynamik, Lectures held in Innsbruck, 1922; Th. v. Karman and T. Levi-Cività, eds., Berlin, Springer, p. 85–96.

This is an analytical study of wave motion in a canal under simplified condition (rectangular channel, horizontal bottom, frictionless motion, with an observer traveling with celerity C and using a coordinate system traveling with him). The velocity potential and streamline function in the form of complex variables are introduced. The movement of a liquid particle during wave propagation is analyzed. The mean discharge through a vertical cross section is determined as $\overline{Q}=CH-q$, in which q is a relative discharge in a moving coordinate system. A proof of water movement due to wave translation is given and an expression for $\overline{Q}$ in integral form is developed. The velocity $V=\overline{Q}/H$, and the application of results to the Airy waves is given. The discussion among Prandtl, Levi-Cività, and von Karman, concerning the use of a moving coordinate system, is summarized.

183. Samsioe, 1924, Dammöpningars avbördning. Mätningsmetod grundad pa vattenstandsvaxlingar i en narbeläger hölga [The discharge through dam openings. Method of computation based on changes in water elevations in nearby pool]: Tekniske Tidskraft [Sweden], 25 Oct.

184. Sen, N., 1924, On the equation of long waves in canals of varying section: Philas. Mag. [London], ser. 6, v. 48, p. 65–78.

Green [Mathematical papers, p. 225] was the earliest investigator of this subject and showed that in the case of progressive waves in a canal with slowly varying sections the elevation of water is inversely proportional to the square root of the breadth and to the fourth root of the average depth at the section. Some special solutions, analogous to those corresponding to the motion of progressive waves, are discussed in the article, and the law of the variations of depth and breadth, for which such types of motion can exist, has been deduced.
[Abstract by F. T. Mavis, Am. Geophys. Union Trans., pt. 4, 1938.]

185. Thijsse, J. Th., 1924, Berechnung der Gezeitenwellen mit betraechtlicher Reibung [The computation of tidal waves with significant friction resistance]: Vortraege aus dem Gebiete der Hydro-and Aerodynamik, Lectures held in Innsbruck 1922; T. V. Karman and T. Levi-Cività, eds., Berlin, Springer, p. 116–122.

Employing Lorentz's linearization of friction losses, the movement of tidal waves is studied, with friction term as kV instead of gV^2/c^2H. An expression for k is given, $[k=8gV\max/3\pi\ c^2H]$, to simplify the formula, $k/n=tg\vartheta$, where ϑ is an auxiliary angle which is a characteristic of water movement in channels. By means of this linearization the continuity and dynamic equations are integrated. Expression for V and H are given in the general exponential form of complex variables, expressing constants in that solution as functions of ϑ, among other given magnitudes. The celerity is given as $C=\sqrt{gH\ \cos\vartheta}/\cos\frac{1}{2}\vartheta$ and considering no friction, $C=\sqrt{gH}$. The friction factor decreases C, inasmuch as $\sqrt{\cos\vartheta}$ is always smaller than $\cos\frac{1}{2}\vartheta$. Application to the Zuider Zee is discussed.

186. Winkel, R., 1924, Aufnahme der beim Schleusen in einer Kanalhaltung entstandenen Senkungswellen [The survey of a negative wave, created in a canal by lock operation]: Die Bautechnik [Germany], year 2, no. 25, p. 251–252.

The negative wave, created by filling the lock 1 in Niederfinow, is followed and surveyed. The wave heights based on the celerity formula are first computed ($h=Q/BC$, and $C=\sqrt{gR}$, R=hydraulic radius), and then compared with the observed values, with good agreement. Using Krey's formula (1912), h is computed and compared to the observed values, also with good agreement.

187. Fantoli, G., 1925, Sul passaggio dell'onda di piena nella supposta rotta di un serbatoio [On the passage of flood wave along a supposed direction of a reservoir]: Ann. di Utilizzazione delle Acque [Italy], fasc. 1.

188. Jeffreys, Harold, 1925, The flow of water in an inclined channel of rectangular section: Philos. Mag. [London], sec. 6, v. 79, p. 793.

A theory has been constructed to account for certain travelling waves observed by Vaughan Cornish in steeply inclined conduits. It appears that the uniform turbulent flow of a stream, with a plane free surface, becomes unstable when the mean slope exceeds 1 in 100 and that it is then replaced by a series of bores travelling faster than the water. Observational evidence is qualititive. Several points remain to be tested.

[Last part of author's summary.]

189. Levi-Cività, Tullio, 1925, Détermination rigoureuse des ondes permanentes d'ampleur finie [Rigorous determination of permanent waves with finite amplitude]: Math. Annalen [Germany], v. 93, p. 264–314.

The periodic waves, irrotational and steady, which propagate without alteration of form on the surface of a liquid infinitely deep, are studied analytically. The author finds that these waves are symmetrical in relation to the vertical lines through the peak and the lowest point but are not symmetrical in relation to the mean horizontal water level, because the height of the peak above this level is greater than the depression of lowest point below this mean level.

190. Lin, Ping-Yi, 1925, Prediction of floods: Iowa Univ., M. S. thesis, p. 202.

On the hypothesis that flood prediction is the most economical means of flood protection, the author considers the flood problem from several angles. Chapter 1 deals with the propagation of flood waves on rivers and presents mathematical derivations of formulas for determining the rate of propagation of flood waves, for computing the velocity of the flood wave, and for predicting the height of the wave. Chapter 2 develops equations for the profile and volume of a roll wave and a translatory wave. Chapter 3 deals with actual floods.

191. Proudman, J., 1925, Tides in a channel: Reprinted from the Philos. Mag., v. 49, p. 465–475.

The present investigation relates to the tidal dynamics of a channel whose section is uniform along the length but of varying depth from side to side. After making general deductions concerning a section of any shape, attention is concentrated on a parabolic section. One result is to show the degree of accuracy involved in the approximations of what may be called the "narrow sea theory," the theory which in recent years has been shown to account for many of the observed features of tides in gulfs, channels, and narrow seas. The essential characteristics of the narrow sea theory is that transverse currents are neglected, so that the motion is assumed to consist of longitudinal oscillations whose currents maintain oscillating transverse gradients in virtue of the earth's rotation. By

introducing geometrical simplicity into the basin, it is possible to take account of the transverse currents and then to compare the results with those obtained on neglecting these currents. If the channel is not too wide or too shallow the degree of accuracy of the narrow sea theory is high, but this degree of accuracy decreases as the effect of the earth's rotation becomes important.

[Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, February 1954.]

192. Eisner, F., 1925–26, Proktisches Beispeil zur Berechnung des Stauschwalles in einem Obergraben bei Vorhandensein einer Heberentlastung [A practical example for the computation of surges in a headrace canal, where a spillway syphon exists]: Wasserkraftjahrbuch, Munchen [Germany].

193. Bernadskiy, N. M., 1926, Teoriya i raschet rechnogo povodka s primenyeniyem k raschety sutochogo regulirovaniya r. Volkhova [Theory and computation of river flood waves with application to the computation of daily regulations of river Volkhov]: Materialy po Issledovaniyu r. Volkhova (Materials on the Investigation of the River Volkhov), no. 17.

The procedures are given for computing unsteady flow by the method of finite differences, as applied to the problems of daily discharge fluctuations on the Volkhov River.

194. Koch. A., and Carstanjen, M., 1926, Von der Bewegung des Wassers und dabei auftretenden Kraeften [Movement of waters and the associated forces]: Berlin Springer-Verlag, sec. 7, p. 132–150.

Discussion is given for waves subject to momentum control that is neglecting energy losses. The celerity of steep positive and negative waves is developed as approximation $C=\pm(V_1+3V_2/4+\sqrt{gH_1}$ where (−) is for a wave of small height traveling upstream, and (+) is for a wave traveling downstream. Friction is neglected. Other aspects of wave celerities are analyzed.

195. Krey, H., 1926, Die Flutwelle in Flussmuendungen und Meeresbuchten [The tidal wave in estuaries and sea bays]: Mitt. der Versuchsanstalt fuer Wasserbau u. Schiffbau, [Berlin], no. 3, Selbstverlag.

196. Makkaveev, V. M., 1926, Gidromekhanicheskiy anali Nevskikh navodneniy [Hydromechanical analysis of Neva inundations]: Izdatel'stov Gidrologischeskogo Inst. [U.S.S.R.], v. 1.

197. Makkaveev, V. M., 1926, Rol'treniya i neprizmatichosti rusla pri dlinnykh volnakh [The role of friction resistance and nonprismatic shape of channel for the long waves]: Izdatel'stov Nauchno-Meliorativnogo Inst. [U.S.S.R.], no. 11–13.

198. Moellner, M., 1926, Die Wellen, Schwingungen und die Naturkraefte [Waves, oscillations and natural forces]: Braunschweig [Germany].

199. Struik, D. J., 1926, Détermination rigoureuse des ondes irrotationnelles périodiques dans un canal à profondeur finie [Accurate determination of irrotational periodic waves in a canal with finite depth]: Math. Annalen [Germany], v. 95, p. 595–634.

The periodic irrotational waves in a straight canal having a horizontal bottom and vertical walls, and in which the liquid moves along the canal, are studied analytically, primarily on the basis of mathematical approach

introduced by Levi-Cività. It is shown that the waves are symmetrical. Other characteristics of waves are treated, such as the equation of free water surface (wave shape), the mean depth in canal, the maximum depth, the maximum depression, the water velocity, the energy, and so forth.

200. Thorade, H., 1926, Fortschreitende Wellen bei Veraenderlicher Wassertiefe [Progressive waves in the changing water depth]: Mitt. Math. Gesell. [Hamburg], v. 6, no. 5, p. 203–225; see also Ann. Hydr., v. 54, p. 217–222, 1926.

Green's law is discussed and found not to be applicable for long waves. Stationary waves as a basic form are discussed. The solutions of wave equations are given in the case of changing depth for both convex and concave conditions. It is concluded that a very long progressive wave, by traveling along a sloping bottom between two points having constant depths of H_1 and H_2, is transformed to a progressive wave of nonuniform celerity, having an amplitude that oscillates between its original amplitude and an amplitude of $\sqrt{H_1/H_2}$ times the original.

201. Weinstein, A., 1926, Sur la vitesse de propagation de l'onde solitaire [On the celerity of the solitary wave]: Reale Accad. Lincei [Rome] Rend., Classe sci. fis.-matematichee naturali, sec. 6, v. 3, p. 463–468.

Using the method of Levi-Cività (1925), the celerity of the symmetrical stationary solitary wave is derived as $C^2 = gH_0[1 + h/H_0 - (21/20) + (h/H_0)^2]$ in a uniform rectangular canal with the initial depth H_0 and wave height h middle point of wave of an infinite length). Since C^2, as estimated by the first two terms, alone is greater than the true C^2, the correction (third term on the right side) is negative, and, for $h/H_0 \leq 1/5$, this term cannot ordinarily be greater than 4 percent of the first two terms.

202. Winkel, R., 1926, Das Verhalten von Hebungs-und Senkungswellen bei verschiedener Fliessbewegung [The behavior of rising and falling waves in the different water flows]: Deutsche Wasserwirtschaft [Germany], year 21, no. 1, Berlin, p. 4–6.

Types of positive and negative waves in a canal are discussed, with the application of basic formula for celerity $C = V \pm \sqrt{gA/B}$.

203. Winkel, R., 1926, Besondere Wellenerscheinungen in Schiffahrts Kanaelen infolge von Schleusungen [Special wave phenomena in the navigation canals due to lock operations]: Die Bautechnik [Germany], year 4, no. 9, p. 110–111.

A full negative wave, created in upstream direction by filling the locks, is discussed. The wave transformation by progressing, its influence on a boat in the canal (oscillations of a boat due to wave movement), and the wave reflection are analyzed.

204. Boess, Paul, 1927, Berechnung der Wasserspiegellage [Computation of water-surface elevations]: 2. Teil, Zeitlich veraenderliche Wasserbewegungen in offenen Gerinnen [Unsteady water movement in open channels]: Forschungsarbeiten aus dem Gebiete des Ingenieur-Wesens, V.D.I. Verlag, G.m.b.H., no. 284, p. 63–96.

Restatement is made of types of surges in channels, with formulas given for known celerity and surge heights. The experiments to verify the surges are described. The computed and observed heights of positive surges created by sudden closure (total or partial) of a gate in a canal are compared both for the subcritical and supercritical flow. The minimum height

$(H+h)$ is given, in the case of total closure with constant Q but changing H, when H is equal to critical depth H_c, $(H+h)$ minimum is equal to $(2.25\ H_c)$, but the height of energy line is only $(1.5\ H)$. Agreement between computed and experimental results is good. In the case of partial closure, the height h for a ΔQ (change of Q) computed by formula agrees with the experimental results. For negative surges, starting from water at rest in channel, the computation gives $V_{max}=0.430\ \sqrt{gH}$; $h=0.477\ H$, in the case of total opening; $Q_{max}=0.225\ B\sqrt{g}\ H^{3/2}$ and the coefficients for an approximation are 0.50, 0.50, and 0.250, respectively, for rectangular channels. The experiments gave somewhat smaller results for slope of channel bottom other than zero. The same is given for sudden partial openings. For celerity, a new formula, $C=4gH/(V+4\sqrt{gH})$, is given, based on experiments, with $\sqrt{gH}$ as a good approximation. Relation of h, Q, and V is discussed. The height of hydraulic jump is developed from the above analysis. The experiments with surges in the headrace canal of the Itter powerplant are described and results given.

205. Dahl, 1927, Om svallninger i kanaler och tuber [On waves in canals and pipes]: Handlinger [Stockholm], no. 63.

206. Hill, R. A., 1928, Graphics of temporary flood storage: Eng. News-Rec., v. 100, p. 657–659.

This paper describes a method, which employs the use of mass curves and a polar diagram of outflow rate, for calculating the balance between reservoir inflow and outflow during floods. The example given is for a complex outflow system. The step-by-step integration by means of mass curves uses the polar diagram to relate slope, or rate of outflow discharge, to storage volume.

207. Puls, L. G., 1928, Construction of floodflow routing curves: U.S. 70th Cong., 1st sess., House Doc. 185, p. 46–52 and U.S. 71st Cong., 2d sess., House Doc. 328, p. 190–191.

The storage equation is used for flood routing along the river. The river is divided into reaches, and for each reach the storage-height curve for the lower end of the reach is determined, as well as the rating curve of outflow discharge, and the two factors, $W+Q/2$ and $W-Q/2$. The routing procedures are explained.

208. Shiroki, S. P., 1929, Sravnitel'naya otsenka metodov opredeleniya kolebaniy uravnya pri regulirovanii raskhoda [Comparative appraisal of the methods for determination of stage fluctuations in runoff regulation]: M. I. I. T. [U.S.S.R.] Trudy (Proc.), no. 11.

209. Forchheimer, Philipp, 1930, Hydraulik [Hydraulics]: 1st ed., Leipzig, Teubner, p. 246–303, 411–432, 465–484.

The basic partial differential equation for unsteady flow and wave celerity are rederived and discussed. The wave energy and wave form are analyzed. Dam breaches and created waves are treated using Ritter's original data. Tidal wave in estuaries is also treated. Flood waves in rivers are analyzed, especially with reference to celerity, maxima values (S, V, Q, H), influence of cross-sectional shape on wave celerity (as the ratio C/V in the function of cross-sectional shape), form changes, and wave attenuations showing $\Delta Q=(B^2Q\Delta x)/[2S(\partial Q/\partial H)^3r]$, where r is curvature radius of the curve $Q=f(x)$ at the peak. The wave-routing method is described in detail by use of the storage equation and mass curves. Other procedures based on inflow and outflow hydrographs are also described. A good historical review of flood-routing procedures for reservoirs is given. The chapter on wave movement treats sea waves. Very extensive bibliographical references are given in footnotes.

210. Makkaveev, V. M., 1930, Teoreticheskoe izuchenie volnoobraznogo dviheniya vodnykh mass [Theoretical study of wave movement of water masses]: Gosudarstveniy Gidrologicheskiy Inst. [U.S.S.R.] Trudy.

211. Besson, F. S., 1931, Reservoir effect upon river floods: The Mil. Engineer, p. 577–580.

The storage-discharge relationship is determined by the use of weighted inflow $(3P_1+P_2)/8$ and weighted outflow $(3Q_1+Q_2)/8$, as a function of $[(3P_1+P_2)/8+W_1]$, where W_1 is the storage at the beginning of the time increment. This weighting of inflows and outflows violates the continuity equation, according to Thomas (see Thomas' paper of 1934).

212. Egiazarov, I. V., 1931, K sutochnomu regulirovaniyu gidroelektricheskikh stantsiy. Eksperimenal'noe issledovanie otritsatel'noy volny izliva v dlinnom lotke [On the daily regulation of hydroelectric stations. Experimental investigations of the movement of negative wave along a canal]: Nauchnomeliorativniy Inst. [Leningrad], Izv., no. 3, (with English summary).

213. Fenchel, W., 1931, Sulle onde di canale di tipo permanente [Canal waves of permanent type]: Reale Accad. Lincei Atti [Rome], v. 13, May 17, p. 740–743.

With reference to Levi-Cività's papers (1907, 1924), it is shown that a wave motion will be of permanent type if the propagation celerity is constant and if the surface profile appears unchanged by wave progression. Levi-Cività's requirement of a constant relative discharge is not necessary.

214. Goodrich, R. D., 1931, Rapid calculation of reservoir discharge: Civil Eng., v. 1, p. 417–418.

The trial-and-error method for flood routing by the use of storage equation $(P-Q=dW/dt)$, for the selected time interval is given, using the outflow-storage factor equal to $(P_1+P_2+W_1-Q_1=W_2+Q_2$, so that W_2 and Q_2 are obtained by successive operations in tabulating the above values.

215. Scobey, F. C., 1931, Unusual flow phenomena: Civil Eng. v. 1, p. 1101–1103.

A brief illustrated discussion of hydraulic jump, stationary waves, overrunning waves, curves, and air entrainment is presented.

216. Shulits, Samuel, 1931, Graphical integration in hydraulic problems: Soc. Civil Engineers [Boston] Jour., v. 18, no. 8, p. 287–300.

In graphical integration use is made of the so-called summation curve, the curve of the integral of the function being investigated. The construction and properties of the summation curve are presented here, and its use is illustrated by two examples. [From author's synopsis.]

217. Thorade, H., 1931, Probleme der Wasserwellen [Problems of water waves]: Hamburg [Germany], Verlag Henri Grand, p. 1–219; also see Probleme der Kosmischen Physik, Heft 13–14, and chap. 32 Flusstiden, 148–152, Hamburg.

This is a clearly written book relating mainly to ocean waves but covering also flood waves and the effect of channel friction on waves. The second part of this work, p. 72–194, treats principally the translation waves. The work of many investigators is reviewed and their equations for wave-celerity are given and explained. The book is comparatively free from involved mathematical discussion. Sixteen pages of references to literature (up to 1930) on sea and canal waves are given at the end of the book.

[Abstract by R. E. Horton.]

218. Velikanov, M. A., 1931, Dvizhenie volny pri proryve plotiny [The wave movement created by dam breach]: Gidrotekhnicheskoe Stoitel'stvo [U.S.S.R.], no. 2, p. 20–21.

The author claims that the crest of a wave due to dam breach moves with celerity $C=\sqrt{gH}\ (1+3h/4H)$. He derives the approximate relationship between wave height and distance traveled by the wave crest.

219. Bakhmetev, B. A., 1932, Hydraulics of Open Channels: New York, McGraw-Hill, Book Co., p. 1–329.

Though this book treats only steady flow, it is useful for the study of those characteristics of channel flows which are common to both steady and unsteady flow.

220. Barrows, H. K., 1932, Velocity of flow in natural streams: Am. Geophys. Union Trans., v. 13, p. 339–349.

Studies of a 230–mile reach of the Delaware River between Trenton, New Jersey, and Downsville, New York, included comparisons of observed and computed times of transit of water. Port Jervis, New York (drainage-area 3070 square miles), approximately midway on the River between Trenton (drainage area 6,800 square miles) and Downsville (drainage area 380 square miles), was taken as the index station and seven assumed index flows from 0.15 to 5.00 sec-feet per square mile were used as the basis for analysis. Many cross sections of the River were taken in the 230–mile reach and stage-discharge curves were made for each profile station. Concordant profiles based on different rates of flow at the index station were constructed to harmonize observations of discharge and profile. Times of transit were computed and corrected on the basis of float measurements which were made simultaneously in five sections of the River averaging about 50 miles each in September 1929, and March 1930. The former float observations were made at an average stage representing 0.45 sec-feet per square mile at Port Jervis, and the latter at a stage representing 3.6 sec-feet per square mile. Computed times of transit were materially longer than those based upon the float measurements, and the computed times were corrected to make them consistent with the float runs in September. Analyses of flood-wave velocities by L. K. Sherman showed that the flood-wave velocity was approximately double the velocity of ordinary flow at the rising stage of the River. Observations of wave velocities in the River below the Hawley Plant of the Pennsylvania Power and Light Company on Wallenpaupack River checked the computed time of flow by Sherman formula fairly well.

[Abstract by F. T. Mavis.]

221. Bednarski, E. J., 1932, Graphical analysis of spillway capacity: Western Construction News, Feb. 25, 113–115 (also erratum, loc. cit., Mar. 25, p. 168).

The graphical solution of the flood-spillway problem is carried out by using the storage equation, the mass curves of inflow and outflow, and the spillway rating curve, expressed in power form.

222. Brown, E. I., 1932, The flow of water in tidal canals: Am. Soc. Civil Engineers Trans., v. 96, p. 749–834.

The author gives a very complete summary of the theories that have been presented at various times and indicates their limitations. He proposes a new theory which assumes the reflection of the canal wave from the basin or reservoir which is fed by the canal, and he states that the degree of reflection will be governed not merely by the canal itself, but also by the basin.

Based on the studies of Bourdelles (1898, 1899, 1900) and Bonnet (1922–1923), the derivations are given for: effect of friction in reducing height of wave in an infinitely long canal; time required for crest of wave to reach a given point; energy of wave; influence of change of width; interference of two opposite waves; propagation of the tide in a canal connected to infinitely large reservoir; propagation of tide in a canal feeding a reservoir of limited capacity; reflection of the canal wave from a tidal reservoir of limited capacity; and propagation of the tide in a canal connecting two bodies of water, each having an independent tide of its own. The theory is applied to the Suez Canal showing comparisons of the computed and observed data.

223. Calame, J., 1932, Calcul de l'onde de translation dans les canaux d'usine [The computation of translation wave in powerplant canals]: Ed. La Concorde, Lausanne [Switzerland].

224. Cherkasov, A. A., 1932, O skorosti dvizheniya volny popuska [On the celerity of release wave]: Vsesoyuzniy Nauchno-issledovatelskiy Inst. Gidrotekhniki i Melioratsii [Moscow] Trudy, v. 7, p. 109–128.

The experiments of wave movement in a dry channel are made in a canal 15.45 meters long, 0.2 meter wide and 0.3 meter deep, with roughness (wood) $n=0.0093$. Three cases of wave movement are studied: $H=0, V=0, S=0$; $H_0=0, S=0$; and $H=0, S>0$. A fourth case is studied, which is the same as the third case but with a partly opened gate. The velocity of the wavehead is given from results as $V=3.1\sqrt{h(2H+h)}/\sqrt{h+0.0025H/h}$, and for $H=0$, and $h>0$, $V=3.1\sqrt{h}=0.7\sqrt{2gh}$.

The celerity of wave is given as $C=f(n, R, V, x, A)$. An empirical formula, based on experimental results, is developed, and the velocities are given for the four above cases.

225. East, R. L., 1932, Spillway-discharge calculations: Civil Eng., v. 2, no. 1, p. 34–36.

The semigraphical flood-routing method discussed was developed by W. A. Robertson. It is similar to the Goodrich method (1931) and is based on the method outlined by R. E. Horton (1918) that employs the simple storage equation $(P-Q=dW/dt)$. Two curves are used: $F_1=W_1-Q_1\Delta t/2$ and $F_2=W_2+Q_2\Delta t/2$ (with $\Delta t=t/12$ in this study), $F_2=P+F_1$ where $P=(P_1+P_2)\Delta t/2$. Values are obtained by a cumulative process of tabulation.

226. Eisner, F., 1932, Stroemungslehre der Rohre und offenen Gerinnen, II Teil (Eisner) Offene Gerinnen [Fluid dynamics of pipe and open channel, part 2 (Eisner) open channel]: Akad. Verlagsgeselschaft m.b.H. [Leipzig].

Paragraphs 6 and 7 of chapter 3 treat translation waves.

227. Lamb, Sir Horace, 1932, Hydrodynamics: 6th ed., New York, Dover Pubs., p. 250–561.

A complete discussion of the hydrodynamics of waves, with many references.

228. Levi-Cività, Tullio, 1932, Caractéristiques des systèmes différentielles et propagation des ondes [Characteristics of differential systems and propagation of waves]: Paris.

229. Meyer-Peter, E., and Favre, H., 1932, Ueber die Eigenschaften von Schwaellen and dei Berechnung von Unterwasserstollen [On the characteristics of surges and the computations for tailwater galleries]: Mitt. der Versuchsanstalt fuer Wasserbau, E.T.H., Schweiz. Bauzietung, v. 100, no. 4, p. 43–50, and no. 5, p. 61–66.

Surges in the tailrace canals of water powerplants are studied for three phases of surge action: (1) when the surge does not touch the ceiling of the gallery; (2) when the surge fills the gallery; and (3) surge action at outlet in the canal. The analytical studies made employ the continuity and momentum equations. The experimental results obtained at the Wettingen water powerplant are added, and comparison is made between computed and experimental results.

230. Sherman, L. K., 1932, Streamflow from rainfall by unitgraph method: Eng. News-Rec., April 7, p. 501–505.

The concept of the unitgraph is presented with an analysis of unitgraph determination and application. The paper discusses the unitgraphs for varying rates of rainfall. It discusses base flow, gives an example of unitgraph (Big Muddy River), and shows the computation of the hydrograph. Percentage of runoff and accuracy of the method are discussed.

231. Takaya, S., 1932, Some problems on the motion of water waves: Geophys. Mag. [Japan], v. 6, p. 347.

A generalization of Green's laws of wave propagation in a canal of gradually varying section having transverse dimensions that vary slightly within the limits of a wavelength.

232. Arakawa, H., 1933, Generalized Green's law on wave motion in a canal of variable section: Geophys. Mag. [Japan], v. 7, p. 319–325.

A generalization of Green's law, demonstrating that: the greater the slope of the canal bottom, the greater becomes the amplitude of the sea wave traveling up the canal; the smaller the variation of canal width, the greater is the amplitude of this wave; the increase of amplitude is greater for the short waves than for the long waves; for small wave oscillations, the amplitude of waves on a sloping canal depends on the bottom slope and on the width of canal.

233. Barrows, H. K., 1933, Reservoir-storage above the spillway-level: Civil Eng., v. 3, p. 233.

A graphical procedure is given for the determination of reservoir storage above the spillway level that reduces the outflow from the reservoir. A simple short method, employing the mass curves of inflow and spillway discharge, is used. (This method was devised by R. S. Holmgren in the 1928–30 investigations of floods in Vermont.) As the starting point, storage W represents the increment of difference in level ΔH. Using the slope of the outflow mass curve for ΔH, a new point on the inflow mass

curve is obtained for the new level $H+\Delta H$, and the computation process is repeated.

234. Bernadsky, N. M., 1933, Rechnaya gidravlika, ee teoriya i metodologiya [River hydraulics, its theory and methodology]: Moscow-Leningrad. Translations from this book *in* Am. Soc. Civil Engineering, Manuals of Engineering Practice, no. 35, p. 53, 1957.

For the computation of elements of the free surface profiles in unsteady flow, the basic curves are developed and followed by the computation of instantaneous regimes. This is a method of plotting the instantaneous water levels in given times, with their control a posteriori, and is an approximate integration of differential equations of unsteady flow in finite differences form, with constant time intervals. The graphs of discharge and level distributions along the channel are obtained in the time moments which are selected in advance. The classical wave celerity formulas are discussed; these, according to the author, are valid only for small wave amplitudes, when flow resistance can be neglected. For wave amplitude greater than the original water depth, the author recommends a new celerity formula derived from continuity equation with the friction resistance taken into account, $C=V\pm\mu\sqrt{g(H_0+h)}$. The value of coefficient μ is given in function of the ratio $H_0/(H_0+h)$. For the wave in a dry bed, $\mu=2$, so that $C=2\sqrt{gh}$.

235. Drisko, J. B., 1933, Report on wave studies: On file with Mass. Inst. Tech., Civil Eng. Dept.

This report contains a summary discussion of various types of waves, for example, solitary, tidal, oscillatory, which is followed by a description of experiments made in a small channel. The experimental work covered solitary waves, waves propagated into a channel by tides, power- (increment-) waves, and drop- (decrement-) waves. Simultaneous stagetime graphs at four points, recorded on a chronograph, accompany the report. [Author's abstract.]

236. Drisko, J. B., 1933, Wave motion in a channel: Am. Geophys. Union Trans., n. 14, p. 516.

This paper discusses briefly the nature of powerwaves (abrupt increase of flow in a channel), mentions their relation to solitary waves of translation, and describes experimental work on various wave-types done in a small channel. The experimental work provided substantiation of J. Scott Russell's work with solitary waves, showed something of the nature of tidal waves propagated into a canal, and of the behavior of power waves. [Author's abstract.]

237. Egiazarov, I. V., 1933, Opisanie laboratoriy gosudarstvennogo nauchnoissledovatel'skogo instituta gidrotekhniki, Leningradskogo gidrotekhnicheskogo Instituta (VTUZ-a) [The description of the laboratory of government scientific institute for hydraulic engineering, or of the Leningrad hydraulic institute (VTUZ)]: Izdatel'stov Nauchno-issledovatel'skiy Inst. Gidrotekhniki [Leningrad], p. 1–192.

The experiments for unsteady flow are described, as are the measuring and recording devices used, and all laboratory activities involved.

238. Gibson, A. H., 1933, Construction and operation of tidal model of the Severn Estuary: Great Britain, H. M. Stationery Office, sec. 9.

The height and rate of travel of the bore is studied in the model of the Severn River. Despite the small dimensions of the model, close agreement is shown between phenomena observed in the river model and in the natural river. The speed of the bore depends on factors as indicated by

$$C=\sqrt{2g(H_0+h)^2/(2H_0+h)}-V,$$

whereas the actual celerity is approximately 95 percent of that given by this formula.

239. Khristianovich, S. A., 1933, Otrazhenie dlinnoy volny konechnoy amplitudy [Reflection of long wave of finite amplitude]: Gosudarstveniy Gidrologicheskiy Inst. [U.S.S.R.] Zapishi, v. 9, p. 27–66.

The paper deals with the reflection of a very long wave of finite amplitude which propagates in a prismatic channel. The method of characteristics is used, and the equations are linearized equations. The reflection caused by a sudden change of cross section is studied. The depths and velocities of the reflected wave correspond to those of the primitive wave. When the reflection is created by a wall, the corresponding depths are related by the equation

$$H_0=HH_1-4\sqrt{H_1H+H},$$

where H=primitive water depth in canal, H_1=depth of wave with fixed direction, and H_0=the corresponding depth in the section where reflection takes place. The solution is valid until the reflected wave is formed.

[From author's abstract, given in French at the end of the paper.]

240. King, W. H., 1933, Translatory waves in open channels: Civil Eng., v. 3, p. 319–321.

In general when the discharge in a channel is suddenly changed a positive or negative accelerating wave is formed. If the change acts to increase the depth, the wave has an abrupt face, and, if it acts to decrease the depth, the wave has a sloping face. A number of formulas for practical application are given. Channel friction has a modifying effect on the height and velocity of a wave and the profile of the water surface behind it. In deep water, for waves of moderate height, the velocity of wave varies approximately as the square root of the maximum depth of flow and can often be determined closely enough by neglecting channel friction. The principles involved in computing the water profile back of a wave by analysis of short reaches are comparatively simple, it is stated, but the computations may be long and tedious.

[Abstract by F. T. Mavis.]

241. Makkaveev, V. M., 1933, Gidromekhanicheskaya zadacha v probleme nagonnykh navodneniy [Hydromechanical task in the problem of surge inundations]: Sbornik rabot po zashchite Leningrada ot navodneniy. (Collection of works treating the protection of Leningrad from inundation) [Leningrad].

242. Puppini, U., 1933, Influenza sui regimi fluviali dell'esercizio non uniforme di impianti idraulici [The influence of unsteady powerplant operations on river regimes]: Elettrotecnica [Italy], v. 20, p. 505–510.

Effects of a periodic change of discharge are propagated along a channel. The paper stresses that the attenuation of a wave thus created is slow and that unsteady operation of a water powerplant will be felt many kilometers

downstream. Integration of partial differential equations is performed analytically and discussed. It is concluded that the velocity, depth, and discharge of waves propagate not only with attenuation, but with a change of form, which tends toward a pure sinusoidal form. An example is given of a flood of Adige River in Trento and Mori.

243. Sherman, L. K., 1933, Formula for wave velocity in open channels verified: Civil Eng., p. 473.

Discussing King's paper (1933), the author gives Graeff's (1875) or Seddon's formula (1899) as $C=(Q_2-Q_1)/(A_2-A_1)$, with its properties emphasized.

244. Thijsse, J. Th., 1933, Influence of the closing of the Zuyder Zee on the tidal regimen along Dutch coasts: Permanent Internat. Assoc. Navigation Cong. Bull., 8th year, no. 15, p. 59–82.

An explanation of the Zuyder Zee problem is given, followed by a study of unsteady flow. The study is confined to determination of the nature of a tidal wave that passes through a channel, setting up frictional resistance to its movement. Using Lorentz's linearization approach, the celerity of the wave is given by a new formula $C=\sqrt{gH \cos \epsilon}/\cos \frac{1}{2}\epsilon$ with $tg\epsilon=(8gVm)/(3\pi nc^2H)$, where $n=0$ for uniform movement and $\epsilon=\pi/2$, and where resistance is nil, $\epsilon=0$.

245. Trifonov, E. K., 1933, Materialy po eksperimentalnomu issledovaniyu dvizhenia polozhitelnoy volny po sukhomu dnu [Experimental investigation of propagation of positive wave along dry bottom]: Nauchno-issledovatelskiy Inst. Gidrotekhniky Izv., (Sci. Research Inst. Hydrotechnics Trans.) [Leningrad], v. 10, p. 169–188 (English summary p. 182–188).

The experiments of wave movement on a dry channel were made in a canal 0.4 meters wide and 30 meters long. The initial wave heights were 30 and 40 centimeters, with canal slope $S=0.004$. Changes of bed materials and of roughness were made. The number and purpose of experiments were limited and only general conclusions were made. Wave profiles were given but the formation of waves was not studied, and the shape of waveheads was determined by rather simple means. The conditions under which the experiments were made are described in detail.

246. Bachet, 1934, Note sur la propagation et l'annonce des crues [Note on the propagation and forecast of floods]: Ponts et Chaussées Annales [Paris], year 104, no. 34, p. 409–465.

The historical development of flood prediction on the Loire by the Federal Service is described. The early predictions were by the double-column tabular method of giving the effects of flood waves from two separate sources from which the maximum could be obtained. On the basis of an extended mathematical analysis, a method of graphical determination of flood heights by the use of transparent scales was developed. The effects of attenuation, overbank storage eddies at confluence, and multiple tributaries are analyzed separately and reduced to a straight-line diagram. In doing this for a large system of tributaries considerable judgment is necessary. An illustration is given of the application of this method to floods on the lower Loire River at Montjean, and the results are compared with actual floods. The positive and negative errors were found to be

evenly divided, in the neighborhood of 0.1 meter, and did not exceed 0.25 meter.

[Abstract by C. A. Wright.]

The discharge of unsteady flow, q, is classified as discharge of steady flow r and complementary discharge ρ (positive or negative). Using the equation $q=k\sqrt{S_0+\partial H/\partial x}$ it is found approximately that $\rho=(r/2S_0)\partial H/\partial x$. Any change of unsteady flow is considered to be accompanied by a vertical and a horizontal change. Passing from discharges to water depths, similar relations are obtained. The confluence of rivers is analyzed, and the relations for three stations are developed. In flood routing for flood forecasts, special templates are constructed, with the methods of construction given, and the example of the Loire River shown.

247. Bonvicini, D., 1934, Sulla propagazione delle perturbazioni di regime nei corsi a pelo libero [Propagation of perturbations in a stream with free surface]: Elettrotecnica [Italy], v. 21, p. 652–656; see Sci. Abs. B., v. 37, p. 692, 1934.

The problem of small fluctuations in the mean velocity and head of water in a free-flowing stream is studied. The loss of energy through friction is estimated according to the Manning formula and account is taken of any variation in the mean radius. It is concluded that waves propagate from upstream to downstream with small alternation. The integration is done by expressing hydraulic magnitudes, composed of values pertinent to steady flow, with additional components due, to unsteady flow. The solution of obtained partial differential equations is given in general form with real and complex numbers, then discussed, and examples are given.

248. Chary, 1934, Influence du calibrage partial d'un cours d'eau sur la propagation des crues [Influence of improvements in a water course on the propagation of flood waves]: Ponts et Chaussées Annales [Paris], year 104, no. 32, p. 394–399.

The object of this study was to determine what effect improvement of a river channel for the reduction of flood stages at one point had upon corresponding river stages at stations downstream. A graphical analysis is given, based upon two successive stations along a river, at which the stage-discharge relation and the inundation volumes are known. Discharge-time curves for a flood are drawn for the upstream station in relation to the downstream station, both before and after the improvements are made. The volume of inundation is determined in each case for a flat flood wave and for a sharp peaked flood wave. An example was given at Beuvron where the river valley inundation was reduced half by the improvements, without increasing the flood stage downstream from Beuvron by an appreciable amount.

[Abstract by C. A. Wright.]

249. Cornish, V., 1934, Ocean waves and kindred geophysical phenomena: Cambridge, Cambridge Univ. Press, p. 1–159.

The third chapter deals with bores and progressive waves in rivers. The creation of a progressive wave observed in the Rapids of Niagara is described. The excess velocity of roll waves over that of the current is in accordance with the formula given by J. S. Russell for the velocity of solitary waves. The friction of the bottom is regarded as the origin of the roll waves.

250. Crengg, H., 1934, Schwallversuche in Oberwassergraben des Murkraftwerkes Mixnitz-Frohnleiten [The experiments with surges in head canal of the hydroelectric power plant on the river Mur, Mixnitz-Frohnleiten]: Die Wasserwirsschaft [Germany], no. 22, p. 213.

The experiments are made for the movement of surges (positive and negative waves) in a canal with bottom width 6.20 meters; depth 4.70 meters; side slope 1:1.5; length, 6,950 meters; bottom slope 0.00025; and maximum discharge $Q=110$ m³ per sec. The experiments made involved nonuniform steady flow, with 0.69 meter and 2.62 meters of backwater at the powerplant, and with surface line slopes of 0.00184 and 0.0000475. The first experiment is made with the discharge decreasing from 94 m³ per sec to zero, in 2 sec (positive surge), and the second experiment with discharge decreasing from 60 m³ per sec to zero, in 2 sec. The results of wave movement are given and discussed.

251. Egiazarov, I. V., 1934, Sutochno regulirovanie Volkhovskoy gidroelektricheskoy stantsii [Analysis of pondage conditions at the Volkhov hydroelectric plant]: Nauchno-issledovatel'sky Inst. Gidrotekhniki Izv. (Sci. Research Inst. Hydrotechnics Trans.) [Leningrad], v. 12, p. 5–23 (brief abstract in English, p. 21–23).

The celerity of disturbances in headrace and tailrace canals is measured. The height of the wave created by discharge change in a powerplant was computed by author's formula (1924) as

$$\Delta h = k\Delta Q / B_m\sqrt{gH_m},$$

where k is a shape coefficient of wave form, and ΔQ is the change of discharge in time Δt, but for small waves

$$h = kW / B_m t\sqrt{gH_m},$$

where h is decrease of level in headrace canal, and W=volume of water passed in trailrace canal for time t. The analysis of level fluctuations in both canals is made on the basis of observed results. The observed wave heights check closely with those computed by the above formulas. Wave propagation in canals covered with ice is investigated.

252. Egiazarov, I. V., 1934, Gidroelektricheskie silovye ustanovki [Hydroelectric power plants, elements of water power development]: 3d ed., Moscow, Leningrad, v. 1, p. 137–152.

The book treats unsteady flow in the canals of water powerplants and presents a summarized exposition on wave movement and methods of computation.

253. Hegly, 1934, Sur la propagation d'une onde solitaire dans un canal réduit, à section trapézoidale [Propagation of a solitary wave in a reduced channel of trapezoidal section]: Acad. sci. [Paris] Comptes rendus, v. 199, p. 826–828.

Experiments using a model of a trapezoidal canal verify the basic formula for wave celerity $C=\sqrt{g(H_0+h)}$, and show small differences between computed and observed celerities.

254. Mastitskiy, N. V., 1934, Graficheskiy method postroeniya mgnovennykh prodol'nykh profiley svobodnoy poverkhnosti pri neustanovivshemsya dvizhenii vody v rechnom rusle [Graphical method of construction of

instantaneous longitudinal profiles of free surface during unsteady flow in the river channel]: Gidrotekhnicheskoe Stroitel'stvo [U.S.S.R.], no. 7, p. 7–14.

This paper presents a graphical flood-routing method, based on a continuity equation, in finite differences form, for selected elementary time intervals and reach lengths that uses the rating curves and the relation of channel storage to the water depth. This method is feasible only for the determination of discharges; for the computation of water depths, additional graphical plottings are necessary. The slope of water surface in the reach is taken into account by deriving the curve

$$2W/\Delta t = f(Q)$$

and by considering the wave celerity in the reach for different depths or discharges. Ten steps are given for the step-by-step computation.

255. Proskuryakov, B. V., 1934, Primer primeneniya metoda proektirovaniya neustanovivshegosiya dvizheniya potoka, predlozhennoga inzh. Bernadskim N. M. K raschetu katastroficheskogo povodka v sluchaye mgnovennog razrusheniya plotiny [An example of application of the method of computation of unsteady flow, proposed by engineer N. M. Bernadsky in paper entitled "To the computation of catastrophic flood in the case of instantaneous breach of a dam"]: Nauchno-issledovatel'skiy Inst. Gidrotekhniky [Leningrad] Izv. (Proc.), v. 11, p. 75–95.

This article contains an example of the method devised by N. M. Bernadsky for unsteady flow computations as applied to a particular case of flow resulting from a break in a dam. In the first part of the article a general description of the method is given. The method consists in simultaneously solving, graphically and by gradual approximations, the equations of continuity and equilibrium. Most terms in the dynamic differential equation for unsteady movement are neglected. The equations have the following shape:

$$AS = (V^2 B_m)/c^2 + (A/g)\partial V/\partial t$$

where $\partial H/\partial t$ = change of elevation in a unit of time; $\partial Q/\partial x$ = change of discharge along a unit of length of the river; B_m = average width of the channel; c = coefficient in Chezy's formula; g = acceleration of gravity; dV/dt = derivative of the velocity. At the same time, marginal conditions for the waves of upper and lower pools must be satisfied. These conditions are as follows: (a) equation of the celerity of propagation of the upper-pool wave, $C=\sqrt{gH}$; (b) equation of the celerity of propagation of the lower-pool wave, $C=2\sqrt{gH}$; (c) condition of continuity, which requires that the discharge of water contained in the propagating wave should be equal to that passed through the terminal part. The condition of constant equality of volumes of water existing in the lower pool and issuing from the upper pool, must be fulfilled when plotting each instantaneous surface level. The procedure consists of plotting the instantaneous surface with subsequent checks and requires satisfaction of all above-mentioned conditions. In the second part of the article a practical example is given of an application of the method to a case of dam breach. In the third part a comparison is made between instantaneous calculated levels and those measured in laboratory.

[From the author's abstract.]

256. Rzhanitsyn, N. A., 1934, Rechnaya gidravlika [River hydraulics]: Moscow Leningrad, pt. 1, p. 96–209.

Comparison of the natural observations of unsteady flow with the results of flood routing by Mastitskiy's method is given in this book. The agreement found was considered to be adequate, in view of likely inaccuracies in observations and in the selection of a reasonable time interval for flood routing. This book is an extensive treatment of unsteady flow, with a general discussion of flood-wave movement, of Mastitskiy's method, Bernadsky's method, and other methods of flood routing. In deriving the outflow hydrograph, the mathematical integration is shown for a given inflow hydrograph and for given storage-discharge relations. The author described his method of graphical integration of the simple equation. The wave created by dam breach is treated in chapter 4, with a summary of the results of previous studies of this subject.

257. Schultze, E., 1934, Die Bestimmung der Abflussverhaeltnisse in Tidegebiet [The determination of flow conditions in the tidal region]: Die Bautechnik [Germany], year 12, no. 34, p. 438–443, and no. 38, p. 493–497.

Results of measurements of velocity of tide movement (by floating object) at five places on a tidal river are given; these results show values obtained for discharge, mean velocity, and cross-sectional area. The current-meter measurements are also given for some tidal waves. The analysis of wave motion is made by two partial differential equations (dynamic and continuity), and harmonic analysis is introduced for tidal waves. The application is discussed, with the solution by numerical approximation, using harmonic analysis (taking into account wave reflection and attenuation), and using the equation for surge movement. Comparison is made of the results obtained by three methods of computation, and the need for model studies emphasized.

258. Thomas, H. A., 1934, The hydraulics of flood movements in rivers: Carnegie Inst. Tech. [Pittsburgh] Eng. Bull., p. 1–70 (1934, 1937). German abstract *in* V. D. I. Zeitschr., v. 79, no. 18, p. 559, 1935.

This report presents a systematic analysis of unsteady flow in rivers and of the approximate flood-routing methods that have been developed. The following are discussed: review of laws of steady and unsteady flows; dynamic propagation of stable wave forms; difficulties of integration by exact methods and boundary conditions; use of hydraulic models for unsteady flow (which is recommended for accurate flood routing); approximate methods of flood routing in uniform channels and in actual rivers. Three approximate methods are analyzed: first approximation, with simple storage equation for reach Δx and time interval Δt, based on the relationship of storage and outflow discharge (method found to be lacking in accuracy); second approximation, in which the slope of the reach is considered to be a straight line, with or without corrections for velocity head and acceleration term; and third approximation employing two differential equations for development of the finite differences method. The applicability of methods is discussed. Model tests are made to check the above methods.

259. Vreedenburgh, C. G. J., 1934, Een eenvondige afleiding van de formule van Airy voor de voort-plantingssnelheid van golven in water met eindige deipte [Simple derivation of Airy equation for celerity of water waves of finite depth]: Ingenieur [Netherlands], v. 49, no. 22, p. 212–213.

A theoretical derivation of celerity formula which is given as $C=\sqrt{(g\lambda/2\pi)\tanh(2\pi H/\lambda)}$, with a relationship between $\delta=H/\lambda$ and the celerity. For $C=\sqrt{gH}$ or $C=\sqrt{g(H_0+h)}$ (J. S. Russell formula), for $1/25<\delta<\frac{1}{2}$, C is the above formula with elliptic trajectories (Airy), and for $\delta>\frac{1}{2}$ $C=\sqrt{g\lambda/2\pi}$ with circular trajectories (Gerstner).

260. Egiazarov, I. V., 1934–35, Regulation of the water level in the reaches of canalized rivers and regulation of the flow below the last lock-dam according to whether the water power is used or not: Permanent Internat. Assoc. Navigation Cong., 16th, Brussels, 1934, Sec. 1, Question 2, and Nauchni-issledovatel'skiy Inst. Gidrotekhniki [Leningrad] Izv. (in Russian), v. 17.

261. Bernard, M. M., 1935, An approach to determinate stream flow: Am. Soc. Civil Engineers Trans., v. 100, p. 347–395.

A continuation of Sherman's study of the unit hydrograph, which introduces the distribution graph as a way of converting rainfall to stream flow. The distribution graph is shown to be a function of watershed characteristics, and the method of developing the distribution graph without recourse to streamflow records is demonstrated by use of the physiographic characteristics of six individual watersheds. Limitations of this method are discussed.

262. Biot, M. A., 1935, Quadratic wave equation-flood waves in a channel with quadratic friction: [U.S.] Natl. Acad. Sci. Proc., v. 21, p. 436–443.

An exact solution of the equation of wave propagation with quadratic damping is demonstrated. It shows that high amplitude waves are more quickly damped and that this damping effect depends on both the volume of wave and the friction coefficient of the channel. The solution may be interpreted physically as representing certain types of flood waves.

263. Brown, E. I., 1935, A study of the effect upon navigation and upon the upkeep of the banks and bed of canals and canalized rivers: a) of undulatory movements and of currents produced, in the reaches adjacent to locks with a high lift, particularly by the rapid filling and emptying of these works; b) of the rise or fall of the water surface caused by changes in the supply, whether natural or artificial, or by the action of the prevailing winds on long reaches: Permanent Internat. Assoc. Navigation Cong., 16th, Brussels, 1935, trans. (discussions).

The results of wave movement due to lock operations in the Panama Canal are given, together with observations of wave movement due to operations of a lock elsewhere in the United States. For the most part, the canal cross sections are large in comparison with the discharges, so that wave heights are relatively small.

264. Clemens, G. R., 1935, The reservoir as a flood-control structure: Am. Soc. Civil Engineers Trans., v. 100, p. 879–927.

A graphical method based on the storage equation, called "reach reservoir method," for flood routing in reservoirs and in valley storage is given. The curves $F_1=W_1+Q_1/2$ and $F_2=W_2-Q_2/2$ are used, as well as $Q=f(H)$ and $W=f(H)$, with known inflows. Curves of different scales can be plotted to increase the accuracy of the method. The time travel of the wave, the local inflows, and the valley-storage effect are discussed.

265. Dachler, Robert, 1935, Betrachtung ueber nicht-stationaeres Fliessen, im besonderem ueber die Hochwasserwelle [Deliberations on unsteady flow, especially the flood wave]: Wasserkraft u. Wasserwirtschaft [Germany], v. 30, no. 8, p. 88–91, and no. 9, p. 103–108.

The instantaneous slope in a cross section is $S=S_f+S_b+S_t$, in which S_f=friction resistance slope, $S_b=(1/2g)\partial(V^2)\partial x$, and $S_t=(1/g)\partial V/\partial t$. The relation of maxima for S, V, Q and H is shown, and the rating curve loop is discussed. The order of magnitude of slopes S_b and S_t is given in an example, and it is shown that they can be neglected when considering friction slope. An approximate graphical procedure is developed for the computation of wave attenuation. The celerity formula $C=(1/B)dQ/dH$ is redeveloped and discussed.

266. Dalle, Valle, and Visentini, M., 1935, Regulation of the water level in the reaches of canalized rivers and regulation of the flow below the last lock dam according to whether the water power is or is not used: Permanent Internat. Assoc. Navigation Cong., 16th, Brussels, 1935, Trans. (discussions).

The summary of experiments with wave movement on the Adige River (Italy) is given. The length of the reach for experiments was 100 km. Waves that are artificially created by sudden water release from the reservoir of Cardano powerplant are measured along the river, and the celerity and wave attenuation are studied.

267. Deymié, Ph., 1935, Propagation d'une intumescence allongée [Propagation of a long wave]: Rev. Gén. l'Hydraulique [France], v. 1, no. 3, p. 138–142.

Results of large scale experiments made on the Seine are discussed. It is rarely possible to operate on this scale, especially when it is necessary to use water carrying a heavy load of traffic. Experimental results agreed well the theoretical calculations. They were also compared with results obtained on the Aar River in Switzerland under similar conditions but with a steeper slope and a bed containing larger sized material. (Another series of observations and calculations for the Seine will follow in a later article.) The celerity computed by formula $C=V+\sqrt{gH}$ as 6.14 m per sec was confirmed by the experiments on the Seine. A function is introduced in the partial differential equation, so that $\partial\varphi/\partial t=Q$, and $\partial\varphi/\partial x=-H$. The partial differential equations of second order, which are linear in relation to second derivatives, are given. The author passes then to the characteristics which are given by three conditions with $dx/dt=V\pm\sqrt{gH}$. He concludes that the celerity of wave front is independent of the form of wave which follows the front, and of the flow resistance (with the assumption that first derivatives in (H, Q)-space stay continuous on the wave front). The discontinuities in h and Q propagate with celerity $V\pm\sqrt{gH}$. The computed h is found to be in agreement with the observed. Supported by analytical analysis, the wave is considered in three parts: front, with attenuation by exponential law $h=ae^{-bx}$; backfront, with attenuation as $e^{-kt}/t^{3/2}$, and central zone of the wave. The theoretical background is the Rieman-Hadamard method of integration.

268. Deymié, Ph., 1935, Note sur la propagation des intumescences allongées [An account of the propagation of elongated intumescences]: Permanent Internat. Assoc. Navigation Cong. Bull., 20 p. 64–70.

The differential equations of unsteady flow are reduced to an Ampere-Monge equation. It is deduced from the characteristics of this later equation that if there is initially a uniform regimen characterized by a velocity V_0, a depth H_0, and a quantity Q_0, the celerity C of the front of an intumescence is $C = V_0 + \sqrt{gH_0}$. This relation is independent of the shape and the height of the intumescence and of the extent of the friction. The propagation of discontinuities in Q and H obey the same law. The law of decrease in the frontal height of a discontinuous intumescence is not analyzed. The law of attenuation of the frontal height h for an intumescence for which the flow superposed on the initial regimen is constant and equal to Q_0 is $h = (Q'_0/V_0)e^{-ax}$ where a is a function of bed-slope S_0, of V_0 and H_0, the velocities and the depths of the primitive regimen, and x is the distance traversed by the front. There can be no steep back face to an intumescence produced by a suddenly applied constant excess flow of short duration.

[Abstract by G. H. Keulegan.]

269. Egiazarov, I. V., 1935, Teoreticheskoe i eksperimentalnoe issledovanie otritsatel'noy volny [Theoretical and experimental investigations of negative wave]: Gidroelektricheskaya Laboratorya (Hydroelectric Laboratory) [Leningrad] Bull. 5, paper 6.

This is an experimental study of the influence of friction and slope upon the propagation and shape of negative wave. The first approximations of these two influences are given by a number of formulas for wave shape and velocity distribution.

270. Evangelisti, Giuseppe, 1935, Lo studio dei fenomeni di colpo d'ariete per mezzo deo calcolo simbolico [The study of phenomena of water hammer treated by symbolic calculation]: Elettrotecnica [Italy], v. 22, p. 134–141.

A method of investigation is explained based on the employment of functional operators, with which can be treated the phenomena of propagation of hydrodynamic perturbations reducible to the equations of vibrating cords. Examples of practical interest are given which can be dealt with by this method in a relatively simple way compared with the usual means employed.

[Abstract by Joseph H. Sorenson.]

271. Favre, H., 1935, Étude théorique et expérimentale des ondes de translation dans les canaux decourverts [The theoretical and experimental study of translation waves in open channels]: Paris, Dunod, p. 1–209. See also a shorter version in Rev. Gen. l'Hydraulique, v. 1, no. 3, p. 157

The works of De Saint-Venant (1871) and Boussinesq (1877) are summarized. Gradually varied waves and surges, as well as their combinations, are analyzed, with some contributions in the discussions by the author. Of principal importance are the reported results of experiments, carried out in a canal, measuring 0.42, 0.40, and 73.58 meters (width, height, and length, respectively). Details of the measurements and experimental work are discussed. The given formulas of second approximation for positive surges are not valid as soon as $h > H/4$; The undular shape of nearly equal waves of wavefront is studied; for H given the ratio of small wavelength to mean height of wave increases by a decrease of mean wave height. For the negative surges the formulas are not applicable to experimental results as soon as $h > H/5$, with the shape of wave front changing by its progression. The experiments are well in agreement with the theory in the case of reflection of surges. For small h, the shapes of surges can be approximated by a straight line. An example for the computation of wave translation in a canal of water powerplant with trapezoidal cross section is given.

272. Heduy, M., 1935, A study of the effect upon navigation and upon the upkeep of the banks and bed of canals and canalized rivers—a symposium [see no. 263]: Permanent Internat. Assoc. Navigation Cong., 16th, Brussels, 1935, Trans. (discussions).

Starting from the elementary theory of wave movement in canals (based on the Lagrange celerity formula and the wave height created by discharge change ΔQ), the results of laboratory model studies (Sauley, Metz) and of observations in nature regarding wave movement are given. The study is made for wave heights that are small in comparison with the water depth, in which case it is found that the wave height is independent of time of creating ΔQ difference.

273. Koval'skiy, M. I., 1935, Issledovanie transformatsii rechnogo povodka graficheskim metodom N. V. Mastitskogo [Investigation of flood routing by Mastitskiy's graphical method]: Gidrotekhnicheskoe Stoitel'stvo [U.S.S.R.], no. 5, p. 7–12.

Natural observations of unsteady flow are compared with the results of flood routing by Mastitskiy's method. Agreement was found to be good within the limits of error incurred in observations and time-interval selection for routing. The Oka River reach from Kaluga to Aleksin is used, and flood waves of 1908 and 1932 are shown as examples. The basis for routing is the storage-discharge relationship, $W=f(Q)$, for the reach of river studied.

274. Kurihara, K., 1935, On the transmissibility of long waves along a canal when there are abrupt changes in depth: Japan Acad. [Japan] Proc., v. 11, no. 8, p. 316–318.

The problem of a partly reflected and partly transmitted water wave occurring at a point where there is an abrupt change in the cross section of the canal is expanded to the problem involving two such points, between which lies a reach of the canal that is either shallower or deeper than the adjoining reaches. It is concluded, by analysis, that the transmissibility is independent of the depth of the intervening reach. The maximum transmissibility occurs when the length of reach is an even multiple of one-fourth of the wavelength in that reach, and the minimum transmissibility occurs when the depth is an odd multiple of one-fourth of the wavelength.

275. Kuskov, L. S., 1935, K voprosu o dvizhenie polozhitel'noy volny po sukhomu dnu [On the problem of movement of positive wave along a dry bed]: Gidrotekhnicheskoe Stroitel'stvo [U.S.S.R.], no. 5.

The methods of Bernadsky (1933) and Cherkasov (1932) are described and analyzed. The author points out that the theoretical coefficient μ in Bernadsky's formula, given as $\mu=2$ for the positive wave moving in a dry bed, coincides with experimental results, if the roughness coefficient $n=0.010$–0.011. For waves moving in a dry bed, the author employs an empirical relation of μ in formula $C=\mu\sqrt{gh}$, and the roughness coefficient n, with mean values for $n=0.010$, $\mu=2.0$; for $n=0.017$, $\mu=1.01$; and for $n=0.0225$, $\mu=0.67$.

276. Lacaze, B., 1935, Une crue artificielle sur l'Aar [An artificial flood on River Aar]: Rev. Industrielle [France], v. 65, no. 2309, p. 161–171.

Report on artificial flood-wave produced on the Aar River, Switzerland, on February 6, 1920, based on report by K. Kobelt, published by Service Federal des Eaux, Switzerland; observations on progress on wave; bearing of observations on effect of pumped storage released into the river.

[Abstract by Joseph H. Sorenson.]

277. Lamoen, J., 1935, La propagation de la marée dans les fleuves maritimes et dans les bras de mer [Propagation of the tide in tidal rivers and in areas of the sea]: Liege Univ., Cours d'hydraulique Appliquée, Nat. Sci. Conf., 2d, Brussels, June 19–23, Contr., p. 15–22.

This article is a mathematical treatment of the tidal phenomena present in tidal rivers; special attention given to the Maritime Scheldt and its tributaries and to the Zuider Zee. The author gives a short summary of the theory of the oscillation of a heavy liquid in an open channel, discusses potential and kinetic energy of waves, and treats the calculation for compound channel cross sections. A short discussion is made of tidal conditions in the Zuider Zee and the Scheldt. Waves of translation are briefly described, and the author closes his paper with a paragraph on model reproduction of tidal rivers.

278. Massé, P. R., 1935, Sur une equation aux derivées partielles de la théorie des intumescences [On an equation with partial derivatives of the theory of intumescences]: Acad. sci. [Paris] Comptes rendus, v. 200, p. 109–110.

By integration of basic equations of unsteady flow, the following conclusions are drawn: the values of discharge, Q are attenuated at the front side of the wave according to exponential law; a flood wave having the initial propagation celerity is transformed by the effect of friction to a wave which travels at the assymptotic celerity $3V/2$, where V=the mean velocity of uniform flow; wave-level increase is governed by the same law as discharge increase, Q/h being equal to the celerity of the front of wave, which changes to $3V/2$ at the middle of intumescence.

279. Massé, P. R., 1935, Sur divers problemes aux limites de la théorie des intumescences [On different problems at the limits of the theory of intumescences]: Acad. sci. [Paris] Comptes rendus, v. 200, p. 376–377.

Referring to an earlier paper (1935, Comptes rendus) it is possible to relate the upstream and downstream wave propagation with the homologous points, $Q'=Qe^{-3ax}$, where Q'=discharge in upstream and Q in downstream direction, a=mean velocity of uniform flow, and x=distance along the canal. This relation is used for the theory of wave reflections. The addition of discharge, occurring in the middle of the canal, and of wave propagation in the canal, when the movement is steady but nonuniform, is studied analytically.

280. Massé, P. R., 1935, Thèse sur l'amortissement des intumescences qui se produisent dans les eaux courantes [Thesis on the attenuation of intumescences produced in flowing water]: Paris, Herman et Cie, Thesis series A, no. 1564, order 2430, p. 1–79.

The fundamental partial differential equations for unsteady flow are rederived, and the basic equation of characteristics is shown. By assuming small differences for depth, velocity, and discharge between steady and unsteady flow, a general partial differential equation of these departures is derived. By assuming a linear-type representation of that equation, the integration is made, and the problem of wave propagation in the downstream direction is solved by using complex variables. Special attention is given to study of the wavefront. The asymptotic intumescence corresponding to an initial limited disturbance is analyzed. These solutions are compared with those of Boussinesq (1877), and the differences are explained. The wave propagates so as to approach celerity to the limit $3V/2$, which makes (HS_0-cU^2) nearly zero, or so approaching the law of floods; C is resistance factor in Chezy's formula. Other problems studied are: Cauchy's problem of injection of water in the middle of channel with resultant creation of two waves; wave reflections; and wave propagation on nonuniform flow. Applications are given, and numerical solutions are shown, in the application of the theory. Experiments made on the Seine River, (those of Châtou), as well as those made on the Aar River, are discribed and compared with the theory.

281. Massé, P. R., 1935, L'amortissement des intumescences [The damping of translatory waves]: Rev. Gén. l'Hydraulique [France], v. 1, no. 6, p. 300–308.

Surges on rivers are studied analytically. The base is the partial differential equation of second order, linear, and having constant coefficients. It is called "equation of the intumescences" by the author. It is expressed in reduced values of time $t'=(\beta Vt)/(gS)$, and length

$$x'=\frac{\beta V}{gS}\sqrt{\frac{gH-\alpha V^2}{\beta}}\,x,$$

as

$$-\frac{\partial^2 Q}{\partial x^2}+2b\frac{\partial^2 Q}{\partial x \partial t}+\frac{\partial^2 Q}{\partial t^2}+2\frac{\partial Q}{\partial t}+3a\frac{\partial Q}{\partial x}=0;$$

where

$$b=1.027a, \text{ and } a=\frac{V\sqrt{\beta}}{\sqrt{gH-\alpha V^2}}, \ \alpha \text{ and } \beta \text{ are}$$

coefficients dependent on velocity distributions and are 1.096 and 1.040, respectively. The characteristics are C_1 and C_2, with $C=b+\sqrt{1+b^2}$. In flood, $C=(3/2)V$. The author uses the integration of the above equation based on the Poincare telegraph-equation method. The boundary conditions are determined by a convenient method of integration in the complex domain. The discussion of differences (of front celerity) between computed and observed values, as well as other aspects of movement of surges is given at the end of the paper.

282. Massé, P. R., 1935, Hydrodynamique fluviale, régimes variables [River hydrodynamics, variable regimes]: Paris, Hermann.

283. Norman, A., 1935, A study of the effect upon navigation and upon the upkeep of the banks and bed of canals and canalized rivers—a symposium [see No. 263]: Permanent Internat. Assoc. Navigation Cong., 16th Brussels, 1935, Trans. (discussions).

The results of experiments with waves produced in the natural canals (Trolhattan, Sweden) by lock operation are given.

284. Posey, C. J., 1935, Slide-rule for routing floods through storage reservoirs or lakes: Eng. News-Rec., v. 114, p. 580–581.

This method involves short, uniform time-increments and requires a separate set of scales for each reservoir having a different volume-depth or outflow-depth relation. Lengths along slide and stock represent total volumes as "day-second-feet," the former bearing a simple scale with graduations defined by $F_2 = W_2 + Q_2t/2$, and the latter bearing two opposed scales with a common origin defined by $I = Pt$, and $F_1 = W_1 - Q_2t/2$, in which P=average inflow rate, t=length of step (time increment), W_1 and W_2 are storage at beginning and end of step, respectively, and Q_1 and Q_2= outflow rates at beginning and end of step, respectively. The outflow rate is assumed to be a known function of the total storage, and the storage equation is of the form $(F_1 + F_2)$. Given the inflow rate and the outflow rate at the beginning of a step, the outflow rate at the end of the step can be obtained directly by means of the slide-rule or by means of a nomograph illustrated.

[Abstract by F. T. Mavis.]

285. Rappert, C., 1935, Nicht-stationaere Abflussvorgaenge in Fluessen [Unsteady flow in rivers]: Ver. Deutsch Ingenieure [Germany], v. 79, no. 18, p. 559.

Thomas' study of the hydraulics of flood movements in rivers (1934) is discussed briefly.

286. Schaffernak, Friedrich, 1935, Hydrographie [Hydrology]: Vienna, Julius Springer, p. 1–438.

Uses of mass curves of inflow or outflow with continuity equation applied to flood routing are discussed (p. 243–253). The stage relationship along a river is shown (p. 287–288 and p. 375–381). Routing of flood waves through reservoirs, based on the continuity equation, is shown for different cases, when some relationships are known and two remaining relationships are to be determined. Two graphical methods are used: (1) inflow and outflow hydrographs and (2) the mass curves of these hydrographs (p. 393–426). Many graphical short cuts are given.

287. Simonov, W. P., 1935, O rasprostranenii dlinnoy volny malogo podema odnogo napravleniya [On the propagation of long waves of small amplitude in one direction]: Zapiski Gosudarsvenogo Gidrologicheskogo Inst. [U.S.S.R.], v. 14, p. 125–199.

The author treats the influence of resistance on long wave propagation in one direction in a prismatic channel. The flow resistance is assumed to be proportional to the mean velocity, and the amplitude of the wave is assumed to be small. The equations of characteristics are developed and used. The method of successive approximation is applied, but the series obtained by this method are inconvenient for the computation. The reconstruction of the series by which they are made convergent and more convenient for the computation, is given in this paper. A particular problem, solved numerically, is given as an example.

288. Sum, T., 1935, A study of the effect upon navigation and upon the upkeep of the banks and bed of canals and canalized rivers—a symposium [see no. 263]: Permanent Internat. Assoc. Navigation Cong., 16th, Brussels, 1935, Trans. (discussions).

The experiments with waves caused by lock operations in the Horin lock in Czechoslovakia are given.

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289. Thijsse, J. Th., 1935, A study of the effect upon navigation and upon the upkeep of the banks and bed of canals and canalized rivers—a symposium [see no. 263]: Permanent Internat. Assoc. Navigation Cong., 16th, Brussels, 1935, Trans. (discussions).

The paper deals with the influence of wave height on the wave celerity and with the attenuation of wave height by flow resistance. For wave heights at the distance x from the wave origin or when wave heights are known, the formula is given (without derivation) as $h=h_0/(1+gh_0x/\phi c^2RH)$, where h_0=maximum wave height, h=any wave height, c=Chezy's coefficient, $\phi=1.5$ (or 3.0), and the influence of flow resistance on celerity is given as

$$C=\sqrt{gH}\sqrt{\cos\,\theta}/\cos\,(\tfrac{1}{2}\theta),\text{ where }\tan\,\theta=3gV_{\max}/8\pi nc^2H,$$

and where n=angular velocity of wave for the formulas of periods $T=2\pi/n$. The formulas were derived in 1924 and 1933. The results agree well with the observations in Zuider Zee. Details are given in the description of the wave reflections caused by narrowing or broadening of a channel, with the use of coefficient $\beta=B_2C_2/B_1C_1$. The comparison with observations is given.

290. Voegerl, Franz, 1935, Beitrag und Beispiel zur Schwallberechnung [Contribution and example for wave computation]: Wasserkraft und Wasserwirtschaft [Munich], v. 30, no. 22, p. 257–262.

The celerities, heights, and velocities of surges produced in canals of trapezoidal cross section are derived as a function of cross-sectional characteristics and as a function of created surges. The celerities are given in a general form, with $C=V\pm\sqrt{gA/B}$ as a special case for small surge heights. The surges in reservoirs and rivers that move along backwater curves are studied, and formulas for wave heights and celerities are derived. An example is given, and the results of wave experiments in the Maxnitz-Forhnleiten Canal are discussed.

291. Weber, C., Kanalwellen mit geringer Wellenhoehe [Canal waves with small wave height]: Zeitschr. Angew. Math. Mech. [Berlin], v. 15, no. 6, p. 380–381.

The potential movement of ideal liquid in a canal is studied analytically, with assumption that the movements are small, so that the equation for motion can be taken without the quadratic term. Three cases are studied: (1) when the gate at the end of canal does not move; (2) when the gate moves periodically; and (3) when the gate is motionless prior to the moment t, then is in motion from t to t_1.

292. Weidner, M. E., 1935, A study of the effect upon navigation and upon the upkeep of the banks and bed of canals and canalized rivers—a symposium [see no. 263]: Permanent Internat. Assoc. Navigation Cong., 16th, Brussels, 1935, Trans., p. 127–136 (discussions).

The results of experiments with wave movements in canals in the natural state are given for the canal from Wesel to Datteln; the canalized part of Neckar; the canalized Main River; and the Danube at the Kochletlock at Passau. No generalization is made, except by comparing the results of observations with the maximum wave height, as computed by formula $h=Q/(B\sqrt{gH}-BV)$, and with the celerity $C=\sqrt{gH}-V$ (the two values were close because the wave heights were small).

293. Angelini, A. M., 1936, Sulla propazione delle perturbazioni nei corsi d'acqua [Propagation of perturbations in water courses]: Elettrotecnica [Italy], v. 23, p. 486–494.

Starting with the simplified equations of disturbed flow in the channels, obtained by Puppini (1933) with the small intumescences, the propagation law for waves of small variations in discharge and depth is determined for any wave shape (periodic or nonperiodic). A graphical procedure is given for the practical solution of problems in order to avoid laborious computations. The graphical procedure is also applied to a more complex and more general case for any variation of form and amplitude. A numerical example is given and some general considerations are derived from the computation. The computation is made by using the method of operators (developed by Heaviside, Giorgi, and others), which is feasible for these problems. Some properties of operational calculus are shown and interpreted.

294. Bolotov, V. V., 1936, Prakticheskiy metod rascheta kolebaniy v byefakh gidrostantsiy pri sutochnom regulirovanii [Practical method of calculating water variations in pools of hydroelectric plant under daily regulation]: Industrial'niy Inst. Trudy [Leningrad], no. 11, Razdel gidrotekhnike, Vypusk, 1, p. 3–32.

The author analyzes the records of the Volkhow and Svir hydroelectric plants. He concludes that the elevations of the water level in the upper and lower pools may be represented as a straight line function depending on two factors: water capacity of the plant and additional water volume received from its upper pool. The author suggests a practical method of adopting, for his formulas, numerical coefficients developed from the operational records. As a result, he recommends this relationship as a method for calculating water-level variations in pools of hydroelectric plants, when their rates of water-capacity variations have been adopted.

[From author's summary.]

295. Brown, E. I., 1936, Flow in tidal rivers: U.S. Engineer School, Fort Belvoir, Va. Engineering Procedure as Applied to Tidal Canals and Estuaries, Pt. 2.

Propagation of the tide in rivers. Theory of river tides. Consideration of termination of estuary in a dead end by a fall or a dam, which will be the case of a fully reflected wave in the indefinite canal. Consideration of case in which estuary merges without interruption into bed of a river free of tidal effects. Consideration of case where maritime portion is limited by dam or other obstruction reflecting primary wave, but with addition of fluvial flow.

[Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, February 1954.]

296. Chertousov, M. D., 1936, Opredelenie maksimal'noy otmetki urovnya vody v derivatsionnykh kanalakh GES pri sbrose nagruzki stantsii [Determination of maximum elevation of water surface in headrace canals of hydroelectric power plants during shutdowns]: Nauchno-issledovatel'skiy Inst. Gidrotekhniki [Leningrad] Izv. (Sci. Research Inst. Hydrotechnics Trans.), v. 19, p. 82–104 (English summary, p. 103–104).

The author contends that the maximum level created in the headrace will be at the upstream end of the canal in the case of the sudden closing of gates of a powerplant. Using Crengg's experiments (1934) and the experiment in the Niva no. 2 canal, it is shown that in actual conditions the rise of the water level in the forebay, near the plant, continues during a sufficiently long period of time after the moment when the positive wave reaches the upstream end of the canal. The maximum elevation in the forebay is higher than that at the upstream end. The author proposes to determine the maximum elevation at the forebays at the time that a negative wave, originating at the upstream end of the canal, reaches the forebay: $H_{max}=H_u+(H_2-H_1)T_2/t$, where H_{max}=maximum elevation at forebay, H_u=elevation at forebay when the positive wave reaches the upstream end, H_2 and H_1, the elevations at two intermediate points of canals when the positive wave passes them, t=travel time of the positive wave between sections 1 and 2, and T_2=travel time of the negative wave along the entire canal.

297. Egiazarov, I. V., 1936, Neustanovishegosya volnovoga dvizheniya v dlinnykh byefakh [Unsteady wave movements in long pools]: Nauchno-issledovatel'skiy Inst. Gidrotekniki [Leningrad] Izv. (Sci. Research Inst. Hydrotechnics Trans.), v. 16 p. 105–138 (English summary, p. 135–138).

The first part is a presentation of the papers read at the 16th Congress of the Permanent International Association for Navigation Congresses. The second part deals with the results of investigation of wave movement, studied theoretically and in nature by the Hydroelectric Laboratory in Leningrad. Intumescences are evidences of degeneration occurring in positive and negative waves. The celerities of these waves are analyzed, with some simplification of earlier formulas, with $C=\sqrt{gH}(1+h'/H)$, where h' is the mean height of wave $(h=3h'/2)$. The reflection of waves is discussed. The comparison of theoretical results with results of observation of waves in canals is made with good agreement. The study is made of the shape of the head of a positive wave, the reflection of waves from great water surfaces, the maximum increase of elevation, and the influence of a broadened part of a canal on waves.

298. Favre, H., 1936, Le problème des vagues [The wave problem]: Schweiz. Bauzeitung [Switzerland], v. 108, no. 1, p. 1–4, and no. 2, p. 18–20.

The propagation of waves in deep and shallow water is studied. The celerity $C=\sqrt{(gL)/2\pi}$, for deep water with length L, is compared with more exact formulas and with results of observations. The reflection of waves is discussed.

299. Fison, David, 1936, Manner of flow of river in flood: Inst. of Engineers [Australia] Jour., v. 5, no. 3, p. 73, and v. 8, no. 5, p. 161–169.

Some views of the manner of flow of a river in flood are advanced in the first part of this paper. In the second part these views are treated somewhat differently, modified, and brought nearer to practical application. It is shown: (1) that the speed of increments of the rate of discharge of a river, through any cross section, is equal to the differential coefficient with regard to height of the rating curve for that cross section, divided by the surface width at the cross section; (2) that there is strong evidence that the rating curves, that is, height against discharge, at average cross sections are curves of sectional areas multiplied by a constant for the section;

(3) that variable flow in a stream can be measured without recourse to current meters or any current measurements whatsoever; and (4) that temporary storage, as separated, by definition, from less frequent classes of valley storage, does not reduce the maximum rate of discharge of a river.

A method of delineating flood flow is illustrated that permits the study of the behavior of flood water in very accurate detail.

300. Horton, R. E., 1936, Natural stream channel-storage: Am. Geophys. Union Trans., pt. II, July, p. 406–418, (corrections 1937, p. 440–441).

The relationship, $Q = KW^M$, between the river discharge Q at the recession part of hydrograph (with ground-water flows deducted) and the total upstream channel storage, W, is analyzed for different streams. The parameters K and M are determined, and their variations are discussed.

301. International Boundary Commission, U.S. and Mexico, 1936, Flow of the Rio Grande and tributary contributions: U.S. Dept. State, Water Bull., no. 6, p. 82–94.

Data is given that shows the approximate travel time and flattening of flood crests on the Rio Grande.

302. Khristianovich, S. A., 1936, Sur le calcul des mouvements non permanents dans les canaux et les rivières [On the computation of unsteady flow in canals and rivers]: Russian edition: O raschete neustanovivshikhsya dvizheniy v kanalakh i rekakh: Hydrological Conference of Baltic Countries, 5th, Finland, 1936, report no. 16, D. p. 1–40; Russian ed.: Gosudarstveniy Gidrologicheskiy Inst. Trudy [U.S.S.R.], v. 5, p. 25–53 1937.

The equations of characteristics are developed from partial differential equations of unsteady flow and are discussed, first with the friction included, then neglected. Four theorems are given for the solution of equations of characteristics, with the analysis of the positive downstream and upstream waves, for mixed waves, and for waves not subject to destruction. The surges as shock waves are studied. The following cases of long waves are discussed: propagation of the waves in one direction, the meeting of two opposite waves, the decomposition of nonpermanent movement in both positive and negative waves, and the reflection of a long wave. The approximate method of computation of a long wave by method of characteristics is given for the case when friction resistance can be neglected; a numerical example is shown.

303. Khristianovich, S. A., 1936, Razlozhenie neustanovivshegosya dvizheniya v kanale na pryamuyu i obratnuyu volnu [Decomposition of unsteady movement in canals in a direct and a reverse wave]: Gosudarstveniy Gidrologicheskiy Inst. [U.S.S.R.], Zapiski, v. 15.

304. Khristianovich, S. A., 1936, O volnakh, voznikayushchik pri razrushenii plotiny [On the waves created by dam breaches]: Gosudarstveniy Gidrologicheskiy Inst. [U.S.S.R.] Zapiski, v. 15.

305. Lamoen, J., 1936, Sur l'hydraulique des fleuves à marées [On the hydraulics of rivers with tidal effects]: Rev. Gen. Hydraulique [France], nos. 10, 11, 12, p. 533–545, 595–600, and 643–654.

The forecast of tidal movements in estuaries after the rivers are corrected (Zuider Zee closure) is the subject of the paper. The general properties of tides in rivers are discussed. The method of determining the instan-

taneous discharges in a tidal river is given. Tidal currents and shapes of beds of tidal rivers are analyzed. Tide graphs are discussed and the theory of water oscillations in a canal is given, as are two partial differential equations for unsteady flow and the expression for potential energy of waves. The law of similitude is given for the studied phenomena. Propagation of tidal waves along rivers is theoretically studied, and the formulas of wave status are developed. After introduction of the flow resistance, the partial differential equation is reduced to that of the telegraph. The case of inclined channel is analyzed. The cases of a horizontal canal with flow resistance taken into consideration; a canal without upstream inflow, the tides of Zuider Zee, and a canal where the upstream inflows may not be neglected are developed and discussed. The results depend mainly on the accuracy of determined flow resistance coefficients.

306. Lane, E. W., and Baldwin, O. T., 1936 Flush wave velocities in sewers: Eng. News-Rec., June 11, p. 848–849.

Describes the results of experiments on waves produced by flush tanks in sewers with very flat gradients, including diagrams showing the formation of the wave front and the velocity of movement. Shows why the wave front moves faster than the velocity of the water forming the wave.

[Abstract by E. W. Lane.]

307. Massé, P. R., 1936, Le problème du mascaret [The problem of tidal bore]: Rev. Gen. Hydraulique [France], v. 2, no. 9, p. 489–498.

The author continues his investigation of this tidal bore as a follow up of his paper of 1935. The assumptions for the equation of intumescences are discussed ($V=V_0+v$, and $H=H_0+h$, in which v and h are small values, as differences among the unsteady and steady flows). Lorentz's introduction of linearization of resistance losses: $kV=(gbV^2)/H$, in which $k=(8gbV_{max})/(3\pi H)$, is discussed; its introduction reduces the equation to the form of the telegraph equation. The history of studies of mascaret (tidal bore) is given. The theory of tidal bore is developed, based on the theory of wave shock in movement of gases (according to Hugeniot). The celerity of the wave front is assumed to be $V+\sqrt{gH}$ and that of the body of the wave to be $3V/2$. The method of characteristics is used. The integrated equation is given, then applied to floods and to tidal movements in rivers. The point x is determined where the singularity in tidal movement appears as a bore. The equation for different cases is discussed, and the applications are made using observed data on tidal bores of the Seine and Dordogne Rivers, which show good agreement when compared with the results computed by this method.

308. Meinzer, O. E., Cady, R. C., Leggette, R. M., and Fishel, V. C., 1936, The channel-storage method of determining effluent seepage: Am. Geophys. Union Trans., v. 17, p. 415–418.

For a selected small river, the relation of the gage height to the total upstream channel storage is derived as a straight line, showing departures up to 7 percent.

309. Pettis, C. R., 1936, Relation of rainfall to flood runoff: Mil. Engineer, v. 28, p. 94–98.

One diagram of a composite flood-wave shows wave velocities in terms of the average velocity of the entire cross-section at crest. This diagram shows that the average ratio of wave-velocity to average discharge-velocity

at crest is 1.35. Much of the discussion is relative to the author's width formula for floods, originated in 1927 and published in Engineering News-Record, June 21, 1934.
[Abstract by F. T. Mavis.]

310. Shilov, V. T., 1936, Issledovanie neustanovivshegosya dvizheniya v derivatsionnom kanale gidrostantsii Niva II [Study of unsteady flow in the canals of water powerplant Niva II]: Gidrotekhnicheskoe Stoitel'stvo [U.S.S.R.], no. 12, p. 26–30.

The experimental results of wave movement in the Niva II powerplant canal are given and then compared with the theoretical formulas for celerity and wave heights. It is found that celerity is in all cases smaller than that computed by the Lagrange formula, and the author supposes that this is due to the neglect of friction resistance in the theoretical studies. The wave heights computed by Johnson's and Egiazarov's formulas were close to the observed values.

311. U.S. Army Corps of Engineers, 1936, Method of flood routing: U.S. Army Corps of Engineers, Zanesville Dist., App. 5 of Survey Rept. on Muskingum River (report not published; mimeo. copies distributed with Corps of Engineers Circular letter R and H, no. 47.)

312. U.S. Army Corps of Engineers, 1936, Flood routing method of computing reservoir effects: U.S. Army Corps of Engineers, "308" report on Ohio River below Wheeling, W. Va., U.S. 74th Cong., 1st sess., House Doc. 306.

Appendix A, Details in connection with flood control, contains the computation of storage curves, routing tables, natural inflow tables, and modified flows.

313. U.S. Army Corps of Engineers, 1936, Method of flood routing: U.S. Army Corps of Engineers, Providence Dist., sec. 1, v. 1 (App. G to Rept. on Survey for Flood Control, Conn. River Valley, 1936), p. 1–13.

The influence of changing slope in a reach for flood routing is herein considered. Though theoretically not correct, the assumption is advanced that the ratio of valley storage to a weighted flow determined from both inflow and outflow is constant for any reach and is dependent upon the physical shape of the valley within the reach. This assumption is shown to be practically acceptable because it permits a good approximation. The principle given as a formula is

$$K=\frac{T[0.5(P_2+P_1)-0.5(Q_2+Q_1)]}{X(P_2-P_1)+(1.0-X)(Q_2-Q_1)}$$

in which K=ratio of storage increment in reach in day-second-feet to corresponding flow increment in second-feet; T=time unit of computation in days or fractions of a day; X=fractions of weighted flow increment that is derived from inflow increment. The general procedure of flood routing is described and application of method to Dover-Newcomerstown reach is given.

314. Bergeron, Louis, 1937, Méthode graphique générale de calcul des propagations d'ondes planes [General graphical method of computation of the propagation of plane waves]: Soc. Ingénieurs Civils [France] Mém., year 90, no. 4, p. 407–497.

This report presents the graphical method of study of wave propagation and is of general interest. In pages 476–480 the positive and negative surges in channels (bore and depression) are studied by this method, using the celerity formula $C=\sqrt{gH}$. An example is given of the movement of a positive wave in a canal, Q, discharge increasing from 0 to Q, and from the time 0 to time, 8 time-units $1/\sqrt{gH_0}$ later, where 1 is a selective unit reach length of the canal, and H_0=the initial depth.

315. Cagniard, L., 1937, Hydrodynamique fluviale; Régimes variables [Fluvial hydrodynamics; variable regimes]: Rev. Gén. Hydraulique [France], v. 3, no. 15, p. 128–136.

The author continues the discussion of Massé's approach to the problem of intumescences, given in his paper of 1935. Using the same basic differential equation and reduced values as Massé used, the author starts with the Carson method of integration and arrives at an expression for elevations h. He first discusses the wave movement in the downstream direction, with wavefront and discontinuities, and the pseudowave called the "flood wave." Then the movement of the wave in the upstream direction is analyzed. The corresponding formulas and characteristic curves are developed for both cases for h and H.

316. Callet, 1937, Note sur la propagation des crues [Note on the propagation of floods]: Ponts et Chaussées Annales [Paris], year 107, no. 3, p. 37–46.

The author starts from Bachet's work (1934) and his formula for discharge of steady flow $\Delta r=(\partial\rho/\partial x)\Delta x$, in which ρ=complementary discharge. Using a reach of the river as an example, he finds that ρ is small, and $\partial H/\partial x$ is only 5 percent of the river slope. By the analysis of other factors, he finds that

$$\Delta r=-(\Delta x/2S_0)\partial[(r/c^2)\partial H/\partial t]/\partial t$$

and with ΔH=difference of levels between two stations, and T=travel time between them, then for Q_{max} and Q_{min}

$$\Delta r=-(1/2\Delta H)\partial[T^2r\partial H/\partial t]/\partial t\cong-(T^2/2\Delta H)r\partial^2H/\partial t^2,$$

with the approximate expression $\Delta r=(-T^2r\gamma)/(2\Delta H)$, where γ=curvature of the stage hydrograph at the peak. The application is given for the Rhine River.

317. Drioli, Carlo, 1937, Esperienze sul moto perturbato nei canali industriali [Experiments on surges in industrial canals]: Energia Elettrica [Italy], v. 14, no. 4, pt. 1, p. 285–305, and app., p. 306–311; and no. 5, pt. 2, p. 382–402.

318. Egiazarov, I. V., 1937, Teoreticheskiy raschet neustanovivshegosya volnovogo dvizheniya v dlinnykh byefakh i ego sopostavlenie s eksperimentom i naturoy [Theoretical computation of unsteady wave movement in long canals and its comparison with experiments and nature]: Gidrotekhnicheskoe Stroitel'stvo [U.S.S.R.], no. 6, p. 36–42, and no. 7, p. 19–25.

The movement of steep positive and negative waves along the pools and channels is studied. The author gives his equation relating H_0, h, and ΔQ, and the equation of traveled length x by the wave in function of all involved parameters. The experimental data is compared with the computed values, with good agreement of two. The reflection of surge waves is studied, as is the fluctuation created by wave reflection. The computa-

tion method for wave movement is given in the case in which a reflection exists. The condition of friction resistance by wave reflection is analyzed. A comparison is given of theoretically determined values with the values from nature, by considering slope, resistance, and reflection. There is sufficient agreement between them. A simplified computational method, using the Maxnitz Canal experiments as an example, is given.

319. Egiazarov, I. V., 1937, Neustanovivsheesya dvizhenie v dlinnykh byefakh [Unsteady wave motion in long pools]: Izv. Nauchno-issledovatel'skogo Inst. Gidrotekhniki [U.S.S.R.], v. 21, p. 57–129.

The first part of the paper is a historical analysis of studies in movement of surge waves that includes a physical picture of surge-wave movement in long pools, and the names for differenct parts of waves are given. There follows a summary of some other of the author's works and of his formulas. The third part deals with experiments of positive and negative waves, which are compared with theoretical results based on the author's formula. Good agreement is shown. The fourth part deals with the total or partial reflection of waves and with the effect of flow resistance on the propagation and fluctuation of waves, created by reflections. The fifth part contains the comparison of results of experiments in large natural channels with theoretical results and results of laboratory experiments. The sixth part gives the simplified computations and shows their practical significance. The last part treats the simularity laws for model studies of unsteady flow and discusses the program of future investigations. Extensive conclusions and summaries are included; a summary in English (p. 129–132) and 131 bibliographical entries (p. 125–129).

320. Goodridge, R. S., 1937 A graphic method of routing floods through reservoirs: Am. Geophys. Union Trans., v. 18, p. 433–440.

A semigraphical method for integrating the storage equation is presented. For given inflow hydrograph, storage-elevation function and outflow discharge-elevation function (rating curve), the outflow hydrograph and storage-time function are determined. The shortcut method described uses a process of direct integration, without employing mass curves. It is based on the use of a selected time unit Δt (which is variable) for the use equation $W=Q\Delta t$ and uses T as a constant (time required to fill a given volume by a given discharge). T depends on the selected increments for storage and discharge.

321. Horton, R. E., 1937, Natural stream channel-storage (second paper): Am. Geophys. Union Trans., v. 18, p. 440–456.

Corrections of the statements in the author's paper of 1936 are given. It is shown that the observed values of M in the storage-outflow discharge function $Q=KW^M$ are in good agreement with the values to be expected from hydraulic considerations.

Data given, derived from analyses of stream rises, show the volume of channel storage during recession when the channel outflow rate is 1 cu ft per sec per mile. It is shown that volume of channel storage is related both to the size of the drainage basin and the stream density. It is also shown that the rating curve of the channel storage-outflow for a complete rise, including both rising and receding stages, usually has the form of a hysteresis loop. It is shown that channel storage was concentrated in a single detention reservoir having the same volume and water-surface area as that of the total channel storage and the same outflow capacity. This relation makes it possible to

treat analytically the subject of outflow graphs as modulated by channel storage. The effect of channel storage in reducing flood crests and regulating outflow is complicated, in most cases, by the presence of groundwater inflow and outflow. Expressions are derived for channel outflow for both the rising and receding sides of the channel outflow hydrograph, and examples are given showing the effect of a constant inflow of ground water.

322. Kertselli, S. A., 1937, Tablitsy dlya vychisleniya skorostey rasprostraneniya frontov voln po potprtomu byefu dlya pryamougol'nogo i parabolicheskogo rusel [Tables for calculations of unsteady flow in open channels of rectangular and parabolic cross section]: Gosudarstveniy Gidrologicheskiy Inst. [U.S.S.R.] Trudy, v. 5, p. 54–61.

This paper contains a table of the integrals

$$A_x(\eta)=\int\frac{\eta^{\frac{x-1}{2}}}{\eta^x-1}\,d\eta, \text{ and } C_x(\eta)=\int\frac{\eta^{x-\frac{1}{2}}}{\eta^x-1}\,d\eta, \text{ for } x=3,\ x=4$$

which are required for the computation of unsteady flow in prismatic open channels, with friction being taken into account. The case $x=3$ pertains to the rectangular cross section, and $x=4$ to the parabolic prismatic cross section. The integrals correspond to the Bakhmetev function formulas.

323. Khristianovich, S. A., 1937, Razrushenie dlinnoy volny odnogo napravleniya [Direction reversal of a long wave with a unique direction]: Gosudarstveniy Gidrologicheskiy Inst. [U.S.S.R.] Trudy, v. 5, p. 62–70.

The paper is a study of unsteady flow in a prismatic channel for the case when a long wave progressing in one direction is reversed. This motion is a "wave in one direction" and is the same as that produced in a channel by a dam breach, or as that of a permanent "shock wave," but the composition of the reversed long wave is changed. The depth of the resultant "shock wave" is not constant, but is variable with time; this variability depends essentially on the wave form that has created the reversal. With time, the "shock wave" depth may approximate zero.

324. Lane, E. W., 1937, Predicting stages for the lower Mississippi; Civil Eng., v. 7, p. 122–125.

This article gives a method of predicting stages on the lower Mississippi, based on the stage-relation method. It describes the method used for reaches between which there are no important tributaries entering the stream and also where important tributaries join. It shows the result of the application of the methods to previous floods and the procedure by which the lowering due to crevasses in past floods can be computed. Data are also given on the time of travel for flood peaks of various heights.

[Author's abstract.]

325. Massé, P. R., 1937, Des intumescences dans les torrents [Translatory waves on torrents]: Rev. Gén. Hydraulique [France], v. 3, no. 18, p. 305–306.

Deymié's (1935) exponential law of wave-front absorption with coefficient $(gS/VC)(1-V/2\sqrt{gh})$ is stressed. It is shown that after travelling some distance the wave front becomes imperceptible. The wave then propagates with $C=3V/2$, in which the attenuation is expressed as the inverse of the square root of the length. In upstream direction, attenuation always follows the exponential law. Wave propagation along torrents is similar to wave propagation on rivers. The author concluded that if $\bar{V}=\sqrt{gH}$, the wave cannot propagate a torrent upstream. He introduced the other limit, $C=2\sqrt{gh}$, above which the torrent is rapid and below which the torrent is moderate. Below the limit the wave attenuates as in a river, but much more slowly. Above the limit the coefficient of absorption changes sign, the asymptotic celerity $3\bar{V}/2$ is greater than $\bar{V}+\sqrt{gh}$, and the wave front increases exponentially, becoming a bore. He suggests the experiments to verify this theory.

326. Massé, P. R., 1937, The problem of river-bores: Permanent Internat. Assoc. Navigation Cong. Bull., no. 23, p. 59–68.

General theory of variable regimens in fluvial hydrodynamics and its application to problem of river-bores. Principal aspects of phenomenon and explanation of same.

[Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, February 1954.]

327. Schoklitsch, Armin, 1937, Hydraulic Structures (translated from the German by Sanuel Shulits): Am. Soc. of Mech. Engineers [New York], v. 1, p. 65–67.

The mass-curve method of graphical integration of the storage-differential equation is presented.

328. Sidenko, A. I., 1937, K analizu neustanovivshegosya dvizheniya po issled ovaniyam v nature [On the analysis of unsteady flow by field measurements]: Izv. Nauchnoissledovatel'skogo Inst. Gidrotekhniki [U.S.S.R.], v. 20, p. 112–131 (English summary, p. 131–133).

Theoretical values of wave motion computed by Egiazarov method (1937) are compared with actual observations at the White Sea Canal. The experiments are made in the canal 1,400 meters long and 42 meters wide, and results for negative wave are compared with those obtained by formula

$$\Delta Q=2bH\sqrt{gH}\,(1-h/H)(1-\sqrt{1-h/H},$$

given by Egiazarov, and $x=(t-t_1)C_1$ with

$$C=3\sqrt{g(H-y)}-2\sqrt{g(H+h)},$$

given by De Saint-Venant (1871). The formulas are given for positive waves employing similar comparisons. A formula is developed for Σy, the height at wavefront. Nine experiments are compared with computed values, and departures are discussed. It is concluded that the two groups of results are in satisfactory agreement, and Egiazarov equations are considered to be a good approximation. The complex wave, considered to be composed of negative and positive waves, can be computed according to Egiazarov method, and for the case of small change of discharge, the friction losses can be neglected for several kilometers. The reflections of waves are studied experimentally, and the prediction of resulting waves, composed of negative and positive parts, and reflections, is discussed.

329. Allen, J., 1938, Experiments on water waves of translation in small channels: Philos. Mag. [London], v. 25, sec. 7, p. 754–768.

In an extremely tortuous channel (exemplified, here, by the Mersey River channel studied at scale model) the wave celerity is dependent upon the depth of the deepest axis of channel (thalweg) and independent of the velocity of the stream in which the wave travels. It is observed that the celerity is greater than that corresponding to the mean depth and less than that corresponding to the greatest depth. In a straight, triangular channel the celerity is that calculated for the average depth, but in the trapezoidal cross section, the effective depth H_e is somewhat greater than the average depth, and $H_e=(bH+\frac{1}{2}SH^2)(b+SH)$, in which b=bottom width, H=depth, S=side-slope. Gibson's celerity formula (1923) is herein confirmed for straight or nearly straight and uniform channels, with depth measured along the thalweg. Celerities of tidal bores computed for the natural river and for its model agree well.

330. Beckman, H. C., 1938, Rate of travel of a change in discharge, with W. B. Langbein's and J. H. Morgan's discussions: U.S. Geol. Survey, Water Resources Bull., p. 61–66, p. 131–132, and p. 196–198.

The use of the celerity formula $C=(1/B)dQ/dH$ is discussed.

331. Belokon', P. N., 1938, O stepeni tochnosti graficheskogo metoda inzh. N. V. Mastitskogo po postroeniyu prodol'nykh profiley svobodnykh poverkhnostey pri neustanovivshemsya dvizhenii vody y rechnom rusle [On the degree of accuracy of the Mastitskiy's graphical method for the construction of water-surface profiles of unsteady flow in the river channel]: Gidrotekhnicheskoe Stroitel'stvo [U.S.S.R.], no. 3, p. 13–14.

Comparison is made of natural observations in unsteady flow with the results of flood routing by Mastitskiy's method. The comparison shows that this method gives results close to those observed. The departures between the two results are within the limits of error of observations. The decrease of time intervals decreases the departures.

332. Brigham, R. H., 1938, Movement of wave crests and troughs in rivers: U.S. Geol. Survey, Water Resources Bull., p. 38–39.

The movement of wave crests created by powerplant operations is compared with the movement of the flood wave in rivers. Differences of celerity and of attenuation of waves are pointed out.

333. Frank, J., and Schueller, J., 1938, Schwingungen in der Zuleitungs—und Ableitungskanaelen von Wasserkraftanlagen [Oscillations in headrace and tailrace canals of water powerplants]: Berlin, Springer, p. 1–200.

The celerity formulas for surges are developed for different cross sections, and families of curves are used to facilitate the computation. Experiments in natural canals are given as support of the formulas. The phenomenon of secondary waves in a positive surge is discussed. The influence of friction on celerity is considered, using Forchheimer's and Favre's methods. The change of waves, reflection, and further progression in narrowing parts of channels is discussed, principally with use of the method of characteristics in simple form. An extensive application of the theory to the powerplant canals is given, as are many examples.

334. Henry, Marc, 1938, Propagation des intumescences dans un canal rectangulaire [Propagation of translatory waves in a rectangular channel]: Rev. Gén. Hydraulique [France], v. 4, no. 19, p. 17–24, and no. 20, p. 65–71.

Starting from Deymié's equation (1935) and his developments, two characteristics are introduced: $y=V\pm C$, and $z=f(V\pm 2C)$. The role of characteristics in propagation is special, namely, when the function z of V and C is demanded, so that its derivative along the direction y and the value

y itself are simultaneously the function of V and C, the above relations for y and z are obtained. On that property the author bases its method of characteristics. The case where $S-(\lambda V^2)/(gR=o$ is analyzed, where $V \pm 2C$=constant on a characteristic (and can change from one to another). The conditions for the discontinuity in the general case are studied, and then the general solution without discontinuity is analyzed. The last part of the report is the study of propagation of discontinuity, and then the results are compared with Bergeron's study (1937) on graphical method of computation of wave propagation.

335. Horton, R. E., 1938, Definitions and classification of flood waves, Permanent Internat. Assoc. Navigation Cong. Bull., no. 25, p. 55–70.

Waves are classified as being either longitudinal or transverse. They are further classified as waves that are subject wholly or mainly to gravitational or momentum control and waves that are largely subject to friction control. These classifications are subdivided with reference to orbital or nontranslatory waves and translatory waves. Wave classification is also made according to occurrence as deep-water or shallow-water waves. Other waves discussed are: seiches, impulse waves (abrupt change of inflow or outflow), solitary waves, power waves (in water-power canals), breaker waves, bores, mud-flow waves, surges, flood waves (normal flood waves, cloudburst flood waves), and alternating flood waves (reflection waves). Definitions are given for single, multiple, and train waves (positive and negative). Waves are then classified according to rate of change of regimen, according to direction of motion of the wave crest relative to the direction of stream flow, and so forth. Classification in eight types of single-faced (monoclinal) waves and eight types of two-faced (or normal) flood waves is discussed.

336. Horton, R. E., 1938, Seddon's and Forchheimer's formulas for crest-velocity of flood-waves subject to channel-friction control: Am. Geophys. Union Trans., v. 19, p. 374–382.

Analysis is made of Seddon's formula for wave-crest celerity $C=(1/B)dQ/dH$, where B is the width of canal; and of Forchheimer's formula $C=(1/B)[(Q_2-Q_1)/(H_2-H_1)]$, where Q_2 and H_2 are upstream constant discharge and depth, and Q_1 and H_1 are downstream constant discharge and depth. Both are derived from continuity equations. Using the actual data for comparison and control it is shown that Forchheimer's formula fits the data better.

337. Horton, R. E., 1938, Channel waves subject chiefly to momentum control: U.S. Dept. Agr., Soil Conserv. Service T. P. 16, p. 1–51, and Permanent Internat. Assoc. Navigation Cong. Bull., no. 27, 1939.

This paper describes a series of experiments made at the author's laboratory, using an experimental slope channel 5⅝ inches wide and 120 feet in length, with recording gages at 25 foot intervals. Water was supplied from a constant-level orifice box, with both screw and level movements, so that the rate of flow could be changed either abruptly or gradually. Altogether about 190 experiments were carried out, the results of which are given in the paper. Cases covered include, for different slopes and depths, the following: instantaneous increase in discharge; instantaneous decrease in discharge; gradual increase in discharge; gradual decrease in discharge; waves with triangular profile; waves with rectangular profile; disturbance waves in still water; and alternating

waves, such as occur when an outlet is opened or closed in a pond. No attempt is made to develop a new formula for wave velocity, but in conjunction with the observed velocities and other data there is given for each experiment the velocity of the wave for the same conditions as computed by two formulas of Bazin-Darcy, by the Leach-King formula, and the Koch and Carstanjen formula, and it is shown that all of these formulas give reasonably good agreement with the experimental data for simple increment waves. The phenomena of alternating waves which travel successively upstream and downstream through a mill-pond are particularly interesting, as it is shown that the water-level in the pond changes abruptly by definite quanta each time the wave changes direction at one end of the channel.

In conclusion, the author points out that in these experiments the effect of channel-friction was apparently relatively small, and waves of the character of those covered by the experiments are subject chiefly to momentum control. In the case of triangular and rectangular waves, the wave-length did not exceed 1000 to 2000 times the wave-height. The author points out that in the case of natural river-floods, the wave-length is often-times of the order of 25,000 to 100,000 times the wave-height, and suggests that there is a gradual transition for waves of the same type as between short waves (that is, wave-length less than 1,000 times wave-height) and long waves, such as occur in river-channels, with an increasing degree of channel friction control and a decreasing effect of momentum as the ratio of wave-length to wave-height increases.

[Author's abstract.]

338. Hung-Chi, Lay, 1938, A digest of the English literature of flood waves: Iowa Univ., M.S. thesis, p. 1–158.

Approximately 70 papers in the English literature of flood waves are reviewed and correlated. Bibliography contains 69 entries.

339. Khristianovich, S. A., 1938, Neustanovivsheesia dvizhenie v kanalakh i rekakh [Unsteady motion in channels and rivers]: Sbornik "Nekatorye novie voprosy mekhaniki sploshnoi sredy" (Several new questions on the mechanics of continuous media). Acad. Sci. Math. Inst. [U.S.S.R.], p. 12–154.

340. Knappen, T. T., 1938, Discussion of the paper "National aspects for flood control" (A symposium): Am. Soc. Civil Engineers Trans., v. 103, p. 667–678.

The value of the experimental approach by use of the model is doubted, and the Muskingum method is emphasized. The ratio of the storage increment ΔW to the weighted flow increment ΔQ is introduced by T. S. Burns, F. B. Harkness and G. T. McCarthy. (See No. 342.) It is shown that

$$K=\frac{\Delta W}{\Delta Q}=\frac{\Delta t[0.5(P_2+P_1)-0.5(Q_2-Q_1)]}{X(P_2-P_1)+(1-X)(Q_2-Q_1)},$$

where X = fraction of the weighted flow increment that is derived from inflow increment; Δt is the time unit; and

$$\Delta Q=(Q_2-Q_1)+X[(P_2-P_1)-(Q_2-Q_1)]$$

is prism storage plus wedge-storage increment.

341. Langbein, W. B., 1938, Some channel-storage studies and their application to the determination of infiltration: Am. Geophys. Union Trans., v. 19, p. 435–437.

The relation between channel storage and discharge is studied, and the characteristics of recession curves are analyzed on the basis of relationships revealed by the consecutive daily discharge. The relation of storage W, discharge q (as the yield in cfs per sq mi), and the river drainage area is studied, showing that M, in the formula $q=DW^{M}$, decreases in the downstream direction. A classification of recession curves is introduced, based on the exponent M, which is representative of the geomorphological characteristics of channels. A new method is derived for the computation of infiltration during rainfall by the determination of the river-inflow hydrograph.

342. McCarthy, G. T., 1938, The unit hydrograph and flood routing: U.S. Engineer School, Fort Belvoir, Va. [1940].

A monograph entitled "The unit-hydrograph and flood-routing" was prepared by Gerald T. McCarthy for presentation at the Conference of the North Atlantic Division of the United States Corps of Engineers at New London, Connecticut, June 24, 1938, this being published in limited edition in the United States Engineer Office at Providence, R.I. This describes the "coefficient" method of flood routing, which is reported to have been used with marked success in recent flood-routing studies, especially those on the Muskingum River System in Ohio and the Connecticut River System in New England. This method is considered to be an improvement over previously used methods, chiefly in making a definite allowance for "Wedge-storage" effects and in more systematic utilization of the available natural flood-flow data. Its original features are said to be especially due to the work of Mr. McCarthy.

[Abstract by H. A. Thomas.]

343. Moots, E. E., 1938, A study in the theory of flood-waves: Am. Geophys. Union Trans., v. 19, p. 383–386.

The hydromechanic equations of a simple flood wave in a wide rectangular channel are developed and discussed. The paper includes: (1) a restatement of the fundamental equations governing a simple flood wave in a wide rectangular channel; (2) the development and solution of the equations of the roll wave as a special case of the flood wave; (3) a discussion of the order of occurrence of the three maxima of mean velocity, discharge, and stage; (4) the determination of the celerity of a flood wave as the rate of movement of a constant flow; (5) the determination of the celerity of a flood wave having small amplitude; (6) the development and solution of the equations of the rising flood wave.

The conclusions are:

1. The profile of the roll wave after equilibrium has been established may be obtained in terms of elementary functions from the general hydrodynamic equations of a simple flood wave.
2. The profile of the monoclinal, or rising flood wave, after equilibrium has been established, may be obtained in terms of elementary functions from the general hydrodynamic equations of a simple flood wave.
3. The celerity of the monoclinal, or rising flood wave in equilibrium, may be obtained from Kleitz's equation for the velocity of a constant flow.

4. The celerity of a flood wave of small height is found to be greater than the mean velocity of the channel in normal flow.

344. Moots, E. E., 1938, A study in flood waves: Iowa Univ. Eng. Studies Bull. 14, p. 1–24.

Two basic differential equations for unsteady flow are redeveloped. The equation for the roll wave (wave of a dam breach) is rederived as $x=(1/S)[H+H_2 \log(1-H/H_2)]$, where x = the abscissa of wave shape, H = ordinate, H_2 = starting depth, and S = river-bed slope. The sequence of attaining maxima is: velocity (V), discharge (Q) and stage (H), in that order, according to Kleitz (1877). For constant Q traveling ($dQ=o$), it is shown that

$$C=[(1/B)\partial Q/\partial t]/(\partial H/\partial t)=V+(\partial V/\partial t)/(\partial H/\partial t)$$

There is a discussion of the changes of C that occur throughout the reach, particularly at the time of occurrence of maximum V, Q, and H. Q_{max}-celerities are given for the wave with constant celerity, the wave with small amplitude, the roll wave and the solitary wave. Equations are derived for mean velocity and shape of the solitary wave. The laboratory tests made for this study, employing a rising wave (from Q_1 to Q_2), show that the celerity of the toe is very close to Russell's and Bazin's celerities for solitary waves.

345. Rouse, H., 1938, Fluid mechanics for hydraulic engineers: New York, McGraw-Hill Book Co., p. 353–402.

Chapter 15 deals with general characteristics of wave phenomena and with properties of gravity waves: celerity formula, velocity due to acceleration by wave movement, characteristic dimensions of surface waves, etc. Chapter 16 deals with channel solitary waves (celerity and shape, generation), with surges (generation, celerity, undular profile, progress of wave with time), roll waves, hydraulic jump as a standing surge, and wave effect on boundaries.

346. Snyder, Franklin, 1938, Synthetic Unit Graphs: Am. Geophys. Union Trans., v. 19, p. 447–454.

The lag of time from center of mass of rainfall to peak of runoff is used as the principal drainage-basin characteristic in the derivation of synthetic unit graphs. The peak rate of the unit graph is expressed as a function of the lag. A distribution graph is determined by means of the lag. By use of the distribution graph and the known peak of runoff, a unit graph can be constructed for the area in question. Use and limitations of the synthetic unit graph in flood forecasting are discussed.

347. Steinberg, I. H., 1938, A method of flood routing: Civil Eng., v. 8, p. 476–477.

This report presents a method for routing floods in the upper Mississippi River. Many outflow-storage and storage-factor (storage+outflow) curves are used, as developed from the general function $W=f(Q, Q_t, M)$, where Q = outflow, Q_t = tributary flow entering river downstream of the reach, the M = the storage factor. Taking constant Q_t or M, two families of curves $W=f(Q)$ are developed. The M parameter family, for Q_t = constant, consists of parallel sloping straight lines, where the slope depends upon the time interval used, and the distance between lines is dependent upon the unit of length of the storage scale ($M=Q/2+W$) or $(-Q/2$

$+M=W$). The intersect of M and Q_t of the two families, gives Q and W. The routing is made by tabulation, using developed families of curves.

348. Taylor, E. H., 1938, Analysis of the positive surge in a rectangular open channel: Civil Eng., N. 8, p. 685–686.

The movement of steep positive wave up the channel, with wave height h, depths H_1 and H_2, and velocities V_1 and V_2, is given by equation $(\delta^3-2\delta^2)/(\delta+1)=4k^2\lambda$, with $V_2=(1-k)V_1$, $\delta=(H_2-H_1)/H_1$, and $\lambda=V_1^2/2gH$. For the hydraulic jump $V_1H_1=V_2H_2$, the equation becomes $\delta^2+3\delta+2=4\lambda$.

349. Bachet and Beau, 1939, Rapport sur la prévision des crues [Report on the flood forecast]: Internat. Assoc. Sci. Hydrology Trans., General Mtg., Washington, D.C., 1939, v. 1, ques. 2, rept. 4, p. 1–19.

The forecast of floods is based on the stages; the actual discharge is divided as: (1) the discharge of steady flow and (2) a complementary discharge (positive or negative) resulting from unsteady flow. A template is presented for flood routing along the river. According to the author, flood forecasting by the given method (when the method is applicable) amounts to the simple integration of partial differential equations.

350. Deymié, Ph., 1939, Propagation d'une intumescence allongée (Problem aval) [Propagation of an elongated intumescence (tailwater problem)]: New York, John Wiley and Sons, p. 537–544 (Internat. Cong. Appl. Mech., 5th Mtg., New York, 1939, Proc.).

The surges created by powerplant releases are studied analytically, and the results are verified by experiments on the Seine River in Paris (1933). Two partial differential equations are integrated using some simplifications. The surge is studied according to its three parts: the wave front, which is alternated by progressing according to exponential law; the residual wave (back part of wave), which is damping asymptotically and which cannot have a "black front"; and the central zone of wave, which is composed of the residual waves of the two other parts. The surge approximates the celerity $3V/2$, when the canal length is infinite. Due to friction, the surge tends to assume the characteristics of the permanent wave. Experimental results at the River Seine confirmed the theory.

351. Egiazarov, I. V., 1939, Proekt tekhnicheskikh uslovij i norm rascheta svobodnoy poverkhnosti potoka pri neustanovivshemsya dvizhenii v otkrytykh dlinnykh byefakh [The project of engineering conditions and norms for the computation of the free surface movement of unsteady flow in the long open channels]: Sbornik: Proekty Tekhnicheskikh Usloviy i Norm Gidrotekhnicheskogo Proektivovaniya (Collection: Projects of Technical Conditions and Norms for Designing in Hydraulic Engineering), [Moscow-Leningrad].

352. Massé, P. R., 1939, Recherches sur la theorie des eaux courantes [Research on the theory of flowing waters]: New York, John Wiley and Sons, p. 545–549, (Internat. Cong. Appl. Mechs., 5th Mtg. New York, 1939, Proc.).

Two general partial differential equations for unsteady flow are integrated in the complex plane [Massé's thesis of 1935]. These lead to the conclusion that: (1) intumescence tends to propagate asymptotically with the celerity

$3V/2$ (known as "law of floods"); (2) for the tidal wave, C, M and N must be expressed as:

$$C = V - \sqrt{gH} < 0;\ M = (3/2C^2)\sqrt{g/H} > 0;$$

and

$$N = (gS/CV)(1 + V/2\sqrt{gH}) < 0,$$

and the condition necessary for conversion of the tidal wave to the bore is

$$S_0 = (\partial H/\partial y)_0 > -N/M.$$

353. Pillsbury, G. B., 1939, Tidal Hydraulics: U.S. Army Corps Engineers, Prof. Paper 34 [1940], Rev. Ed. 1956.

354. Rutter, E. J., Graves, Q. B., and Snyder, F. F., 1939, Flood routing (with discussions): Am. Soc. Civil Engineers Trans., v. 104, p. 275–313.

This paper describes the procedure developed by the engineers of the Tennessee Valley Authority for routing flood through the Tennessee River in order to predetermine the effects of various dams in modifying flood-stages. While in their broad aspects, the flood-routing methods described in this paper are not essentially different from ones which have been used in other previous investigations, nevertheless a description of the application and extension of these methods to meet the special problems encountered on the complex river-system of the Tennessee constitutes an important contribution to the literature on flood-waves and reflects current progress in the development of practical methods to analyze flood-wave phenomena on actual river-systems.
[Abstract by H. A. Thomas.]

Routing was accomplished by dividing the river length into reaches and applying the storage equation to each reach. Total inflow into each reach was determined day by day from the routed outflow of the adjacent upstream reach, published discharges of metered streams flowing into the reach, and inflow estimated from rainfall on the unmeasured areas draining into the reach. Total flood volume at each dam site was assumed to be known. The storage in the reach for various flows are determined from topographic maps and cross sections of the river valley in conjunction with flow profiles determined from backwater curves. The outflow was then derived from the relation of storage and discharge for the reach.
[From author's conclusions.]

In the discussion (p. 197), R. D. Goodrich proposed the use of first (Δ) and second (Δ^2) differences of inflow and outflow, when floods rise and fall rapidly; C. O. Wisler proposed the plotting of curves ($W + Q\Delta t$) and ($W - Q\Delta t$), R. W. Powell suggested the use of $W = f(0.4P + Q + W)$, in which 0.4 is the weight for inflow.

355. Supino, Giulio, 1939, Sur la propagation des ondes dans les canaux [On the wave propagation in canals]: Rev. Gén. Hydraulique [France], v. 5, no. 29, p. 260–262.

The integration of two partial differential equations is made by linearization of equations for small wave heights ($H = H_0 + h$, $V = V_0 + v$), with $S_0 = kV_0^2$. By neglecting the small, second-order terms of the second member, the general form of solution is given and discussed. The solution of the complete equation is obtained by means of successive approximations, and two solutions of linearized and of complete equations are compared. The author concludes that the attenuation of the wave as

obtained by linearized equation is smaller than the attenuation which results from the integration of the complete equation.

356. Zoch, R. T., 1939, The mathematical synthesis of the flood hydrograph: Am. Geophys. Union Trans., v. 20, p. 207–218. See also the author's earlier papers: Monthly Weather Review, Sept. 1934, Apr. 1936, Apr. 1937.

In studying the function of rainfall infiltration, the flood hydrograph is analyzed for the synthesized evidences of this function. Parts of the hydrograph are shown to represent the concentration, saturation, transition and depletion phases of flood runoff. General aspects of the mathematical approach to the rainfall-runoff relationship are discussed.

357. Allen, J., and Matheson, J. L., 1940, An experimental investigation of the propagation of tides in parallel and in convergent channels: Inst. Civil Engineers [England] Jour., p. 107–118.

The propagation of tidal waves in parallel and convergent channels is studied experimentally and compared with the theoretical formulas. The results demonstrate that, within certain limits, close agreement exists between theory and experiments, and they confirm that the amplification is greater in a parallel inlet than in a convergent inlet of equal channel length and mean depth. No significant difference was observed in the behavior of two convergent channels, one of 6° and the other of of 13° apex angle.

358. Dantscher, Kasper, 1940, Wanderwellen in Schiffahrtskanaelen [Traveling waves in navigation canals]: Wasserkraft u. Wasserwirtschaft [Munich] v. 35, no. 7, p. 145–147.

This is a discussion, with examples, of the traveling of surges in navigation channels, and especially the influence of the velocities produced by wave progression on the resistance to ship movements.

359. Dantscher, Kasper, 1940, Die Wanderwelle in Schiffahrtskanal [The traveling wave in a navigation canal]: Wasserkraft u. Wasserwirtschaft [Munich], v. 35, no. 10, p. 226–229.

The study of surge propagation in a navigation canal, where the surge must travel through transitions from a large to a narrow or narrow to large section of canal, is given. The celerities for two sections are developed and related to section characteristics. The change of wave heights due to the change of cross sections is determined. The velocities in the canal, produced by movement of the changing waves, are determined and their influence on navigation is discussed.

360. Keulegan, G. H., and Patterson, G. W., 1940, Mathematical theory of irrotational translation waves: U.S. Nat. Bur. Standards Jour. Research, v. 27, p. 47–101.

This paper is the first of a series dealing with the motion of flood waves and other waves of translation in open channels. The case treated is that of waves for which the forces of fluid friction are negligible with respect to the inertia and gravitational forces. The irrotational motion of a perfect liquid in a horizontal rectangular canal when the original surface is disturbed is investigated on the assumption that the horizontal velocity in a cross section is approximately uniform. The results are also applicable to motion in a canal of uniform slope containing water originally moving with a

uniform velocity. Special emphasis is laid on disturbances which are propagated without change of form, and in these cases formulas are derived for the wave profile and velocity of propagation. Formulas are also derived which give the deformation, energy, motion of the center of gravity, and moment of instability of an arbitrary intumescence. Consideration is given to the maximum height of a wave of a permanent form. Formulas have been compared with the available experimental data. Of special interest is the comparison of the shape of the undulations composing the head of an initial surge with the characteristics of the cnoidal wave. [Authors' abstract.]

361. Keulegan, G. H., and Patterson, G. W., 1940, A criterion for instability of flow in steep channels: Am. Geophys. Union Trans., v. 21, p. 594–596.

In the case of a symmetrical wave traveling along a wide channel having bottom slope i, depth H_0, and mean velocity V_0, before the wave arrives, the criterion developed for instability of wave (steepening of wave with time and distance travelled) is

$$i>(9/8\lambda_0, \text{ or } V_0^2>(9/4)gH_0,$$

where $gH_0i=(\lambda_0/2)V_0^2$.
This formula is based on Manning's formula. H. Jeffreys (1925) and H. A. Thomas (1939) developed the criterion $i>2\lambda$, based on Chezy's formula.

362. Laden, N. R., Reilly, T. L., and Minnotte, J. S., 1940, Synthetic unit hydrograph, distribution graphs and flood routing in the Upper Ohio River Basin: Am. Geophys. Union Trans., v. 21, p. 649–659.

The coefficient method or Muskingum method for flood routing is discussed. The authors present an outline of the method used in determining the unrated tributary inflows. The unit-graph and synthetic-graph methods are discussed. The adopted method uses McCarthy's method and Snyder's synthetic distribution graph for determination of inflow hydrographs.

363. Langbein, W. B., 1940, Channel storage and unit hydrograph studies: Am. Geophys. Union Trans., v. 21, p. 620–627.

The paper discusses the channel phase in rainfall-runoff relation. The storage of the reach, given by McCarthy as $W=K[xP+(1-x)Q]$, is studied through the analysis of coefficients K and x, and the equation is modified as

$$S_t=K[xP_{(t-2Kx)}+1-x)Q_{(t+2Kx)}],$$

where S_t is storage of a reach in time t; P is the inflow at time preceding t by $2Kx$, and Q is outflow following t by $2Kx$. An approximate form $S_t=KQ_{(t+2Kx)}$ is suggested. The use of the lag interval $(2Kx)$ is discussed, and examples are given.

364. Meleshchenko, N. T., 1940, Primenenie teorii dlinnykh voln maloy amplitudy k voprosam sutochnogo regulirovaniya [The application of the theory of long waves of small amplitude to the problems of daily regulations]: Vsesoyuzniy Nauchno-issledovatel'skiy Inst. Gidrotekkniki i Melioratsii, Izv. [U.S.S.R.], v. 28.

365. Posey, C. J., and Fu Te, I., 1940, Functional design of flood control reservoirs: Am. Soc. Civil Engineers Trans., v. 105, p. 1638–1674.

The functional design of a flood control reservoir has as its objective the determination of the relation between the storage space that must be provided and the corresponding reduction of the flood peak. The method of incorporating this relation into the study of the economic balance of a flood control system has been explained in detail by Sherman M. Woodward, M. Am. Soc. C.E. Mr. Woodward's "five-sixths rule," which heretofore has provided the only direct method for the functional design computations, is limited in its application to reservoirs of such proportions that the maximum outflow is small compared with the average inflow during the flood, and to reservoirs with orifice-type outlets and a certain type of depth-capacity relationship. In this paper the writers have generalized Mr. Woodward's method and have extended it to apply to reservoirs with either orifice- or weir-type outlets, in valleys of a wide range of morphological configuration. Although the relationships derived can be used in the design of multiple-purpose reservoirs, the present discussion is restricted to reservoirs for flood control only.

[Authors' synopsis.]

366. Ramponi, Francesco, 1940, Resultati sperimentali sulla propagazione delle perturbazioni di regime nei canali [The experimental results of the propagation of regimen changes in channels]: Energia Elettrica [Italy], v. 17, no. 11, p. 643–653.

The author first gives a summary of his note on theoretical treatment of wave propagation in open channel, as presented to Second Congress of V.M.I. in Bologna, Italy, 1940. Starting with a particular solution of partial differential equation for unsteady flow in channel, he derives the parameters for that particular solution. As the final result, he gives the celerity and wave attenuation as function of period variation. He describes his experiments with flood waves on the Isarco and Adige Rivers and gives his results for celerities and attenuations, with the parameter R in his solution being computed. The relation of R to period $T=2\pi/\omega$ is given for the experimental data. The second group of experiments, performed in the laboratory canal, is a study of short-period disturbances. The experiments and devices used are described. The results are given in terms of the relation of celerity or attenuation to the period T, showing an increase of attenuation by an increase of amplitude, and a decrease of celerity by an increase of period T.

367. Tarpley, J. F., 1940, A new integrating machine: Mil. Engineer, v. 32, no. 181, p. 39–43.

This paper describes the use of a mechanical integrator to be used in routing floods through reservoirs. Based on the simple storage equation, the device employs drums, gears, and a differential motor. The operator is required to follow three curves in order to translate the recordings on three drums and obtain the simultaneous values of: $P=f(t)$-inflow hydrograph, $Q=f(H)$-outflow rating curve, and $W=f(H)$-storage function. At the same time, the integrator is giving values of $Q=f(t)$ and $H=f(t)$ on two other drums.

368. Thomas, H. A., 1940, Graphical integration of flood wave equations: Am. Geophys. Union Trans., v. 21, p. 596–602.

The two partial differential equations for unsteady flow are rederived. A trial-and-error process of determining local inflow from given stage pro-

files for different time instants is given by using the stage-surface profile for a constant discharge as the reference base, from which the depth h is measured. By the repeating process, finite relations of $Q=f(t)$ for given stations and $A=f(x)$ for given instants are determined. Having Δt and Δx for both families of curves, $\Sigma\Delta Q$ and ΔW are obtained, where $\Sigma\Delta Q$ is the difference between outflow and inflow and ΔW is the storage of a reach Δx during Δt. Local inflow $\Sigma P=\Delta W-\Sigma\Delta Q$. Solution of the trial-and-error process in the case of known local inflow is also given. The author recognizes that the method is laborious, but considers it justified.

369. Thomas, H. A., 1940, The propagation of waves in steep prismatic conduits: Iowa Univ., Hydraulic Conf. Proc. Eng. Studies Bull. 20, p. 214–229.

Roll waves on a steep channel are analyzed by application of the moving-belt analogy. The hydraulic theory of flow on the surface of a moving-belt conveyor is developed. The formulas are given for different computed quantities. Stationary wave trains on a moving belt are analyzed, and relationships are given for the various characteristics of wave train and channel.

370. Volkov, I. M., 1940, Resul'taty naturnykh nablyudeniy popuskovykh voln napolneniya na rr. Topse i Tsareve [The results of observations in nature of filling release waves on the rivers Topsa and Tsareva]: Sbornik Arhangel'skogo Lesotekhnicheskogo Inst. [U.S.S.R.].

371. Arkhangel'skiy, V. A., 1941, K raschetam neustanovivshegosya dvizheniya and v kanalakh i rekakh [On the computation of unsteady flow in canals and rivers]: Akad. Nauk [U.S.S.R.], Inst. Mekhaniki, Inzhenernyy Sbornik, v. 1, no. 1, p. 129–136.

A general grapho-analytical procedure used as a finite-differences method for approximate integration of two partial differential equations for unsteady flow is given, when velocity and acceleration heads are neglected in the momentum equation. The channel is divided in reaches Δx, and time interval Δt for integration. For each reach the relations $H_2''=f(Q_2'')$ and $H_2'=f(Q_2')$ are developed with index 2 referring to the end of time-interval Δt, and Q', H' for the beginning and Q'' and H'' for the end of the reach Δx. Special families of curves [$H=f(Q)$ for given ΔH or H''] are to be developed for each reach in order to facilitate the integration. A combined integration graph is given for many reaches. The method is discussed as applied to the system of several channels and to surges. The author's integration shows that the results agree well with observations.

372. Arkangel'skiy, V. A., 1941, Obzor sposobov proektirovaniya rezhima otkrytykh vodotokov v sluchae neustanovivshegosya dvizheniya [Review of methods of computing unsteady flow of water in channels]: Akad. Nauk [U.S.S.R.], Inst. Mekhaniki, Inzhenernyy Sbornik, v. 2, p. 321–326.

The methods of integrating two partial differential equations for unsteady flow, either by the method of characteristics or by the method of finite differences as applied to the two partial differential equations are discussed. The author discusses his method of finite differences (1941), which neglects the inertia term, obtaining

$$\partial H=V^2\,\Delta x/C^2R$$

with c=Chezy's coefficient. If the inertia term cannot be neglected, it is used as a correction

$$\Delta\partial H=\frac{V_2^2-V_1^2}{2g}-\frac{1}{g}\frac{\Delta Q}{\Delta t}\frac{\Delta x}{A}$$

where $\Delta\partial H$ is to be added to ∂H. The initial and boundary conditions are discussed, and the results are compared with the observed flood waves.

373. Arredi, Filippo, 1941, Sulle onde di traslazione nei canali [On the translatory waves in channels]: Acqua [Italy], no. 5.

374. Coulson, C. A., 1941, Waves: London, Oliver and Boyd, p. 60–86.

This book is a mathematical account of the common types of wave motion, including waves in liquids. Following a summary of hydrodynamic formulas, application to tidal waves and surface waves is shown. The equations for tidal waves in a canal are given, defining celerity, shape development, and particle paths of such waves.

375. Dietz, D. N., 1941, A new method for calculating the conduct of translation waves in prismatic canals: Physica [Netherlands], v. 8, no. 2, p. 177–195.

The author improves the wave-celerity formula by including two resistance terms, so that

$$C=\sqrt{gH}\left[1+\frac{3}{2}\eta-\frac{g\sqrt{gH}}{2C^2H}\left(\int A dt+B\frac{dt}{d\eta}\right)\right],$$

where c=Eytelwein's resistance coefficient, C=celerity of a given height, $\eta=(Z-H)/H$ with Z momentary average depth, and with

$$A=\eta^2/\sqrt{1+\eta};\ B=\eta^2\sqrt{1+\eta}.$$

The author concludes that without the influence of resistance each positive wave would become steeper in front and would be flattened in back.

The resistance effect upon the wave is shown mainly by the term

$$(-g\sqrt{gH}\,B/2C^2H)(dt/d\eta),$$

indicating that the wave will be retarded at the front and accelerated at the back, and that effect increases as the slope S of the water surface decreases.

376. Gilcrest, B. R., and Marsh, L. E., 1941, Channel storage and discharge relations in the lower Ohio River Valley: Am. Geophys. Union Trans., v. 22, p. 637–649.

The procedure for development of storage discharge relation is treated with basic equation $Q_2-Q_1=(t/K)(P_1-Q_1)$, where t=time in days and K=ratio between corresponding increments of storage W and discharge Q for steady flow ($K=dW/dQ$). Examples of routings based on that equation are given. Coefficients C_0, C_1, and C_2, related to t, K, P, and N (a ratio of wedge storage to corresponding difference $P-Q$, with discharge assumed to vary with distance), are determined and analyzed.

377. Horton, R. E., 1941, Virtual channel-inflow graphs: Am. Geophys. Union Trans., p. 811–820.

The virtual channel-inflow graph is conceived as a synthetic inflow graph, which meets four required conditions and, by the routing procedure,

is produced as a likeness of the outflow graph. The four conditions required are: (1) inflow stops at second inflection point of outflow graph; (2) inflow graph intersects outflow graph at its maximum; (3) both graphs start at the same time; (4) their volumes are equal. Simple forms of synthetic inflow graphs are discussed and channel storage is analyzed by the use of virtual-inflow graphs. The conditions under which these graphs are valid and should be used are discussed; examples are given.

378. Horton, R. E., 1941, Flood crest reduction by channel storage: Am. Geophys. Union Trans., p. 820–836.

Using his virtual-inflow graph (1941) the author gives the ratio of inflow and outflow crests as $Q_m/P_m = S(1-W_m/W)$, where W_m=maximum channel storage, W=volume of flood wave, and $S=1-(1-c)t_0/b$, where t_0=time of occurrence of channel-outflow crest, b=time base of channel virtual inflow graph, and c=ratio of total outflow W_1 at crest time to the area of the triangle $[c=W_1/(Q_m t_0/2)]$. The above equation is applied to many examples. For maximum advantage in the use of this equation many more determinations of W_m/W, c, and t_0/b are needed to contribute to its application. The relationship of S and c is given as determined from above equation and from observed data; good agreement is shown.

379. Messina, U., 1941, Il grado di approssimazione di una formula semplice per la determinazione delle altezze delle onde di traslazione nei canali [Degree of approximation of a simple formula for determination of translation wave heights in channels]: Acqua [Italy], no. 10.

380. Meyer, O. H., 1941, Flood-routing on the Sacramento River: Am. Geophys. Union Trans., p. 118–124.

In considering the judgments to be made in preparation for a flood-routing system, the author discusses such criteria for selection of river reaches as location of major inflows, critical points (stream junctions, flow divisions, and weirs), and check-points (actual gage records). The method of estimating local inflows by synthetic unit hydrographs and by discharge correlation is discussed. The channel storage-outflow discharge relationship is determined both by the flow-line method and by empirical curves (which were preferred to the flow-line method). A method is given for using the rating curve of the upstream station to develop discharge-rating curves and storage-outflow curves for locations downstream where only gage-height records are available. Three methods of routing are used: (1) the lag method, using six-hour time intervals; (2) the Steinberg method; (3) the flood-routing machine, for long reaches (employing a family of storage-outflow curves having inflow as the parameters). Equal preference was given to the first and third methods; the Steinberg method was replaced by the labor-saving machine.

381. Meyer, O. H., 1941, Simplified flood routing: Civil Eng., v. 11, no. 5, p. 306–307.

The routing procedure is made in tabular form by using the simple storage equation, and storage-discharge relationship, based on the method of derivation given by Rutter, Graves, and Snyder (1939).

382. Rakhmanov, A. N., 1941, O neustanovivshemsya dvizhenii v nizhnikh i verkhnikh byefakh rechnykh gidrouzlov [On unsteady flow in headrace and tailrace canals of hydraulic structures]: Vsesoyuzniy Nauchno-

issledovatel'skiy Inst. Gidrotekhinki i Melioratsii, Izv. [U.S.S.R.], v. 30.

The method developed by the author is often used for flood routing in natural water courses. Two functions $H=f_1(x,t)$ and $Q=f_2(x,t)$ are graphically represented. These functions having been derived from initial and boundary conditions that are more or less arbitrarily used (derived from previously solved parts of the flood-routing solution). Control is obtained by using two differential equations for unsteady flow. Basically, the storage equation is applied, thus giving to the method characteristics of the volumetric approach. For the long channel with gradually changing wave, results by this approach show good agreement with the observations. This method is used by the author for flood routing on the Volga, Kama, and other rivers with a flat slope. Regardless of the somewhat arbitrary nature of the approach, the results have proved to be sufficiently accurate for many flood-routing purposes.

383. Schultze, Edgar, 1941, Die Berechnung der Gezeiten in Flussmuendungen [Calculation of tides in estuaries]: Die Bautechnik, v. 19, no. 12/13, p. 135–150, Mar. 21. [U.S. Army, Corps Engineers, Waterways Expt. Sta., Translation no. 43–17, Dec. 1942, by H. B. Edwards].

Theoretical mathematical discussion of methods of computing tide levels in river estuaries.

[Annotations in Bibliography on Tidal Hydraulics, Corps of Engineers, February 1954.]

384. Tatum, F. E., 1941, Flood routing by method of successive averages: U.S. Army Corps Engineers, Rock Island Dist. [Illinois], p. 1–10.

The equation for a hydrograph recession curve is $Q_1/Q_2=K^{(t_2-t_1)}$, where t_1 and t_2 represent the times measured from the second inflection point of hydrograph, and Q_1 and Q_2 are discharges at t_1 and t_2, respectively. All recession curves having the same K will be parallel on semilog paper. K is generally constant for a particular river station, except that uneven rainfall distribution changes the K-value somewhat. Because K remains constant for a short pattern, the storage-discharge functions are identical for all reaches of equal time of travel, and for two recession curves (P and Q) with equal K, $Q_2=(P_1+P_2)/2$. The first step is to determine the length of the time period. The time of travel should not be too long, because the obtained ordinates would not determine the true shape of the hydrograph, especially at critical points. (A long reach can be broken into small subreaches.) The second step is to determine the number of routing steps. Two general guides for procedure are: to determine the number of time units (periods) between the peaks, then to double this number; to route a few critical points on the recession curve and by trial to determine which number of routing reaches causes the recession curve to coincide with the recession curve of the downstream hydrograph (when there is considerable inflow). This method becomes a single procedure of obtaining the average of the inflow at the beginning and the end of the period. Application of the method is shown.

385. Turner, H. M., and Burdoin, A., 1941, The flood hydrograph: Soc. Civil Engineers [Boston] Jour., v. 28, p. 232–281.

386. Wilson, W. T., 1941, A graphical flood-routing method: Am. Geophys. Union Trans., v. 21, p. 893–898.

To evaluate the influence of storage in a reach, the author suggests that the attenuation be determined by routing a "storage component" (as though a reach were a reservoir) and a "translation component." The actual graph would be between the two, but no method is given for determining the exact amount of attenuation between two limits. The time-conversion factor is analyzed and, as Langbein gives in discussion, is equal to the time distance between centers of gravity of inflow and outflow hydrographs.

387. Woodward, S. M., and Posey, C. J., 1941, Hydraulics of steady flow in open channels: New York, John Wiley & Sons, p. 133–145.

This paper describes a method of flood routing that uses the storage equation and two auxiliary functions: $(W+\frac{1}{2}Q\Delta t)$ and $(W-\frac{1}{2}Q\Delta t)$. The computation is performed with Posey's slide rule. A slide-rule computation of pool-level routing that employs a variable stage-discharge relationship is described. The use of slide rule is discussed.

388. Arredi, Filippo, 1942, Metodo del calcalo della propagazione delle perturbazioni graduali nelle correnti aperte [Method of computation for propagation of gradual perturbation in open channels]: Acqua [Italy], no. 2.

389. Barrows, H. K., 1942, A study of valley storage and its effect upon the flood hydrograph: Am. Geophys. Union Trans., v. 23, p. 483–488.

The influence of valley storage upon flood hydrographs is discussed together with analysis of observed data. The recession curve is used for analysis of valley storage and permits computation of the inflow hydrograph from the actual flood hydrograph.

390. Evangelisti, Giuseppe, 1942, Sulla propagazione delle piccole onde nei canali a sezione variabile [On the propagation of small waves in canals with variable cross section]: Annali di Matematica Pura ed Applicata [Italy], Jan. 25.

391. Ippen, T. A., 1942, Gas-wave analogies in open-channel flow: Iowa Univ., 2d Hydraulic Conf. Proc., Eng. Studies, Bull. 27, p. 248–265.

The paper summarizes the general results of the theoretical and experimental investigations of shooting or rapid flow in hydraulic structures, bringing these results to the attention of hydraulic engineers as useful tools. Analogy is drawn between supersonic flow of gages and supercritical flow of water. Special attention is given to standing waves. Wavefronts of small height and assumed constant energy are analyzed. The method of characteristics is applied to supercritical flow with constant energy, and the method of using shock polars for supercritical flow with high wavefronts and energy dissipation is presented. The application of theory is discussed.

392. Langbein, W. B., 1942, Storage in relation to flood waves, Part XI–H of Hydrology (O. L. Meinzer, ed.): Physics of the Earth, IX, New York, McGraw-Hill Book Co., p. 561–572.

The storage of a river reach and its influence on flood waves are discussed. The general relation of storage and discharge is

$$W=\frac{b}{a}\left[xP^{m/n}+(1-x)Q^{m/n}\right],$$

in which constants a and n express the stage-discharge characteristics of the control sections at the respective ends of the reach, and b and m evaluate the mean storage-volume characteristics, with $Q=aH^n$ and $W=bH^m$. The coefficients K and x in Muskingum formula are analyzed, with x defining also the attenuation of flood wave (known as attenuation factor). Other relationships of storage in river reaches are discussed.

393. Levin, Léon, 1942, Méthode graphique de calcul du mouvement non-permanent dans les canaux en ecoulement libre [Graphical method for computation of unsteady flow in open channels]: Génie Civil [France], v. 119, no. 11–12, p. 109–113.

The Bergeron method (an alternative of the method of characteristics) is used for travel of surges along a channel. The effect of resistance along a reach is replaced by the concentrated head loss (KQ^2) at the end of the reach. The experiments in a canal have shown that this method gives results that are in close agreement with experimental results only for cases involving very small wave heights.

The author introduces the effect of friction resistance by means of a parameter depending on depth H. When the resistance along the channel cannot be considered to be negligible, the singular concentrated resistance at the end of each reach replaces the effect of channel resistance. Experiments made in the hydraulic laboratory of Hydro-technical Society of France, in Grenoble, have confirmed the author's approach.

394. Linsley, R. K., 1942, River forecasting methods: U.S. Weather Bur. Man., Nov., p. 1–100 [reprinted 1945].

In the author's discussion entitled "translation of hydrograph downstream," the gage relationships are given with and without backwater effect. Peak-discharge relations are given and discussed, and the lagging of the hydrograph is explained. The streamflow-routing method under consideration is based on the storage equation and relates average storage to levels of outflow and inflow. The method of storage factors ($W-Q_1/2$) and ($W+Q_2/2$) is used.

395. Posey, C. J., 1942, Flood wave characteristics as related to flood routing: Iowa Univ., 2d Hydraulic Conf. Proc., Eng. Studies, Bull. 27, p. 224–233.

The Jones and Grimm formulas for the relation of discharges in steady and unsteady flows are compared analytically, and the Jones formula is proven to be less accurate than the Grimm formula. A flood-routing method is described, based on the storage equation, that uses a double family of curves with the storage factor $(W+\frac{1}{2}Q\Delta t)=f(\Delta H)$, where $\Delta H=$ fall in the reach. For one family of curves, depth (H) at the end of the reach is a parameter. The other family of curves uses discharge (Q) at the end of reach as a parameter. The routing table is given.

396. Réméniéras, Gaston, 1942, Adaptation de la méthode Bergeron à l'étude de la propagation des intumescences [Adaptation of Bergeron method to the study of wave propagation]: Unpublished manuscript to be found in Archives of "La Soc. Hydrotech. France," [Paris].

The Bergeron method is adapted for unsteady flow in channels. The author discusses the use of $\sqrt{H}$ instead of H in the equation of characteristics, noting the advantage of the straight line of graph $\sqrt{H}=f(V)$ which has a slope of 45°. This concept was later developed in detail by Craya.

397. Shepley, J. M., and Walton, C. B., 1942, Solving reservoir problems with circular point-by-point computer: Civil Eng., v. 12, p. 154–155.

The use of a device patterned after the circular slide rule is described for flood routing based on the storage equation. The storage factors $F_1 = W/\Delta t + Q/2$ and $F_2 = W/\Delta t - Q/2$ are used. Reservoir elevations are represented by concentric circles. The average inflows are measured as the angular distance between two index lines. A limited amount of numerical integration is required.

398. U.S. Beach Erosion Board, 1942, A summary of the theory of oscillatory waves: U.S. Beach Erosion Board Tech. Rept. 2, p. 1–43.

This summary of the theories of oscillatory wave motion on a free water surface includes studies made of wave characteristics, small amplitude waves, waves of finite amplitude, wave groups and transmission of energy, waves on a sloping bottom, wave refraction, and damping of oscillatory waves. Much of the theory discussed requires experimental verification, and mention is made of the extent of confirmation obtained.

399. Wisler, C. O., and Brater, E. F., 1942, Am. Soc. Civil Engineers Trans., v. 107, p. 1519–1562.

A method of flood routing is described, the successful use of which depends only upon the availability of dependable streamflow records during a typical flood at various points on the main stream or on the tributaries whose flow is to be routed downstream. No cross sections of stream channel or velocities of flow are required. Nor are discharge records on all of the tributaries needed. A hydrograph of inflow from the unmeasured area is directly computed. This flow and that at each of the upstream stations is then routed downstream.

These routed flows show the extent to which each of the upper tributaries contribute to the flood peak at each downstream point. A check on the accuracy of the results is provided by adding the routed flows and comparing the resulting hydrograph with the actual records.

The entire procedure is based upon the storage equation and upon the principle that, for all high stages, there is a straight line relationship between the volume of storage contained in any reach of river channel and the sum of the inflow rate at the upper end and the outflow rate at the lower end of that reach. Except perhaps for unusual channel conditions, this relationship holds true.

[Authors' synopsis.]

400. Arkhangel'skiy, V. A., 1943, Effektivnyy priem raschetov neustanoivshegosya dvizheniya v otkrytykh vodotokakh pri spokoynykh izmeneniyakh rezhima [An effective method of calculating unsteady flows in open channels]: Akad. Nauk [U.S.S.R.], Inst. Mekhaniki, Inzhenyernyy Sbornik, v. 3, no. 1, p. 124–127.

This paper presents improvements that shorten the author's method (1941) for integrating partial differential equations of unsteady flow by a method of finite differences. Both computation methods neglect the inertia term $(1/g)\partial V/\partial t$.

401. Keulegan, G. H., and Patterson, G. W., 1943, Effect of turbulence and channel slope on translation waves: U.S. Natl. Bur. Standards, Jour. Research Paper 1544, v. 30, p. 461–512.

This paper is the second of a series dealing with the motion of flood waves and other waves of translation in open channels. The first paper considered waves controlled solely by inertia forces; the present one is an analysis of the combined effects of turbulent friction and inertia. The basic equation of motion for gradually varied unsteady flow in prismatic channels is derived from fundamental principles. The effect of the velocity distribution in the original undisturbed current on the motion of short waves is investigated and the effects of wave height, curvature of profile, and fluid friction on the celerity of a wave-volume element is analyzed in detail. The deformation of a straight sloping front and the change of height of an abrupt wave front is treated. Special emphasis is laid on disturbances of negligible curvature and practical methods of handling engineering problems arising in connection with the operation of locks or hydroelectirc canals are given.

[Authors' abstract.]

402. Turner, H. M., 1943, The flood hydrograph and valley storage: Am. Geophys Union Trans., v. 24, p. 609–618.

The difference shown between the rising limb of the actual hydrograph and of the computed hydrograph (using storage computed from recession curves) is greater than the river stage at which the rise begins. This difference is also greater for flood hydrographs having the longest time between the rising and falling limbs. In such cases, the greatest difference between actual and computed hydrographs occurs at the lowest stages.

403. Herbert, D. J., and Lowe, F. C., 1944, Progress report on model studies of the Sacramento-San Joaquin Delta, Central Valley Project, California: U.S. Bur. Reclamation, Hydraulic Lab. Rept. no. Hyd–142, Apr. 10.

404. Kohler, M. A., 1944, The use of crest-stage relations in forecasting the rise and fall of the flood hydrograph: U.S. Weather Bur., Hydrol. Director Office (mimeo.), Aug., p. 1–19.

In certain areas, where the stage-discharge relation changes frequently with changes in channel regime, all forecasting procedures must necessarily be expressed in terms of discharge. If these changes are gradual, however, the procedure may be developed in terms of stage from the most recent data, and revised as subsequent floods provide additional data.

The procedures described in the following paragraphs are, in fact, solutions of the storage equation; that is outflow is equal to inflow plus or minus changes in storage.

Within a fixed period of time the change in storage is determined by the reach characteristics, which are constant, and the four variables P_1, P_2, Q_1, and Q_2. The procedures are based on a statistical determination of factors inherent in discharge and stage records. The constants of the regression equations themselves reflect physical characteristics which determine the relative storage in any reach.

The reader is encouraged to demonstrate to himself the physical basis for the procedure in its application to the solution of the storage equation in a particular reach for which the storage volume is actually known.

[From author's synopsis.]

405. Kohler, M. A., 1944, A forecasting technique for routing and combining flow in terms of stage: Am. Geophys. Union Trans., p. 1030–1035.

A method of flood forecasting for downstream points on large rivers is given, using records of river stages and particularly those showing change

in stage during the period equal to the wave-propagation time between two mainstream stations. During this time of wave propagation between the two stations, stage departures that occur in straight-line relationship represent change in stage on major tributaries. As an example of the method, the Mississippi River is used with the Arkansas River and White River as tributaries.

406. Linsley, R. K., 1944, The development of flow and stage relations for river forecasting: U.S. Weather Bur., Feb. (mimeo.), p. 1–14.

This paper discusses choice of the gage relation to be used as the forecast tool, primarily for crest-stage predictions, describing the simple gage relationships that may be developed (stage relations, flow relations, and flow-stage relations). It further discusses selection of data, cautioning that the upstream stages used to relate to stages at a downstream point must be those that actually influence the crest at the downstream point. It is noted that in order to obtain a true stage relationship, the two stages used for establishing a particular point on the curve of relationship must be separated by a time interval equal to the time of travel.

407. Linsley, R. K., 1944, Use of nomographs in solving stream-flow routing problems: Civil Eng., v. 14, no. 5, p. 209–210.

The nomograms are designed to speed the solutions of storage differential equation in the form

$$\bar{P}\Delta t+(W_1-\tfrac{1}{2}Q_1\Delta t)=W_2+\tfrac{1}{2}Q_2\Delta t.$$

Three vertical straight lines are used with scales shown, left and right of the line, for variables: line I, with variables $(W_1-\tfrac{1}{2}Q_1+\Delta t)$ and Q_1; line II, with variables Q_2 and $(W_2+\tfrac{1}{2}Q_2\Delta t)$; and line III, with relationship to runoff during the time interval Δt, inches over the basin, and total runoff during the time interval. The variables can be set up along the three lines in different ways. Each river needs individual treatment in planning the setup of scales.

408. Clark, C. O., 1945, Flood-storage accounting: Am. Geophys. Union Trans., p. 1016–1030.

Flood-storage accounting is the flood-routing procedure applied in reverse, and it uses known hydrographs to calculate the runoff on the ground. It is based on the fundamental storage equation, inflow minus outflow = the change of storage. The Muskingum flood-routing method is used for the computation of inflow hydrographs in major and minor watercourses. The effective rainfall which produces the inflow hydrograph is derived by the use of the unit hydrograph.

409. Clark, C. O., 1945, Storage and the unit hydrograph: Am. Soc. Civil Engineers Trans., v. 110, p. 1419–1488.

The purpose of this paper is to clarify the inherent relationship between fundamental tools (storage and unit hydrograph) and to show how this relationship may be used to derive accurate unit hydrographs for very short periods of initial runoff which accurately reflect the influence of shape of drainage area upon the shape of the hydrograph, allow the segregation of elements of the hydrograph attributable to particular components of the drainage area, and permit the definition of the calculations on intangible factors of personal judgment.

After showing that the constant time units characteristics of unit hydrographs are the characteristics induced by storage capacity of the streams and that storage capacity and discharge capacity are each a limiting factor on the other, the paper illustrates the incorporation of these characteristics in hydrograph calculations.

[From author's synopsis.]

410. Cooperrider, C. K., Cassidy, H. O., and Niederhof, C. H., 1945, Forecasting stream flow of the Salt River, Arizona: Am. Geophys. Union Trans., v. 26, p. 275–281.

The forecast is made for a small tributary stream on the basis of that stream's flow correlation with Salt River. An equation is given which defines the standard error of forecast. The estimated or forecasted flow of test cases came within 2.2 percent of the actual flow.

411. De Marchi, Giulio, 1945, Onde di depressione provocate da apertura di paratoia in un canale indefinito [Depression waves created by opening of gates in canal of infinite length]: Energia Elettrica [Italy], v. 22, no. 1–2, p. 1–13.

The experiments by Egiazarov have shown the possibility of applying the Saint-Venant theory to the movement of depression waves created by sudden opening of a gate at the extremity of a canal of indefinite length with rectangular cross section. The outflow discharge is smaller than the discharge which corresponds to the maximum energy level in the canal. The Saint-Venant theory is applied, and the results are discussed for the case of the sudden total opening of a gate, as wide and deep as the canal. The equation is obtained: $H=(1/9g)(2\sqrt{gH_0}-x/t)^2$, between depth H and the initial depth H_0, with x=distance from gate, and t=time. The profiles of waves are given, and the results of experiments by Trifonov are compared with computed data, and discussed. The same negative waves are studied in the cases of a bottom gate, or a surface reversible gate, with sudden openings, as well as the case with a gradual opening of gates.

412. Neyrpic, 1945, Report on Special Rhine River flood model experiments: Ateliers Neyret-Beylier and Piccard-Pitet (actually Neyrpic), Grenoble [France].

Report describes a model study of dam breaches and sudden water release on the Rhine River. Experiments are described, and results are given.

413. Vedernikov, V. V., 1945, Conditions at the front of a translation wave disturbing the steady motion of a real fluid [Usloviya na fronte volny popuska, narushayushchey ustanovivsheesya dvizhenie real'noy zhidkosti]: Akad. Nauk [U.S.S.R.] Doklady, v. 48, no. 4, p. 239–242 (version in English).

The trajectory of the front of a translation wave disturbing a steady flow is a characteristic by means of which the partial derivative cannot generally be computed. If the condition establishing a relation between the independent variables and the first derivatives is fulfilled, then one of the partial derivatives remains arbitrary, and it is possible to compute the derivatives by means of the characteristics. It is shown that this condition is fulfilled at the front of a wave disturbing a steady flow. A convenient way of computing the derivatives is indicated and the formulas are derived for computing the derivatives of the first and second order in a partial differential equation for unsteady flow in a prismatic channel for any distance from the initial section.

414. Vedernikov, V. V., 1945, V raschetu neustanovivshegosya dvizheniya zhidkosti v otkrytom rusle [On the computation of unsteady motion of liquid in an open channel]: Akad. Nauk [U.S.S.R.], Izv., Otdelenie Tekhnicheskikh Nauk, no. 4, p. 499–503.

The Khristianovich approach to the Massau method of characteristics is discussed. In the formulas developed by Khristianovich (1938), the term $K=g(S_0-V^2/c^2R)$ must be determined in the region of computational differences of characteristics. This author's study makes it possible to obtain the formula with a change of K in the first approximation. The basis for this solution is the Boussinesq assumption of a linear-resistance law for waves of small amplitude. In differing from the Boussinesq approach, the author treats the wave of finite amplitude, with the linearization of resistance in a more general form, in the region of computational differences of characteristics.

415. Wilkinson, J. H., 1945, Translatory waves in natural channels: Am. Soc. Civil Engineers Trans., v. 110, p. 1203–1236.

The phenomena of translatory wave travel in natural channels have been studied for many years without entirely satisfactory results. Published studies have been largely theoretical, supplemented by laboratory tests in some instances, but with few observed data on waves in natural channels. Stream flow regulation by storage reservoirs in the Tennessee River Basin during the past few years has developed numerous translatory waves of various magnitudes. Under such conditions, waves can be selected for study which travel considerable distances without being appreciably affected by tributary inflows.

This paper presents data on natural channel translatory waves on the Clinch and the lower Tennessee rivers, and on Wheeler Reservoir in Alabama for a considerable range of flow in each type of channel. Theoretical velocities are computed for these waves by formulas and are compared with observed velocities. Results indicate that wave velocities on the Clinch River can be computed with reasonable accuracy, and that velocities for the rising face of a wave on the lower Tennessee River can be approximated. Computed velocities for waves in Wheeler Reservoir are in poor agreement with observed velocities.

[Author's synopsis.]

416. Arkhangel'skiy, V. A., 1946, Dvizhenie v forme preryvnoy volny v otkrytykh vodotokakh [Flow in the form of a breaker]: Akad. Nauk [U.S.S.R.], Inst. Mekhaniki, Inzhenernyy Sbornik, v. 2, no. 2, p. 119–126.

The problem of the steep wave (surge) moving on a long wave is studied by combining the method of characteristics, given for Z (level) and Q (discharge) in the x-, t-plane, with application of the author's graphical method of finite differences. The trial- and error-procedure of obtaining the celerity function $C=f(t)$ in x-, t-plane is given for steep wave movement on the unsteady flow of a long wave. The case of steady flow is introduced. The time when the steep wavefront will arrive at a given place, x, is computed by the use of developed procedures, and some corrections are introduced.

417. Cheng, H. M., 1946, A graphical solution for flood routing problems: Civil Eng. [London], v. 16, Mar., p. 126–128; Corrigenda, p. 404, Sept.

The storage equation $P-Q=dW/dt$, for finite Δt, is integrated graphically using the form

$$P\Delta t+W_1-\tfrac{1}{2}Q_1\Delta t=W_2+\tfrac{1}{2}Q_2\Delta t.$$

Three graphical methods of integrating are given: (1) using factors $W-\tfrac{1}{2}Q\Delta t$, and $W+\tfrac{1}{2}Q\Delta t$; (2) using only factor $W+\tfrac{1}{2}Q\Delta t$; and (3) using only $W=f(Q)$ and three parallel lines for $P\Delta t$, $+\tfrac{1}{2}Q\Delta t$ and $-\tfrac{1}{2}Q\Delta t$.

418. Craya, A., 1946, Calcul graphique des régimes variables dans les canaux [Graphical computation of variable regimes of flows in channels]: Houille Blanche [France], new ser., no. 1, Nov. 1945–Jan. 1946; p. 19–38, and no. 2, Mar. 1946, p. 117–130 (Partly translated into English by A. Duncan, TVA).

Graphical methods based on characteristics are studied. For the train of elementary waves it is found that $V-2\sqrt{gH}=$ constant for the downstream wave and $V+2\sqrt{gH}=$constant for a wave in upstream direction. These equations give the characteristics (lines) in the coordinate system $(V, \sqrt{gH})$, with slopes 0.5 and -0.5. For the observer traveling upstream with the wave train and with the celerity $V-C$, the velocity and wave height vary as $V-2C=$ constant, with $C=\sqrt{gH}$. For the observer traveling downstream with celerity $V+C$, the relation is $V+2C=$constant. Waves resulting from the merging of two waves approaching from opposite directions are studied in terms of their characteristics. The graphical representation is given with straight-line characteristics. Problems are solved or cases involving: the jump in bottom; the bottom having continuous slope; friction losses due to flow resistance; both sloping bottom and flow resistance; changing cross section; wave translation; prismatic channel of any cross section; the boundary conditions; propagation of elementary wave (estimate of wavefront in the case of canal which is initially without water movement, where flow is uniform or variable); and shock waves.

419. Craya, A., 1946, La propagation des ondes dans les écoulements à surface libre graduellement variée [Wave propagation in the flow with free, gradually varied surface]: Centre de Perfectionnement Tech., Maison de la Chimie [France], Jan., no. 1549.

420. Puppini, Umberto, 1946, Sulla forma dell'equazione del regime vario nelle correnti liquide [On the form of equation for unsteady flow]: Energia Elettrica [Italy], no. 7, p. 261–265.

The equation of unsteady flow is presented in two forms, the first being obtained by the terms of the equation relating to the filament of velocity flux in the field of discharge, and the second being obtained by means of the field "cross section of flow" determination. It is demonstrated that the first form is preferable, either because of its immediate significance of energy, or because it may be adapted to a simplified form which leads, in an approximate way, to an equation of the usual practical form.

421. Ré, R., 1946, Étude du lacher instantané d'une retenue d'eau dans un canal par la méthode graphique [A study of sudden water release from a body of water to a canal by the graphical method]: Houille Blanche [France], no. 3, p. 181–187.

Craya's development of method of characteristics (1945–46) is applied to Rhine River for the purpose of wave-height forecast in case of dam breach. The problem is presented in a schematic form. The principles of integration by characteristics are given in summary form with special

attention given to shock waves (bores). The graphical procedure of integrating is described, and precautionary advice is given. The results are considered and conclusions drawn at the end. Charts giving practical illustrations of the method accompany the paper.

422. U.S. Engineer School, 1946, Engineering construction, flood control, (Chap. 5, Flood routing, and Chap. 6, Reservoirs): U.S. Engineer School, Fort Belvoir, Va., p. 127–224.

A definitive discussion of flood routing is given, including generalized theories of flood wave and channel. Pauls method for flood routing (with factors $W+Q/2$, $W-Q/2$) is described; Steinberg's and McCarthy's (Muskingum) methods are given. All three methods are based on the storage equation. Special attention is given to the effects of levees, floodways, and channel improvement. With reference to reservoirs, the storage equation is solved by trial-and-error, Goodridge, graphical, and mass-curve methods. Integrating machines and various aspects of flood routing through reservoirs are discussed in detail.

423. Zamarin, E. A., 1946, Dvizhenie povodka pri proryve plotiny [Movement of flood wave after the dam breach]: Nauchnye Zapiski (scientific proceedings) Moskovskogo Gidromeliorativnogo Inst. [U.S.S.R.].

424. Anonymous, 1947, Electrical computer aids flood studies: Civil Eng., v. 17, no. 3, p. 18.

This is a brief note describing an electrical analog device employing an oscilloscope to observe the change in shape of flood waves.

425. Arkhangel'skiy, V. A., 1947, Raschety neustanovivshegosiya dvizheniya v otkrytykh vodotokakh [The computations of unsteady flow in open channels]: Akad. Nauk [U.S.S.R.], p. 1–134.

This is a complete treatment of the author's contributions to the study of unsteady flow. There is a general discussion of unsteady flow and of the differential equations for unsteady flow in prismatic channels. Khristianovich's approach to the integration of these equations by the method of characteristics is given in detail. The author's method of expressing finite-difference equations, or the method of instantaneous regimes, is described. By this method the inertia terms in the Saint-Venant equations are ignored or replaced by a correction coefficient. The variables are assumed to change linearly within the time Δt, and reach Δx. The boundary conditions are discussed. The procedure of integration is explained. Both the method of characteristics and the method of finite differences are then applied to unsteady flow in form of surges, with an analysis of wave reflection given. The sources of data and the representations of the data are discussed, and examples of computation by both methods are given. Also examples of flood-wave transformation are shown for the case of dam breach and of powerplant operation, with unsteady channel flow. All seven chapters have summaries in English.

426. Bruman, J. R., 1947, Application of the water channel—compressible gas analogy: North Am. Aviation, Inc., Rept. NA–47–87.

This report contains a brief discussion of the analogy between flow of a free-surface liquid (that is, under action of gravity) in a shallow channel and the two-dimensional flow of a compressible gas.

Results of reasearch to determine the optimum conditions of water depth and model size are presented. The basic equipment used is a shallow basin about four feet wide by twenty feet long, traversed by a light carriage on which models are mounted. Findings of this work are embodied in a set of design recommendations for a complete water channel set up.

[From the summary in the paper.]

427. Dean, W. R., 1947, Note on waves on the surface of running water: Philos. Soc. [Cambridge] Proc., v. 43, p. 96–99.

A solution is given for local disturbance induced on the surface of a steady uniform flow by application of a constant pressure. Treatment of the problem is purely analytical.

428. Dronkers, J. J., 1947, Methoden van getijberekening [Methods of tidal calculation]: De Ingenieur [Netherland], no. 49, Bouw. -en Water-Bouwkunde 13, p. 129–188.

Two methods of tidal calculations are given, the choice of method being dependent upon the desired accuracy of results and on the nature of the rivers or estuaries in question. They are called the linear and the exact methods. The exact method takes into account the irregularities of rivers or estuaries, but when rivers are considered regular, the linear method gives more rapid but less accurate solutions. The exact method uses finite differences, whereas, the linear method uses the Fourier-series for tides, with integration along the regular channel.

429. Haines, N. S., 1947, Varying reservoir levels and discharges forecast by new method: Eng. News—Rec., v. 138, June 26, p. 69–71.

A method of routing in reservoirs that uses rate of rise of the water surface as a function of inflow rate and elevation of pool.

430. Holsters, H., 1947, Le calcul du mouvement non-permanent dans les rivières par le méthode dite des lignes d'influence [The computation of unsteady flow in rivers by the so-called "influence-lines" method]: Rev. Gén. l'Hydraulique [France], v. 13, no. 37, p. 36–39; no. 38, p. 93–94; no. 39, p. 121–130; no. 40, p. 202–206; and no. 41, p. 237–245.

The method of characteristics for integration of unsteady flow equations, developed by Massau (1899–1905), is adapted for practical purposes. The author calls the characteristics "the influence lines" in order to distinguish his construction of lines from the rigorously exact lines (characteristics). The base of the computation method is a net of characteristics that correspond to zero velocity of stream and to a depth invariable in time (that is, the mean depth for a tidal river). The theory of Massau is presented. Using two partial differential equations and two total differentials and determinants, the following two characteristics are obtained: $dx = +\sqrt{gH}\,dt$, and (level) $dh = \pm dQ/B\sqrt{gH} \pm Q|Q|/c^2B^2H^2$. If $(\partial h/\partial t)_0$ along a propagated discontinuity is greater than $V(V+2\sqrt{gH})(\sqrt{gH}-V)/3c^2H$, there will be a finite wave discontinuity. The method of influence lines is given. Wind influence, Chezy coefficient, and many applications are studied. These include modification of permanent regime, tidal movement in rivers, the analysis of influence on floods on a main river by a dam and reservoir on a tributary river, problems relating to the Escant River, and polders. Model verifications are analyzed.

431. Lin, Pin-Nam, 1947, Unsteady flow problems from Massau's line of attack: Iowa Univ. M.S. thesis.

In this paper is attempted the introduction of a workable scheme to adapt Massau's theoretical treatment to practical problems. The paper

consists of three parts, the first part being devoted to a brief account and discussion of Massau's treatment, the second part to the introduction of a graphical solution, and the third part to two numerical examples.
[From author's abstract.]

432. Mastitskiy, N. V., 1947, Priblizhennyy graficheskiy metod rascheta neustanovivshegosya dvizheniya potoka [Approximate graphical method of computation of unsteady flow]: Tran. Neustanovivsheesya Dvizhenie Vodnogo Potoka v Otkrytom Rusle, Akad. Nauk [U.S.S.R.].

The method developed by the author (1934) is improved, and the computation of basic hydraulic elements of movement is made by unique construction.

433. Nosek, T. M., and Dice, R. I., 1947, A theoretical study of flood waves resulting from sudden dam destruction: Mass. Inst. Technology, M.S. thesis, p. 1–28.

For the study of dam breach waves the Craya method (1946) was used in the solution of two simulated dam failures for which very accurate model results were available. The modified Craya method was used for routing of the wave resulting from the failure of St. Francis Dam, for which accurate records exist. The Craya method was found to be the best theoretical method, in the absence of model studies, for predicting flood-wave phenomena resulting from sudden dam destruction. This method gave results within 5 percent of accurate model tests for relatively uniform channel cross section, slope, and friction factor. The modified Craya method gave results within 10 percent of the recorded maximum water elevations resulting from the failure of St. Francis Dam.

434. Potapov, M. V., 1947, Priblizhennyy gidravlicheskiy method rascheta dvizheniya volny popuska [Approximate hydraulic method of computation of the movement of release waves]: Sbornik Neustanovivshesya Dvizhenie Vodnogo Potoka v Otkrytom Rusle, Akad. Nauk [U.S.S.R.]. Reprinted *in* Sochineniya [Works], v. 3, p. 449–471, 1951.

For simple schematic flood waves (triangular, trapezoidal), flood-wave attenuation, which occurs as a function of channel length, is shown as the change of the peak discharge and of maximum stages. A correction coefficient is introduced in simple formulas, and a discussion of the method of determination of the cofficient is given. The application is presented for any channel cross-sectional shape and for the parabolic shape. The time of arrival of the maxima is computed, and the manner of determining the hydrographs and the graphs of instantaneous regimes is shown. The channel with changing hydraulic elements is discussed.

435. Schultz, E. R., 1947, Preliminary flood-routing studies for the Yangtze River Gorge Dam: Col. Univ. M.S. thesis.

436. Steinberg, I. H., 1947, A flood routing device: Am. Geophys. Union Trans., v. 28, p. 247–254.

A flood-routing device, based on the simple storage equation and the coefficient method for the storage-discharge relation of a river reach, is developed by the use of two slides to which are attached arms with adjustable slopes. One of the slides has a linear scale graduated in inches and tenths of an inch. The equation for routing is given and the application is discussed. Two examples are given.

437. Stroband, H. J., 1947, Een bijdrage tot de kennis van de getijberebing op benedenrivieren en zeearmen [A contribution to the knowledge of tidal calculations for river and sea estuaries]: De Ingenieur, no. 36, Bouw-en Water-Bouwkunde 10, p. 89–95.

An approximation is added to the method of linearizing the quadratic resistance term, in order to account for the effects of partial tides as well as the main tide.

438. Ursell, F., 1947, The effect of a fixed vertical barrier on surface waves in deep water: Philos. Soc. [Cambridge] Proc., v. 43, no. 3, p. 374–382.

The two-dimensional reflection of surface waves from a vertical barrier in deep water is studied theoretically.

439. U.S. Bureau of Reclamation, 1947, Flood routing: U.S. Bur. Reclamation Manual, chap. 6.10, pt. 6, Flood Hydrology, IV, Water Studies, Dec. 30.

Different simple methods of flood routing along channels are described: Puls method (and example), modified Puls method (and example), Sternberg method (using storage factor, and example), Rutter, Groves, and Snyder method, and Muskingum method. Flood routing through reservoirs is briefly discussed with reference to the modified Puls method and the trial-and-error method.

440. U.S. Geological Survey, 1947, Channel storage and flood routing: U.S. Geol. Survey, Water Resources Div., Handb. Hydrologists, p. 1–16.

The general relation between upstream and downstream hydrographs is discussed. Included are: stage-storage and discharge-storage methods of flood routing, and computation of storage volume in the coefficient (Muskingum) method, with evaluation of coefficient K and of x. Applications are shown.

441. Vedernikov, V. V., 1947, Volni popuskov real'noy zhidkosti [Release waves of the real liquid]: Sbornik Neustanovivsheesia dvizhenie vodnogo potoka v otkritom rusle [Transactions, Unsteady movement of the water current in an open channel]. Akad. Nauk [U.S.S.R.].

Special attention is given to the ways that wave movement of real liquids differs from wave movement of ideal liquids. The author gives the criteria for the stability of initial flow, of attenuation, and for breaking of translation waves. By theoretical analysis of unstable initial flow and the formation of unattenuated waves, the author comes to the conclusion that the classification of flow as subcritical and supercritical is not sufficient. He adds also "oversupercritical flow." When applied to large rectangular channels, the criteria are expressed as:

(1) subcritical $V_0 < V_c = \sqrt{gH_m}$; (2) supercritical $V_c \leq V_0 < V_c'' = 3\sqrt{gH_m}/2$; and (3) oversupercritical $V_0 > V_c''$,

with $H_m = A/B$. For the first two classifications, waves attenuate when they propagate, the flow is stable, and the wave breaks only if its creation is sufficiently sudden. In the case of oversupercritical flow, every disturbed surface is not attenuated, but is increased resulting in formation of new shorter waves having greater heights, so that the wave breaks by progressing. Oversupercritical flow is not stable, and the waves created contract. It is characteristic of such flow that the negative wave does not become less steep, but may, on the contrary, become steeper by wave translation.

442. Veen Van, Johan, 1947, The calculation of tides in new channels: Am. Geophys. Union Trans., v. 28, p. 861–866.

The numerical solution of the large number of simultaneous partial differential equations used in the theoretical design of the Netherlands waterways has previously been done manually at enormous expense of time and money. This paper shows how the same ends may be accomplished by an electrical network analogy.

[From Bibliography on Tidal Hydraulics, Corps of Engineers, February 1954.]

443. Bachet, 1948, Méthodes graphiques d'annonce des crues [Graphical methods of flood forecast]: Houille Blanche [France], no. B, p. 720–728.

The same concept is used as in the author's earlier paper (1934), namely, that a discharge (q) is the sum of the steady flow (r) and the changing discharge with $q=r+\rho$. It is developed that $dx=(dr/dH)(dt/B)$, and $\partial\rho=-(\partial\rho/\partial x)dx$, so that the wave is propagated by a translation $(1/B)dr/dH$ and a vertical change $\partial r=-(\partial\rho/\partial x)dx$, called "attenuation." The fundamental equation is $B\partial H/dt+\partial\rho/\partial x=o$, where ∂H is the height variation due to attenuation. If two cross sections of the propagating wave are under consideration, the lost volume due to attenuation is equal to the sum of complimentary discharges lost from those sections during the time period considered. The complementary discharge is found to be $\rho=(1/2S)(dH/dr)(r\partial r/\partial x)$. The flood routing is based on translation and attenuation factors. The functions $r=f_1(x-Ct)$ and $\rho=f_2(x-Ct)$ are introduced. The procedure for practical flood routing is developed, based on the formula $\Delta H=\theta\tau(\partial^2 H/\partial t^2)$, where θ and τ are two constants to be computed. The templates are given for the graphical flood routing.

444. Barrows, H. K., 1948, Floods: Their Hydrology and Control (Flood routing and effect of reservoirs upon flood peaks, p. 73–79): New York, McGraw-Hill Book Co., p. 1–432.

The Muskingum flood-routing method and procedure of routing is given and the effect of reservoirs on flood-peak attenuation is briefly discussed.

445. Bourguignon, M. P., 1948, Relevés d'intumescences dans les ovarages d'amanée et de restitution de l'usine de Kembs [Surge records in the head-race and tail-race canals of the Kembs water powerplant]: Houille Blanche [France], no. B, p. 699–719.

The records which have been obtained give an approximate confirmation of the results of theoretical and experimental studies made up to 1947. It is found that the propagation celerities of waves approximate Boussinesq's formula

$$C=V\pm\sqrt{gH}[1+(1-2H/3B)3h/4H],$$

where C=propagation celerity, V=velocity in the canal, H=mean depth of water, h=height of wavefront, B=width of water surface. Observations for secondary waves agree with Favre's results. Wavefronts have the tendency to attenuate and to approach the solitary wave, in this respect.

446. Courant, R., and Friedrichs, K. O., 1948, Supersonic flow and shock waves: New York Inter-science Publishers Inc.

The shallow water theory is briefly discussed on pages 32–35. The treatment of supersonic flow and shock waves is useful whenever an analogy can be drawn to compare waves in compressible fluids with the water waves.

447. Culp, M. M., 1948, The effect of spillway storage on the design of upstream reservoirs: Agr. Eng., v. 28, p. 344–346.

Starting from the simple triangular shape of the inflow hydrograph and the spillway storage as a function of reservoir characteristics, a formula for the ratio $\beta = Q_m/P_m$ (peak of outflow to peak of inflow), is given as

$$\beta = 1.25 \pm \sqrt{(18/AD)(ad + 0.85d^2\sqrt{a/S}) + 0.06},$$

where A=area of river basin in acres; D=average depth of flood runoff in inches, a=area of reservoir surface at crest-level of spillway in acres, d=stage in feet above the spillway crest; S=average slope of banks of reservoir through range of stage d, in percent.

448. Friedrichs, K. O., 1948, On the derivation of the shallow water theory (An appendix to the article: The formation of breakers and bores, by J. J. Stoker): New York Univ. Commun. Pure and Appl. Math., v. 1, no. 1, p. 81–85.

The shallow-water theory is derived by means of a formal perturbation procedure that is essentially a development with respect to a parameter $\sigma = Kh$, in which K is the maximum initial curvature of the free surface and h is the depth. When σ is small, the water is said to be shallow. It is shown that in the derivation of higher order approximations for flows in shallow open channels, it is not necessary to make as many special physical assumptions as were made by Boussinesq (1877).

449. Gil'denblat, Ya. D., Makulov, V. V., and Semikolenov, A. S., 1948, Neustanovivshiysya rezhim nizhnego byefa GES [Unsteady flow regime in the tail-race canal of hydroelectric power stations]: Sbornik Problemy regulirovanya rechnogo stoka [Collection: Problems of river flow regulation]: Akad. Nauk [U.S.S.R.], no. 2, p. 43–142.

The first part is the investigation of organization, methodology, and results of observations of the hydraulic regime of tailrace levels of hydroelectric power stations during daily regulations. The second part is the comparison of the theoretical results with the data of observations in the nature. The method of finite differences (instantaneous regimes) applied to two partial differential equations and the method of characteristics are used; the results of both methods are compared with the observations. The departures are relatively small.

450. Keller, J. B., 1948, The solitary wave and periodic waves in shallow water: New York Univ. Commun. Appl. Math, v. 1, no. 4, p. 323–342.

The author contends that the existence of the solitary wave has not yet been proved mathematically and that the method of proceeding to higher approximations is also obscure. He further contends that when the actual solutions for stationary waves are found, progressive waves may be obtained from them by the addition of constant fluid velocity. This paper discusses waves of permanent type in shallow water. The method is that of expending systematically the solution of the exact hydrodynamic problem in powers of a dimensionless parameter $\sigma = (\omega h)$, where h is the depth of undisturbed fluid, and ω is the curvature at some point on the surface. The solution of permanent form is given by the first and second approximations.

451. Keulegan, G. H., 1948, Gradual damping of solitary waves: U.S. Natl. Bur. Standards Jour. Research, v. 40, p. 487–498.

The problem of damping due to the viscous action of translation waves is treated. A short explanation of Boussinesq's boundary layer theory for wave motion is given and expressions for the damping of rectangular and solitary waves are derived. Russell's experimental results for solitary waves are compared with the theory, and satisfactory agreement is found to exist. This verification permits the use of the resultant formulas in model tests on harbors, and so forth, to correct for the dissipative effects of damping of shallow-water waves.

452. Lemoine, R., 1948, Sur les ondes positives de translation dans les canaux et sur le ressaut ondulé de faible amplitude [On the positive translation waves in the canals and on the undular hydraulic jump with small amplitude]: Houille Blanche [France], no. 2, p. 183–185.

The yielding of waves in a powerplant's headrace canal, as a consequence of a sudden switch-off of the units, is analyzed for the Kembs water powerplant. According to the surge theories (and methods by Massé and Craya), an increase in the front's steepness could be anticipated. Experimental results show that no such phenomenon occurs, and undular waves of the jump are observed. Based on Favre's work (1935), the undular jump traveling upstream is analyzed and explained. The difference is emphasized between a positive wave propagated upstream in water at rest and a positive wave (more stable) traveling on flowing water.

453. Linsley, R. K., Foskett, L. W., and Kohler, M. A., 1948, Use of electrical analogy in flood wave analysis: Internat. Assoc. Sci. Hydrology, Oslo, Aug. 1948, Comptes rendus et rapports, no. 19, v. 1, p. 221–227.

Electrical analogy is used to solve the flood-routing problem of the storage equation, using the Muskingum method. It is shown that $W=K[XP+(1-X)Q]$, where K=storage factor with dimension of time, and X=weighting factor for inflow. The electric current flowing into a condenser is equal to the time rate of change of charge of W on the condenser (dW/dt), and the total charge on the condensers C_1 and C_2, at any time, depends on the potential drop across them. It is proved that the electrical storage in the devised circuit is analogous to the channel storage assumed in the Muskingum method, when $C_1=C_2$ and when the two resistances are equal, or $R_1=R_3$. The method is described, and examples of flood routing for different K and X are given.

454. Linsley, R. K., Foskett, L. W., and Kohler, M. A., 1948, Electronic device speeds flood routing: Eng. News-Rec., v. 141, no. 26, p. 64–65.

This is a description of an electronic device for speeding flood-routing computations based on the Muskingum method of expressing the storage-flow relationship. The device includes condensers, resistors, self-balancing potentiometers, and other equipment.

455. Meleshchenko, N. T., and Yakubov, M. S., 1948, Metodika raschetov neustanovishegosya dvizheniya v otkrytykh ruslakh po methodu S. A. Khristianovicha [The methodology of computation of uusteady water movement in open channels by the method of S. A. Khristianovich]: Vsesoyuzniy Nauchno-issledovatel'niy Inst. Gidrotekhniki [U.S.S.R.] Izv. (Trans.), v. 38, p. 29–70.

This paper results from studies made during the period 1938–40. The basic two partial differential equations and elementary derivation of equations of characteristics for prismatic channels are given. The types

of equations of characteristics for prismatic channels (simple equations), and nonprismatic channels, as well as for natural channels (complex equations), are developed. The finite differences for numerical computation by method of characteristics are introduced, employing characteristics in the plane (x,t) either as straight lines or as curves. The computation applicable in the case of great resistance forces is analyzed with discussion of initial conditions and boundary conditions, for plane (x,t) considered for both steady and unsteady flow. The cases of lateral inflows, and channel junction are discussed. Computation of unsteady flow in prismatic and natural channels by means of the method of characteristics is shown in two examples. A simplified method for computation of daily water regulation is given and a nomographic procedure for computing unsteady flow in a prismatic channel is developed, and an example of use for daily regulation in powerplant canals is given. The method of characteristics is discussed, noting the mechanical character of the computation when employed for standard operations, and noting further that the method is practical for use in those cases when forces of inertia prevail over resistance forces.

456. Meleshchenko, N. T., and Yankubov, M. S., 1948, Methodika rascheta preryvnoy volny v prizmaticheskom rusle [The methodology of computation of positive surges in the prismatic channel]: Izv. Vsesoyuznogo Nauchno-issledovatel'nogo Inst. Gidrotekhniki [U.S.S.R.], v. 38, p. 71–94.

This work was done in 1940. It is the continuation of work done in 1938–40 and published in 1948 on the method of characteristics. The movement of a positive surge (traveling bore) is studied and computed by method of characteristics. First, the equations for steep positive wave traveling downstream are derived in a simplified form, in which $C=V_m+aC_m$, where a is a function of H_1 and H_2 (depths in front of and behind the surge), $V_m=\frac{1}{2}(V_1+V_2)$, and $C_m=\frac{1}{2}(C_1+C_2)$, where $C_1=\sqrt{gH_1}$ and $C_2=\sqrt{gH_2}$. The procedure of wave computation is given, as is discussion of initial and boundary conditions and an example of computation. An approximate and simplified computation of wavefront deformation is derived, inasmuch as the more exact procedure is laborious. Also, a simplified computation of the change of free water surface at the place of created surge is developed. An example of simplified computation of positive surge waves is given for powerplant operations.

457. Putman, J. H., 1948, Unsteady flow in open channels: Am. Geophys. Union Trans., v. 29, p. 227–232.

Massau's method (1900) of characteristics in integrating two basic partial-differential equations is given as a short summary.

458. Stoker, J. J., 1948, The formation of breakers and bores; the theory of nonlinear wave propagation in shallow water and open channels: New York Univ., Commun. Appl. Math., v. 1, no. 1, p. 1–87.

As this subject is also treated extensively in the author's book "Water Waves" (1957), only the content of the paper is here reported. The author's contribution is mainly in the mathematical treatment of breaker and bore formation, particularly in his use of gas-wave versus water-wave analogy. Basic differential equations are given in the introduction. The paper discusses: nonlinear shallow-water theory; integration of differential

equations by the method of characteristics; the motion of a single wave; propagation of pulses into still water of constant depth; propagation of depression waves into still water of constant depth; shock conditions; constant shocks as bores, hydraulic jump, reflection from a rigid wall; the breaking of a dam; flood waves in rivers; breaking of waves in shallow water of uniform depth; and breaking of waves on a uniformly sloping beach.

459. Veen Van, Johan, 1948, Estimates of tidal currents: Panama Canal Spec. Eng. Div., I.C.S. Memo 299, Feb. 18.

Results of computation of velocities to be expected in an open Panama Sea-Level Canal. Studies made by Dr. Van Veen, Chief Engineer at the Rijkswaterstaat, The Hague, Netherlands, from data furnished by the Special Engineering Division, Panama Canal. Comparison between velocities computed by Dr. Van Veen, and those of the Special Engineering Division using Gen. Pillsbury's method. Includes article "Analogy between Tides and A.C. Electricity" by Dr. Van Veen.

[Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, February 1954.]

460. Viti, M., 1948, Le equazioni differenziali del moto lineare non permanente per i liquidi incomprimibili [The differential equations of the linear unsteady motion for noncompressible liquids]: Acqua [Italy], Mar., no. 1–3.

461. Barber, N. F., 1949, Behaviour of waves on tidal streams: Royal Soc. [London], ser. A, v. 198, no. 1052, p. 81–93.

Discussion of the manner in which waves change characteristics when passing through regions where water has streaming motion; application to tidal streams where velocity depends on both time and position; and experimental data.

[Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, February 1954.]

462. Citrini, Dulio, 1949, Sull'attenuazione di un'onda positiva ad opera di uno sfioratore laterale [On the attenuation of a positive wave affected by a side spillway]: Energia Elettrica [Italy], v. 26, no. 10, p. 589–599. Reprinted as Memorie e Studi, Istituto di Idraulica e Costruzione Idrauliche, Milano, no. 76, 1949.

The passage of a positive (downstream or upstream directed) wave along an open channel which has a lateral spillway is studied by the use of characteristics. Although the quantitative results are considered to be only approximate, the study presents clear evidence of interrelationships of wave movement (particularly wave attenuation) and the geometric and hydraulic elements which characterize the stream current and the spillway. The method of characteristic lines is given and applied. The conclusion is that the spillway does not influence the wave celerity. Relative-value relationships, giving attenuation of depth or discharge, are developed as a function of the ratio of spillway length to canal width.

463. Clark, C. O., 1949, Application of flood storage accounting methods: Am. Geophys. Union Trans., v. 30, p. 528–532.

Employing his flood storage accounting method, discussed in an earlier paper (1945), the author demonstrates its use in the day-by-day application to forecast flow of the Roanoke River.

464. Crossley, H. E., Jr., 1949, The analogy between surface shock waves in a liquid and shocks in compressible gases. Experimental study of hydraulic jump interactions: Calif. Inst. Technology Lab. Rept. N–54.

465. Dmitriyev, G. T., 1949, Vychisleniye karakteristik ustanovivshegosya plavno izmenyayushchegosya dvizheniya v prizmaticheskikh ruslakh [Computation of characteristics of a steady but gradually varying movement in prismatic channels]: Akad. Nauk [U.S.S.R.], Doklady, v. 68, no. 5, p. 825–827.

Using Agroskin's method, as developed from the theory of steady but gradually varying flow, characteristics of the differential equations for unsteady flow are analyzed for two waves of opposite flow direction. The analysis is given for three different slopes of canal bottom, S: $S=0$; $S>0$; and $S<0$. The wavefront shape is derived for the waves.

466. Dressler, R. F., 1949, Mathematical solution of the problem of roll-waves in inclined open channels: New York, Interscience Publ. Inc, New York Univ. Inst. Math. Sci., Commun. Pure Appl. Math., v. 2, no. 2, 3, p. 149–194.

The purpose of this paper is to obtain solutions which are periodic with respect to distance, describing the phenomenon called "roll-waves." The discontinuous periodic solutions can be constructed by joining together sections of a continuous solution through shocks (or "bores"). It is shown first that no continuous solutions can be periodic and that only one special continuous solution can be used as the basis for constructing discontinuous periodic solutions. The analysis is based upon the non-linear partial differential equations of the "shallow water theory," augmented by the Chezy formula to allow for turbulent resistance. The Bresse profile equation is obtained in a form applicable for progressing wave flows. Shock conditions are derived for the case of an arbitrary continuous channel bed and for a flow subject to a resisting force. It is proved that roll-waves cannot occur if the resistance is zero or if the resistance exceeds a certain critical value. As the resistance decreases, the size of the waves decreases also; and if the resistance becomes too large, the profiles reverse their direction and can no longer be joined by shocks. This critical value is reached when the (dimensionless) resistance coefficient equals one-fourth the value of the channel slope. The resistance must also act in such a manner that it decreases as the water depth increases. The analysis proves that the ratio of wave height to wave length of roll-waves is always independent of the speed of the waves. Explicit expressions for water height and shock height as functions of wave length are derived. The investigation studies the static discharge rate as a function of the wave speed, and asymptotic formulas for the wave speed in terms of the average discharge rate are derived. Twelve sets of curves are presented. For prescribed values of slope, resistance, and wave speed, there is a one-parameter family of roll-wave solutions. If the wave length is also prescribed, the solution will then be unique.

[From author's summary.]

467. Dronkers, J. J., 1949, Aperçu des methodes pour la determination du movement de marée dans les embouchures et les fleuves à marée néerlandais [Review of methods for the determination of tidal movements in the entrances and in tidal rivers of Netherlands]: Internat. Cong. Navigation, Lisbon, rap 17c, sec. II., quest. 1, p. 159–178.

468. Johnstone, D., and Cross, W. P., 1949, Elements of applied hydrology, Chap. 7, Flood routing: New York, The Ronald Press Co., p. 160–191.

The chapter includes discussion of the simple storage equation; an example of flood routing through reservoirs and retarding basins with storage as a function of discharge alone; an example of flood routing by use of a mass diagram and a storage-factor ($2W/\Delta t+Q$) curve; discussion of flood routing in a stream, where storage is used as a function of inflow and outflow, derivation is given of the storage relationship for a reach, and an example of the flood routing is shown. Special problems are discussed, such as the accounting for intermediate inflow and backwater effects. Effects of flood-control projects are shown by computations. Mechanical aids and short cuts are discussed.

469. Keller, J. B., 1949, The solitary wave and periodic waves in shallow water: Acad. Sci. [New York] Annals, v. 51, act. 3, p. 345–350.

This paper gives a brief summary of studies of the permanent waves, with reference to the solitary permanent wave. The author concludes that the existence of the solitary permanent wave has not yet been proved mathematically. He claims that solutions are found for stationary waves and that progressive waves are obtained from them by the addition of a constant fluid velocity. The shape of the wave is developed and discussed. The author theorizes that, if the water at infinity were stationary, the water under the crest of a solitary wave would be moving. According to the solutions of Korteweg and De Vries (1895), and Rayleigh (1911) the water under the crest of a solitary wave would then be stationary.

470. Kreisel, G., 1949, Surface waves: Appl. Math. Quart., v. 7, p. 21–44.

This paper is a mathematical treatment of the asymptotic behavior of wave motions and the definition of the reflection coefficient, the objective being to determine the reflection from obstacles in arbitrary depth. Such reflected and transmitted wave trains are determined as coming up against obstacles, which lie in the bottom of a canal or are fixed in the surface. The simple harmonic oscillations are considered.

471. Lamoen, J., 1949, Tides and current velocities in a sea-level canal: Eng. [London], v. 168, no. 4357, p. 97–99.

The method of characteristics is herein used for solving the tidal-current problem, based on Massau's (1900) and Holster's (1947) studies of this method, but the application given differs slightly from that in Holster's original text. More emphasis is placed on the method of treatment than on the theory. The example of an open-sea Panama Canal is shown, as is the good agreement between the computed results and results of the model test (Meyers-Shultz, 1949), within the framework of assumptions made. The process of computation is useful whenever the method of characteristics is used for flood routing, using the same basic assumptions. The linear law of friction resistance is a basic assumption of this computation process.

472. Linsley, R. K., Kohler, M. A., and Paulhus, J. L. H., 1949, Applied Hydrology: 1st ed., New York, McGraw-Hill Book Co., p. 465–541.

Waves of translation and of oscillation are considered here. Translation waves treated include monoclinal rising waves, abrupt translatory waves, surges, flood waves due to dam failure and seiches. Oscillatory waves and tides are briefly discussed. Natural flood waves and channel storage are discussed, and it is shown how the channel storage function can be derived from hydrographs. With reference to routing based on storage equations, the following procedures are given: Muskingum, semigraphical, graphical, and by routing machines. Nonstorage routing procedures discussed (based on correlations and on successive averages) include methods of adjustment for local inflow, variable stage-discharge relations, and gage relations. Stage routing is shown, as are the typical curves used. Travel time is analyzed.

473. Lowell, S. C., 1949, The propagation of waves in shallow waters: New York Univ, Ph. D. thesis.

474. Meyers, J. S., and Schultz, E. R., 1949, Tidal currents: Am. Soc. Civil Engineers Trans., v. 114, p. 665–684.

The tidal currents in an open-sea Panama Canal are treated by means of a small-scale model. The study is chiefly concerned with velocities of currents.

475. Munk, W. H., 1949, The solitary wave theory and its application to surf problems: Acad. Sci. [New York] Annals, v. 51, act. 3, p. 376–424.

Testing of the solitary wave theory, as applied to surf problems, was performed by field observations, by laboratory experiments, and by theoretical approaches.

The results of observations made to test the solitary wave theory exhibit so much scatter that they cannot be said to constitute a confirmation of theory comparable to an inductive laboratory confirmation of physical law. However, verifying evidence is strong enough to indicate that the solitary wave theory provides a useful approach to the study of various surf phenomena.

The difficulties of obtaining reliable observations of some of the features described are very great. In the field, extreme variability in the wave train itself is likely to obscure meaningful relationships. In the laboratory, it is difficult to avoid interference by reflected waves and surging. Because of these inherent uncertainties in measurements, a theoretical framework is almost essential for establishing useful relationships.

476. Ramponi, Francesco, 1949, Modello di un'onda di piena fluviale [Model of a river flood wave]: Energia Elettrica [Italy], no. 11, 12, p. 699–700.

A discussion of the selection of satisfactory scales to use for deformed river models, having a fixed bed, as exemplified in the case of a reach of the Adige River for the flood regime. The relation of scales for different hydraulic elements is given as the study result.

477. Ruggiero, C., 1949, Ondes graduelles dans les cours d'eau [Gradual waves in channels]: Intern. Assoc. Hydraulic Research, Convention, Grenoble [France], 1949, Rept.

478. Scimemi, Ettore, 1949, Risultati sperimentali sulla propagazione delle perturbazioni di regime nei canali [Experimental results of the propagation of regimen perturbations in channels]: Energia Elettrica [Italy], no. 11, 12, p. 691–696.

The paper treats the results of a study of wave movement that involves experiments in nature (canal of powerplant Cismon-Moline, Italy). Model experiments are described, and the computations of wave characteristics given. The canal of the powerplant and of the experimental laboratory (canal model 1:25) is described, as are the experiments. Forchheimer's

formula for wave celerity is used. The results of two types of experiments are compared with the results by computation. The maximum wave height, as computed by the Forchheimer formula for initial positive wave, may increase by a combination of reflected waves, but can be substantially decreased if the spillways are well designed. Wave profiles for different types of spillways are given.

479. Sokolovskiy, D. L., 1949, O metodakh ucheta transformatsii maksimumov v prudakh pri proektivovanii iskusstvennykh sooruzheniy [On the methods of computation of transformation of flood peaks in the pools for the design of artificial structures]: Stroitel'stvo Dorog [Highway Construction] [U.S.S.R.], no. 4.

480. Sorenson, K. E., 1949, Curves solve reservoir flood-routing equations: Civil Eng., v. 19, no. 11, p. 56–57.

The flood routing is based on the storage equation, arranged in the form $(W_1+\frac{1}{2}(Q_1\Delta t)+[\frac{1}{2}(P_1+P_2)-Q_1]\,\Delta t)=W_2+\frac{1}{2}(Q_2\Delta t)$. The following curves are used: $W+\frac{1}{2}(Q\Delta t)=f(H)$, $W=f(H)$, $Q=f(H)$, and $P=\frac{1}{2}(P_1+P_2)=f(t)$, $Q=f[W+\frac{1}{2}(Q\Delta t)]$. The functions $H=f(t)$, and $Q=f(t)$ are computed. A graphical method is used, and a combined graphical and nomographic solution is shown.

481. Stoker, J. J., 1949, The breaking of waves in shallow water: Acad. Sci. [New York] Annals, v. 51, act. 3, p. 360–375.

The principal objective of the paper is to interpret certain well known phenomena in gas dynamics in terms of water-wave phenomena, considering, particularly, the analogy between the shock wave in compressible gas and a breaker of water wave. The method of characteristics is used for solutions. The author explains why waves break in shallow water and applies his theory to problems found in studying the down-river progress of flood waves and surges. He concludes that waves break in different ways, depending upon the particular circumstances, among which an important factor is the comparison between the depth of water and the wavelength.

482. U.S. Army Corps of Engineers, 1949, Hydraulic design; surges in canals: U.S. Army Corps Engineers, Civil Works Construction, Eng. Manual, pt. 106, chap. 6, p. 1–13.

The chapter is confined to the problem of surges that occur in navigation canals as a result of lock operations. The generation of surges is discussed. The analysis of reflection, stability, and longevity of surges is given. The surge control and model test are presented briefly. A table at the end of chapter gives equations of the physical characteristics for four kinds of surge: positive upstream and downstream surges, and negative upstream and downstream surges.

483. Volker, F., and Schoenfeld, J. C., 1949, Le régime d'une rivière sous l'effet de débits variables [The regime of a river under the variable flows]: Internat. Cong. Navigation, Lisbon, 1949, rap. 17e, sec. 1, quest. 3, p. 59–78.

484. Adler, G. F. W., 1950, Model tests on Clachan underground power station: English Elec. Jour. [Great Britain], v. 11, no. 4, p. 119–127.

Experiments on wave movement in tailrace underground channels having free water surface are described and analyzed. Calculation of wave

heights and celerities by the momentum method gives reliable results in the case of reasonably uniform velocity distribution and a low rate-of-change of discharge. The real values are somewhat lower than the computed because of friction and wave reflection.

485. Arthur, R. S., 1950, Refraction of shallow water waves: The combined effect of currents and underwater topography: Am. Geophys. Union Trans., v. 31, p. 549–552.

A solution is given for determining the refraction effect according to Fermat's Principle for the case of shallow water waves moving in any given distribution of current and depth. Application is made to an analytic model of an intense rip current, and the results are compared to actual rips. [Author's abstract.]

486. Bergeron, Louis, 1950, Du coup de belier en hydraulique au coup de foudre en électricité. Méthode graphique générale [From the water hammer in hydraulics to the voltage surge along electrical power lines]: Paris, Dunod, p. 284–289.

The same graphical construction that is used to follow wave progression in the case of a water hammer in a conduit is applied to surge movement in an open surface canal. This is an application of the method of characteristics, used in a simplified form.

487. Biesel, F., 1950, Étude théorique de la houle en eau courante [Theoretical study of wave in running water]: Houille Blanche [France], no. A, p. 279–285.

The propagation of a wave in channels with running water is studied analytically employing approximations of the first order. This is a two-dimensional problem considering an ideal fluid in a large channel. The author's conclusions are as follows:

1. For very short waves, the difference between wave propagation celerity and the velocity of flowing water at the surface is equivalent to the propagation celerity of very short waves in still water.
2. The difference between propagation celerity of long waves and the mean velocity of flowing water is generally greater than the propagation celerity of long waves in water at rest.
3. This difference is greater by the amount of difference in surface and bottom velocities.
4. In general, the mean critical velocity is slightly greater than $\sqrt{gH}$.

488. Citrini, Dulio, 1950, Sull'efficacia di uno sfioratore laterale nelle manovre di arresto completo [On the effectiveness of a side weir in the unsteady motion following full rejection of load]: Energia Elettrica [Italy], v. 27, no. 2, 1950 p. 77–80. Reprinted as Memorie e Studi, Inst. di Idraulica e Costruzione Idrauliche, Milano, no. 78, 1950.

The validity of a formula for computing the attenuation of a positive wave in a canal, the result of the operation of a lateral spillway, is proved in the limits defined in the paper, and on the basis of the recent experiments. This formula was developed in an earlier study (Citrini, 1949). A graph is developed from the same formula, which is of immediate practical value for special operations producing complete stoppage of flow in the canal. [Author's summary translated.]

489. Finzi, Bruno, 1950, Caratteristiche dei sistemi differenziali e propagazione ondosa [The characteristics of differential systems and wave propagation]: Energia Elettrica [Italy], v. 22, no. 4, p. 189–195.

This theoretical study contains: an analysis of wavefronts that considers the function of wavefront movement and develops celerity

$$C = -(\partial\tau/\partial t)/|grad\tau|;$$

a discussion of the method of characteristics; a discussion of the kinematic and dynamic conditions relating to discontinuities; analysis of the propagation of sound in liquids; and analysis of the propagation of tidal waves, in which is developed the wave celerity formula $C=\sqrt{g(H+h)}$.

490. Gentilini, Bruno, 1950, L'azione di uno sfioratore laterale sull'onda positive assendente in un canale (risultati di nuove experienze) [The effect of a side weir on a positive ascending wave in a canal (the results of the new experiments)]: Energia Elettrica [Italy], v. 27, no. 1, p. 1–10. Reprinted in Memorie e Studi, Istituto di Idraulica e Contruzione Idrauliche, Milano, no. 79, 1950.

The effect of a lateral longitudinal spillway on a translation wave (positive upstream) is studied experimentally, in the case when the wave is created by sudden stoppage of flow in a canal which has this lateral spillway. Conditions of the experiments are described and results given for the spillway effects on wave attenuation. The lateral spillway produces an attenuation which corresponds to that computed by the theoretical approach (Drioli, 1937). No difference in wave attenuation is noted, whether the spillway is on one side of the canal or whether there are two spillways, one on each side, having a total length equal to that of the single lateral spillway.

491. Gilcrest, B. R., 1950, Flood routing: Engineering Hydraulics (H. Rouse, ed.), New York, John Wiley and Sons, Chap. 10, p. 635–710.

The chapter is divided in four parts:

1. Introduction, which treats the general flow-routing problems and their classification;
2. Mathematics of flood routing, which treats the differential equations for unsteady flow, rate of travel of flood waves, wave attenuation, discharge in unsteady flow, two approximation methods based on neglect of the momentum equation, and the third approximation method, based on two differential equations;
3. Routing of floods through reservoirs, which treats the reservoir-storage characteristics, the use of storage factors (or working values $W+\frac{1}{2}Q\Delta t$ or $W/\Delta t+Q/2$); the use of slide rules for routing, the various factors affecting reservoir storage, the mechanisms for routing; and
4. Routing of floods through open channels which treats the stage-discharge-storage relations, attenuation of flood waves, application of coefficients in Muskingum method, the analysis of wedge storage, routing through junctions, complete methods of solution, solution by the method of characteristics, and the mechanical and electrical devices for flood routing.

492. Guerrero, E. C., 1950, Analisis del paso de avenidas por vasos reguladores [The passage of floods through control reservoirs]: Ingeniera Hidraulica en Mexico, v. 4, no. 4, p. 21–28.

Graphical and numerical flood-routing methods based on the storage equation and currently in use are described and applied. Particular attention is given to the Goodridge method.

493. Keulegan, G. H., 1950, Wave motion: Engineering Hydraulics (H. Rouse, ed.), New York, John Wiley and Sons, Chap. 11, p. 711–768.

The chapter contains 7 parts: (1) preliminary considerations, which treats the classification of waves and method of analysis; (2) shallow-water waves, which treats translation waves, solitary waves, negative waves, and boundary resistance; (3) deep-water waves, which treats waves of finite amplitudes, configuration of waves at maximum height, and effect of internal resistance; (4) oscillatory waves in shallow water; (5) transformation of waves, which discusses shallow-water waves in channels of variable cross section, deep-water waves over sloping beds, and wave pressure on vertical walls; (6) open-channel surges, which shows surge characteristics, propagation of a discontinuous surge front, intermittent surges on steep slopes, and propagation of surges on a dry bed; and (7) internal waves due to the variation of density, the propagation of interfacial waves and their stability, and density currents.

494. Kritskiy, S. N., and Menkel', M. F., 1950 [Russian], Gidrologicheskie osnovy rechnoy gidrotekhniki [Hydrologic basis of river hydraulics engineering]: Akad. Nauk [U.S.S.R.].

495. Seemann, D., 1950, Die Kriegsbeschaedigungen der Edertalsperrmauer, die Wiederherstellungsarbeiten und die angestellten Untersuchungen ueber die Standfestigkeit der Mauer [War damage to the Eder Dam; repair work and testing the stability of the dam]: Die Wasserwirtschaft [Germany], year 41, p. 1–7 and p. 49–55.

Stage hydrographs of the flood wave, created by the destruction of the dam, are given for the dam site and for six other sites 51 to 362 kilometers downstream.

496. Sretenskiy, L. N., 1950, Volny [Waves]: In "Mekhanika v SSSR za tridtsat let (1917–47)", Gosudarstvennoe Izdatel'stvo Tekhniko-teoreticheskoy Literatury [Moscow, Leningrad], p. 279–299 (Hydrodynamics).

The contributions by the scientists of U.S.S.R. in the field of wave theory are reviewed from the standpoint of the mathematical approaches employed. Included are discussions of the results of studies of waves of finite amplitude, infinitely small waves (two dimensional problems, and three dimensional problems), and the theory of tidal waves. The references included cover a total of 80 Russian theoretical studies in the field of wave theory.

497. Supino, G., 1950, Sur l'amortissement des intumescences dans les canaux [On the attenuation of steep waves in channels]: Rev. Gén. l'Hydraulique [Paris], no. 57, p. 144–147.

It is shown by analysis that it is possible for a wave progressing downstream in rapid flow, $V^2/gH>2.25$, to increase in height. This theory, also held by Massé, could be confirmed only by experience. By use of Lamoen's method of linearization, only attenuation of the wave could be shown. This study recommends the author's first method of linearization.

498. Supino, Giulio, 1950, La propagazione delle onde nei canali [Wave propagation in channels]: Energia Elettrica [Italy], v. 22, no. 4, p. 196–205.

This paper studies the wave propagation in channels with free surface flow using the two hydraulic equations (continuity and dynamic). It is shown: (1) that the different forms of dynamic equation are essentially equivalent for the study of linearized waves; (2) that it is possible to integrate the nonlinear equation in some cases; (3) that the inconvenience stressed by Bonvicini (1934) can be eliminated; this is for the wave which stops attenuating for the slopes 2.6 times the critical slope; (4) that some results can be obtained very rapidly by the theory of characteristics, and some limitations are put forward for the use of this theory.
[Author's summary translated.]

499. Wicker, C. F., and Rosenzweig, O., 1950, Theories on tidal hydraulics: Comm. Tidal Hydraulics, Rep. 1, p. 101–125.

500. Chow, Ven Te, 1951, A practical procedure of flood routing: Civil Engineering and Public Works Rev. [Great Britain], v. 46, no. 542, p. 586–588. Reprinted as: A practical procedure for flood routing, Ill. Ill. Univ. Civil Eng. Studies, Hydr. Eng. Ser., no. 1, 1951.

The basic storage equation is used in the form

$$2W_1/\Delta T - Q_1 + P_1 + P_2 = 2W_2/\Delta T + Q_2.$$

The historical data of floods in a reach are used for the construction of characteristics curves. The ΔT unit is used to that $2/\Delta T = 1$, and curves $W+Q$ and $W-Q$ are used. An example is given. Instead of using loop curves for storage of river reaches (for rising and falling stages), a mean $W+Q=f(W)$ curve is used, and a graphical procedure is given for computing the mean curve. Five related factors or conditions for use of the above procedure are discussed: selection of reaches, routing period, storage function, flow less than critical, and local inflows.

501. Daily, J. W., and Stephan, S. C., 1951, Characteristics of the solitary wave: Am. Soc. Civil Engineers Trans. v. 118, p. 575–587.

The solitary wave is analyzed with reference to its wave classification and discussed in terms of its three principal characteristics of celerity, form, and velocity of liquid particles. The energy loss due to friction is also discussed. Results of earlier studies of these properties are reviewed. Experiments made at the Massachusetts Institute of Technology concerning these properties are discussed in comparison with the theoretical studies, showing that the experiments produced a value for celerity that is somewhat smaller than that obtained by theoretical formulas and produced an expression for wave form that is very close to the Boussinesq sech-function. The study further concludes that theories for motion and displacement of liquid particles have been confirmed only qualitatively, and that 90 percent of wave energy is contained within the limits $y/h \geq 0.46$.

502. Drouhin, Mallet, and Pacquant, 1951, Contribution à l'étude des débits de crue et du dimensionnement des évacuateurs [Contribution of the study of flood discharge and the design of spillways]: Internat. Comm. Large Dams, 4th Cong., New Delhi, 1951, Trans. v. 2, p. 461–578.

For the determination of spillway characteristics, a parameter $r = P_{max}/Q_{max}$ is introduced in the formula. The storage function is used, and a method of computing spillway length is given. If, however, Q_{max} is imposed, the maximum water stage at the dam can be determined by the formula developed. Graphical methods for flood routing are given, using the

storage factors $W+\frac{1}{2}Q\Delta t$ and $W-\frac{1}{2}\Delta t$ in different combinations, based on the storage equation.

503. Dupouy, M., 1951, Ondes de crues et reglettes Bachet [Flood waves and Bachet templates]: Houille Blanche [France], no. A, p. 230–231.

Comparison is made of the transitory regimes of telegraphic signals with unsteady flow, which linearizes hydraulic regimes by taking small increments ∂Q of discharge as discontinuous changes and derives the relation between a hydrograph $Q=f(t)$ and the so-called "percussional admittance" of signal transmission. The Bachet templates (1934) for flood forecasting can theoretically be based on this relationship. Rating-curve relation used for plotting curves of templates is introduced, and use of the templates is discussed. The author makes a theoretical approach from other angles to derive templates for flood routing.

504. Eckart, C., 1951, Surface waves on water of variable depth: Scripps Ins. Oceanography, Lecture Notes, Fall Semester 1950–51, Ref. 51.12, Wave Rept. 100.

Starting from the linearized equations of hydrodynamics, it is possible to give a logically consistent treatment of the propagation of waves in water of variable depth. This theory does not, in all respects, conform to observation; it was the intention to exhibit the assumptions that underlie the theory, so that later investigators might alter them with a view to improving the theory. The later parts of Section 7 show in detail how the theory fails when the water depth becomes very small.

[From author's preface.]

505. Frank, J., 1951, Betrachtungen ueber den Ausfluss beim Bruch von Stauwaenden [Considerations on the outflow from dam breaches]: Schweizer. Bauzeitung [Zurich], no. 29, p. 401–406.

Outflow hydrographs resulting from dam breaches are studied by a simple analytical approach. Six assumptions are made: friction along the reservoir and singular resistances are negligible; the breach is rectangular; the crest is horizontal and perpendicular to flow; the reservoir has a constant mean depth, and horizontal bottom; width of the water surface is either very great or infinite; the water in the reservoir is at rest before the breach occurs and there is no tailwater effect. Starting from De Saint-Venant and Ritter equations, new equations are developed for the maximum discharge. The Eder Dam breach is used to evaluate the maximum discharge by means of the developed formulas.

506. Freeman, J. C., Jr., 1951, The solution of nonlinear meteorological problems by the method of characteristics: Am. Meteorol. Soc. Compendium Meteorol. p. 421–433.

507. Harkness, F. B., 1951, Harkness flood router; Device for graphically representing flood conditions; specification of construction and operation: U.S. Patent Office, Patent File no. 2, 550, 692.

A mechanical flood-routing device is described that is based on the continuity (storage) equation. Starting from the inflow discharge hydrograph, and considering the storage-out-flow conditions, the flood-routing device traces directly the outflow discharge hydrograph.

508. Hayami, Sho., 1951, On the propagation of flood waves: Kyoto Univ. [Japan], Disaster Prevention Research Inst., Bull. 1, p. 1–16.

Introducing the effect of longitudinal diffusion, caused by the mixing into the equation of continuity and assuming the mean flow, taken over a suitable range, to be steady and uniform, the differential equation of flood waves was derived. It is an equation of diffusion containing a term of advection. As the equation is nonlinear, an approximate method of solution was discussed and solutions were obtained under several conditions. They explain well the properties of flood waves. The approximate equation of flood waves is linear, and a flood of any form is, therefore, supposed to be composed of many elementary flood waves of simple character: unit graph, or unit flood. A method of computing the unit graph was described and some numerical examples were shown. In the last, some of the results of observations made on an artificial unit flood in the Yedo River were compared with the theoretical computations. Their agreement is excellent.

[From author's synopsis.]

509. Hayashi, Taizo, 1951, Mathematical Theory of Flood Waves: Japanese Natl. Cong. Appl. Mech., 1st, Proc., p. 431–436.

This paper deals with the mathematical theory of flood waves in open channels of uniform rectangular cross section. The basic equations used are the equation of motion for gradually varied unsteady flow and the equation of continuity. Solution is carried out by a method of successive approximations with respect to a small parameter σ, which is defined by

$$\sigma = (\partial^2 H/\partial t^2)_0/gs_0{}^2$$

where the numerator is the vertical acceleration of the water level of the peak of a flood wave at a cross section, say at the origin, g=the intensity of gravity and S_0=slope of channel. By the second approximation, the law of diminution of flood waves is given, where the factor of a diminution is expressed as a function of a Froude number, the width of the channel, and the small parameter σ. The law is compared with available data of field experiments.

[Author's abstract.]

510. Hayashi, Taizo, 1951, Mathematical study of the motion of intumescences in open channels of uniform slope: Japan Soc. Civil Engineers Trans. [Tokyo], no. 11, p. 1–11.

This paper is the first of a series dealing with the motion of translation waves in open channels. The case treated is that of an arbitrary intumescence with a small height in a uniform channel with a rectangular cross section containing water originally moving with a uniform velocity. Approximate formulas are derived which give the deformation and motion of the intumescence. The case of the intumescence of the form of a rectangular type is also dealt with and for that the form of the rigorous solution is shown. Special emphasis is laid on the dispersive property of intumescences. A method of illustration of the dispersion is presented and both the propagation speed and the dispersion of intumescences of any arbitrary shape are illustrated. The results are all derived with the method of operational calculus on the plane of the complex variables.

[Author's English synopsis.]

511. Iwagaki, Yuichi, 1951, Theory of flow on road surface: Kyoto Univ. [Japan] Eng. Fac. Mem., v. 13, no. 3, p. 139–147.

By solving the momentum equation of a thin sheet flow on road surface numerically with the condition of continuity obtained under the condition that rain falls on road uniformly, water depth and mean velocity of thin sheet flow and also frictional velocity related with soil erosion of road surface are computed, and then the effects of camber shape and longitudinal slope of road surface on its drainage and stabilization are discussed.

[Author's abstract.]

512. Levin, Léon, 1951, Étude experimental du régime transitoire engendré par la rupture d'un barrage [The experimental study of the transient regime created by a dam breach]: Acad. sci. [Paris] Comptes rendus, v. 233, p. 646–648.

Shape of the wave travelling downstream immediately subsequent to breach of the dam is studied experimentally in a small canal 0.115 meter wide by 2.0 meters long. The study is particularly concerned with effects of canal shape, original depth of water below dam, and friction. It is considered that the first two factors are the most important and that the influence of friction can be neglected. The wave shapes are classified in four categories.

513. Levin, Léon, 1951, Eksperimentalna hidraulika [Experimental hydraulics], Part 4, Unsteady flow in canals and rivers: Belgrade [Yugoslavia], p. 161–216.

Analysis is made of waves in channels that develop by progressing, and classical theories of surges and gradually varied waves are discussed. The methods of finite differences and characteristics are given. One chapter is devoted to a discussion of the author's graphical method, outlined in detail in his later papers (1952). Special attention is given to the similitude between models and prototypes for the experimental studies of unsteady flow. Various model techniques are discussed in terms of the experiment used by the author and others.

514. Nekrassov, A. I., 1951, Tochnaya teoriya voln ustanovivshegosya vida na poverkhnosti tyazheloy zhidkosti [The exact theory of steady waves on the surface of a heavy fluid]: Izdat. Akad. Nauk [Moscow], p. 1–94.

The first chapter deals with surface waves in water of infinite depth. The second chapter treats mathematically the solitary wave of shallow water, starting from the theory and formulas for deep-water surface waves. The paper concludes with treatment of a class of nonlinear integral equations.

515. Potapov, M. V., 1951, Sochineniya [Works]; Book 3 on runoff regulation: Sel'khozgiz, Moscow, v. 3, p. 1–471.

In discussing various treatments of unsteady flow, the author states that the use of finite-differences methods, taking into account the inertia terms, is a very tedious procedure. This method requires detailed topographic data of the channel, and 8 to 10 man-days of work, under the supervision of a highly specialized engineer, for the computation and plotting of the graphs depicting the travel of one wave.

Use of mass-curves of hydrographs is discussed (p. 81–99). The integration of storage equation for spillway discharges is given in approximate form for tabular procedure, as well as for more accurate tabular procedure. The author's graphical method, using curves $W=f(Q/2)$, is given as in a simple graphical method for equal Δt (also in nomographical form). Ap-

proximate analytical methods are discussed and compared (p. 179–205), with reference to the principles of unsteady flow in channels (p. 205–215). The effect of lateral inflow and backcurves is analyzed (p. 215–218).

516. Ransford, G. D., 1951, A contribution to the first-order theory of translation waves: Houille Blanche [France], no. 5, p. 758–763.

The Massé equation for the dynamic partial differential equation of unsteady flow is used in the form where Q, H and V are replaced by Q_0+q, H_0+h, and V_0+v, where g, h, and v are quantities that differ very little for unsteady flow, and where reduced variables are used in dimensionless form. The purpose of this equation was to obtain a solution of the type $f_p(x)e^{pt}$, by integration. The author considers that this type of solution is incomplete, because singular solutions also exist in the form $f_p(x)te^{pt}$. This is proved in the paper. By comparing results of an example, the theory of waves of the first order needs to be modified along the lines indicated above, and according to the Boussinesq formula, the change in shape of the front of wave is due to the term: te^{pt}.

517. Rich, R. G., 1951, Hydraulic transients, Chapter 8, Surges in power canals and tidal harmonics: New York, McGraw-Hill Book Co., p. 214–242.

Johnson's treatment (1922) of the rejection and demand surges in power canals is given, with formulas for changes in levels and for celerity. The effects on surges of channel-bottom slope and channel friction in the power canal is discussed. The numerical computation of the surge in power canal by the trial-and-error method is given. The movement of tidal waves along the channel is presented in general and for the special case of the channel having a lock at one end. The method of Brown (1932) for tidal wave movement in the channel and an example of computation is given. The infinite-series solutions of tidal harmonics is given at the end of the chapter.

518. Schoenfeld, J. C., 1951, Propagation of tides and similar waves: 'S-Gravenhage, Ministerie Van Verkeer en Waterstaad Staatsdrukkerij-en Uitgeverijbedrijf [Netherlands], p. 1–232.

This Ph. D. thesis is one of most extensive works on the unsteady tidal and water movement in channels. Theoretical and practical aspects of integration problem are given, especially by method of characteristics. Chapter 1, Introduction, gives or derives the basic theories and differential equations and discusses them. Chapter 2 gives the mathematical and physical approach and describes the theory and procedure of the method of characteristics for integration of the partial differential equation. Chapter 3 analyzes the use of characteristics in wave propagation and studies various methods for wave development in propagation as well as reflection of waves. Chapter 4 deals with harmonic analysis of wave motions. Chapter 5 considers the relation of characteristics and harmonic conceptions for linear effects of wave propagation. Chapter 6 treats the same relation, but for nonlinear effects. This theoretical treatment constitutes Part I, which deals mostly with mathematical and physical aspects of waves. In Part II the empirical approach is emphasized. Chapter 7 deals with laboratory experiments of tidal motion in channels. Chapter 8 discusses computations of wave propagation by the characteristics method. Chapter 9 deals with harmonic computation-methods, and Chapter 10 with continuity-dynamic computation methods (iterative and difference methods); Chapter 11 is a comparative study and discussion of computation methods. The appendices contain: Chapter 12, some practical integration problems and their solution by characteristics; Chapter 13, some considerations on the accuracy of the experiments and the computations; and Chapter 14, some auxiliary analyses in connection with the computation.

519. Schoenfeld, J. C., 1951, Distortion of long waves; equilibrium and stability: Internat. Assoc. Sci. Hydrology, U.G.G.I., v. 4, p. 140–157.

Two causes of distortion of long waves are considered. The differences in celerities (dispersion) tend to steepen the wavefront and to flatten the waveback; the friction resistance tends to flatten both the wavefront and waveback (attenuation), or to steepen them (amplification). The wavefront of the long wave either develops into a surge, or a decline (tidal waves, flood waves, waves in power and navigation canals). The conditions of wave equilibrium are studied. If $V<C$ at the toe of a wave, $C=2\sqrt{gH}/[1+(2H/k)\ (dk/dH)]$, in which k=coefficient in Chezy formula, and the wave front is stable. When $V>C$, the waveback is stable. The waves having constant profile are determined analytically: waves traveling down a river, waves traveling up a river, and waves in a canal having a horizontal bottom. The stability of different waves is studied, and criteria for stability are developed.

520. Swain, F. E., 1951, Determination of flows in interconrected estuarine channels produced by the combined effects of tidal fluctuations and gravity flows: Am. Geophys. Union Trans., v. 32, p. 653–672.

A method is presented for determining the variable flows produced in an estuarine channel system by gravity flows from the uplands and the effects of tidal heights and phases throughout the system. The procedure makes use of a theory developed to establish the magnitude of "surge" or "bore" waves produced in canals by a disturbance of flow. The tidal height variations are represented as a series of incremental changes in water surface height which in turn produce small bore waves. The effect of channel friction, length, and cross section are taken into account. A suggested procedure for solving a given problem is presented together with an illustrative example which gives the results of a study made for the delta region of the Sacramerto and San Joaquin Rivers in central California.

[Author's abstract.]

521. Tison, L. J., 1951, Le propagation des ondes de crue dans une région lacustre [Flood-wave propagation in a lacustrine region]: Centre d'études de recherches et d'essais scientifiques des constructions du génie civil et d'hydraulique fluviale Bull. (Centr. Rech. Const. Bull.), [Liege, Belgium], v. 5, p. 371–386.

Starting from De Marchi's study (1948) and Fantoli's basic approach to the integration of storage equation, by using a sine function for the inflow hydrograph in the reservoir, the formulas for the peak discharge attenuation and time lag between the peaks of inflow and outflow discharge hydrographs are applied to the lakes of Sotkamo River Basin.

522. Baines, W. D., 1952, Water surface elevations and tidal discharges in the Fraser River Estuary, Jan. 23, 24, 1952: Canada, Natl. Research Council rept. MH–32, Apr. 8.

Report on a survey made during the low-water period of the Fraser River and presentation of results obtained for water surface elevations, local low and high water, and total discharge. Method of computing tidal discharge is included.

[Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, June 1955.]

523. Biesel, F., 1952, Study of wave propagation in water of gradually varying depth: U.S. Natl. Bur. Standards, Circ. 521, p. 243–253.

The present paper is a theoretical study of periodic waves progressing in water of variable depth. Although the theory can be extended to three-dimensional motion, it will only consider the two-dimensional case which can be briefly described as that of waves advancing in a straight channel with rectangular cross section and variable depth. This problem has already been studied by various authors, but the solutions obtained have always been restricted to flat sea bottoms of constant slope and can be worked out in practice only for a limited number of singular values of the bottom slope (cf. the works of Miche, Stoker, Loewy, etc.) or for small bottom slopes and very small relative depth (Lowell, Miche). The solution obtained by the present theory is subject to the restriction that the slope of the bottom is so small that its square is negligible. Bottom curvature and higher derivatives of depth with respect to distance along the wave course are also neglected. Provided these assumptions are satisfied, the bottom may have any shape whatsoever.

In the first part of this paper, investigations are limited to first order theories, that is, the squares of the velocities due to the presence of waves are neglected (this is the case for all previous theories of waves on sloping bottoms). In the second part, a second-order correction is introduced, and it is shown by numerical examples that this correction may be important in some respects.

[Author's abstract.]

524. Blackmore, W. E., 1952, Line diagrams for problems of storage: Water Power [London], Aug.–Sept., p. 299–303, and p. 355–357.

A graphical method based on the storage equation and called z-line (zig-zag line) is described. The procedure is based on a constant reservoir area A (small spillway fluctuations). The time intervals (T) are constant, and a slope line $dH/dQ=T/2A$ is used. The graphical construction uses Q_1 from the rating curve $Q=f(H)$, then $P=(P_1+P_2)/2$ from inflow hydrograph $P=f(t)$ and the slope line $dH/dQ=T/2A$ to obtain the point Q^2, and so forth. A similar procedure, starting from an estuary tidal curve, is used to determine levels in tidal basins. Other hydraulic problems are solved by this method.

525. Clausnitzer, R., 1952, Der Schwellbetrieb in Flusskraftwerken, seine Moeglichkeiten und Vorteile [The possibilities and advantages in the use of canals as pools with wave movement in the operation of water power plants]: Die Wasserwirtschaft [Germany], year 42, no. 11, p. 342–347.

By considering canal storage to be effectively the san e as pool storage in the case of powerplant operation that creates wave movement along the canals, experimental data are here given that describe wave movement in the Iller powerplant canal, particularly the change of water levels during the operation of the plant

526. Craya, A., 1952, The criterion for the possibility of roll-wave formation: U.S. Natl. Bur. Standards Circ. 521, p. 141–151.

The theoretical interpretation of roll waves, proceeding from the equations of De Saint-Venant, can follow two primary lines of approach: The first considers the stability of flow in a channel, i.e., the damping or amplification of a perturbance of small initial amplitude; the second proceeds from a systematic analysis of the quasisteady regime introduced by H. A. Thomas.

The object of the study is to analyze, compare, and clarify these two points of view for a prismatic channel of arbitary cross section and for resistance laws of general form. Both lead to the same criterion for the condition of roll-wave formation, which coincides for particular conditions with the formula obtained by Vedernikov from a somewhat different stability derivation.

[Author's abstract.]

527. Cuenod, Michel, and Gardel, André, 1952, Étude des ondes de translation de faible amplitude dans les cas des canaux d'amenée des usines hydro-électriques [Study of translation waves of small amplitude in the canals of water powerplants]: Bull. Tech. de la Suisse Romande [Switzerland], year 78, no. 7, p. 93–102.

Using operational calculus for study of the movement of waves of small amplitude in channels, the authors have developed a general analytical solution that taken into account the resistance factor. The boundary conditions at upstream and downstream ends of powerplant canals are given, and the particular solution in operational form is developed. For a sudden change occurring at the end of a canal, the inverse transformation of Laplace is used for functions obtained. The wave elevation in a canal is thus obtained as the function of position and time. Canals of both infinite and finite length are treated, reflection waves are taken into account in the last case and friction resistance is or is not considered. The study is related to the stability of powerplant regulation.

528. Daily, J. W., and Stephan, S. C., Jr., 1952, The solitary wave; its celerity, profile, internal velocities and amplitude attenuation: Mass. Inst. Technology Hydrodynamics Lab. Tech. Rept. 8, p. 1–56.

History, definition, and previous investigations of the solitary wave are first considered. A table summarizes results of previous investigations in terms of the celerity, profile, fluid element velocity, and attenuation, as found by each investigation. Experimental setups are described and discussed. Included is a summarizing report and discussion of experimental data with respect to the following topics: (1) Profile—the deviations from the Boussinesq theoretical profiles lead to the conclusion that the amplitude and depths experimentally examined vary, plotting around the theoretical profile. (2) Celerity—the equations of celerity presented by Rayleigh and Boussinesq are close to experimental results for all values of h/H_0, in which h is the amplitude. (3) Attenuation—due to friction both the amplitude and wave volume decrease (as is concluded by thorough study performed both analytically and experimentally. (4) Particle path and velocity—the theoretical values of fluid-element velocity are greater for velocities at the center of the wave than those values found by experiments, and the difference increases with an increase of h/H_0. The maximum amplitude value is $h/H_0=0.6$, approximately. The friction effects on wave attenuation are studied experimentally for two values of roughness, but the results are not generalized.

529. Davies, T. V., 1952, Symmetrical, finite amplitude gravity waves: U.S. Natl. Bur. Standards Circ. 521, p. 55–60.

The problem of the steady propagation of gravity waves of finite amplitude in a tank with a horizontal base is a nonlinear problem of a special type. The stream function satisfies a linear partial differential equation and the nonlinearity enters through the free surface boundary condition. The classical theory converts the nonlinear boundary condition into a linear condition using the perturbation method and is therefore restricted to infinitesimal amplitude waves. It is possible, however, to deal with the finite amplitude problems by replacing the exact nonlinear condition by a new nonlinear condition that is a close approximation and for which the exact solution may be determined. This enables one to discuss all waves up to and including the limiting case when breaking occurs at the crest. The Levi-Civita approach is used for this purpose, and the application of the method to the solution of the solitary wave problem is demonstrated.

[From author's abstract.]

530. Davis, C. V., 1952, Handbook of Applied Hydraulics: 2d ed., New York, McGraw-Hill Book Co.

The Muskingum flood-routing method along channels (by P. Z. Kirpich, p. 1197–1198), and reservoir flood routing as graphical procedure by use of storage equation (by K. E. Sorensen, p. 1235–1236) are given.

531. Dressler, R. F., 1952, Hydraulic resistance effect upon the dam-break functions: U.S. Nat. Bur. Standards Jour. Research v. 49, no. 3, p. 217–225.

The dam-break solution, a known centered simple-wave when resistance is neglected, is studied with the Chezy resistance formula added to the nonlinear shallow-water equations. Resistance transforms the wavefront from a characteristic curve into an envelope of characteristics. The flow near the tip differs from the other parts, due to a distinct boundary-layer type of region adjacent to the wavefront envelope. Then a perturbation leads to a system of partial differential equations with variable coefficients. Initial conditions are derived for the singularity at the origin. By studying its characteristic equations, this system is solved explicitly for the correction functions. Except at the tip, resistance raises the water surface and lowers velocities. These functions, no longer simple waves, possess concurrent straight characteristics lines that map into another set of the same type. The critical flow locus moves downstream, faster for more resistance and discharge rates are reduced. The method fails in the tip layer because the asymptotic expansions for the first derivatives lose validity there. Estimates are made indirectly for the wavefront velocity by observing where the boundary-layer becomes predominant.

[Author's abstract.]

532. Dressler, R. F., 1952, Stability of uniform flow and roll-wave formation: U.S. Natl. Bur. Standards Cir. 521, p. 237–241.

A uniform open channel flow down an inclined aqueduct sometimes changes into a complicated periodic motion with a progressing-wave profile, the profile moving downstream faster than the water particles. This phenomenon called "roll-waves" is investigated by three completely different methods:

(1) The construction of actual roll-wave solutions is effected by joining certain special continuous solutions together by means of moving dis-

continuities or bores. Of the three shock conditions of mass, momentum, and energy, the energy condition requires the use of a resistance term whose magnitude varies inversely with the hydraulic radius, as well as directly with the square of the velocity; otherwise, the equations are not sufficiently non-linear to produce roll-wave solutions. These waves cannot exist if resistance is absent, or if it exceeds a certain critical value. A two-parameter family of solutions is obtained; in all cases, the flow is subcritical at the peaks and supercritical in the valleys.

(2) By a perturbation procedure using maximum curvature of the profile as expansion parameter, continuous periodic progressing waves are found in terms of elliptic functions. These waves possess greater profile curvature than the discontinuous solutions of the non-linear shallow water theory. The perturbation also yields a two-parameter family of waves; these also are supercritical at peaks and subcritical in the valleys.

(3) Using a stability analysis which observes the location of roots in the right or left half-plane, the same critical condition on resistance causing instability is obtained as in Section 2. This method likewise shows that a resistance term varying only with velocity will produce uniform flows which are always stable, hence no roll-waves. A more general analysis indicates that instability may occur whenever resistance varies directly with any power of velocity, and inversely with any nonzero power of the hydraulic radius.

[Author's abstract.]

533. Eckart, C., 1952, The propagation of gravity waves from deep to shallow water: U.S. Natl. Bur. Standards Circ. 521, p. 165–173.

An approximate wave equation has been derived for the propagation of gravity waves on water of any depth. Although the precise conditions for the validity of the approximations have not been established, they appear to be justifiable in a wide variety of cases.

[From author's abstract.]

534. Hensen, W., 1952, Ueber die Fortschrittsgeschwindigkeit der Tidewelle in einem Flusse [On the celerity of tidal wave in a river]: Hann. Versuchsanstalt fuer Grundbau u. Wasserbau Mitt. [Franzius-Inst. der Hoch-Schule], Hanover [Germany], v. 2, p. 89–106.

535. Hunt, J. N., 1952, Viscous damping of waves over an inclined bed in a channel of finite width: Houille Blanche [France], no. 6, p. 836–842. [French, English].

The calculation of energy dissipation in the layers of water near the bed and near the walls is made for wave propagation in a canal of finite and uniform width, when the wavelength is large in comparison with the water depth. An expression for the damping of wave amplitude is deduced, which confirms results already obtained for the case in which the canal width is infinite. The example given is for the case in which the canal bed has a slight slope.

536. Kalinin, G. P., 1952, Osnovy methodiki kratkosrochnykh prognozov vodnogo rezhima [The basis of the methods for short-term forecast of runoff regime]: Tsentralniy Inst. Prognozov Trudy (Central Inst. for Forecase Proc.) [U.S.S.R.], no. 28 (55).

537. King, R. E., 1952, Stage predictions for flood control operations: Am. Soc. Civil Engineers Trans., v. 117, p. 690–698.

Extensive study of the problem of flood-stage forecasting for the lower Mississippi River has led to the following conclusions:

1. Reliable forecasts for the lower Mississippi River cannot be made by using any rigid formula or procedure.
2. Procedures which do not evaluate the effects of current channel conditions, tributary or diversion flows, storage, and precipitation will not give reliable forecasts.
3. Procedures dependent on the use of volumetric principles will give better results than those which are based entirely on stage heights.
4. Stages on the lower Mississippi River can be forecast with sufficient accuracy to warrant the use of these forecasts as the basis for planning and executing flood-fighting activities.

[Author's conclusions.]

538. Kotik, J., 1952, Existence and uniqueness theorems for linear water waves: Mass. Inst. Technology.

539. Kritskiy, S. N., and Menkel', M. F., 1952, [Russian], Vodokhozyaystvennye raschety Regulirovanie polovodiy i povodkov [Water resources computations; Regulation of floods and floodwaves]: Gidrometeoizdat [Leningrad], p. 225–261.

The schematic or constructed discharge hydrographs are discussed for cases in which the return period for both peak and volume of flood are known. In other cases a binomial curve for discharge hydrograph is assumed (using the Euler integral of second order). The continuity and dynamic equations of unsteady flow are discussed, especially with reference to the hydraulic characteristics and relations for a given channel reach, the initial and the boundary conditions, and various methods of integrating the two partial differential equations. Particular attention is given to the approximate methods of flood routing. Wave celerity is studied as the sum of water velocity and the expression $A(dV/dA)$, and the celerity formula is developed as

$$C=V\left[1+\frac{2}{3}\left(1-\frac{H_m}{B}\frac{dB}{dH}\right)\right]$$

For three types of channel cross section it is shown that: rectangular, $C=1.67\ V$; triangular, $C=1.33\ V$; and for the section composed of two quadratic parabolas (with the sharp point at the junction), $C=1.22V$. For simple schematic hydrographs the decrease of peak by storage is related to the flood volume and storage volume.

540. Lacombe, H., 1952, Quelques aspects du problème des marées fluviales et de la formation du mascaret [Some aspects of the problem of tides in rivers and of the bore formation]: C.O.E.C. Bull., p. 228–251.

541. Levin, Léon, 1952, Elements de depart du mouvement nonpermanent à la suite d'une rupture de barrage [Basic principles of the start of unsteady flow following the breach of a dam]: Électrotech., Hydraulique, Radio [France], no. 172.

This paper describes the experimental results already given in author's other papers and in his book.

542. Levin, Léon, 1952, Razvoy valova od rushenya visokikh brana [Evolution of waves created by the breach of large dams]: Yugoslav Natl. Comm. Large Dams, 2d Mtg., 1952, Trans., p. 104–118. (Translated into

English by Military Hydrology and Hydraulics Division, Army Map Service, Washington, D.C., Jan. 1955.)

This is an experimental study dealing with the evaluation and shape of waves created by dam breach. A graphical method of flood routing is used, based on a modified method of characteristics which employs the form coefficient $\epsilon = A/HB$, making it possible to use straight-line characteristics for any cross section form. The comparison of wave-shape detail, permitted by the graphical method developed by the author, is given special attention with respect to use in the model study of dam breach involving Medyuvrshye dam and reservoir.

543. Levin, Léon, 1952, Mouvement nonpermanent sur les cours d'eau à la suite de rupture de barrage [Unsteady flow in rivers created by dam breach]: Rev. Gén. l'Hydraulique [Paris], no. 72, p. 297–315.

The method for computing unsteady flow by De Saint-Venant's equation should not be used where the wave heights are great in comparison to the initial river depths (Favre limits the application to the ratio $h/H = 1/5$). Wave profiles after dam breach are given for different tailwater depths, resulting from both experimental and analytical study. A new graphical method based on the characteristics is developed which is suitable for large ratios of h/H (12–15, of which are experimentally verified). The method is applicable to any cross section characterized by the form coefficient $\epsilon = A/BH$. A graph is given for computation of height coefficient and celerity coefficient for a given value of ϵ. Integration is by straight lines only. An example of computation is shown for the dam and reservoir Medjuvrshye (Yugoslavia) and the experimental results of the model study are compared with results by the graphical method; good agreement is obtained.

544. Lin, Pin-Nam, 1952, Numerical analysis of continuous unsteady flow in open channels: Am. Geophys. Union Trans., v. 33, p. 226–234 (discussion by J. C. Schoenfeld and Pin-Nam Lin, v. 34, p. 792–795, 1953).

In this paper is presented a simplified method of solving Massau's equations of characteristics for unsteady flow having a continuous surface profile. This method eliminates much of the trial-and-error process formerly required.

Although Massau's rigorous analysis of unsteady flow in open channels was published some 50 years ago, its practical application has not been very extensive; for, among other reasons, the application of Massau's method is handicapped by the tiring labor involved in the trial method of solving his equations of characteristics. By developing a scheme of using a constant time interval for each step of computation, it is shown that the trial-and-error process can be avoided, except in the laying out of characteristics in the x-t plane. The latter operation being guided either by the characteristics laid out in a previous step or by the initial condition, is generally easy to perform. Since the method has the distinct feature of using a constant value of Δt throughout a step of computation, it may be called the method of constant Δt.

[From author's abstract and introduction.]

545. Nougaro, J., 1952, Recherches expérimentales sur les intumescences dans les canaux découverts [Experimental research on steep waves in open channels]: Acad. Sci. [Paris] Comptes rendus, v. 235, no. 15, p. 788–791.

The celerities of steep waves in canals are first determined by the graphical method of characteristics, and are then verified both by experiments in nature (as exemplified by operations of the Camon and Valentine powerplants on the Garonne River) and by experiments employing a model canal. Results agree well.

546. Ogiyevskiy, A. V., 1952, Gidrologiya Sushi Sel'khozgiz [U.S.S.R.].

Methods are given for computing the maximum reservoir outflow to be accounted for in emptying reservoir storage preparatory to storing flood inflows. The inflows are represented by schematic hydrographs, either triangular or trapezoidal in shape. Employing the schematic form of inflow hydrograph, the routing equation is shown to be:

$$Q_m = P_m(1+\alpha)[1 - W_s/(W_f - W_r)]$$

where Q_m=peak of outflow, P_m=peak of inflow, W_s=storage between the level of spillway crest and the maximum attained level, W_r=preflood reduced storage, W_f=flood-wave volume, $\alpha = t/T$ (t=duration of flood maximum of inflow hydrograph having trapezoidal shape, and T=the duration of base of inflow hydrograph; t=0 and α=0 for triangular shape).

547. Paynter, H. M., 1952, Methods and results from M.I.T. studies in unsteady flow: Soc. Civil Engineers [Boston] Jour., v. 39, no. 2, p. 120–165.

Part 2 treats the graphical solutions of transient problems by slopeline method, in which integration of function $y=f(x)$ by differences $\Delta y = \frac{1}{2}(f_1 + f_2)\ \Delta x$, where f_1 and f_2 are the values of f for the beginning and the end of Δx. It is shown that

$$\Delta y = \Delta y_1 + \Delta y_2 = (\Delta x/2) f_1 + (\Delta x/2) f_2.$$

The values Δy_1 and Δy_2 are determined from slopes $\pm \frac{1}{2}\Delta x$ and values f_1 and f_2. The simple graphical integration is shown. The graphical integration of function $dy/dx = f(x) - g(y)$ is shown in a similar way. This technique is applied to flood routing through reservoirs using the equation $dW/dt = P(t) - Q(t)$, or using the inflow hydrograph $P(t)$ and the storage-discharge relationship $Q(W)$. The integration of Δt gives $Q(t)$ directly. The second-order differential equation is also solved graphically and applied to surge tanks. Part 3 treats flood routing by admittance methods using the electrical network theory, where impedance and admittance functions characterize the behavior of transients and oscillations in a linear system. By using Duhamel's superposition theorem, the admittance has analogy in river channel storage and travel time, and the input in the inflow and the output in the outflow being analagous. The relation is given for coefficient routing method (Muskingum), where the admittance is analogous to the function of time. The routing procedure is outlined in the case of a given admittance. The admittance for floods is given as the sum of time delay and storage effect.

548. Pohle, F. V., 1952, Motion of water due to breaking of a dam and related problems: U.S. Nat. Bur. Standards Circ. 521, p. 47–53.

The two-dimensional hydrodynamical equations of motion, expressed in Lagrangian representation, are used to investigate the motion of an ideal fluid. This representation has the far-reaching advantage in problems with time-dependent free boundaries that the independent space variables

(a, b) are the initial coordinates of the particles: the region occupied by (a, b) is thus a fixed region independent of t.

The displacements $X(a, b; t)$, $Y(a, b; t)$ and the pressure $P(a, b; t)$ are expanded in powers of the time t. Equating to zero the coefficients of powers of t leads to a systematic procedure for the determination of the successive terms in the three expansions. Each term is a solution of the Poisson equation, in which the nonhomogeneous terms are known functions of previously determined quantities. In all cases considered, higher approximations require determination of a Green's function, which is the same for all approximations.

The method is applied to the initial stages of the breaking of a dam. [Author's abstract.]

549. Roseau, M., 1952, Réflexion des ondes dans un canal de profondeur variable [Wave reflection in a canal of variable depth]: Acad. sci. [Paris] Comptes rendus, v. 234, no. 3, p. 297–299.

An analytical study of wave reflection that computes the reflection coefficient of waves and studies the relationship of amplitude of stationary waves at the extremes of a canal of indefinite length to constant and infinite depth.

550. Russell, R. C. H., and Macmillan, D. H., 1952, Waves and tides: Hutchison's Sci. Tech. Pubs. [London], p. 1–348.

Contents of Part I, Waves, by R. C. H. Russell: Characteristics of ocean waves. Ideal waves. The generation of waves by wind. Waves near the shore. Reflection, diffraction, refraction and wave-induced currents. Movement of material by the sea. The effect of wave action on structures. Wave measuring. Part II, Tides, By D. H. Macmillan.

[Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, February 1954.]

551. Sokolovskiy, D. L., 1952, Rechnoy stok, metody issledovanniy i raschetov [River runoff, methods of investigation and computation]: Gidromet, Izd. [Leningrad], chap. 7, 8, p. 226–348.

Among other aspects of flood investigation, the book deals with the approximation of hydrographs by analytical expressions, travel time of flood waves, and flood forecasting.

552. Tsinger, V. N., 1952, Uchet srabotki vodokhranilishch pri raschetakh snizheniya maksimal'nykh raskhodov [Taking into account the lowering of reservoir levels in the computation of the decrease of maximum outflow discharge]: Gidrotekhnika i Melioratsiya [U.S.S.R.], no. 11.

Formulas are developed for computation of maximum discharges over spillways for the case involving lowering of the reservoir levels prior to flood inflow, and assuming inflow hydrographs of simple triangular and trapezoidal shapes.

553. Uchida, Shigeo, 1952, On the analysis of flood wave in a reservoir by the method of characteristics: Japanese Natl. Cong. for Appl. Mech., 2d, Proc., p. 271–276, (published by Science Council of Japan [Tokyo], May 1953).

A generalized form of characteristic equations for unsteady flow in open channels having varying cross sections is introduced for the purpose of analyzing the reservoir effects. The method uses a simplified form for cross sections that employs the hydrograph plane for construction of overflow conditions. An example is given of the calculation of flood-wave movement through an idealized reservoir. Computed hydrographs of uniform-channel routing are compared with observed hydrographs; fair agreement is shown. Deformation of flood waves in reservoirs is discussed and examples are given.

554. Ursell, F., 1952, Discrete and continuous spectra in the theory of gravity waves: U.S. Natl. Bur. Standards Circ. 521, p. 1–5.

It is shown how systems with discrete and continuous spectra differ in their physical behavior. It has generally been assumed that waves in an infinite canal have a continuous spectrum; examples are here given to show that the spectrum is mixed on a sloping beach and when there are internal boundaries. The corresponding oscillations are three-dimensional. The relation of the theory of spectra to the problem of uniqueness is discussed.

[Author's abstract.]

555. Zheleznyak, I. A., 1952, Opredelyeniye raschetnikh raskhodov vodosbrosnykh sooruzheniy s uchetom akkumulatsii [The determination of discharges over spillways when the storage is taken into consideration]: Gidrotekhnika i Melioratsiya [U.S.S.R.], no. 1.

The method of computing maximum outflow discharge is given for the case involving preflood reduction of reservoir storage and a spillway crest that is equipped with gates.

556. Apté, A., 1953, Sur une solution approchée du probléme de l'onde solitaire [On an approximate solution of the problem of solitary wave]: Acad. sci. [Paris] Comptes rendus, v. 236, no. 26, p. 2477–2479.

This paper presents an analytical approach to the problem of solitary wave having crest at angular point.

557. Baines, W. D., 1953, Survey of tidal effects of flow in the Fraser River Estuary, June 10 and 11, 1952: Canada Natl. Research Council rept. MH–40, Apr. 10.

Description of survey made on two typical days during 1952 freshet. Presentation of measured water surface elevations and computed discharges at 25 stations in the estuary. Curves of local low and high tides are included and are compared to results obtained in a similar survey on January 23 and 24, 1952. Description of the dynamics of the flow as indicated by results so far obtained.

[Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, June 1955.]

558. Becker, E., 1953, Analogie zwischen Wasserschwall and Verdichtungsstoss [Analogy between surge and shock wave of compressible fluid]: Ingenieur Archiv [Germany], v. 21, no. 1, p. 42–54.

When applied for gas, dynamic approximations, the analogy between surge and shock waves does not hold because the thermodynamic conditions applicable to the two fluids are different. The fact that water surge is not an exact discontinuity accounts for this difference. The conditions required for analogy are discussed. Surge formulas are given for small and great surges occurring in very shallow water as well as in deep water.

559. Bergeron, Louis, 1953, Méthode graphique pour le calcul des ondes de translation [Graphical method for the computation of translatory waves]: Soc. Française des Mécaniciens [France] Bull. 7.

560. Burns, J. C., 1953, Long waves in running water: Philos. Soc. [Cambridge] Proc., v. 49, p. 695–706.

Classical shallow-water theory for the propagation of long waves in running water is modified by the inclusion of the effects of the vorticity present in the main stream as the result of the action of viscosity. When this vorticity is assumed constant, a non-linear theory can be used, but for more general velocity distributions in the main stream it is necessary to linearize the problem.

In the linearized theory, a general equation is obtained connecting the wave velocity with the velocity in the undisturbed stream and this is solved in several special cases. It is shown generally that the wave velocity relative to the mean flow is always greater than the value given by the classical theory. The wave velocity relative to the bottom of the stream has two values, one less than the minimum stream velocity and the other greater than the maximum stream velocity.

[Author's abstract.]

561. DeMarchi, Giulio, 1953, Action of side weirs and tilting gates on translation waves in canals: Minn. Internat. Hydraulics Conf. (Joint Mtg. Internat. Assoc. Hydraulic Research and Hydraulics Div. of Am. Soc. Civil Engineers), Minneapolis, 1953, Proc., p. 537–545.

This is an experimental study of the surge-reduction effect of a side weir, placed along a canal, and of frontal tilting gages in the forebay of a powerplant. The wave heights reduced by side spillways are computed by the Citrini method (1949, 1950) and compared with the experimental results, thus verifying the consistency of the Citrini theory. The tilting gate can be effective if operated either before or at the same time as the turbine operations.

562. DeMarchi, Giulio, 1953, Azione de uno sfioratore a ventola sull'onda positiva provocata dall'arresto delle macchine nel canale adduttore di un impianto idroelettrico [On the action of a side weir, controlled by a tilting gate, upon the positive translation wave generated in the canal of a powerplant by the sudden closing of the turbines]: Energia Elettrica [Milano], v. 30, no. 12, p. 12–20. Reprinted as Istituto di Idraulica e Costurzioni Idrauliche [Milano] Memorie e studi, no. 110, 1953: [On model tests for the Tornavento Power Plant].

A report is given concerning a first series of experiments carried out at the Hydraulic Laboratory of the Politenico of Milano on a model of the canal forebay and power-station of the Tornavento hydroelectric plant. The aim of the experiments was to determine the characteristics of the ascending positive wave generated in the canal by the closing of the turbines and to study the action exerted upon such wave by the sudden opening of the tilting-gates of the spillway in the forebay. The experimental layout and the instrumentation are described and the results obtained with a special water-level recorder placed in the canal are presented and discussed. The conclusion is that, in the case studied, the effect of the gates is substantially one of stabilizing the reduction of the wave height due to the widening of the canal at the location of the forebay.

[Author's abstract.]

563. Dmitriyev, A. A., 1953, Prokhozhdeniye dlinnykh voln cherez prepyatstviya pri chastichnom otrazhenii [Passage of long waves over obstacles in the case of a partial reflection with the given reflection coefficients]: Akad. Nauk [U.S.S.R.] Doklady, v. 15, no. 4, p. 509–512.

This is an analytical study of wave reflection and wave transmission in the case of one concentrated resistance and of two resistances, located L and $2L$ distances from the coordinate origin. It is concluded that, under actual conditions, the dissipation of energy must decrease with the coefficient of wave transmission.

564. Dressler, R. F., and Pohle, F. V., 1953, Resistance effects on hydraulic instability: New York, Interscience Publ. Inc., New York Univ., Inst. Math. Sci., Commun. Pure Appl. Math., v. 6, no. 1, p. 93–96.

The Vedernikov and Craya results of condition for instability are obtained by other methods and assumptions. The developed criterion for Chezy's formula

$$V > \frac{n}{m}\sqrt{gH\cos\theta}$$

is for Manning resistance formula $V^2 = 9/4\ gH\cos\theta$. Any flow governed by a resistance varying with any power of velocity, but independent of depth, must remain stable. The next result is that both effects of depth and velocity dependence for resistance, operating simultaneously, are needed to produce instability.

565. Frank, Josef, 1953, Die Hydraulik der Durchlaufspeicherung [The hydraulics of using pools in chain for flow regulations]: Der Bauingenieur [Germany], year 28, no. 2, p. 39–44 and 87–91.

Analysis is made of the movement of positive and negative surge waves in reservoirs used by powerplants as regulating pools for their daily operations. The celerity formulas, in approximately exact form, serve for routing of surges. Wave crossings, new waves occurring in opposite directions, and wave reflection at the end of pools are studied; new water depths are determined. Secondary phenomena are discussed. A graphical procedure is given, as is a detailed numerical example.

566. Glover, R. E., Herbert, D. J., and Daum, C. R., 1953, Electrical analogies and electronic computers (a symposium); Application to a hydraulic problem: Am. Soc. Civil Engineers Trans., v. 118, p. 1010–1027.

This paper discusses the general conditions of the problem of flow distribution in a network of estuarine channels to which an analog computer model was applied. After developing the analog requirements, the model is described, with emphasis on the electronic circuit that provides the required square-law resistance. The equations correlating electrical and hydraulic quantities are developed from the basic electrical and hydraulic relationships. Finally, the methods by which the required boundary conditions were duplicated are discussed.

[Author's abstract.]

567. Hayashi, Taizo, 1953, Mathematical theory and experiment of flood waves: Japan Soc. Civil Engineers Trans. [Tokyo], no. 18, p. 13–26.

This paper describes mathematical theory and laboratory experiment of the flood wave in uniform open channel. The theory is developed by the method of successive approximations with respect to a small parameter

σ which is defined by $\sigma=\sqrt{-\ddot{F}(0)/g/S_0}$, where $\ddot{F}(0)$ denotes the vertical accelerative rate of rising of the stage of the flood wave at the moment of passage of its crest at a cross section of the channel, g the acceleration of gravity and S_0 slope of the channel. The laboratory experiment was conducted primarily for the verification of the theory. Flood wave with any desired value was generated in flowing water with any desired value of Froude number and specifically, the rate of attenuation and the speed of propagation, both of which were related to the crest of the wave, were measured and are compared with those given by the theory.

[Author's abstract.]

568. Holsters, H., 1953, Le calcul du movement non-permanent dans les rivières par la méthode dite des "lignes d'influence" [The computation of unsteady motion in river channels by the method called "influence lines"]: Houille Blanche [France], no. 4, p. 495–509.

This paper on nonpermanent flow in rivers contains: fundamental equations, ignoring secondary terms of inertia and their solution by methods of finite differences (exact methods); a simplified method called "influence lines," (as characteristics in simplified form are called); repercussions of the arbitrary choice of mean depth and of velocity on the calculated evolution, first, with reference to an elementary wave profile, and second with reference to tidal curves (choice of too large a value for depth regularizes the profile without modifying the general form, while choice of a too small a value for depth leads to rapidly diverging oscillations). Examples of calculation are given. Influence of secondary terms of inertia is studied. The paper discusses the errors introduced by simplifications in the form of influence lines (introduced first by author's paper in 1947).

569. Hopkins, C. D., Jr., 1953, The celerity of very large flood waves in the Arkansas Basin, with discussion: Am. Geophys. Union Trans., v. 34, p. 594–596 (discussions: v. 35, p. 513–518).

Numerous streams in the Arkansas River basin provide the data shown as crest discharge-travel time and crest stage-travel time relationships, supporting the theory that large flood waves travel much more rapidly than flood waves of moderate size.

570. Ichiye, Takashi, 1953, On the abnormal high waters in rivers: Oceanographical Mag., v. 5, no. 1, p. 45–60.

The rivers and estuaries in the city of Osaka often overflow, chiefly due to the meteorological high waters caused by typhoons, and cause serious damage to structures and other facilities. The properties of flood waves of meteorological tides progressing up the rivers and some discussion about them are presented. Data collected during three typhoons: Muroto on Sept. 21, 1934; Jane on Sept. 3, 1950; and Ruth on Oct. 15, 1951, were were used as a basis for the theoretical computations.

[Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, May 1959.]

571. Keulegan, G. H., 1953, Characteristics of internal solitary waves: U.S. Natl. Bur. Standards Jour. Research, v. 51, no. 3, p. 133–140.

This is an application of the method of approximations initiated by Boussinesq to the disturbances of the interface points for waves of permanent form and the interval solitary wave. The system considered is a layer of liquid on another layer of greater density, the liquids of the layers being initially at rest and of constant total depth. The form of the wave is established. The dependence of wave velocity on wave height, on density differences, and on layer thickness is determined.

[Author's abstract.]

572. Kohler, M. A., 1953, Electrical analogies and electronic computers (a symposium); Application to stream-flow routing: Am. Soc. Civil Engineers Trans., v. 118, p. 1028–1045.

Late in 1948, the Weather Bureau, United States Department of Commerce, developed an electronic device for stream-flow routing that has proved to be highly effective in the preparation of river stage forecasts. Although originally designed for routing flows from point to point along a stream, subsequent studies indicate that the equipment is equally applicable to the direct routing of effective rainfall (runoff) over relatively large basins. This application of the flow analog and the conditions under which the original circuit fails to provide a satisfactory reproduction of the outflow hydrograph are discussed in this paper. The basis for the circuit employed in the analog and the method of operating the equipment are also discussed briefly.

[Author's synopsis.]

573. Koženy, Josef, 1953, Hydraulik, F/III, Die nichtstationaere Bewegung: [Hydraulics, Chapter F/III, The unsteady flow]: Vienna, Springer-Verlag, p. 251–267.

Starting from two partial differential equations for unsteady flow, the wave heights are expressed as

$$y=A \sin (\pi/L)[x-(V_0+\sqrt{gH_0})t]+\exp. (-\psi_0 V_0 t/2H_0),$$

in which L=wave length, ψ_0=resistance coefficient, and where V_0 and H_0 are values for a steady flow. The case is analyzed when the curvature of surface profile is taken into consideration, and especially its influence on the wave celerity. The energy of wave is given. Surges (bore and depression) are treated in a way similar to that of De Saint-Venant (1871) and of Boess; examples are given for the computation of height, celerity, and velocity of waves. The changes in positive and negative surges as they progress is studied, and formulas for height, celerity, and time of the wave break are given. The change of cross section and its effect on surges is analyzed, and examples of lock operations are given.

574. Labaye, G., and Duranton, R., 1953, Utilisation des galéries souterraines en réservoirs d'éclusées [Use of tunnels as storage reservoirs]: Houille Blanche [France], no. B, p. 735–746.

The use of tunnels as storage reservoirs is studied through the determination of initial surface (after the tunnel ceases to be under pressure). Study is made of unsteady transitory movement until critical regime is attained. Results are generalized.

575. Mason, M., 1953, Surface water wave theories: Am. Soc. Civil Engineers Trans., v. 118, p. 546–574.

This paper discusses the characteristics of oscillatory surface waves and summarizes the development of pertinent theory. The more important equations that characterize wave formation and movement are presented. The method by which waves are believed to be generated is described, and a theory of the growth of waves is formulated. Several charts

provide convenient means of determining wave characteristics and wave effects from a knowledge of the limiting factors. Other subjects, including wave refraction, diffration, and reflection, are also briefly treated.

[From author's abstract.]

576. Messerle, H. K., 1953, Electronic high speed simulation of hydraulic problems: Inst. Engineers [Australia] Jour., v. 25, no. 3, p. 35–41.

An electronic high speed simulator has been developed at the University of Melbourne for the solution of the pondage equation relating the waterflow and the water level in a reservoir. The instrument has been used for the investigation of general flood-routing problems and is described in this paper.

The solution of more complex hydraulic problems is investigated also, and it is shown that the principles used for the simulation of the pondage equation can be extended quite readily to the analysis of general hydraulic systems.

The experimental results obtained on the pondage simulator have been sufficiently accurate for practical purposes and the setting up and solution of any particular pondage problem for a number of boundary conditions can be carried out within about 10 minutes. The effects of varying the circuit parameters can be investigated rapidly once a problem has been set up. It is also important to one reservoir, one inflow and one outflow function, which can be solved with fixed dimensional computed ranges simply by applying the ideal of analogue models. Several independent inflows arising from various tributaries can be allowed for by providing additional inflow generators.

[From author's conclusions.]

577. Milyukov, P. I., 1953, O sootnoshenii mezhdu skorostyu dobeganiya ravnogo raskhoda i skorostyu techeniya [On the relation between the celerity of the uniform discharge and the velocity of flow]: Meteorologiya i Gidrologiya [U.S.S.R.], no. 8.

578. Nougaro, Jean, 1953, Theoretical and experimental studies of the propagation of translation waves in open channels: Minn. Internat. Hydraulics Convention (Joint Mg. Internat. Assoc. Hydraulic Research and Hydraulics Div. of Am. Soc. Civil Engineers), 1953, Proc., p. 555–559.

A graphical method for predicting the rise in the water level upon passage of a translation wave is presented.

The classical characteristics lines of Bergeron are replaced by characteristic curves which eliminate most of the hypothesis postulated by Bergeron.

A system of curves on tracing paper moving on the proper coordinates particularly facilitates the graphical construction. The author points out how to take into account the head losses, the slope of the channel bed, and special conditions resulting from the section change of a bifurcation, a large basin, etc.

The author points out that many experiments justify the application of the formula $C=\sqrt{gH}\pm V$ and indicates some application of his graphical method.

The results obtained are in accordance with those experimentally obtained.

[Author's abstract.]

579. Nougaro, Jean, 1953, Recherches expérimentales sur les intumescences dans les canaux découverts [Experimental research on the intumescences in open channels]: Soc. Française des Mécaniciens Bull. [France], no. 9, 3d trimestre, p. 23–35.

580. Nougaro, Jean, 1953, Étude théorique et expérimentale de la propagation des intumescences dans les canaux découverts [Theoretical and experimental study of the propagation of translation waves in open channels]: Pubs. Sci. et Techs. du Ministère de l'Air [France], no. 284, p. 1–155.

581. Pleshkov, Ya. F., 1953, Vliyanie predpavodochnoy srabotki vodokhranilishcha na raschetnyy maksimal'nyy raskhod [The influence of preflood decrease of storage in reservoirs on the maximum outflow discharge to be computed]: Gidrotekhnicheskoe Stroitel'stvo [U.S.S.R.], no. 9.

The influence of the preflood decrease of storage on maximum outflow discharge over a free spillway is studied; it is concluded that in some cases the peak outflow may not be reduced, or may even be increased.

582. Riggs, H. C., 1953, A method of forecasting low flow streams: Am. Geophys. Union Trans., v. 34, p. 427–434.

The normal base-flow depletion curve is used, and examples are shown, to provide a method of forecasting low flow.

583. Schoenfeld, J. C., 1953, Inleiding tot de methode der Karakteristieken [Introduction to the method of characteristics]: Rijkswaterstaat commun. [Den Haag, Netherlands], Nota C.S.D 53–2.

584. Serre, F., 1953, Contribution à l'étude des écoulements permanents et variables dans les canaux [Contribution to the study of permanent and nonpermanent flows in channels]: Houille Blanche [France], p. 374–388 and p. 830–872 (unsteady flow, p. 857–872).

Study of nonpermanent flow showing derivation of partial differential equations of flow. The advantages which can be gained in the study of swells, waves, and breaking jumps under variable flow are emphasized. The assumption of uniform distribution of velocities is made. The differential equations for the waves having constant celerity are derived, and are similar to differential equations derived for steady flow. In a canal having initial constant velocity V and depth H, there can be an infinity of waves of finite amplitude (h), each having a celerity C_r and the length $2L$. The solitary wave is that where celerity, $C_r = \pm\sqrt{gH} = \pm\sqrt{g(H+h)}$, is maximum.

585. Shuleykin, V. V., 1953, Razrushenie voln pod deystviem melkovodya [Breaking of waves under the effect of shallow water]: Akad. Nauk [U.S.S.R.] Doklady, v. 43, no. 3, p. 463–466.

The conditions are studied under which a wave would break on a sloping breach of shallow water. The profile of a wave before breaking is analytically determined and found to compare well with the wave profile photographed in the model basin. The stability and instability of wave profiles are discussed at the end of paper.

586. Sorenson, K. E., 1953, Graphical solution of hydraulic problems: Am. Soc. Civil Engineers Trans., v. 118, p. 61–77.

Graphical solution of first-order differential equations and application of that procedure to various hydraulic problems, including flood routing through reservoirs, is shown.

587. Stoker, J. J. 1953, Numerical solution of flood prediction and river regulation problems; Derivation of basic theory and formulation of numerical methods of attack: New York Univ., Inst., Math. Sci. Rep. IMM–200.

The report treats rederivation of two partial differential equations for unsteady flow and discusses the numerical methods to be used in integrating these equations for solutions to practical problems. Thomas's approach (1937), using the finite-differences method, which is laborious to apply by standard computation methods, is improved by the use of digital computers and by development of appropriate numerical procedures. The experience and calculating equipment of gas dynamics (with the same equations applicable for unsteady flow of water in channels) assist in problem solutions. The numerical procedures of solving equations by the method of characteristics is discarded, and a method of finite differences of integration in (x,t)-plane, covered by a rectangular net of points, is derived, wherein the derivatives in differential equations are replaced by difference quotients. The relation of Δx and Δt is determined by the use of characteristics, so that the value Δt is limited as soon as Δx is selected.

588. Stoker, J. J., 1953, Unsteady waves on a running stream: New York Univ., Comm. Pure and Appl. Math., v. 6, p. 471–481.

The problem is investigated mathematically in terms of the classical hydrodynamic theory of irrotational flow in an incompressible perfect fluid. The dimensionless parameter gH/V^2 is used for the discussion of the problem. The solution of the unsteady wave problem is developed, and the state of steadiness is discussed as the limit of unsteady motion for infinite time.

589. Supino, Giulio, 1953, La propagazione delle onde di translazione [The propagation of translation waves]: Energia Elettrica [Italy], v. 30, no. 4, p. 201–210.

The solution of partial differential equations, already known for rectangular channels, is extended to apply to any prismatic channel. Assuming that the changes of discharge and area are small, in the comparison of unsteady flow with steady flow, and assuming neglect of friction, the Boussinesq celerity formula is obtained and extended for any prismatic channel. Translation waves are studied with respect to nonuniform steady flow. The linearized solution for the wave of the regime is obtained (being the solution that gives the same celerity for the water depth and for the discharge). The wave attenuation is determined, and an example of its use is given. The waves having different celerities for depths and discharges are also studied. The initial conditions are discussed.

590. Supino, Giulio, 1953, Propagazione di piccole onde su un moto-base permanente [The propagation of small waves on a basic nonuniform unsteady flow]: Energia Elettrica [Italy], v. 30, no. 6, p. 333–340.

This paper is a continuation of a previous paper by the author (Energia Elettrica, 1953). The movement of small waves on a nonuniform steady flow is studied, especially in the case of supercritical flow. Starting from the general partial differential equation for discharge and area changes due

to unsteady flow and for short travel time of wave, the method of characteristics is used for derivation of the equation along a "band of characteristic." From this band of characteristic the celerity of wavefront is determined for changing velocity, area, and width for a nonuniform steady flow. This method is not appropriately applied to long waves, for which the earlier paper (1953) gives the best solution. In the case of small waves, the solution for coefficients in partial differential equation is developed for known stage and discharge hydrographs at the initial point (as is the case for supercritical flow).

591. Supino, Giulio, 1953, La riduzione delle piene del Reno da Cento alla Bastia [The attenuation of River Reno flood from Cento to Bastia]: Giornale del Genio Civile [Italy], year 91, no. 1–2, p. 34–40.

The continuity equation ($P-Q=dW/dt$) is integrated by use of a sine function for P. The results developed for Q are applied to the Reno River in a reach of 57.4 kilometers that is practically without tributaries. The results are compared with the field data. It is concluded that the flood of 1951 has the nature of a rare flood, not in terms of volume, but in terms of the type of flood wave.

592. Tsinger, V. N., 1953, Raschet snizheniya maksimal'nykh raskhodov vodokhranilisch s uchetom predpavodochnoy srabotki [The computation of the decrease of maximum outflow discharge from reservoirs when reservoir level is decreased prior to the flood]: Gidrotekhnicheskoe Stroitel'stvo [U.S.S.R.], no. 4.

Maximum discharge at the free spillway, when the inflow hydrograph is approximated by a triangle, is developed as $Q_m=(W_f-W_r-W_s)P_m/(W_f-\sqrt{KW_fW_r})$, where Q_m=maximum outflow, P_m=maximum inflow discharge, W_r=emptied storage before the flood inflows and the spill of water begins; W_s=volume of stored water between the level of spillway and the maximum attained level; W_f=flood volume; and $K=t_1/T$, ratio of the duration of flood peak to total duration of flood.

593. Ursell, F., 1953, The long-wave paradox in the theory of gravity waves: Philos. Soc. [Cambridge] Proc., v. 49, p. 685–694.

The theory of long waves in shallow water under gravity employs two different approaches, which have given rise to a well known paradox remarked by Stokes, but hitherto not fully resolved. On the one hand it was shown by Airy (1845) that, if the pressure at any point in the fluid is equal to the hydrostatic head due to the column of water above it, then no wave form can be propagated without change in shallow water of uniform depth; on the other hand, it was shown by Rayleigh's theory (1876) of the solitary wave that this conclusion may be incorrect. In the present paper an attempt is made to elucidate the paradox. Waves of small amplitude h and large horizontal wavelength λ (compared with the depth H of the water) are studied, and is shown that Airy's conclusions is valid if $h\lambda^2/H^3$ is large, whereas the solitary wave has $h\lambda^2/H^3$ of order unity. Equations of motion are derived corresponding to large, moderate and small values of $h\lambda^2/H^3$; these can be summarized in a single equation for the profile $y(x,t)$:

$$\frac{\partial y^2}{\partial t^2}-gH\frac{\partial^2 y}{\partial x^2}=gH\frac{\partial^2}{\partial x^2}\left(\frac{3}{2}\frac{y^2}{H}+\frac{H^2}{3}\frac{\partial^2 y}{\partial x^2}\right)$$

due to Boussinesq (3). It is also shown that the linearized theory of surface

waves is valid only if h/λ and $h\lambda^2/H_3$ are both small. Some remarks are made on the generation of the solitary wave, and on the breaking of waves on a shelving beach.

[Author's abstract.]

594. U.S. Army Corps of Engineers, Waterways Experiment Station, 1953, Study of hydrographs in triangular flume: U.S. Army Corps of Engineers, Waterway Expt. Sta., Interim rept. 1. folder 1, July 1953, p. 4–6; folder 2, figs. 5–411; Interim rept. 2, folder 1, p. 1–6; folder 2, figs. 14–405.

Steep-front triangular waves are studied experimentally in a triangular flume, for the purpose of obtaining data to assist in the determination of coefficients and curves for routing floods with steep fronts. This information is needed for study of the effects on wave attenuation of channel slope, roughness, and discharge. The experiments are described, and percentages of discharge reduction along the flume are given. The experimental results are given in figures and tables.

595. U.S. Army Corps of Engineers, 1953, Routing of floods through river channels: U.S. Army Corps of Engineers Manual, Sept. 1953, pt. 114, Chap. 8, p. 1–33, pls. 1–14.

Two basic types of approximate flood-routing methods are demonstrated: (1) "storage routing" methods, in which energy factors are largely neglected and only the effects of storage in the intervening reach and local inflows are considered in estimating changes in the flood wave as it passes through a reach of channel; and (2) methods of approximating flood-wave shape at the lower end of a reach by time displacement of values of average inflow into the reach. Of the storage-routing methods, the coefficient (or Muskingum) method is given principal attention; other methods are explained or compared in terms of the factors determining these routing coefficients.

596. Wells, L. W., 1953, Some remarks on shallow water theory: New York Univ. Inst. Math. and Mech. Rept. IMM–NYU–198, p. 1–18.

The report is a simplified analytical treatment of the approximate theory derived from the exact hydrodynamic theory of gravity waves of small surface amplitude, which makes the assumption that the length of wave is sufficiently large when compared with the depth of water. Results obtained are of general interest and application to shallow-water theory in treating the reflection of waves from an obstacle of finite length.

597. Zheleznyak, I. A., 1953, Opredelennie maksimal'nykh raskhodov s uchetom reguliruyushchey emkosti vodokhranilishcha [The determination of maximum discharges by taking into account the regulating storage]: Gidrotekhnicheskoe Stroitel'stvo [U.S.S.R.], no 9, p. 21–25.

In the case of spillways with gates on the crest, preflood reduction of reservoir storage does not reduce the peak of outflow discharge, when $W_r < W_n$, where W_r=preflood reduction of storage and W_n=the volume of inflow hydrograph from the beginning of inflow to the moment when the outflow is equal to the discharge of normal level (with all gates open).

598. Appleby, F. V., 1954, Runoff dynamics; A heat conduction analogue of storage flow in channel networks: Internat. Assoc. of Sci. Hydrology, Rome, 1954, Proc., no. 38, v. 3, p. 338–348.

The normal differential equation of storage flow is developed into a form analogous to that for linear heat flow, where storage corresponds with temperature. Inherent in the resulting equations are two parameters, the outflow velocity V and the storage transit factor K, which is a response time and mean time of concentration of the network. The product of these two factors is a characteristic length $\bar{e}$, the product of this length and the surface width of flow at a control section measures the total water surface area in the system; or, if combined with area of flow at the outfall, the volume of storage. V^2K is directly analogous to diffusivity in heat flow and a measure of the power of the system to eliminate its accumulated flood waters. Time factors in general are discussed, with formulae for the lag of peak flow and the regulation of the peak under unit runoff conditions. Examples are given using runoff data from large catchment areas and urban drainage networks. The development of a heat conduction analogue as a working instrument is described, as is its application to studies in general of the hydrograph of flow, and in particular the study of brief inter-periodic correlation between runoff and rainfall.

[Author's abstract.]

599. Benjamin, T. B., and Lighthill, M. J., 1954, On cnoidal waves and bores: Royal Soc. [London] Proc., v. 224, no. 1159, p. 448–460.

From the theory of stationary wave trains a new presentation of cnoidal wave theory is given. The theory permits study of the development of the wave train behind a bore. Such a wave train, which is present in all but the very strong bores, is shown to be capable of absorbing almost all the energy that, according to classical theory, is liberated at the bore, although some minute residual dissipation of energy still appears to be necessary.

600. Chitale, S. V., 1954, Bores in tidal rivers with special reference to Hooghly: Irrig. and Power [India], v. 11, no. 1, p. 110–120.

Includes discussions and presents equations pertaining to formation and propagation of the bore, variations in the bore height, and speed of the bore.

[Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, June 1955.]

601. Cocchi, G., 1954, Sulla propagazione delle onde in alveo a forte pendenza [On wave propagation in channels of steep slope]: Accad. Sci. dell'Ist. Bologna [Italy].

602. Dressler, R. F., 1954, Comparison of theories and experiments for the hydraulic dam-break wave: Internat. Assoc. Sci. Pubs. no. 38, v. 3, p. 319–328.

Experimental data obtained at the U.S. Bureau of Standards are presented for trajectories of the forward wave front, the backward wave front, and for flow profiles, over long time intervals for three different channel bottoms which range from smooth to very rough Chezy coefficients. The roughness has been calibrated over a wide range of discharge rates in each case. The experiments are compared with previous data of Schoklitsch and Egiazarov; they are then compared with mathematical results obtained by Dressler and Whitham, based upon two different approaches. In one theory, the tin region is neglected; in the other, the tip is analyzed and the agreement with each other. The experimental data deviate in a consistent way from these theoretical results, indicating more generally that the Chezy resistance function, normally measured for steady flows,

may be inadequate to describe a highly unsteady flow or the nature of turbulent resistence in a tip region. The experiments show a division of the flow into distinct regimes, depending initially upon laminar viscous effects only, then upon inviscid hydrostatic pressure effects only, and finally upon turbulent resistive action. These transitions occur over a very short time interval; quantitative data for this was obtained from high speed motion picture photography. This furnishes estimates on the time necessary for the formation of fully developed turbulence arising from the state of rest. Wave profiles immediately after gate opening are also compared with theoretical results of Pohle, based upon an analysis of velocity-potential flow in Lagrangian coordinates.
[Author's abstract.]

603. Friedrichs, K. O., and Hyers, D. H., 1954, The existence of solitary waves: New York Univ., Commun. on Pure and Appl. Math., v. 7, p. 517–550.

The authors prove the existance of stable solitary waves in a direct mathematical manner, assuming that, aside from pressure forces, gravity is the only force acting on the fluid.

604. Gherardelli, Luigi, 1954, Sul moto vario in canali prismatici attorno al regime critico [On the unsteady movement in prismatic canals around the critical regime]: Energia Elettrica [Italy], v. 31, no. 1, p. 34–38.

Using the equations of characteristics and their properties, the propagation of waves is studied for the case when the water depth is close to the critical depth H_c. It is shown that the wave has oscillations as soon as the discharge and depth approach those of critical flow. With respect to rivers, critical flow is a zone rather than a determined depth or discharge. In this region the given function $F_1(x-Ct)$ is indeterminate, a casual function, having anomalies that may be of sizable amplitude. The physical aspects of phenomena taking place in rising and falling limbs of a hydrograph, when the depth passes the critical depth, are discussed, and the patterns of secondary oscillations are outlined.

605. Haws, E. T., 1954, Surges and waves in open channels: Water Power [London], v. 6, no. 11, p. 419–422.

The surges occurring in powerplant channels are studied analytically. The rise in water level resulting from a positive upstream surge traveling on the downstream-flowing current is derived; particular attention is given to the level rise that occurs after the surge has passed. A similar analysis made for a variable channel section. The celerity for very low waves is derived as approximately that of the classical form $C=-V+\sqrt{gH_m}$. Slow changes in flow and superelevation of water surface occurring at bends in the channel are discussed.

606. Hoffman, G. R., 1954, Tidal calculations applied to the Estuary of the River Great Ouse: Inst. Civil Engineers Proc., v. 3, no. 3, p. 809–829.

The paper describes calculations made to reproduce tidal levels observed during a flood of the River Great Ouse on 16 March 1937. It outlines briefly the method of calculation and gives an account of the various alternatives tried before agreement between observed and calculated levels was finally obtained. In the lower part of the estuary, where the tidal range is large compared with the mean depth at half tide, the losses at bridge piers, bends, and obstructions had to be separated from the boundary roughness loss which was represented by a Manning type equation. In

addition, it was found necessary to average the surveyed cross sections in a reach by a rather more elaborate method than that generally used. It is concluded that tidal calculations in relatively deep estuaries give reasonably accurate results and that in certain cases the method may compare favorably, both in time and cost, with model experiments. In shallow estuaries, however, the accuracy of predicted levels is reduced.
[Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, June 1955.]

607. Isaacson, E., Stoker, J. J., and Troesch, B. A., 1954, Numerical solution of flood prediction and river regulation problems (Ohio-Mississippi floods), report II: New York Univ., Inst. Math. Sci. Rept. IMM-NYU-205, p. 1–46.

Problems of flood prediction and regulation on the Ohio and Mississippi rivers are studied by means of the finite-differences method (Stoker, 1953) on the digital computer, UNIVAC. Flood problems are posed, and techniques for solution are formulated employing a computational model of the Ohio River. Description is given of the finite differences employed and of the mesh system used for Δt and Δx. Special attention is given to problems resulting from junction of the Ohio and Mississippi rivers. Expansion of solutions approximately of the first characteristics, and theoretical analyses of steady flows and of steady progressing waves are shown in two appendices.

608. Ishihara, T., Hayami, Sho., and Hayami, Shi., 1954, On the electronic analog computer for flood routing: Japan Acad. Sci. Proc., v. 30, no. 9, p. 891–895.

Starting from Hayami's theory (1952) of flood waves, in which the basic difference equation is approximately

$$\frac{\partial H}{\partial t}+D\frac{\partial H^{3/2}}{\partial x}=\mu\frac{\partial^2 H}{\partial x^2},$$

where D=a constant determined by the hydraulic resistance and bottom slope of channel, and μ=numerical constant defined by the various irregularities of river reach, the electronic analog computer is developed, in order to avoid the defect of principle that is inherent in the Muskingum method. The computer that has been developed to solve the above equation is described, and examples of its application to river flood routing are given. The estimates of D and μ are discussed.

609. Iwagaki, Y., and Sueishi, T., 1954, Approximate method for calculation of unsteady flow in open channels with lateral flow: Japanese Natl. Cong. for Appl. Mech., 4th, Proc., rept. II/3, p. 235–240 (published by Science Council of Japan [Tokyo], March 1955).

The authors propose an approximate method for calculation of unsteady flow in open channels with lateral inflow, using the characteristic curves. Hydrographs resulting from abrupt increase and decrease of rate of lateral inflow are obtained by this method, and the calculated hydrographs are compared with experimental results.

[Abstract in the report.]

610. Iwagaki, Y., and Sueishi, T., 1954, On the unsteady flow in open channels with uniform lateral inflow: Japan Soc. Civil Engineers Proc. [Tokyo], v. 39, no. 11, p. 575–583.

611. Moklyak, V. I., 1954, Rozrakhunok neodnochasovogo dobichannya vodi pri nerivnomirnomu raspodili shvidskosti v pototsi [Computation of unequal travel times of water with nonuniform distribution of velocities in channel]: Akad. Nauk [Ukrainian S.S.R.] Doklady, no. 4.

Because the velocity distribution in a cross section is not uniform, the water passing a given cross section at a given moment will arrive at the downstream section at different times. Special curves are derived for a rectangular section for the purpose of computing travel time for different parts of initial discharge having nearly the same velocity in a cross section.

612. Morikawa, G. K., 1954, On the theory of flood waves in rivers: New York Univ., Inst. Math. Sci. Rept. IMM–NYU–210, p. 1–24.

Using a coordinate system that moves downstream with C, the celerity of steady progressing wave, it is found that for the monoclinal wave (H_1, H^2; V_1, V^2) there is

$$C/V_1=[(H_2/H_1)^{3/2}-1]/[(H_2/H_1)-1].$$

Using the stretching transformation for the combined space and time variables, a natural way of obtaining appropriate asymptotic representations is derived. The best stretching transformation is given for flood routing.

613. Nougaro, Jean, 1954, Pribilizhno odredyivanye visine nestatsionarnikh talasa u otvorenom kanalu [An approximate computation of surge heights in an open canal]: First Mtg. Hydraulics, Belgrade, 1954, Proc., p. 73–78.

It is useful sometimes for a preliminary study to show the approximate value of the wave height in an open canal. For this purpose, equations are set up and then expressed in terms of graphs. The author is considering cases occurring most frequently in the operation of hydroelectric power plants, i.e.: (1) Sudden stopping of turbines, for the wave in headwater canal; (2) Sudden starting of the turbines, for the wave in headwater canal; (3) Linear starting of the turbines, for the tailwater canal, taking or not taking into account the influence of the mean velocity of the water flow on the celerity of wave propagation. Numerous experimental data show the feasibility of use of graphs, given in this paper.

[From author's abstract.]

614. Nougaro, Jean, 1954, Influencia de una singularidad de un canal sobre la propagacion de intumescencias [Influence of a singularity of a canal on the propagation of intumescenses]: Facultad de Ingenieria [Montevideo] Bull., v. 5, no. 14, p. 349–390.

When the resistance to water movement is taken into account, then an approximate celerity formula for rectangular channel is derived as

$$C=\sqrt{g(H+h)/(1+ghL/n^2HR^{4/3})}\pm V$$

where V=mean velocity for depth H, H=original depth in canal, h=wave height (surge), n=Strickler's roughness coefficient, R=hydraulic radius, and L=length of canal. For a singularity having the head loss h', the propagation celerity through the irregularity is shown by approximation to be

$$C=\sqrt{g[H-h'(H/h-h/H+2)]}\pm V.$$

The approximation is verified experimentally. It is also verified experimentally that a jump in bottom of ΔZ gives a change in h (wave height) by $\Delta h = -V^2 \Delta Z / (-V^2 + gH)$. Equations for a trapezoidal canal are also given.

615. Penati, Savio, 1954, Azione di uno sfioratore a ventola sull'onda positiva provocata dall'arresto delle macchine nel canale adduttore di un impiante idroelettrico [On the action of a side weir, controlled by a tilting gate, upon the positive translation wave generated in the canal of a power plant by the sudden closing of the turbines]: Energia Elettrica [Italy], v. 31, no. 10, p. 733–741. Reprinted as Istituto di Idraulica e Costruzioni Idrauliche [Milano] Memorie e studi, no. 115, 1954 [On new model tests for the Tornavento Power Plant].

This study gives the second series of experiments made using a model of Tornavento powerplant. Description is given of the instruments and the procedure used in registering water levels when surges occur due to sudden stoppage of powerplant operation, for the case when a tilting gate with spillway in the channel forebay is used. These results, employing more accurate techniques, confirm the previous results. They show that the surge reduction is appreciable, but that the secondary oscillations depend on the time duration of closure of powerplant, and they propagate farther upstream as this closure time increases. The result of measurement of surge clearly confirm the values obtained by formula derived theoretically.

616. Schoenfeld, J. C., 1954, Analogy of hydraulic, mechanical, acoustic, and electric systems: App. Sci. Research., Sec. B., p. 417–450.

617. Supino, Giulio, 1954, Le oscillazioni del risalto idraulico durante una propagazione ondosa [The oscillations of hydraulic jump during wave propagation]: Energia Elettrica [Italy], v. 31, no. 2, p. 90–94.

This paper is a study of the movement of the hydraulic jump due to wave propagation and its passage through the hydraulic jump. The study is limited to the rectangular and very wide channel. The position of the jump as it occurs in nonuniform steady flow is first determined. Analysis is then made of the effect, upon position of the jump, of a wave that is generated upstream from the jump and moves downstream. The analytical approximation is obtained. If the jump is to remain at the same place during wave movements, the downstream change of level must be regulated to produce this condition.

618. Tifonov, E. K., 1954, Neustanovivsheesya dvizhenie vody v otkrytykh ruslakh pri nulevoy nachal'noy glubnne, -volna posukhu [Unsteady flow in open channels with zero initial depth, for wave in dry bed]: V.N.I.I.G. im. Vedeneeva (manuscript), [Leningrad].

The celerity of positive downstream surges, in dry bed, is studied experimentally in a canal 75 meters long. Results show that the celerity of wavefront decreases with an increase of distance travelled by the surge, and that it approaches asymptotically the velocity V of steady flow corresponding to the constant discharge which is introduced at the beginning of the canal. The mean velocity at the beginning of the canal approaches also the mean velocity of steady flow, but the celerity of surge is somewhat greater than the velocity at the beginning of the canal. However, the difference is small, decreasing with an increase of distance travelled by the

surge. (This summary and tables of results are given in the paper of Kovalenko, 1957.

619. Ambraseys, N. W., 1955, Application of the continuity equation for sewer system computations: Internat. Assoc. Hydraulic Research, 6th Gen. Mtg., The Hague, 1955, Proc., v. 4, p. D3–1 to D3–10.

A method for the determination of sewage-pipe diameters is developed, based on the use of the storage in the system and employing the balance equation for inflow minus outflow equals the stored water in a given time unit.

620. Bata, G., 1955, Utilisation des valeurs réduites dans le domaine des régimes transitories des canaux ouverts et leur application dans le cas des galéries utilisées en réservoirs d'eclusées [Use of relative values for unsteady flow in open channels and their application in the case of tunnels used as storage reservoirs]: Internat. Assoc. Hydraulic Research, 6th Gen. Mtg., The Hague, 1955, Proc., v. 4, p. D8–1 to D8–10.

Two forms of relative values are studied for unsteady flow. In the first group the relative (or reduced) values are obtained in De Saint-Venant's equations by using Q and H of the initial steady uniform flow. In the second case the dimensionless terms are those resulting from Froude's similitude, the reference length being the height H of the steady flow. The regions are precisely defined for suitability of groups of values to be used, and applications are made to the problem of emptying a tunnel being used as a storage reservoir. Use of relative values for unsteady flow in channels was suggested by use of relative values for unsteady flow in conduits under pressure.

621. Binnie, A. M., Orkney, J. C., 1955, Experiments on the flow of water from a reservoir through an open horizontal channel; II, The formation of hydraulic jump; Royal Soc. [London] Proc., ser. A, v. 230, no. 1181, p. 237–246.

By experimental study of the hydraulic jump it is revealed that there is little difference between the height of the leading wave in the smooth undular jump and the theoretical height of a solitary wave formed at the same Froude number. Rayleigh's classical theory (1914) gave a fair approximation of mean depth in the undular train of waves.

622. Bose, N. K., and Sinha, G., 1955, A study of discharge variation in tidal channels: Internat. Assoc. Hydraulic Research, 6th Gen. Mtg., The Hague, 1955, Proc., v. 1, Tidal Hydraulics and Tidal Models, Paper A2.

Hydraulic observations taken in 1954 in a tidal creek near Calcutta are analyzed with the purpose of finding a relation between the wetted perimeter P and the discharge Q, similar to Lacey's formula for uniform flow in stable alluvial channels: P equals $c\sqrt{Q}$, where c is a constant. For the ebb flow such a fixed relation was found at the beginning of this stage of the flow. For the flood flow, however, this relation between P and Q was found during the greater part of the flow. Further analyses are being continued and the relations between P and Q will be examined more closely.

[Abstract from Bibliography on tidal hydraulics, Corps of Engineers, May 1959.]

623. Dahl, N. J., 1955, On non-permanent flow in open canals: Internat. Assoc. Hydraulic Research, 6th Gen. Mtg., The Hague, 1955, Proc., v. 4, p. D19–1 to D19–16.

Cases are treated for oscillations without the loss of energy and oscillations without distortion. The general case is stated, and a particular solution is shown; examples are given. The theoretical results and previous full-scale test results are compared and discussed. The following propagation celerities are given: (1) wave in deep water, $C \approx g\sqrt{l/2\pi}$, where l-wavelength; (2) wave in shallow water without distortion, $C = V + \sqrt{gH}$; (3) wave in shallow water with distortion (very long waves), $C \approx 3V/2$, where V=original mean velocity of water in the channel.

624. Dronkers, J. J., and Schoenfeld, J. C., 1955, Tidal computations in shallow waters: Am. Soc. Civil Engineers, Proc., v. 81, Separate 714 (discussions, Paper 841, p. 41, and HY 5, Paper 1082–3). Also Rijkswaterstaat commun. no. 1, [Den Haag, Netherlands] 1959.

The computation of tidal elevations and currents in shallow coastal waters may serve various practical purposes. Mathematically the problem is so involved that no simple procedure to be generally accepted exists. There are mainly three ways of approach which are illustrated in this paper by expounding the computation methods developed in Europe, in particular in the Netherlands. It depends on the sort of problem to be solved which method should be adopted as the most appropriate. Contents: Introduction (purposes of computation; nature of the problem; historical survey of tidal computation; list of basic symbols). The Basis of Tidal Computations (schematization of an estuary; the differential equations; particularizing conditions). Integration by harmonic components (confined to the periodic tide; equations are linearized making it possible to consider the tide as sinusoidal; nonlinear terms are treated and the interaction of harmonic components is investigated; are developed more in detail). Direct integration (finite difference methods are discussed, both in the original quad-scheme and in the more recent cross-scheme; principles of the more refined methods of power series and iteration are discussed, followed by various applications of these methods). Integration along characteristics (principle of the characteristics and their bearing on the phenomenon of propagation discussed in connection with linear equations without resistance; amendments to be made in order to deal with the resistance are discussed; the variability of the velocities of propagation is dealt with; shock wave conditions are considered).

[Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, May 1957.]

625. Dziatlik, H., 1955, Model elektryczny rzeki [Electrical model for a river]: Przeglad Elektrotechniczny [Poland], 21 July 1955, p. 447–445.

Unsteady flow in open channels is considered to be an important and difficult problem of modern hydroengineering. Elucidation of this problem depends on the solution of De Saint-Venant differential equations, but the various graphical and analytical integration methods hitherto practiced are, as a rule, troublesome or inaccurate. The new method is based on the contention that the equations for a long-distance electric transmission line having variable parameters can be made to be analogous in all respects, so that even the most complex cases of unsteady flow in rivers can be studied according to the transient phenomena in long-distance lines. The analogy

of electric and hydraulic variables is given by comparing basic differential equations.

626. Einstein, H. A., and Fuchs, R. A., 1955, Computation of tides and tidal currents—U.S. Practice: Am. Soc. Civil Engineers Proc., v. 81, Separate 715.

The authors have recently made a survey of past and present calculation methods used for the prediction of tidal stages and flows in canals and estuaries. The results of the study conducted under contract for the Committee on Tidal Hydraulics, Corps of Engineers, U.S. Army, are in part, offered in this paper. The report is limited to practice in the United States as developed and applied before the Second World War. Describes and discusses prominent older methods and indicates where the methods may be modified for added accuracy or simplicity. A resume of the complicating aspects of the tidal flow problem in general and the various effects and causes which must go into a mathematical description of such flows is given. Discusses the differential equations of tidal flow and presents the equations. The various solutions of the equations, obtained by the introduction of simplifying assumptions, which are presented and discussed are: Parson's harmonic theory (Parsons, W. B., The Cape Cod Canal); Brown's reflected wave theory (Brown, E. I., Flow of Water in Tidal Canals); and Pillsbury's theory (Pillsbury, G. B., Tidal Hydraulics). [Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, May 1957.]

627. Evangelisti, Giuseppe, 1955, On the tidal waves in a canal with variable cross-section: Internat. Assoc. Hydraulic Research, 6th Gen. Mtg., The Hague, 1955, Proc. v. 1, p. A10–1 to A10–2.

The theoretical approach to wave movement in a canal having variable cross-sections, showing that the wave-phase velocity is independent of wave frequency, is given by $\sqrt{gH}$, as if the canal were uniform. The waves undergo appreciable distortion, and their amplitude varies along the canal in such a way that a quantity (proportional to the energy of the wave) remains constant.

628. Hunt, J. N., 1955, On the solitary wave of finite amplitude [A proposed l'onde solitaire d'amplitude finie]: Houille Blanche [France], no. 2, p. 197–203.

It is shown that the equation for the profile of the solitary wave and the velocity of propagation can be obtained by a convenient process of successive approximations without determining the velocity potential throughout the plane of flow. The method employs an operational form of the free-surface condition given by Levi-Cività (1911). The wave profile and velocity are found to the third order in the ratio h/H, where h=the wave amplitude and H=the undisturbed depth. For small values of this ratio the solution reduces to that of Boussinesq. The velocity equation is in good agreement with measurements reported by Daily and Stephan (1953):

$$C^2=gH\left[1+\frac{h}{H}-\frac{1}{20}\left(\frac{h}{H}\right)^2\frac{3}{70}\left(\frac{h}{H}\right)^3\right], \text{ where}$$

$(h/H)_{max}=0.92$; comparison with $C/\sqrt{gH}$ gives the difference 0.6 percent for $h/H=0.5$ and 2 percent for $h/H=0.92$.

629. Ippen, A. T., Kulin, Gershon, and Raza, M. A., 1955, Damping characteristics of the solitary wave: Mass, Inst. Technology Hydrodynamics Lab. Tech. Rept. 16, p. 1–40.

This report presents the results of an investigation of the damping of solitary waves in water of constant depth. The damping was studied principally by direct observation of the decrease in amplitude of waves as they proceeded up and down a 32-foot Lucite tank. Also reported is a preliminary investigation of the damping process by direct measurement of boundary layer velocities with a specially designed differential gage.

Damping was observed over bottoms roughened with uniform sands of diameter ranging from 0.0056 feet to 0.0198 feet, and the results are compared with previously published smooth-bottom data. Still-water depths ranged from 0.17 feet to 0.40 feet. Approximate expressions for wave Reynolds number and boundary layer thickness are derived and the latter is compared wlth available experimental results.

It was found that in a tank of the size used in this investigation, Reynolds number sufficiently high to permit the existence of true roughwall friction in the roughened bottom runs were not developed.

Results of near-bottom velocity observations with a differential gage verified the existence of a boundary layer increasing in thickness from front to rear of the wave.

[From author's abstract.]

630. Ishihara, Tojiro, and Ishihara, Yasuo, 1955, On an electronic analog computer for flood routing: Japan Soc. Civil Engineers [Tokyo] Trans., no. 24, p. 44–57.

The flood is a very complex phenomenon. According to Dr. Hayami's theory of flood waves, however, which takes into account the complex features of river, the fundamental equation becomes, approximately

$$\frac{\partial H}{\partial t}+A\,\frac{\partial H^{3/2}}{\partial x}=\mu\frac{\partial^2 H}{\partial x^2}$$

where H: water depth, t: time, x: distance, and A, μ: numerical constants. In this paper, a method to initiate this equation by electric circuits and to construct them was mentioned, that is, an electronic analog computer was designed and constructed. Using this computer, the authors obtained some solutions of flood waves, and discussed the characteristics of flood and the significance of constants A and of the equation mentioned above. As an example, the results of actual flood routing for the Kiso River were explained and it was found that this computer produced with good accuracy.

[Author's English synopsis.]

631. Iwasa, Y., 1955, Analytical considerations on cnoidal and solitary waves: Kyoto Univ. [Japan] Eng. Fac. Mem., v. 17, no. 4, p. 264–276.

The mathematical theory on progressive translation waves of the permanent type is developed. The analysis of the second approximation based upon the momentum and energy approaches, considering the effects of the vertical acceleration, shows that the surface profile of waves is expressed in terms of an elliptic function which is designated as the cnoidal-function, the name being derived therefrom as analogous to sinusoidal waves, and the solitary wave is included as a special case of the cnoidal waves.

It should be noticed that the behavior of cnoidal waves may be reduced to that of classical sinusoidal waves as a special limiting approximation. [Author's abstract.]

632. Kazarnovskiy, Yu. E., 1955, Uchet transformatsii pavodka v vodokhranilishchakh s sifonnymi vodosbrosami [Flood-wave transformation in reservoirs with syphon spillways]: Gidrotekhnika i Melioratsiya [U.S.S.R.], no. 8, p. 55–57.

In flood routing based on Kocherin's formula for the triangular-shaped hydrograph, the author develops the following formula for damping of flood peak (routed through a reservoir having a free spillway automatically operated):

$$\gamma=\frac{Q_m}{P_m}=1-\frac{W_1}{W},$$

where P_m=peak of inflow, Q_m=peak of outflow hydrograph, W=volume of flood wave, W_1=volume between the maximum and starting (normal) reservoir levels. For routing through a reservoir having syphon spillways, the formula is: $[(1+K)/2K](1-\sqrt{W_1/W}$, where $K=Q_0/Q_m$, Q_0 being the outflow for normal water level in reservoir. Because, for the most part, $K>0.9$, $[(1+K)/2K]\approx 1$, so that $\gamma=1-\sqrt{W_1/W}$.

633. Kovács, György, 1955, Az árhullámok levonulására jellemző hidrológiai mennyiségek meghatározása [Determination of the hydraulic magnitudes characterizing the propagation of flood waves]: Hidrológiai Közlöny [Hungary], v. 35, no. 11–12, p. 394–423. (English summary p. 422–423.)

This study uses the Bernoulli equation for unsteady flow and the continuity equation. The basis of the study is the analysis of rating curves, especially of loop curves pertaining to flood waves. The parts of the loop corresponding to a rising limb (monoclinal) and falling limb (also monoclinal) are specially treated and fitted by power function, and then applied to one-peak floods. The actual data of the Tisza and Danube Rivers support all derivations for rating curves and flood waves. The author considers that this procedure enables the determination of time and local variations of discharge for flood waves.

634. Ligthill, M. J., and Whitham, G. B., 1955, On kinematic waves I, Flood movements in long rivers: Royal Soc. [London] Proc., A 229, p. 281–316.

The kinematic waves are first defined as destinct from the dynamic waves. The flood waves in long rivers are treated mostly as kinematic waves, although dynamic waves also appear. From the full equation of motion for an idealized problem, it is shown that at Froude numbers appropriate to flood waves, the dynamic waves are rapidly attenuated, and the main disturbance is carried downstream by the kinematic waves. The kinematic shock wave (monoclinal flood wave) is investigated. The final section treats the application of the theory of kinematic waves to the determination of flood movement.

635. Moklyak, V. I., 1955, K raschetu transformatsii povodkov v estastvenny ruslakh [On the flood routing in natural channels]: Ukrainskiy N.I.G.M.I. [U.S.S.R.], Trudy (Proc.), Vypusk (issue) no. 3.

636. Nash, J. E., and Farrell, J. P., 1955, Graphical solution of the flood routing equation for linear storage-discharge relation: Am. Geophys. Union Trans., v. 36, p. 319–320.

Starting from the equation $W=KQ$ and $P=Q+KdQ/dt$, the experiment relation of Q and P is developed analytically as $P-Q=(P-Q_0)_e-t/K$, where Q_0 is the outflow for $t=0$. A template is developed for $Q=Q_{0e}-t/K$, and the routing for constant P is given as an example of the template's use.

637. Nougaro, Jean, 1955, Méthode graphique pour le calcul de la propagation des intumescences dans les canaux decouverts [Graphical method for the computation of propagation of tranlatory waves in open channels]: Internat. Assoc. Hydraulic Research, 6th Gen. Mtg., The Hague, 1955, Proc., v. 4, p. D5–1 to D5–15.

The paper deals with the principle of a new graphical method for the study of intumescences. It is based on and deduced from Bergeron's graphical method, by using linear characteristics $(1/B)(1/[V \pm C])$ in coordinate system (Q, H). These straight lines can be replaced in a rectangular channel by the characteristic curves

$$Q+Q'xQ_0 \log H \pm (2/3)BH\sqrt{gH}.$$

The applications of this method are made for both laboratory and powerplant canals, and the results justify the application, according to the author. This method allows an approximate relation

$$\frac{H_1}{H_0}=f\left(\frac{C}{V}\right),$$

where H_0 is the initial water depth in rectangular channel, and H_1 is the depth after the passage of wave, especially for the sudden changes in water powerplant operations and for surges created in both headwater and tailwater canals.

[From author's abstract.]

638. Proudman, J., 1955, The propagation of tide and surge in an estuary: Royal Soc. [London] Proc., A, v. 231, p. 8–24.

Theoretical investigation of the distribution, along an estuary, of a combination of tide and surge which have been generated in the open sea. The following results relate to the same sequence of meteorological conditions over the sea. For a single progressive wave, the height of a surge whose maximum occurs near to the time of tidal low water, and decreases as the range of tide increases. To the order of approximation followed in the paper, these differences are due to friction and increases with distance from the sea. For a standing oscillation, the following results relate to the head of the estuary. When the primary surge rises to its maximum more rapidly than it falls from it, and when this maximum occurs near to the time of tidal high water, the effect of shallow water is to make the surge increase as the range of tide increases, and the effect of friction is to make the surge decrease as the range of tide increases. On applying the formulae of the paper relating to progressive waves to the Thames Estuary, it appears that, owing to the small depth of water at mean level, the details only hold for a few miles from the sea. But the tendencies enumerated should hold all the way to London.

[Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers; February 1954.]

639. Proudman, J., 1955, The effect of friction on a progressive wave of tide and surge in an estuary: Royal Soc. [London] Proc., A, v. 233, p. 407–418.

This is a second paper (for the first see: The propagation of tide and surge in an estuary, by J. Proudman) on the theoretical distribution, along an estuary, of a combination of tide and surge which have been generated in the open sea. It differs from the first paper in that, in the basic equations, the nonlinear inertia term is neglected, and only one phase of the wave is considered at a time. Also, the formulae are now valid for any height of tide or surge at the mouth of the estuary and for any distance from the sea. For the same sequence of meteorological conditions over the sea and when the height of surge at the mouth of the estuary is greater than a certain fraction of the range of tide there, the surge up the estuary is less at the time of high water than it is at the time of low water, and it decreases as the range of tide increases. This is a wide extension of the range of validity of one of the results of the first paper.

[Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, February 1954.]

640. Richards, B. D., 1955, Flood estimation and control; Ch. XI, Flood Control: London, Chapman and Hall, Ltd., p. 134–153.

This paper descusses the flood absorption effect of regulating reservoirs having various kinds of outlets, constant or changing inflows, and flood-storage basins.

641. Sato, S., Kikkawa, H., and Kishi, T., 1955, On the hydraulic characteristics of the tidal reach of the Tone River: Internat. Assoc. Hydraulic Research, 6th Gen. Mtg., The Hague, 1955, Proc., v. 1, Paper A6.

This paper reports the results of field observations made in the tidal reach of the Tone River in 1950 and 1954. It discusses: (1) the characteristic of the equation of motion, velocity profile, and the propagation velocity of river tide; (2) attenuation of tidal amplitude along the river course; (3) the characteristics of the suspended load transportation and the shearing force acting on river bed; and (4) the turbulent mixing between salt and fresh water, and the intrusion velocity of the salt water.

[Abstract from Bibliography on Tidal Hydraulics Corps of Engineers, May 1959.]

642. Schoenfeld, J. C., 1955, Theoretical considerations on an experimental bore: Intnat. Assoc. Hydraulic Research, 6th Gen. Mtg., The Hague, 1955, Proc., v. 1, p. A15–1 to A15–12.

Both the propagation and the shape of the bore are considered thoretically, and these considerations are checked by experimental evidence obtained on a tide with a bore in a laboratory canal.

The propagation is treated by considering the bore as a mobile hydraulic jump. The theory is checked twice, first by combining the data on the bore when it passed the observation places, and secondly by computing the whole tide with the bore by the method of characteristics.

The influence of the vertical accelerations on the shape of the bore is treated by considerating elementary short wave solutions of sinusoidal and exponential form. The bore is compared to the solitary wave. The influence of capillarity is discussed. The theory is checked by means of length profiles of the observed bore. Finally an energy budget is re-

constructed, which yields the amount of deceleration losses. The breaking of the bore is discussed.

[Author's abstract.]

643. Schoenfeld, J. C., 1955, Discussion of the paper "Graphical method for calculating the propagation of translatory waves in open channels" by Nougaro, J: Intnat. Assoc. Hydraulic Research, 6th Gen. Mtg., The Hague, 1955, Proc., v. 4, p. III–3 to III–5.

Use of the method of characteristics for integration of two De Saint-Venant equations is discussed. Because of the symmetry of differential equations and contresubcharacteristics (characteristics projected on surfaces of dependent variables), it is shown that it is best to use H and V (depth and velocity) or H_e and Q (total energy head and discharge) as dependent variables, rather than to use other combinations. It is advantageous to use the combination H_e and Q when the cross-sectional area varies with the length of channel.

644. Schoenfeld, J. C., 1955, Discussions on the papers "Nonpermanent flow in open canals" by N. J. Dahl and "On tidal waves in a canal with variable cross-section" by G. Evangelisti: Intnat. Assoc. Hydraulic Research, 6th Gen. Mtg., The Hague, 1955, Proc., v. 1, p. III–3 to III–6.

Long waves in channels undergo two distinct forms of distortion, as caused by : (1) friction, which is expressed by the zero-order terms, linear or nonlinear, in the differential equations; and (2) variability of cross-sectional area with height of water, which is expressed by the nonlinear character of the first-order terms. In this view, the author's studies and results are compared with the results of Evangelisti and of Dahl.

645. Schoenfeld, J. C., 1955, Tides in funnel-shaped channels: Rijkswaterstaat Commun. [Den Haag, Netherlands], Nota CSD 55–16.

646. Schoenfeld, J. C., 1955, Getijberekening door Integratie langs Karakteristieken [Tidal computation by integration along characteristics]: Rijkswaterstaat Commun. [Den Haag, Netherlands], Nota CSD 55–14, p. 13–18.

647. Sueishi, Tomitaro, 1955, On the runoff analysis by the method of characteristics [Hydraulic studies on the runoff phenomena of rain water, 2d report]: Japan Soc. Civil Engineers [Tokyo] Trans., no. 29, p. 74–87.

In this paper, the approximate method calculating the unsteady flow in open channels with uniform lateral inflow using the characteristic curves, described in the author's previous paper, is practically applied to the analysis of runoff phenomena in natural rivers, and a new method estimating the runoff from rainfall is established.

Application of this method to runoff analysis in the Daido River in Japan shows that the method of characteristics is very available for the estimation of the runoff in rivers with relatively steep slopes.

[Author's abstract in English].

648. Yuichi, Iwagaki, 1955, Fundamental studies of the runoff analysis by characteristics: Kyoto Disaster Prevention Research Inst., Kyoto Univ. [Japan], Bull. 10, p. 1–25.

In treating the hydraulic analysis of surface runoff phenomena, the author's basic idea is expressed as follows: the surface runoff mechanism of rain water in a mountainous district consists of the combination of overland

flow and flow in open channels with lateral inflow. Consequently, the unsteady flows with lateral inflow must be solved for various conditions, individually.

In this paper, an approximate method for calculation of unsteady flow in open channels with lateral inflow, using the characteristics, is presented. Hydrographs resulting from the abrupt increase and decrease of rate of lateral inflow are obtained by this approach, and moreover the calculated hydrograph are compared with the experimental results.

[Author's synopsis.]

649. Whitham, G. B., 1955, The effects of hydraulic resistance in the dam-break problem: Royal Soc. [London] Proc., v. 227. no. 1170, p. 399–407.

When resistance is neglected, the solution of the simple dam-break problem is readily obtained on the basis of shallow-water theory, and the results are well known. However, near the head of the wave, where the water surface meets the ground, resistance effects cannot be neglected; there is, in fact, a type of boundary layer near the wave-front. In this paper, the Pohlhausen method (which is used in conventional boundary-layer problems) is applied to study of the effect of this "boundary layer." In particular, the retardation of the wavefront behind the position predicted by the simple theory is found.
[Author's abstract.]

650. Zheleznyak, I. A., 1955, Raschet transormatsii maksimal'nykh raskhodov s uchetom predpavodkovogo oporozhnyeniya vodokhranilishcha [Computation of maximum discharge modification taking into account the emptying of a reservoir before the flood]: Akad. Nauk [Ukrainian] S.S.R. Research and Computations in Hydrology, v. 13 (XX), p. 112–121.

The study takes into account the effect on reduction of the maximum outflow discharge of prior emptying of a part of storage. Tsinger's formula (1953) is discussed, and its misconceptions are stressed. Pleshkov's (1953) position is criticized, and an explanation is given for those instances when reduction in the peak will not take place. Assuming preflood reduction of storage, approximate formulas for maximum outflow are developed for free spillway operation and for operation of gages on the crest. The author employs Kocherin's assumption of the linear function of the outflow rating curves and states that the formulas permit an error of several percentage points. The limitations of this procedure are discussed, and it is shown that the procedure is valid only if W_r (reduced storage) is smaller than W_n (see Zheleznyak, 1953).

651. Abbott, M. R., 1956, A Theory of the propagation of bores in channels and rivers: Philos. Soc. [Cambridge] Proc., v. 52, no. 2, p. 344–362.

A theory is presented of the nonlinear propagation of waves and bores in channels of varying cross section with a basic steady flow governed by frictional resistance; this corresponds to the flow in tidal rivers. The theory provides a condition on the tidal range required to produce a bore, in terms of the geometry and friction parameters of the river, and the propagation of such a bore is then described. The theory is applied to the Severn River and the results agree satisfactorily with observation. Results for the special case of waves moving into still water in a channel of varying section are also noted in detail.

[Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, May 1959.]

652. Adachi, Shohei, 1956, On the propagation of flood wave in the transient region between the river channel and the storage region of reservoir: Japanese Natl. Cong. Appl. Mech. Proc., rept. II/27, p. 367–371 (published by Science Council of Japan [Tokyo], March 1957).

A flood wave has the character of a long wave in the storage region where the slope of water surface is nearly always zero. The propagation velocity of a flood wave is usually larger in this region than in the river channel. In the actual reservoir, however, the storage effect acts to delay the propagation of a flood wave. Therefore, between the river channel and the storage region of a reservoir, there must be the transient region where the propagation of a flood wave may be retarded by the storage effect. Authors have confirmed the existence of this region by the model experiment of Maruyama Reservoir in the Kiso River.

[Abstract in the report.]

653. Alekseev, G. A., 1956, Priblizhennye methody rascheta transformatsii pavodka vodokhranilishchem na osnove skhematizatsii gidrografov pritoka i sbrosa [Approximate methods of flood routing through reservoirs by using the schematic hydrographs of inflow and outflow]: Gosudarstveniy Gidrologicheskiy Inst. [U.S.S.R.] Trudy, No. 52 (106).

654. Arsenishvili, K. I., 1956, Kriterii volnoobrazovaniya v kanalakh s bol'shimy uklonami [Criteria of wave formation in canals with great slopes]: Gidrotekhnicheskoe Stroitel'stvo [U.S.S.R.], no. 3, p. 41–44.

Results of experimental studies of roll-wave formations in steep channels are reported. The experimental check of the existing criteria (Vedernikov, Mostkov, Keuligan-Patterson, Thomas, Dressler, and so forth) of roll-wave formation shows that these criteria, in some cases, neither describe the true character of wave movement nor take into consideration the shape of the cross section. The author gives the following empirical criteria: Criterion A_1: $0.02 > S_0 > 0.30$; Criterion A_2: $H_0/P \geq 0.10$, where S_0 = bottom slope, H_0 = depth for full canal with steady flow, and P = wetted parameters for steady flow. Criterion A_2 determines the regime, without roll waves. for slopes of waves according to Criterion A_1.

655. Blind, Hans, 1956, Nichtstationaere Stroemungen in Unterwasserstollen [Unsteady flow in tailrace outlet tunnels]: In Veroeffentlichungen zur Erforschung der Druckstossprobleme [F. Toelke, ed.], Berlin, Springer, p. 67–108.

This theoretical and experimental study of water regimen in the tailrace tunnels of underground water powerplants also treats the movement of surges (bores and depressions) along the circular tunnels, when water is flowing as free surface flow. The celerity of waves, which is the criterion for change of surges by progression, is analyzed both by the theoretical and experimental method.

656. Chow, Ven Te, 1956, Hydrologic studies of floods in the United States (Flood-routing): Internat. Assoc. Sci. Hydrology, U.G.G.I., 1956, v. 2, pub. 41, p. 143–147.

This is a short review of flood-routing methods in the United States, including: (1) analytical, graphical, semigraphical, nomographical, slide-

rule, circular computers, and Muskingum methods; (2) mechanical and electronic instrumental routing procedures; (3) model routing; and (4) stage routing.

657. Cuénod, M., 1956, Contribution à l'étude des crues-Détermination de la rélation dynamique entre les précipitations et le débit des cours d'eau au moyen du calcul à l'aide des suites [Contribution to the study of floods; calculation, by using a series method, of the dynamic relation between rainfall and river flow]: Houille Blanche [France], no. 3, p. 391–402.

The device of the index (or characteristics) hydrograph is introduced to represent the rising limb of a hydrograph resulting from a given value of precipitation that starts suddenly and persists indefinitely. The hydrographs are derived by using a series of units for the precipitation and the corresponding series of resulting discharges. This method is a variation of the classical unit hydrograph theory. An example of its use for the Krummbach River is given and discussed.

658. Dooge, J. C. I., 1956, Synthetic unit hydrographs based on triangular inflow: Iowa Univ., Dept. Mech. and Hydraulics, M.S. thesis, p. 1–103.

This study is concerned with the problem of synthetic unit hydrographs, that is, the prediction of surface runoff on the basis of only such information as is available from a topographic map of the catchment area. A theory of the unit hydrograph is presented which links the unit hydrograph principle with the presence of linear storage in the catchment area. The nature of the runoff process and the morphology of natural catchments are discussed in order to provide a basis for a synthetic method. It is argued that the instantaneous unit hydrograph can be produced by routing a triangular inflow through a single element of linear reservoir storage, and hydrographs derived in this way are shown to give excellent agreement with both the dimensions and shape of unit hydrographs derived by other means. In the final chapter a practical procedure is outlined and two examples are worked.

659. Egiazarin, B. O., 1956, Priblizhennyy raschet neustanovivshegosya dvizheniya vody v nizhnem b'efe GES pri sutochnom regulirovanii [An approximate method of computation of unsteady flow in tailwater channels, made by employing use of powerplant records]: Gidrotekhnicheskoe Stroitel'stvo [U.S.S.R.], no. 7, p. 48–51.

This approximate method is derived from the actual data obtained on powerplant canals and laboratory channels. The formulas for the parameter a (a degree of wave deformation) are derived for different cases. A practical procedure is given for use of formulas in computing wave characteristics.

660. Escande, L., and Nougaro, J., 1956, Régime variable dans un canal d'amenée associé à une galérie en charge [Unsteady flow in a headrace canal leading to a pressure tunnel]: Houille Blanche [France], no. 2, p. 240–249.

This report is a study of two problems relating to waves in a headrace canal that is connected to a pressure tunnel having a mass-oscillations setup. The first problem deals with a tunnel-surge-tank system connected to an open canal. The second problem deals with a syphon set in a headrace canal. The use of theoretical, analytical, and graphical methods is demonstrated. The results thus obtained show satisfactory agreement with the experimental results.

661. Gil'denblat, Ya. D., 1956, Nekatorye voprosy, svyazannye s raschetami neustanovivshegosya dvizheniya i otsenkoy raspolagaemykh po naporu pikovykh moshchnostey gidroelektrostantsii [Some questions related to the computation of unsteady flow and the estimation of peak power loads of water powerplants, as determined by amount of head available]: Akad. Nauk [U.S.S.R.], Problemy regulirovaya rechnogo stoka [Problems of river runoff regulations], v. 6, p. 263–277.

Unsteady movement in the tailwater of powerplants, as represented by the unsteady-load diagram, is studied in order to analyze power loads. The boundary conditions at the powerplant and at the beginning of tailwater channel are discussed. A study is made of the position of positive surges in each moment, which information is necessary for selecting Δt and Δx for flood routing. Rating curves at selected stations are used for analysis. A formula for the optimum length of reach is given, and the selection of reaches and of time interval is discussed. A method of accounting for winter conditions is shown. An approximate (straight-line) relationship between the decrease of powerplant head (by tailwater-level increase) and the difference between peak load and mean load is developed, as are other significant relationships.

662. Isaacson, E., Stoker, J. J., and Troesch, B. A., 1956, Numerical solution of flood prediction and river regulation problems, Report III: New York Univ., Inst. Math. Sci. Rept. 6 IMM–NYU–235, p. 1–70.

The flood-routing procedure employing the finite-differences method, and using the digital computer UNIVAC (Stoker 1953), is applied as a numerical solution for prediction of the 1945 and 1948 floods in the Ohio River, for prediction of the 1947 flood through the junction of the Ohio and Mississippi Rivers, and of the floods of 1950 and 1948 through the Kentucky Reservoir. The factors affecting accuracy are discussed, and suggestions are given for improvement of the numerical methods. The model studies of river flood movements are compared with numerical studies using digital computers.

663. Ishihara, T., Hayami, Sho., and Hayami, Shi., 1956, Electronic analog computer for flood flows: Regional Tech. Conf. Water Resources Devel. in Asia and Far East, Proc., ECAFE, Folod Control Ser. no. 9, Bangkok, p. 170–174.

Based on Hayami's equations (1954) for flood flows, the electronic analog computer is developed and described. Its suitability for accounting for the nonlinear resistance losses along the river is notable.

664. Ishihara, Tojiro, and Ishihara, Yasuo, 1956, Electronic analog computer for flood flows in the Yodo River: Japan Soc. Civil Engineers Proc. [Tokyo], v. 41, no. 8, p. 21–24.

665. Iwasa, Y., 1956, Hydraulic characteristics of solitary waves: Japan Soc. Civil Engineers Ann. Convention [Tokyo], May 1956.

666. Jaeger, Charles, 1956, Engineering Fluid Mechanics: [English ed. translated by P. O. Wolf] London Blackie and Son Limited, chap. 8, p. 359–392. German edition, Basle, Switzerland, Verlag Birkhaeuser, 1949.

The chapter on unsteady flows discusses propagation of small waves on flowing water and surges in open channels. It contains analysis of surges if change of discharge is known and if change of depth at inlet is known. A graphical method of analysis is given for translatory waves in open

channels (see Craya, 1945, 1946), and for underground powerstations (tailrace canals).

667. Kinosita, T., 1956, Hydrodynamic study of the flood flow: Internat. Assoc. Sci. Hydrology, U.G.G.I., Symposia Darcy, Dijon [France], 1956, v. 2, pub. 41, p. 173–183.

The paper deals with numerical integration of two partial differential equations for unsteady flow. By neglecting the inertia term ($\partial V/\partial t + V\partial V/\partial x = 0$), the boundary condition is determined by water depth alone, and Q is determined from a momentum equation. A mesh point scheme of Δx and Δt is given for the finite differences method of integrating two equations, in which emphasis is on the use of mesh points for boundary and for initial conditions. An expression is given for the accumulation of errors due to replacement of differentials by differences. The restrictions for Δt are introduced (for any cross section and for rectangular and triangular channels). The examples for river and model channel are given. The peak velocity is studied as a function of wave steepness.

668. Kohler, M. A., 1956, River and water-supply forecasting: Acad. Sci. [New York] Trans., ser. 2, v. 18, no. 8, p. 732–745.

This is a general discussion of flood forecasting which treats the subject of streamflow routing. The basic storage equation is used, and the storage of a reach is expressed as $S = K[xP + (1-x)Q]$, in which K and x are experimental constants.

669. Kozák, Miklós, Néhány a nempermanens szabadfélszinu vizmozgás számitására szólgálo eljárás ismertetése [Some methods for the computation of unsteady flow with free water surface]: Hidrológiai Közlöny [Hungary], v. 36, no. 1, p. 17–32.

The present paper discusses methods of analysis of unsteady, gradually varying, free surface flow. The following conclusions can be drawn: (1) Because of its mathematical exactness the method of characteristics is advantageous, provided that all the relevant hydraulic coefficients can be determined exactly according to actual conditions. Greatest care should therefore be exercised in applying this method to watercourses having greatly varying cross sections. Irregularities of the bed and particularly wave reflections occurring at river bends in natural watercourses made the reliability of the establishment of wave front velocities highly questionable. Values obtained by computation and by actual measurements, however, show a fair agreement up to lengths of 50 kilometers. (2) The method of finite differences is applicable to all cases. A disadvantage of this method is its reduced exactness, which, however, is sufficient for practical purposes. Actual computation work is at the same time facilitated.

[From author's summary, given in English.]

670. Kritskiy, S. N., and Menkel', M. F., 1956, O rasplastyvanii volny vysokikh vod pri prodvizhenii po prizmaticheskomu ruslu [Attenuation of the flood wave in the movement along the prismatic channel]: Akad. Nauk [U.S.S.R.], Problemy regulirovaniya rechnogo stoka [Problems of river runoff regulations], v. 6, p. 248–262.

A schematic approach is used to study wave attenuation along a river channel. The maximum discharge attenuation is analysed in terms of the path x, maximum discharge Q_0 at starting station, roughness coefficient n,

volume of flood wave W, and river-bed slope S_0. A simple starting wave is assumed, having $H_{max}=H$ and triangular shape of the length s on each side of H. An approximate formula is derived which gives the peak at the section x as:

$$Q=Q_0\sqrt{1/(1+2Q_0^2r^2x/W^2S_0^2)}$$

where area $A=aH^{m+1}$, $H=$depth, $W=2aH^{m+1}s/(m+2)$. It is concluded that the rate of maximum discharge attenuation along river increases with sharpness of hydrograph (characterized by Q_0/W) and with increase of channel roughness, but decreases with an increase of river slope S_0. The routing of a simple triangular wave by instantaneous wave profiles (method of finite differences) is used to derive attenuation. Results show a close check of the developed formula.

671. Long, R.R., 1956, Long waves in a two-fluid system: Jour. Meteorology, v. 13, no. 1, p. 70–74.

The differential equations governing the unsteady motion of a system of two superimposed liquids are integrated by use of the method of characteristics. The solution differs significantly from that obtained by assuming a very thin lower layer.

If the basic shear across the interface is small, a wave breaks forward (as an ordinary water wave) only if the amplitude is fairly small and if the lower layer is thinner than the upper layer. If the upper fluid is thinner, the wave breaks backward. The speed of the various points of the wave does not vary monotonically with amplitude.

If a shear exists, higher velocities in the upper layer in the direction of wave. The opposite shear may cause a wave to break backward.

[Author's abstract.]

672. Long, R. R., 1956, Solitary waves in one- and two-fluid systems: Tellus, v. 8, no. 4, p. 460–471.

A theoretical discussion is given of the solitary wave. Part I is concerned with the wave on a water surface; Part II considers the solitary wave at the interface of two superimposed liquids of different density, bounded above and below by rigid surfaces.

In Part I higher approximations are obtained for the speed of propagation, and limits to the wave speed are derived from the momentum theorem. The results include a correction to the second approximation of Weinstein.

In Part II the first two approximations to the wave speed are derived. The equations of the second approximation yield limits on the wave amplitude.

[Author's abstract.]

673. Messerle, H. K., 1956, Differential analyser solution of hydraulic problems in hydroelectric systems: Houille Blanche [France], no. 6, p. 813–836.

The use of automatic computers in hydraulic analysis is discussed in this paper, and special reference is made to the mechanical differential analyser, which has been found particularly suitable for problems arising in hydraulic systems. The programming of a mechanical differential analyser for the solution of general hydraulic problems is described, and it is shown how the results obtained may be used to determine the optimum design fea ures.

One of the problems analyzed on the mechanical differential analyser of the C.S.I.R.O., Mathematical Instruments Section, University of Sydney,

is the diversion of the Tooma River, a project of the Snowy Mountains Hydro-Electric Authority in Australia. This investigation is discussed in detail as a typical example.

The paper is useful for the study of advantages and disadvantages, as well as for study of the assumptions made in the use of mechanical differential analyser.

674. Nougaro, Jean, 1956, Réflexion et transmission d'une intumescence à un changement de section dans un canal découvert [Reflection and transmission of a wave at a change of cross section in an open channel]: Acad. sci. [Paris] Comptes rendus, v. 243, no. 15, p. 1016–1019.

The method is given for determination of the waves reflected and transmitted as a result of sudden change of canal cross section, in a transitory movement. Formulas for the new depth are developed and are checked by experiments; adequate agreement is shown.

675. Nougaro, Jean, 1956, Sur l'amortissement de la hauteur d'une intumescence positive dans un canal découvert a fond horizontal [On the attenuation of the height of a positive wave in an open canal with horizontal bottom]: Acad. sci. [Paris] Comptes rendus, v. 242, no. 10, p. 1263–1265.

An analytical method using characteristics (straight lines) is developed for computing the damping of a steep wave as it progresses both in upstream and downstream directions. The characteristics resulting from friction loss is introduced as a first approximation. Expressions for change in Δh are developed, and it is concluded that the damping of a positive surge in downstream direction is smaller than that of a positive wave in upstream direction. The formulas can be used for any cross section if the equivalent depth in rectangular channel is used.

676. Nougaro, Jean, 1956, Recherches expérimentales sur l'amortissement des intumescences dans les canaux découverts [Experimental research of the wave absorbtion in the open channels]: Acad. sci. [Paris] Comptes rendus, v. 242, no. 16, p. 1953–1956.

It is shown that if a discharge is changed from Q_0 to Q_1 (from V_0 to V_1) in a channel from

$$\Delta h/h = (V - V_1)/V_0,$$

where Δh is the damping of created wave of height h, and V is the velocity at the point where the damping is Δh. Graphical solutions are given for three cases and compared with the experiments in a model 3 percent size of the Palaminy powerplant canal. Comparison is also made with the experiments envolving natural canal (Pebernat), and good agreement is shown.

677. Nougaro, Jean, 1956, Influencia de una singularidad de un canal, sobre la propagacion de intumescencias [The influence on wave propagation of a singularity in a canal]: Bol. de la Facultad de Ingeneria y Agrimensura de Montevideo [Uraguay], v. 5, no. 14, p. 349–390.

The wave propagation in a canal becomes a complex study when it involves a singularity in resistance to flow. The paper deals with the influence of this singularity on wave celerity and with the variation of elevation created by the disturbance. For a large resistance or construction, the approximate celerity is $C = V \pm \sqrt{g(H+h)}/\sqrt{1+ghB/Hn^2R^{4/3}}$. The formula is checked by experimental results. A similar expression is

given for rapid change in bottom level. The problem of unsteady flow in a canal attached to the tunnel of a powerplant is studied, as is the influence on unsteady flow of a syphon in the middle of a canal.

678. Pan-Zhu, E., 1956, Raschet neustanovivshegosya dvizhenya v otkrytykh vodotokakh po metodu karakteristik [Computation of unsteady movement in surface channels by method of characteristics]: Akad. Nauk [U.S.S.R.] Trans., Div. Tech. Sci. (Izv.), no. 4, p. 42–57.

A simplified approach to the method of characteristics for integrating the partial differential equation of unsteady flow is given. From he equations of characteristics in differential form, the form of finite differences for all four variables (x, t, V, H) is introduced. An auxiliary function F is introduced, as function of Δx, Δt and ΔV. The depth H_x is expressed as a function of depths H_a and H_b (at points a and b) and of F, and nomograms for solving the relation, and for determining F, are given. The nomographical solution for many relationships is a feature of this approach. The wave-computation procedure is given by steps, with computation examples. The method was applied to regular channel, however, the possible application of the method for natural channels is discussed.

679. Ransford, G. D., 1956, The propagation and the reflection of abrupt translation waves in still water of steadily decreasing depth: Houille Blanche [France], no. 3, p. 406–414.

By analysis and employing the method of characteristics (according to Massau), the author concludes that the movement of a positive wave, having discontinuity or finite wave slope at the wave tip, at the point of departure, will always conclude by breaking in a channel having either horizontal or sloping bottom in the direction of movement. The negative wave tip with the same departure condition will flatten out indefinitely in the horizontal channel, but in the sloping channel the situation depends on the slope relationship of the wavefront and the bottom of the channel. For $\partial Z/\partial S$ (wavefront slope) $<i/2$, the wave steepens: for $\partial Z/\partial S=i/2$ the wave stays unchanged, and for $\partial Z/\partial S>i/2$ the wave flattens out, where i=bottom slope.

680. Rockwood, D. M., and Hildebrand, C. E., 1956, An electronic analog for multiple-stage reservoir type storage routing: U.S. Army Corps Engineers, Civil Works Inv., Proj. CW 171, Tech. Bull. 18, 27 Mar. p. 1–12.

A working model of an electronic analog for multiple-stage reservoir type storage routing has been built and successfully tested. It may be used for routing rainfall or snowmelt runoff or streamflow through basin, channel, or reservoir storage. The advantages of this analog over other types currently in use are: (1) Flexibility of operation—a variable number of stages can be coupled, the time of storage of any of the stages can be varied during the routing: and (2) Simplicity and low cost—the parts for the working model described herein are all standard radio components and cost less than $100 exclusive of the recording potentiometer.

[From author's summary.]

681. Rudinger, George, 1956, Boundary conditions in nonsteady flow: Internat. Convention Appl. Mech, 9th, Brussells, Proc., v. 3, p. 152–164 [1957].

Problems involving quasi-one-dimensional nonsteady flow in a duct require the knowledge of the conditions that govern the reflection of pres-

sure waves from various flow boundaries. Some of these boundary conditions are accurately known while others must be treated by means of suitable assumptions. A brief review of the present knowledge in this field is presented. For wave reflections from an open end of a duct, it has been customary to assume that the boundary conditions in nonsteady flow are the same as in steady flow. Actually, the steady-flow boundary conditions require some time to re-establish themselves after having been disturbed by an incident wave.

[Part of author's abstract.]

682. Schoenfeld, J. C., 1956, Getijberekening met de ARMAC [Tidal computation by the ARMAC]: Rijkswaterstaat Commun. [Den Haag, Netherlands], Nota CSD 56–9.

683. Semenido, V. I., 1956, Proverka raschetov neustanovivshegosiya dvizheniya v otkrytom rusle [Verification of computation of unsteady movement in open channel]: Gidrotekhnika i Melioratsiya [U.S.S.R.], no. 7, p. 51–52.

The celerity of surges is measured in a canal 7,000 meters long, about 16 feet deep, and about 60 to 70 square meters in cross section. The discharge is increased from 165 to 210 cubic meters per second, and then, after 16 minutes, is decreased to 165. Results obtained are compared with those of a grapho-analytical computation by V. A. Arkhangelskiy and with the celerity computed by the formula $C=V+\sqrt{gA/B}$. Good agreement is shown between measured and computed celerities.

684. Supino, G., 1956, Onde di ampiezza crescente su moto-base permanente [The waves with increasing height on the nonuniform steady baseflow]: Energia Elettrica [Italy], v. 33, no. 11, p. 1–7.

This paper is the continuation of earlier papers published in Energia Elettrica (1953, 1954) and treats analytically the case of the wave having increasing height, which is compared with results of experiments made in a laboratory canal. The wave movement of small waves is generally studied in comparison to basic nonuniform steady flow, when the heights of discharges propagate with the same difference of phase from infinity upstream to infinity downstream, or the other way around. This study eliminates first and third assumptions and continues the treatment given in 1953, it gives a new theoretical solution. Among other results that are verified experimentally is the conclusion that in the propagation of wave, a wave exists with an important characteristic, that of increasing amplitude, starting from value zero.

685. Szesztay, K., 1956, Some methodical problems of flood forecasting in drainage basins of great extension: Internat. Assoc. Sci. Hydrology, U.G.G.I., Symposia Darcy, Dijon [France], 1956, v. 2, pub. 14, p. 198–216.

This is a description of a method of forecasting stages that is based on water-level relationships. The selection of parameters is analyzed, and by a coaxial method of graphical correlation, the forecast graphs are developed.

686. Tsinger, V. N., 1956, Transformatsiya maksimalnykh raskhodov vodokhranilishchami [Transformation of maximum flood discharges by reservoirs]: Gosudarstveniy Gidrologicheskiy Inst. [U.S.S.R.] Trudy, no. 52 (106).

687. U.S. Soil Conservation Service, 1956, Flood routing through reservoirs: U.S. Soil Conserv. Service, Eng. Handb. Hydraulics, sec. 5, pt. 5.8, p. 1–14.

Two flood-routing methods based on the storage equation are formulated: (1) a graphical method, using storage-discharge curves for reservoirs and mass curves for inflow and outflow; (2) a semigraphical method, using the storage-discharge relation for reservoir inflow hydrograph, and the conversion-time interval, T, as time required for a flow as measured by one unit (say, 1 inch) of ordinate on the flow scale to accumulate to the same unit (1 inch) of storage on the storage scale.

688. Afanasev, A. I., 1957, Nekatorye voprosy ucheta transformatsii povodochnoy volny pri gidrologicheskikh prognozakh po zapasam vody v ruslovoy seti na primere r. Dunaya [Some problems of computation in flood routing for the hydrological forecasts by using the water storage in river channels (River Danube used as an example)]: Tsentralniy Inst. Prognozov [U.S.S.R.] Trudy, No. 59.

689. Befani, A. N., 1957, Voprosy teorii i rascheta stoka [Problems of theory and computation of runoff]: Meteorologiya i Gidrologiya [U.S.S.R.], no. 12, p. 6–15.

This paper commences with a general discussion of the water-balance equation, the genesis of water concentration, and rainfall-runoff relationships (given by formulas). Surface runoff is analyzed by means of the differential equation of flow on a sloping surface. The continuity equation of river flow, considering lateral inflow per unit of channel length, is given as $\partial Q/\partial x + \partial A/\partial t = q$.

Wave celerity is derived as a function of water velocity together with functions of coefficients representing form of longitudinal profiles, relative damping, and relative nonuniform side inflow. The formulas for flood peaks q_{max}, (yield) or Q_{max} are given for different practical cases, taking into account the coefficient of transformation by river channel storage. The formulas for spring runoff, underground discharge, seasonal and annual flows are also given.

690. Blackmore, W. E., 1957, The slope-template method for some hydraulic problems: Water Power [London], March, p. 83–88.

The solution starts with two functions, $X=f_1(t)$ and $Y=f_2(t)$, related as $dY/dt=f_3(X-Y)$. Given the curve representative of the variation X, the slope-template method allows the immediate plotting of the curve of Y. The template (a device consisting of a rectangular piece of transparent celluloid) is described. The above equation is used for mass curves of inflows and outflows, and for routing floods on the basis that the outflow discharge (slope on the mass curve of outflow) is related to the difference of accumulated volume of inflow and outflow. The template has a sloping line for Q, and a curve to determine the relation of Q and $W=P\Delta t-Q\Delta t$. If the hydrographs are used instead of mass curves, a curve of template is drawn giving the rate of increase of outflow against $(Q-P)$. Generally a family of template-drawn curves, one curve for each value of outflow, is used. The method of constructing the curves is given.

691. Bonnet, L., 1957, Contribution à l'étude des fleuves à marée. Probleme des discontinuites [Contribution to the study of tidal rivers. Problem of discontinuities]: Travaux Publics de Belgique Annales, Mem. [Belgium], no. 5, p. 5–18.

This study treats the effects on river tidal movement of marked changes occurring in river cross sections, due to widening or narrowing of the section or deepening of the channel. Simple formulas are developed for the effect of these discontinuities on tidal-river waves, on wave energy and amplitude. The results are verified, in a tidal-river model, for abrupt widening, narrowing, heightening, or deepening of channel. Good agreement is shown between results by formulas and results of experiments in a model.

692. Butler, S. S., 1957, Engineering Hydrology: Englewood Cliffs, N.J., Prentice-Hall, p. 203–212.

A brief treatment of mass curves and flood routing, which gives emphasis to the modified Puls method of flood routing.

693. Chertousov, M. D., 1957, Spetsialniy kurs [Hydraulics, special course, chapter 13 on unsteady water movement in open channels]: Gosenergoizdat [Moscow], p. 391–453,

This is an exposition on waves in channel; special emphasis is on the work done by Russian hydraulicians. The fundamental partial differential equations are derived and discussed; special attention is given to the integration of the equations in special cases. The celerity and discharge of waves are given. Waves of small amplitude, particularly those created by pondage in powerplant operation, are treated, and cases are discussed. Emphasis is on practical application.

694. Doodson, A. T., 1957, The analysis and prediction of tides in shallow water: Internat. Hydrographic Rev., May.

The considerations that have led to the development of practicable methods of overcoming the difficulties in predicting shallow water tides are explained. The harmonic method of analysis and prediction of tides used for deep water cannot be used for shallow water with any degree of accuracy. The mode of generation of the shallow water tides and the interrelations of the harmonic constituents are discussed. The problem of predicting the times and heights of high and low water is then discussed with only a minimum of mathematical explanation. Instructions for analysis and prediction are given which can be followed by computers who are little concerned with theory. These methods, which are evolved at the Liverpool Observatory and Tidal Institute, have been in use since 1926.

[Abstract from Bibliography on tidal hydraulics, Corps of Engineers, May 1959.]

695. Druml, F. U., and Lawler, E. A., 1957, Application of differential analyser and digital computers to hydraulic problems: Am. Soc. Civil Engineers Mtg., Jackson, Miss., 1957, p. 1–9.

The economic feasibility of the use of computers is briefly described. The use of UNIVAC I for solution of two partial differential equations for unsteady flow is discussed. Use of the Goodyear electronic differential analyser is described using Muskingum routing equations applied to determine movement of surges in sewers, and using the storage equation to determine pool elevations. The Burroughs E–101 computer is discussed with respect to use for unit hydrograph computations and in statistical studies.

696. Escoffier, F. F., 1957, Determining the coefficients for the Muskingum method directly from the physical characteristics of a uniform channel (and linearizing the friction term, and the similarity of the flood

wave and canonical heat equations): U.S. Army Corps of Engineers, Mil. Hydrology Bull. 10, Apps. B and C, p. 37–71.

The quantities K, X, C_2 for use in the conventional Muskingum flood-routing method are derived directly from the physical characteristics of a uniform channel. Certain sources of error in the conventional method are indicated and a modified method is then developed to eliminate them. It is shown that for a given stage there exists an ideal length of reach for use in the modified Muskingum method and it is suggested that this same ideal length will prove useful in the conventional method as well. The first portion of Appendix C shows how the friction term of the flood wave equation used in Appendix B can be converted to a linear form, thereby facilitating the process by which the flood wave equation can be converted to a form similar to the canonical heat equation. The second portion of Appendix C demonstrates how, by a simple change of variables, the flood wave equation can be converted to the classic canonical heat equation.

[Author's abstract.]

697. Frank, Joseph, 1957, Nichtstationaere Vorgaenge in den Zuleitungs-und Ableitungskanaelen von Wasserkraftwerken [Unsteady phenomena in the tailrace and headrace canals of water powerplants]: 2d. ed., Berlin, Springer, p. 1–333.

This is an extention of the work done collaboratively by this author and J. Schueller (first edition, 1938). This study elaborates on the method of characteristics used for the transformation of waves in narrow parts of channel and gives a more detailed analysis than the first study.

698. Haindl. K., 1957, Translachni vlny na nadkitickem proudeni [Translation waves in supercritical flow]: Vodni Hospodarstvi [Czechoslovakia], no. 5, p. 127–128.

This paper discusses the occurrence of secondary waves that propagate in the direction of slope of the channel, creating complications in a canal. The theory of these waves, results of observations made by specialists in the U.S.S.R., and measures taken to avoid occurrence of such waves are discussed.

699. Ippen, A. T., and Kulin, Gershon, 1957, The effect of boundary resistance on solitary waves: Houille Blanche [France], no. 3, p. 390–407.

This paper reports on the results of an experimental investigation of the attenuation of solitary wave amplitude conducted in the Hydrodynamics Laboratory of the Massachusetts Institute of Technology. The tests were in a lucite channel 32 ft. long and 16½ in. wide in which initial water depths ranged from approximately 0.2 ft to 0.4 ft. Some runs were made with the tank bottom artificially roughened with gravel of uniform diameter. Attenuation results are compared with available theory.

In another phase of the study, it was attempted to approach the damping problem by direct measurement of the transient boundary layer near the bottom. A special differential gage for measuring the low transient velocities was designed, and the preliminary results obtained with it are described.

700. Ippen, A. T., and Mitchell, M. M., 1957, The damping of the solitary wave from boundary shear measurements: Mass. Inst. Technology, Hydrodynamics Lab. Tech. Rept. 23, p. 1–50.

Due to the unsteady nature of the flow under the solitary wave, the instantaneous shear stresses cannot adequately be described by steady-state relations except at the point of zero acceleration under the wave crest.

In considering the entire wave, the effects of the unsteady motion cancel out so that an average resistance coefficient can be related to a wave Reynolds number. Correlation to the Blasius theory for steady laminar flow over a flat plat was obtained.

As for the attentuation runs, much higher damping developed from the rough bottom than from the smooth bottom. The rate of damping could be related systematically to the absolute roughness and the wave properties.

The unsteady nature of the flow causes inertial as well as viscous effects in the shear measurements, which are explained by virtual mass effects on the roughness particles. These local inertial effects cannot be separated from the local viscous effects, but cancel out when the entire wave is considered.

The average resistance coefficients obtained from the various waves for the two roughnesses fall mostly into the transition range of Reynolds numbers beyond which the resistance coefficients seem less dependent on viscosity.

[From authors' abstract.]

701. Ishihara, Yasuo, 1957, On the application of an electronic analog computer for flood routing to actual rivers: Japan Soc. Civil Engineers Trans. [Tokyo], no. 43, p. 43–47.

The principle of an electronic analog computer for flood routing, based upon Dr. Hayami's excellent theory, and its characters were already explained in the author's previous papers. In this paper, the methods of application of this computer to actual rivers with various figures, inflow from tributaries, outflow to distributaries, retardation pool for flood protection, etc., are discussed and it is found to be applicable to actual rivers, changing the constants contained in the computer. As some examples of application, the results of flood routing in the Kiso and Yodo Rivers are shown.

[Synopsis in English at the end of transactions.]

702. Iwasa, Y., 1957, Attenuation of solitary waves on a smooth bed: Am. Soc. Civil Engineers Trans., v. 124, p. 193–206.

The hydraulic characteristics of solitary waves have been considered in a preceding paper (1955). In this paper, further development of the study of solitary waves is treated. Especially considered is the mathematical analysis of the attentuation process of solitary waves. The theoretical results are compared to experimental data of Russell and Ippen.

It should also be noticed that the total kinetic energy minus the potential energy is twice the amount of the kinetic energy of the vertical motion, as Starr (1947) has already verified.

[Author's summary.]

703. Kalinin, G. P., 1957, Runoff calculations and forecast according to water storage in the river network and to its water inflow: Internat. Assoc. Sci. Hydrology, U.G.G.I., Mtg., Toronto, 1957, v. 3, pub. 45, p. 78–88.

For flood-forecast purposes, water stored in the channel network of a river is studied with respect to its effect upon runoff and particularly upon flood peaks. According to the author, the approximations made by use of simple formulas, are justified by the disadvantage of exact routing methods.

704. Kalinin, G. P., and Milyukov, P. I., 1957, O raschete neustanovivshegosya dvizheniya vody v otkrytykh ruslakh [On the computation of unsteady flow in open channels]: Meteorologiya i Gidrologiya [U.S.S.R.], no. 10, p. 10–18.

An approximate treatment of wave movement in river channels that neglects the inertia terms and assumes that the discharge Q_i, at a point in unsteady flow, is $Q_i = Q_0(1+\frac{1}{2}\Delta S/S_0)$, where Q_0=steady flow for the surface slope (S_0) and ΔS=additional slope ($S_i = S_0 + \Delta S$). When $\Delta S = 0.1S_0$, the error is 1 percent, and when $\Delta S = +2S_i$, the error is 6 percent. When the discharge is the same as in steady flow, the length L is found to be $L = Q_0/(2S_0\partial Q_0/\partial H)$. This length is called the "characteristic reach of river channel" and can be taken as constant, regardless of change of levels, that involves only small errors. Also, the storage-discharge relationship is approximately constant for the length L. The example of water powerplant at Ivankovo is analyzed with characteristic reaches. By assuming that storage-discharge relation is linear, the coefficient of proportionality is analyzed, and it is shown that the outflow must be an exponential function for constant inflow. Assuming constant inflow during time intervals Δt, it is possible to determine the approximate outflows. The author considers the accuracy of this approximation to be close to that of exact methods.

705. Kishi, Tsutomu, 1957, On the highest progressive wave in shallow water: Public Works Research Inst. Jour. Research [Tokyo], March, v. 2, Research paper 4, p. 93–97.

The highest progressive wave in shallow water has been one of the most important problems not only in the dynamical point of view but also in the engineering purpose.

The author successfully applies the Rayleigh method to calculate the momentum which is contained in a wave, and consequently to give the expression of the highest wave.

The analysis gives good agreement with Messrs. F. Suquet and A. Wallet's experiment.

Also some characteristics of the highest wave are discussed in this paper. [Author's summary.]

706. Kivisild, H. R., 1957, Hydraulic studies in estuaries: Conf. Coastal Engineering, 6th, Gainesville, Palm Beach, and Miami Beach, Fla., Procs., p. 562–572.

There is a region in estuaries where water velocities are far below critical and where sea-level variations greatly affect the hydraulic conditions, but where still-distinct channels exist. In this region the water levels are usually as much influenced by tides and meteorological conditions as by river discharges. Floods may arise from high discharges as well as from storm surges. In this study relations are presented where hydrological and meteorological factors are included. The affected area is treated as a system of channels with more or less unidirectional flow in each. Frequently the flow conditions vary considerably over the length or width of a sea or river arm. The determination of hydraulic parameters is, therefore, quite difficult. In this paper, methods for a rational estimate of parameters have been shown. Using these parameters, the influences of channel topography, river flow, and meteorology are considered in a system of equations. These equations are transposed to an applicable form for

integration by finite differences, which in dynamic cases could be carried out along characteristics.

[Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, May 1959.]

707. Kovalenko, E. P., 1957, Nekatorye sluchai rascheta pryamoy polozhitel'noy volny popuska [Some cases of computation of positive release waves]: Akad. Nauk [Minsk, U.S.S.R.], Inst. Energetiki Trudy [Inst. Energetics Trans.], v. 2, p. 165–183.

The author states that the methods of characteristics and of finite differences are difficult to apply, and that there is need for more simple computing procedures to apply to the practical problems of movement of surges along canals. Accordingly, he proposes an approximate method for the computation of a positive surge traveling along a dry natural channel of the regular form and having constant bottom slope. The discharge at the upstream end is increased from zero to discharge, Q. The celerity of the surge created will decrease and, by progressing, will approximate the velocity of steady movement for the discharge, Q. According to the author, this assumption is supported by experiments of Tifonov (1954). The length of the steep wavefront will increase with progression, and after some distance the form will be stabilized. The same results are obtained by the experiments of Volkov (1953), in a canal of 24.57 meters. Moots' work (1938) is criticized, and particularly his formula for the progress of wave. The author derives the relationship $Q_x=Q_0+C\Delta A_x$, where C=celerity, Q_0=initial discharge, ΔA_x=change of cross section by the increase of discharge, and Q_x=the discharge at the moment considered. The celerity $C=(Q_1-Q_0)/(A_1-A)$, where A_1=cross section for steady movement with Q_1 and A=cross section at the considered moment. Using two partial differential equations (neglecting the inertia term), the following formula is obtained:

$$dx=\frac{c^2RdA}{\left[S_0c^2r-\left(\frac{Q_0+C\Delta A_x}{A_0+\Delta A_x}\right)^2\right]B}$$

in which c and R are from Chezy's formula, and S_0=bottom slope. This formula is applied to different channel forms, and is compared with the experiments; good agreement is obtained. Examples of computations are shown.

708. Levi, I. I., 1957, Dinamika ruslovikh potokov [Dynamics of channel flow; on hydraulic regimes of channel flows]: Gos. Energeticheskoe Izdatel'stvo [Moscow, Leningrad], p. 15–34.

The author presents the fundamental features of unsteady flow and characteristics of channels. The method of characteristics, as elaborated by Khristianovich, is given.

709. Mayer, P. G., 1957, A study of roll waves and slug flows in inclined open channels: Cornell Univ., Ph. D. thesis.

710. Morikawa, G. K., 1957, Non-linear diffusion of flood waves in rivers: New York Univ., Commun. Pure and Appl. Math., v. 10, no. 2, p. 291–303.

This report presents a consideration of surface waves in which resistance plays an important but not completely dominant role. By use of the stretching transformations the asymptotic representation and the conse-

quent asymptotic solutions of the two basic partial differential equations are obtained.

711. Nougaro, Jean, and Barbe, A., 1957, Amortissement, réflexion et transmission des intumescences dans les canaux découverts [Absorption, reflection and transmission of steep waves in open canals]: Internat. Assoc. Hydraulic Research, 7th Convention, Lisbon, 1957, sec. D, p. D36, 1-21.

This paper first treats the influence of the roughness of the canal on loss of wave energy, determining the decrease of wave celerity in the two following cases: (1) high linear head losses, (2) local head loss. The decrease of the amplitude of the wave is defined by means of very simple relations, deduced from a graphical method improved by the author and applicable in the case of a canal having a horizontal apron, and also in the case of a canal having a certain slope. Tests have been made to confirm the theoretical studies for the different cases considered. The paper next treats the reflection and transmission of the waves in a canal where a change in section has occurred. Relations permitting determination of the fractions of the waves which are reflected and transmitted have been established; they are based on the fundamental theorems of hydraulics and on graphical methods. Tests made in the Toulouse Laboratory canals confirmed the solutions proposed.

[Adapted from authors' abstract.]

712. Ray, W. E., and Mondschein, H. F., 1957, A method of forecasting stages on flat rivers: Am. Geophys. Union Trans., v. 38, p. 698-707.

In the course of daily forecasting of river stages at selected points along the Illinois and Mississippi Rivers, it has been noticed that conventional mainstem forecasting procedures proved somewhat inadequate in cases where rivers were generally flat. Such methods usually involve multiple graphical correlation of two inflow with two outflow stage values, or Muskingum routing methods. In the study reported here, an entirely new approach to the problem on the Illinois River was developed. It involved the use of a downstream stage parameter as an index of slope and channel storage, and when used in conjunction with modifications of standard procedures for predicting discharge inflows, could more adequately reproduce the observed stage hydrograph at the upstream point. Results achieved using this new approach are demonstrated in two actual flood rises.

[Authors' abstract.]

713. Sandover, J. A., and Zienkiewicz, O. C., 1957, Experiments on surge waves: Water Power [London], v. 9, no. 11, p. 418-424.

A description of experiments made to ascertain whether the profiles of various types of surge waves, recorded in a model channel, correspond with those deduced analytically. The undular wavefront is studied. The experiments are described in detail. The dimensionless parameters are compared, taking into account the original depth, the maximum and minimum depth, length of the first undular wave, and surge celerity. Favre's and Lemoine's theoretical results for solitary wave and steep-front wave are compared with experimental results. Suggestions are given for the computation of surge celerity (by simple hydraulic jump formula) and for computation of the maximum height of undular wave (by simple solitary-wave expression).

714. Schoenfeld, J. C., 1957 Analogue methods for storm surge problems: Rijkswaterstaat Commun. [Den Haag, Netherlands], Nota CSD 57–5.

715. Stoker, J. J., 1957, Water Waves: New York, Inter-science Publishers, p. 1–567.

The book is a comprehensive account of the mathematical theory of wave motion in liquids having a free surface and subjected to gravitational and other forces. Applications to a wide variety of physical problems are given. As a thorough discussion of wave theory, the book treats types of physical and mathematical problems involved as well as methods used. Its four parts consider: (1) basic hydrodynamics theory, (2) approximate wave theory based on small wave amplitude, (3) waves in shallow water approximated by nonlinear theory, with amplitudes not necessarily small, and (4) exact theory, using the exact nonlinear free-surface conditions.

Special attention is given to the methods of finite differences and characteristics of flood waves. Dam-breach waves are analyzed mathematically in full extent.

716. Syrov, Yu. P., and Kovalenko, E. P., 1957, Neustanovivshesya rezhimy v byefakh malykh gidroelekrostantsiy [Unsteady regimes in the headwater and tailwater canals of small water powerplants]: Akad. Nauk [Minsk, U.S.S.R.], Inst. Energetiki Trudy [Inst. Energetics Trans.], v. 2, p. 148-164.

The propagation of surges along the headwater and tailwater canals of small water powerplants is registered at selected stations. It is concluded that Lagrange's formula, $C=V+\sqrt{gH_m}$, for small depths gives values greater than those measured. A correction coefficient α is introducted so that $C=V+\alpha\sqrt{gH_m}$. H_m=mean depth before the wave arrives, and V= mean water velocity, α ranging in the cases studied from approximately 0.38 to 0.50 for the positive surges, and 0.24 to 0.42 for the negative surges.

717. U.S. Army Corps of Engineers, 1957, Flow through a breached dam: U.S. Army Corps Engineers, Wash. Dist., Mil. Hydrology Bull. 9, June.

This bulletin describes the computation of outflow resulting from dam breach under the following conditions of breaching: (1) relatively small breaches of various shapes; and (2) Relatively large breaches of rectangular shape, in which frictional resistance of flow through the reservoir becomes an important factor.

718. U.S. Army Corps of Engineers, 1957, Artificial flood waves: U.S. Army Corps Engineers, Wash. Dist., Mil. Hydrology Bull. 10, June.

The bulletin provides a simplified and practical method for predicting the magnitude, progress, and other hydraulic characteristics of an artificial flood wave produced by dam destruction. This method attempts to improve upon the Muskingum method of flood routing and to produce results that are sufficiently accurate for military purposes.

719. U.S. Army Corps of Engineers, 1957, Regulation of stream flow for military purposes: U.S. Army Corps Engineers, Wash. Dist., Mil. Hydrology Bull. 11, June.

This bulletin presents computation methods of operation of outlet dams for controlled variation in river depth, width, and velocity at downstream points. Also included are methods for determining the magnitude, duration, and timing of reservoir releases to obtain the desired hydraulic effects at downstream points.

720. U.S. Soil Conserv. Service, 1957, Flood routing: U.S. Soil Conserv. Service, Eng. Handb., Hydrology, sec. 4, suppl. A, pt. 3, 17, p. 1–28.

In the subchapter on flood routing, three methods, based on the storage equation, are given: (1) storage-indication method, which uses two storage factors ($W/\Delta t - Q/2$) and ($W/\Delta t + Q/2$), and tabular integration; (2) Goodrich-Wisler method, which differs from the above method only in the plotting and the use of the working curves (storage factors); and (3) Wilson method, a graphical method that gives results equal to the above methods, when the storage factors, plotted on log paper, approximate a straight line, with slope as unity. Other methods for approximate routing are given.

721. Viti, Mario, 1957, Contributo allo stuido del moto vario nei canali [Contribution to the sudy of unsteady movement in channels]: Ingegnere [Italy], v. 31, no. 10, p. 903–914.

This paper treats the differential equations of unsteady flow by using two characteristics. Characteristics lines are given and discussed, and their advantages are stressed. The simplification introduced is discussed, and the integration of equations for characteristics is given. The numerical applications follow the theoretical analysis. Special attention is given in the paper to the treatment of varied flow by use of characteristics.

722. Yamada, H., 1957, On the highest solitary wave: Research Inst. Appl. Mech. Rept. [Japan], v. 5, no. 18, p. 53–67.

723. Zienkiewicz, O. C., and Sandover, J. A., 1957, The undular surge wave: Internat. Assoc. Hydraulic Research, 7th Gen. Mtg., Lisbon, 1957, Proc., v. 2, sec. D.

When sudden changes of flow occur in a channel, the well-known phenomenon of surge waves results. For a certain ratio of the average depths on either side of such a surge the profile resembles a breaking hydraulic jump. Similarity as in the case of such undular, positive waves moving in still water of constant depth is studied. Accurate measurements of profiles are taken throughout the undular range, enlarging and supplementing the original results of H. Favre. It is found that the height of the first wave crest in all tests closely approximates to the height of a solitary wave moving with the same velocity. The wavelength and the heights of the succeeding troughs and crests are considerably influenced by channel friction. A development of the variable flow equations originated by F. Serre allows a step-by-step computation to be made for the whole of the wave profiles. With a suitable choice of friction coefficient remarkable agreement between computed and measured profiles has been obtained. [Authors' abstract.]

724. Dressler, R. F., 1958, Unsteady non-linear waves in sloping channels: U.S. Natl. Bur. Standards Rept. 5768, p. 1–14.

It is shown in general that the exact solution to every non-degenerate unsteady water wave problem in a straight channel inclined at arbitrary slope, governed by the non-linear hydraulic equations, can be obtained in terms of the complete elliptic integral of the second kin, E. By means of a non-Newtonian reference frame, every such wave problem for a sloping channel can be replaced by an associated problem for a horizontal channel. For the latter, the partial differential equations become reducible and thus permit hydrograph inversion. The Riemann integration method for the resulting Euler-Poisson equation yields an auxiliary function for these hydraulic problems which is transformable into a Legendre function and

then into the elliptic integral. In particular, the procedure is applied to obtain the exact solution for the water wave in a sloping channel produced by sudden release of the triangular wedge of water (the reservoir) initially at rest behind a vertical wall. The behavior of the solution is exhibited for convenience in two level-line charts, and representative wave profiles and velocity distributions are presented.
[Author's abstract.]

725. Felkel, K., 1958, Die Berechnung der nicht-stationaeren Fliessbewegung des Wassers in offenen Gerinnen [Computation of unsteady flow in open channels]: Die Bautechnik [Germany], year 35, no. 6, p. 216–222.

An approximation of the computation of unsteady flow in a river reach that is made of the assumption that the discharge distribution along the reach can be represented as $Q_x = Q - B\Delta H \Delta L/\Delta t$, where Q_x is discharge, ΔL is distance from the outlet end, and Q is the discharge at the end. For $\Delta L = L$ (length of reach), $Q_x = P$. The method of computation based on this assumption is outlined. The movement of surges is analyzed, taking into account the effect of friction resistance.

726. Gregor, L., 1958, Modifikace Boussinesqovy rovnice nestacionarniho proudeni [The modification of Boussinesq's equation for unsteady flow]: Vodni Hospodarstvi [Czechoslovakia], v. 81, no. 3, p. 74–78.

Analysis of the equation for unsteady flow in open ehannels is given; the significance of critical flow is emphasized. Losses due to friction are considered to be nearly the same for unsteady flow as for steady flow. The Froude number is used as the criterion for the degree of wave deformation. It is shown that Forchheimer's solution for wave deformation can be derived from the continuity equation. Kleitz' solutions are discussed, and Boussinesq's equation for unsteady flow is extended.

727. Guerrini, Pietro, 1958, Sopra il calcolo delle onde di translazione nei canali prismatici [On the computation of translation waves in prismatic channels]: Energia Elettrica [Italy], v. 35, no. 2, p. 125–131.

This paper presents simplifications of and additions to the Craya method of using characteristics lines for computation of unsteady flow in channels. It is shown that characteristic curve lines may be replaced by straight lines. If the mean depth is $H_m = A/B$, and depth is H, then for every cross section (rectangular, parabolic, triangular) having constant dH/dH_m, characteristic curve lines can be reduced to straight lines. Polygonal cross sections, however, would require tedious integration, as is shown by the tabulation of results given.

728. Halek, V., 1958, Zkushenosti z resheni hydraulickych problemu methodou elektroanalogie [Experience with the solutions of hydraulic problems employing the method of electrical analogy]: Vodni Hospodarstvi [Czechoslovakia], no. 10, p. 297–305.

The author describes the electric analogy apparatus which has been designed at the Hydraulic Research Institute in Brno for purposes of research. The paper sums up the experience gained in measuring and preparing electrical models and indicates new ways of simplifying experimental techniques. Solutions of problems involving unsteady flows are also discussed.

729. Isaacon, E., Stoker, J. J., and Troesch, A., 1958, Numerical solution of flow problems in rivers: Am. Soc. Civil Engineers Proc., v. 84, no. HY 5, Paper 1810, p. 1–18.

The numerical method of integrating two basic partial differential equations by the finite-differences procedure is described. The computation is done by a digital computer, UNIVAC. Examples show that the numerical computation is flexible. By this procedure the method of characteristics is used to determine the time-difference range for Δt, when Δx is selected, and there is good analysis of the mesh of points (Δt, Δx) in the plane (t, x). Also, the method permits analysis of flow at river junctions (where backwater effects may pertain) and is therefore favored over flood routing methods based on the continuity equation alone. Comparison of flood waves computed on UNIVAC and observed flood waves gives good agreement in all cases when background data is sufficient and accurate.

730. Kalinin, G. P., and Milyukov, P. I., 1958, O raschete neustanovivshegosya dvizheniya vody po ruslam pri pomoshchi krivykh dobeganiya [On the computation of unsteady water flow along the channels by the use of reach-travel curves]: Meteorologiya i Gidrologiya [U.S.S.R.], no. 7, p. 18–25.

The flood-routing starts from the characteristic reach introduced by the authors (1957) with $L=(Q_0/S_0)\partial H/\partial Q_0$, computed from steady flow. Storage W is related to Q as $Q=W/t$, where t is constant, and is the same for all characteristic reaches. Using these simplifications and assumptions the storage equation is integrated, starting from the inflow discharge and developing the outflow discharge at the end of the reach. The reach-travel curve is called $F(t)=Q/P$, where P is inflow and Q is outflow of a group of reaches. The formula for Q for n-characteristic reaches is developed and $F(t)$ comptued, where the peak of $F(t)$ increases with an increase of number of reaches. The formulas for $F(t)$ are given, Poisson's function being used as a standard curve. This method is applied with good agreement to the data for unsteady flow in the Irankovo powerplant canal.

731. Kawabata, S., 1958, Study of tidal river. Graphical solution of unsteady flow and its application: Public Work Research Inst. Rept., Ministry of Construction [Japan], v. 99, March, p. 1–11.

732. Kohler, M. A., 1958, Mechanical analogs aid graphical flood routing: Am. Soc. Civil Engineers Proc., v. 84, no. HY 2, Paper 1585, p. 1–14.

A new graphical procedure of flood routing that is aided by the use of mechanic analogs is described.

Graphical techniques for routing directly on the plotted hydrograph charts have been discussed for (1) the case of simple linear storage-discharge function, (2) the case of the Muskingum equation, (3) the case of linear storage function in conjunction with constant lag (translation), and (4) the case of both lag and the storage factor varying as complex functions of flow in the reach. The fourth category represents what is believed to be one of the most flexible methods of routing yet devised. Amazingly reliable results have been obtained for reaches which are incompatible with storage functions of the Muskingum and other recognized methods of routing. The time required for procedure development compared favorably with any other known technique. Moreover, the graphical solutions illustrated are operationally rapid and reliable. The basic

features of mechanical analogs comparable to the graphical solutions described have also been discussed. An analog which will facilitate variable K and lag has now been constructed and is undergoing tests. [Author's summary and conclusions.]

733. Lawler, E. A., and Druml, F. U., 1958, Hydraulic problem solution on electronic computers: Am. Soc. Civil Engineers Proc., v. 84, no. WW I, paper 1515, p. 1–38.

This paper discusses the application of three types of electronic computers and demonstrates the wide range of problems readily solved by these modern tools. Tedious, cumbersome, conventional methods of solution are replaced by quicker and frequently more accurate methods. The use of computer Univac I for flood routing by finite difference method (New York Univ. studies); the use of Goodyear Electronic Differential Analyzer (1954) for flood routing by Muskingum method for surges in sewers and for fluctuations in pool elevations and others; and the use of computer Burroughs E–101 for unit-hydrograph computations and for statistical computations, are discussed. The conditions under which the computers are useful are analyzed at the beginning of paper.

734. Linsley, R. K., Kohler, M. A., and Paulhus, J. L. H., 1958, Hydrology for Engineers: New York, McGraw-Hill Book Co., p. 216–244.

This is a condensed exposition of methods of flood routing, and the major emphasis is on flood routing procedures which are based either on the storage equation (for both reservoirs and channels), or on stage relations. The routing aids are included.

735. Moklyak, V. I., 1958, K osnovam rascheta transformatsii pavodkov [On the basis of flood routing]: Meteorologiya i Gidrologiya [U.S.S.R.], no. 1, p. 37–40.

According to the author, one of the main causes of the attenuation of waves is the difference in travel required for the component parts of the initial discharge occurring in the starting cross section. This difference is due to the varying velocities at which the water travels in different filaments. The travel velocities from one cross section to another are introduced, and special travel curves are developed. This approach is applied to the results of experiments (Gil'denblat and others, 1948), and good agreement is shown with experimental observed values of attenuation. This study emphasizes the nonuniformity of velocity distributions in a cross section.

736. Nougaro, Jean, and Duffour, Pierre, 1958, Méthode graphique pour le calcul de la propagation des intumescences dans un canal de section quelconque [The graphical method for computation of wave propagation in a channel of any cross-sectional shape]: Acad. Sci. [Paris] Comptes rendus, v. 247, no. 12, p. 860–861.

The graphical method of computing wave propagation, as developed by Nougaro (1955, Comptes Rendus, v. 240, p. 1689) for rectangular channels, and applied with some modifications and approximations to trapezoidal channels, is here extended to channels of any cross-sectional shape, in which B (width of channel) assumes the function of area (or depth).

737. Nougaro, Jean, and Duffour, Pierre, 1958, Recherches experimentales sur les intumescences dans les canaux de section quelconque. Application de la methode graphique S(Q) pour le calcul de la propagation des

intumescences [Experimental research on the intumescences in channels with any cross section. Application of graphical method $S(Q)$ for the computation of propagation of intumescences]: Acad. sci. [Paris] Comptes rendus, v. 247, no. 25, p. 2292–2295.

The experiments of movement of surges in laboratory channels that are models of powerplant canals and galleries, are described. The sudden or linear changes of powerplant operations are studied, and the observed waves are compared with the waves computed by graphical constructions, the $H(Q)$ and $S(Q)$ methods as proposed by Nougaro (1955). The comparison shows good agreement of the two sets of results, thus justifying use of the proposed graphical method.

738. Pezzoli, G., 1958, Osservazioni sulle perturbazioni nei canali a pelo libero e spora alcuni aspetti particolari dei moti ondosi [Observations on the disturbances in canals with free water surface, and on some particular aspects of undular motions]: Energia Elettrica [Italy], no. 1, p. 1–8.

The "small wave" concept is considered unclear in earlier works and is defined more precisely by using three parameters: height of disturbance or small wave amplitude (h), water depth (H), and wavelength (L). The direction of propagation of small waves (being either in the downstream direction or in both directions) is shown to be no criterion for determination of the type of flow (whether subcritical or supercritical). For S_f=bottom slope, and S_c=slope, in the case of critical velocity, it is derived that small waves will not propagate upstream in moving water when $(S_f/S_c)>(L/2\pi H)$, which is valid for $(L/H)\leq 2.7$. For L/H in limits 2.7–25, and for $(L/H)>25$, the other conditions are given. The problem of genesis of uniform velocity of the transporting water, associated with any wave motion, is studied. The author contends that this generally neglected aspect of wave motion is of interest in revealing how periodic disturbances tend to propagate on particular profiles of steady flow, including uniform flow.

739. Proudman, J., 1958, On the series that represent tides and surges in an estuary: Jour. Fluid Mech. [Great Britain], v. 3, pt. 4, p. 411–417.

Paper is concerned with a progressive wave of general form in an infinitely long estuary of uniform cross section when there is a permanent current independent of the wave. The only approximation is the neglect of friction. Explicit formulas in the form of infinite series are found for the surface elevation and for the current. In the special case in which there is no permanent current and when the oscillation at the mouth of the estuary reduces to a single harmonic constitutent, the first five harmonic shallow water constituents at any place up the estuary are evaluated.
[Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, May 1959.]

740. Rockwood, D. M., 1958, Columbia Basin stream flow routing computer: Am. Soc. Civil Engineers Trans., v. 126, pt. 4, Paper 3119, p. 32–56.

A method for using digital computers for stream-flow routing in Columbia Basin is described. Water excesses of rainfall and snowmelt are routed by subbasins to synthesize streamflow, which in turn is routed through lakes and channels. A new routing technique is introduced by use of the digital computer. The storage equation is used and linear relation of storage and outflow discharge is applied, $T_s=W/Q=dW/dH)/(dQ/dH)$. Basic codes and routines, input data, derivation of basin storage coefficients

($T_s = KQ_t^{-0.2}$ for Columbia), derivation of channel storage coefficients and lake-routing coefficients are treated. The method provides the framework for completely automatic determination of the components of streamflow in the Columbia River Basin, and the method is feasible only with an electronic computer. (Discussion by Willard M. Snyder, D. M. Rockwood, and Author's closure.)

741. Scholer, H. A., 1958, Tides in rivers and coastal inlets: Inst. Engineers [Australia] Jour., v. 30, no. 4–5, p. 125–136.

The object of the paper is (a) to describe features of tides in rivers and coastal inlets that might be of interest to the engineer and the hydrographic surveyor and (b) to present formulae and methods of computation for predicting tidal behavior. In Section I, after the introduction, some general features of ocean tides are described. A method of determining representative ocean tides for the purpose of estimating tidal behavior in rivers and coastal inlets is then presented. A general mathematical discussion follows, and the equations of tidal flow, on which the paper is based, are derived. The basic equations are applied to three special cases of tidal flow, which serve to bring out salient features of tidal behavior in rivers and coastal inlets. General methods of evaluating friction coefficients from analyses of tidal observations are applied to the Clarence River. In Section 2 a general method of numerical computation of tides in rivers and coastal inlets is presented, and an outline given of its application to tides in a single channel, a channel with tributaries, and a branched channel. [Abstract from Bibliography on Tidal Hydraulics, Corps of Engineers, May 1959.]

742. Sinha, G., 1958, Mathematical computation of tidal propagation in estuarine rivers: Irrig. and Power [India], v. 15, no. 1, p. 85–99.

The question of improvement of tidal rivers is usually investigated with the help of hydraulic scale models, since direct mathematical treatment is not possible owing to the complicated structure of the mathematics of tidal mechanics. However, there are certain inherent limitations of tidal models which make their operation more difficult than models of nontidal rivers. This naturally affects the results obtained from the model experiments to some extent. There is, however, no other way of verifying these results. This has encouraged attempts at mathematical treatments with the help of simplified approximation of the channel contour, and methods of mathematical computation of the tidal propagation have been evolved in different countries, especially in the Netherlands where the sea and tidal action are of profound importance. Of the different methods of computation the iteration has been found to be particularly useful and has been applied successfully for computations of many rivers. The paper gives, in brief, an idea of the simplifications introduced, the different methods of solution and their applications. In particular, the solution by the iteration method and its practical application have been discussed. [Abstract from Bibliography on Tidal hydraulics, Corps of Engineers, May 1959.]

743. Volkov, I. M., 1958, Obzor rezultatov issledovaniy dvizheniya volny popuska po sukhomu ruslu [Survey of the results from the investigation of release-wave movement along a dry river bed]: Akad. Nauk [Kazakh. S.S.R.], Inst. Energetiki Trudy, v. 1, p. 3–18

This is a short review of most of the important works on unsteady flow in U.S.S.R., from about 1930–48. It is concluded that a small amount of attenuation of waves occurs in movement along the dry channel. The characteristics of progressing wave heads in dry channel (infiltration, eddies, air-water mixture, friction losses, excessive erosion, and sediment-transport capacity) are outlined and discussed; particular attention is directed toward torrents. Bernadski's (1933) formula for celerity of waves in dry channels is derived ($C=2\sqrt{gh}$) and analyzed. Some of Bernadski's assumptions are criticized. The use of the formula $C=2\sqrt{gh}$ is considered to be good, but due to the attenuation of wave head it is difficult to determine the height h. The results of Tikhonov's paper (1933) and Cherkasov's (1932) paper are discussed, and many objections made to the assumptions, results, and conclusions. The results of many papers dealing with unsteady flow in dry channels are analyzed, the objective being the solution of the problems of rivers used for floating logs. The status of the problem of wave movement in dry channels is given and is clearly differentiated from the problem of wave movement in full flowing channel. The need for more extensive systematic experiments is emphasized.

744. Yamada, Hikoji, 1958, Permanent gravity waves on water of uniform depth: Research Inst. Appl. Mech., Rept. [Japan], v. 6, no. 23, p. 127–139.

Levi-Cività's formulation (as an eigenvalue problem) of permanent water-wave problem, which deals with a field function, holomorphic in the unit circle and satisfying a certain boundary condition on the circumference, are extended to the periodic waves in a canal of uniform depth. Deep-water waves and solitary waves are two special cases. The application of an iterative procedure on this formulation supplied sufficiently accurate results. An example is given. Some discussions on mass transport and eigenvalue are also included.

[Author's summary.]

745. Yano, Katsumasa, 1958, On the characteristics of the flood stream: Kyoto Univ. [Japan], Disaster Prevention Research Inst., Ann. no. 2.

In this paper, flood flow in river channels are theoretically solved under the assumptions that the acceleration term in the equation of motion is negligible and the Froude number is approximately constant in the small reach of river channel. The theoretical solution is too complex to be applied to real rivers, so the method of the practical numerical calculation using the table of Gauss' error function is proposed.

In conclusion, the flattening of the flood becomes remarkable with the increase in slope and resistance of river bed, but it is almost independent of initial water depth. The velocity of propagation of flood peak becomes large with the increase in slope of river bed and initial water depth.

746. Yuan-Po, Kou, 1958, Investigation of storage effects of reservoirs subjected to superfloods: Iowa Univ. M. S. thesis.

A theoretical study to find out what effect reservoirs of various characteristics will have in decreasing the peak of increasingly greater superfloods. A quick, approximate method has been devised which takes the principal variables into account.

747. Hensen, W., 1958–59, Die Berechnung von Tidewellen in Tidefluessen [Computation of tidal waves in tidal rivers]: Die Kueste [Germany], no. 7, p. 1–19.

748. Brabant, C. E., 1959, Electronic computers in flood control studies: Mil. Engineer, v. 51, no. 342, p. 293–296.

The following computer topics are discussed: Programs, results obtained, improvements in application, preparation of a program, and time and ost advantages.

749. Charrueau, A., 1959, Sur les ondes de choc [On the shock waves]: Ponts et Chaussées [France], Annales no. 5, p. 497–549.

750. Chow, Ven Te, 1959, Open-Channel Hydraulics, Part V. Unsteady Flow: New York, McGraw-Hill Book Co., p. 523–621.

Part V of the book contains three chapters concerning the unsteady flow, each accompanied by long reference list (total 163 references). Chapter 18 treats gradually varied unsteady flow, considering continuity of unsteady flow, dynamic equation, monoclinal rising wave, dynamic equation for uniformly progressive flow (showing the wave profile of this flow), wave propagation, solution of the unsteady surface flow. Chapter 19 treats rapidly varied unsteady flow, considering uniformly progressive flow, the waving hydraulic jump, positive surges, negative surges, surges in power-plant and navigation canals, surges through channel transitions and channel junctions, and pulsating flow. Chapter 20 treats flood routing, considering the method of characteristics, the method of diffusion analogy, including principles and methods of hydrologic routing, and a simple hydrologic method of routing.

751. Darman, Z. I., 1959, O sposobakh predskazaniya srokov nastupleniya pika polovodya na ravninnykh rekakh [On the procedures of forecasting the times of peak discharge of floods on the lowland rivers]: Tsentralniy Inst. Prognozov Trudy [Central Inst. for Forecast, Trans.] [U.S.S.R.], fasc. 82, p. 64–82.

For the purpose of forecasting the time of occurrence of flood peaks on small rivers due to snowmelt, a study is made of the travel time of peaks of small rivers, where time of concentration of flood flow generally varies with the magnitude of the flood. For some rivers, the duration of peak period is found to be related to the magnitude of the peak, a decrease in period length being shown for an increase of peak discharge.

752. Dooge, J. C. I., 1959, A general theory of the unit hydrograph: Am. Geophys. Union Trans., v. 64, p. 241–256.

On the basic assumption that the reservoir (storage) action in a catchment can be separated from translation of waves, the general equation of the unit hydrograph is given in function of the main parameters. The derivation is analytical, and existing solutions are discussed for different shapes of catchments. The general solution and the errors of approximation are given.

753. Dronkers, J. K., and Schoenfeld, J. C., 1959, Tidal computations in shallow water: Rijkswaterstaat Commun. [Den Haag, Netherlands], no. 1, p. 4–60.

754. Eagleson, P. S., 1959, The damping of oscillatory waves by laminar boundary layers: U.S. Beach Erosion Board Tech. Memo. 117, p. 1–38.

The report presents the results of an analytical and experimental investigation of the shearing stresses exerted on a smooth bottom by the passage of oscillatory water waves. Average resistance coefficients and damping coefficients were derived in terms of the pertinent physical properties of the waves, using existing small amplitude wave theory and assuming nonseparating flow in a laminar boundary layer. The measured bottom shearing stresses greatly exceeded those predicted by theory for the range of waves tested.

755. Einstein, H. A., and Harder, J. A., 1959, Electric analog model of a tidal estuary: Am. Soc. Civil Engineers Trans., v. 126, pt. 4, Paper 3277, p. 855–869.

Five hundred miles of channel in the Delta Region, California, which are subject to sea water encroachment, may be protected by salinity barriers and a master levee system. An electric analog model developed at the University of California, Berkeley, Calif., predicts tidal amplitude and flows resulting from such modifications to the hydraulic system.
[Authors' synopsis.]

756. Felkel, K., 1959, Beitrag zur berechnung von schwall und sunk [Contribution to the computation of the positive and negative bore]: Die Wasserwirtschaft [Germany], no. 9, p. 242–246.

757. Guyot, M. Th., Nougaro, Jean, and Thirriot, Claude, 1959, Méthode de calcul numérique de la variation de niveau consécutif au passage d'une intumescence [Method for numerical computation of consecutive variation of elevation during the passage of an intumescence]: Acad. sci. [Paris] Comptes rendus, v. 248, no. 21, p. 2950–2952.

A method of numerical computation for the elevation or depression of consecutive water levels is presented for the passage of an intumescence. This method is a numerical transposition of the graphical method proposed by one of the authors (J. Nougaro, 1955, the Hague).
[Translated synopsis from the paper.]

758. Guyot, M. Th., Nougaro, Jean, and Thirriot, Claude, 1959, Calcul numérique des régimes transistoires dans les canaux découverts [Numerical computation of transient regimes in open channels]: Acad. sci. [Paris] Comptes rendus, v. 249, p. 1858–1860.

The method of numerical computation already described (Comptes rendus, no. 248, p. 2950) has been applied to diverse laboratory experiments, and two examples are given: (1) the sudden stop of flow at the downstream end of a horizontal section having rectangular cross section; (2) the sudden stop of flow at the downstream end of a horizontal canal having trapezoidal cross section.

759. Hayashi, T., and Nougaro, Jean, 1959, Sur la similitude des régimes non permanents dans les canaux découverts [On the similitude of unsteady regimes in open channels]: Acad. sci. [Paris] Comptes rendus, v. 249, no. 12, p. 1028–1030.

Different scales have been used for the similitude of nonpermanent regimes in open channels. The method proposed is being justified by the comparison of theoretical and experimental results, these last being derived by the similitude of laboratory experiments.

760. Hill, P. G., and Stenning, A. H., 1959, Laminar boundary layers in oscillatory flow: Am. Soc. Mech. Engineers, Paper 59–A–265, p. 1–17.

761. Holsters, H., 1959, Stabilité et convergence dans les calculs numériques du mouvement non permanent dans les rivièrés par des "méthodes

pas-à-pas" [Stability and convergence in the numerical computations of unsteady movement in rivers by the step method]: C.E.R.E.S. [Belgium], v. 10, p. 389–450.

762. Ishihara, Yasuo, 1959, On an electronic analog computer for runoff: Japan Soc. Civil Engineers Trans., no. 60, January.

The fundamental equation is derived in diffusion type including convection by introducing the longitudinal mixing process of flows as in flood routing presented by Hayami, and by considering the areal fluctuations of rainfalls. The equation is two dimensional and nonlinear.

Integrating the above equation along contour lines of catchment basin and after some approximations, the modified equation, which is one-dimensional but nonlinear, is derived. The electric circuit of the direct analog computer, therefore, is virtually equivalent to the one for flood routing.

Some examples of runoff estimations in the Yura River Valley, which is the model drainage basin having about 350 sq km are conclusively described. The results by this computer method indicate a good coincidence with the actual hydrographs in this basin.

763. Kalinin, G. P., and Milyukov, P. I., 1959, Prognoz i raschet uravney vody nizhe GES na osnove vodomernikh nablyudeniy [The forecast and computation of water levels downstream from powerplants, on the basis of the stage observations]: Meteorologiya i Gidrologiya [U.S.S.R.], no. 8, p. 13–19.

A brief discussion of forecasting methods based on stage observations is given. An analytical method is presented, based on a procedure employing a short time interval Δt, during which it can be assumed that travel time $\tau = dW/dQ$ is constant (W=storage of the reach, Q=outflow discharge of the reach) and that there is a linear change of inflow (q) along the reach. For $\Delta t < \tau$ the continuity equation gives $\Delta Q = (q_m - Q)K_1$, in which $K_1 = 1 - \exp(-\Delta t/\tau)$, q_m=mean q. K_1 is to be determined: (1) by $W = f(Q)$, which is determined from channel morphology; (2) from the change of discharge in the upstream and downstream gage stations as $K_1 = \Delta Q/(q_m - Q)$; (3) by the travel time of individual flood wave phases; (4) by an empirical approach. These various ways are discussed, and practical application is made.

764. Kunstatsky, J., 1959, Pohyb translacni vlny v prazdnem koryte [Movement of a translation wave in dry bed]: Vodohospodarsky Casopis [Czechoslovakia], v. 8, no. 4, p. 287–299.

765. Laboratoire National d'Hydraulique, 1959, Mouvement nonpermanent dans les canaux à section prismatique et variable quelconque [Unsteady movement in channels with prismatic or any variable cross section]: Lab. Natl. Hydraulique, Châtou [France], Rept. T218B.

766. Laboratoire National d'Hydraulique, 1959, Intumescence consécutive à une variation brusque de débit [Intumescence following a sudden discharge change]: Lab. Natl. Hydraulique, Châtou [France], Rept. 1, T220B.

767. Laurenson, E. M., 1959, Storage analysis and flood routing in long river reaches: Jour. Geophys. Research, v. 64, no. 12, p. 2423–2433.

Numerous flood-routing studies have indicated that the maximum length of reach through which a flood can be routed in a single step is such that

time of travel through the reach does not exceed about half of the period of rise of the inflow flood. This paper presents a method of analyzing the storage characteristics of reaches considerably longer than this by arbitrarily inserting a number of hydrographs between the inflow and outflow hydrographs in such a way as to produce single-valued storage-discharge relations for the shorter reaches so formed. These storage-discharge relations are then used in conjunction with a graphical flood-routing procedure to route the inflow successively through the several short reaches and so reproduce the outflow hydrograph.

[Abstract from Journal of Geophysical Research, December 1959.]

768. Liggett, J. A., 1959, Unsteady open channel flow with lateral inflow: Stanford Univ. Tech. Rept. 2, p. 1–73.

769. Ludewig, Dietrich, 1959, Rechenschieber zur Berechnung des Seerueckhaltes [Slide rule for the computation of storage in reservoirs]: Wasserwirtschaft-Wassertechnik [Germany], no. 3, p. 100–103.

Based on the storage equation and the use of time intervals Δt, the slide rule is designed to compute the storage in the movement of a flood wave through the reservoir. Scales on the fixed part of the slide rule are used thus: the lower scale for P (inflow) and Q (outflow), and the upper scale for H_1 and H_2 (the heights at the beginning and the end of Δt); scales on the moving part are used thus: the lower scale for $W/\Delta t$ and the upper scale for H_1 and H_2. In this first form the slide rule is used for two procedures: when different openings of an outlet under pressure are operated and when both the spillway and bottom outlet are operated. In a second form the slide rule is used for fixed outlet shapes (without moving gates or valves), and the Δt is constant.

770. Mayer, P. G., 1959, Roll waves and slug flows in inclined open channels: Am. Soc. Civil Engineers Trans., Paper 3158, p. 505–564 [1961].

Roll waves and slug flows are established as two distinctly different wave patterns and are phenomenologically studied in an inclined open channel. The basic characteristics of flow are expressed in terms of pertinent physical properties. A theory is presented and reference is made to similar phenomena in allied fields in which mathematical analyses exist. It is hoped that this study will extend the present knowledge (1961) of unsteady phenomena in open channel flow. (Discussions by F. F. Escoffier, R. H. Taylor, J. F. Kennedy; Tojiro Ishihara, Yuichi Iwagaki, and Yoshiaki Iwasa; and author's closure.)

[Author's abstract.]

771. Mkhitaryan, A. M., 1959, O valnakh na bystrotoke [On the waves in a steep canal]: Akad. Nauk [U.S.S.R.], Izv., Odelenie Tekhnicheskikh Nauk, Energetika i Avtomatika, no. 1, p. 90–99.

Waves created in steep channels are analyzed. The fundamental equations are given, as are the conditions for the creation of wavetrain, the solution of fundamental equations, and analysis of results. The celerity of waves is derived, examples are given, and conclusions are drawn.

772. Nash, J. E., 1959, A note on the Muskingum flood-routing method: Jour. Geophys. Research, v. 64, no. 8, p. 1053–1056.

An exact method of solution of the flood-routing equation, when the storage is a linear function of weighted inflow and outflow, is developed. This operation is shown to be equivalent to routing a multiple of the inflow

through reservoir storage and subtracting the excess inflow. Modified coefficients for the Muskingum equation are developed which do not depend on the routing interval being small relative to K.
[Author's abstract.]

773. Nechaeva, N. S., 1959, Naibolee rasprostranennye v S. SH. A. methody rascheta transformatsii voln povodkov pri peremeshchenii po rechnomu rusle [The methods most currently used in the U.S.A. for the flood routing along rivers]: Tsentralniy Inst. Prognozov [U.S.S.R.], Trudy (Proc.), no. 94, p. 79–91.

774. Pavlov, G. G., 1959, Opredelenie vysoty podpornoy volny v derivatsii GES pri nalichii sliva [Determination of the height of a surge wave in the canal of a water powerplant, equipped with spillway]: Gidrotekhnicheskoe Stroitel'stvo [U.S.S.R.], no. 4, p. 43–45.

A graphical method is geven for the computation of wave height in the case of a canal having a spillway, where a decrease in powerplant discharge creates the spill of water and a small wave traveling upstream. From spillway rating and the channel-wave characteristics $y=f(Q)$, where y=wave height, and Q=change of discharge passing under the wave, the intersection of curves gives the wave height and Q.

775. Poggi, B., 1959, Correnti stratificate. Onde di traslazione in alvei prismatice [The stratified flows. The translation waves in prismatic channels]: Energia Elettrica [Italy], no. 8, p. 685–691.

776. Rubbert, F. K., 1959, Die Tiberechnung als Problem der numerischen Analysis [Tidal computation as a problem of numerical analysis, a report of the principles and methods]: Mitteilungsblatt der Bundes Anstalt fuer Wasserbau [Karlsruhe, Germany], v. 12, p. 29–51.

777. Sapozhnikov, V. I., 1959, Ob ispol'zovanii krivykh dobeganiya vody dlya prognoza stoka [The use of curves of water travel time for the flow forecast]: Tsentralniy Inst. Prognozov [U.S.S.R.] Trudy (Proc.), no. 84, p. 54–64.

The curves of water-concentration, or water-travel, time from a part of a river basin to the gaging station are used as a basis for flow forecast. The study is analytical, emphasizing the problems usually associated with the synthetic hydrographs and synthetic unit hydrographs.

778. Schnoor, E., 1959, Anwendung des "Differenzenverfahrens" bei der Tidewellenberechnung in den von den gezeiten beeinflukten Fluessen [Application of "finite-differences method" in tidal wave computation in the tidal rivers]: Der Bauingenieur (Germany), v. 34, no. 6, p. 231–240. Discussion of this paper by D. Rose, Bauingenieur [Germany], v. 35, no. 8, p. 327.

The finite-differences method as applied to the two De Saint-Venant partial differential equations is given and discussed. Once Δx is selected, the selection of Δt is limited by giving the limit value for $\Delta t/\Delta x$. The procedure and the comparison with tidal model experiments are discussed.

779. Schoenfeld, J. C., and Verhagen, C. M., 1959, Development of the tidal analogue technique in Holland: Internat. Days for Analog Computations, Strasburg [France], Sept. 1–6, 1958, p. 376–380.

780. Scholer, H. A., and Germanis, E., 1959, Unsteady flow in rivers and artificial canals: Inst. Engineers, Civil Engineering Trans. [Australia], v. CE I, no. 1, p. 27–37.

This paper is concerned with unsteady flows in rivers and artificial canals produced by disturbances of the order of magnitude of floods and tides. With such disturbances the vertical accelerations of the water particles are negligible compared with the horizontal accelerations.

The object of this paper is to demonstrate a new approach to problems of unsteady flow in rivers and artificial canals. Cases of unsteady flow are described. General formulas, based on the method of characteristic lines, are developed from the point of view of programming for an electronic digital computer. A specific problem is selected and the general scheme of programming this problem for the computer is described.

[From authors' summary and introduction.]

781. Sekerz-Zen'kovich, T. Ja., 1959, Propagation of a free tidal wave in a canal of variable depth: Akad. Nauk [U.S.S.R.], Morskoy Gidrofisicheskiy Inst. Trudy, v. 18, p. 85–93.

782. Thirriot, Claude, 1959, Sur les Phénomènes de propagation d'une onde de choc dans une galerie de fuite d'usine hydroelectrique souterraine. [On the phenomena of shock-wave propagation in a tailrace gallery of an underground water power station]: Acad. sci. [Paris] Comptes rendus, v. 249, p. 2716–2718.

The case of a positive intumescence moving in a gallery, when the bore partly touches the ceiling of the gallery, is analyzed analytically by neglecting the effect of the air on the water mass.

783. Tonini, Dino, 1959, Su di una generalizzazione della equazione di continuita nel campo idrologico [A generalization of the continuity equation in hydrology]: Italian Natl. Hydraulics Conf. Rept., p. 1–17 (manuscript presented to conference).

The continuity equation

$$\partial Q/\partial x+\partial A/\partial t=0$$

is analyzed, and some characteristics of specific volumes and times are introduced and discussed. Integration of the above storage or continuity equation is given for specific inflow hydrographs, and the resulting outflow hydrographs are developed, taking into account the type of storage-discharge relationship involved.

784. U.S. Army Corps of Engineers, 1959, Design memorandum no. 8, Columbia Lock and Dam, Control and regulation of Walter F. George powerhouse inflows analyzed by means of Univac I electronic computer; Appendix, Univac I computer program for routing flows through Columbia Reservoirs; Part I, Program setup, code and operating instructions;Part II, Flow Charts: U.S. Army Corps Engineers, Mobile Dist., December.

This is a study outlining the basic regulation plan for the Columbia pool under normal operating conditions. The study includes an analysis of the effects of these inflows on stages and mean velocities at regular intervals along the entire length of the pool. The analysis is based on the results of wave-routing computations performed by means of a UNIVAC I elec-

tronic computer. Memorandum No. 8 presents the UNIVAC program in detail, including the basic equations, flow charts, annotated code, and operating instructions.

785. Uspenskiy, P. N., 1959, Wave propagation in a channel subject to rotational motion: Akad. Nauk [U.S.S.R.], Morskoy Gidrofisicheskiy Inst. Trudy, v. 15, p. 17–33.

786. Vapnyar, D. U., 1959, Vliyanie treniya na dvizhenie svobodnoy prilivnoy volny v kanale postoyannogo secheniya [Effect of resistance on the movement of tidal wave in a channel of constant cross section]: Meteorologiya i Gidrologiya [U.S.S.R.], no. 7, p. 21–25.

The form of a wave of type $y=y_0 \cos \alpha t$ that is propagated in a narrow channel is given as an equation that is the product of exponential and cosine functions. The celerity is given as $C=\sqrt{2gH}/(\sqrt{1+\mu}+1)$, where $\mu=k/\alpha$, in which k=coefficient of friction. The friction has negligible influence on C for $H<100$ m. The friction has greater influence on the short wave than on the long wave.

787. Whittington, R. B., 1959, Reservoir flood influx problems: Inst. Water Engineers Jour. [Great Britain], v. 13, no. 7, p. 628–632.

788. Yevdjevich, V. M., 1959, Computation of the outflow from a breached dam: U.S. Natl. Bur. Standards Rept. 6473, p. 1–27.

The openings of breached dams are classified as small, medium and large openings. For the first two, the procedures are given for computation of the outflow hydrographs after the breach, using Savage River Reservoir, Maryland, as an example.

789. Yevdjevich, V. M., 1959, Analytical integration of the differential equation for water storage: U.S. Natl. Bur. of Standards Jour. Research, B. Math. and Math. Phys., 63 B, no. 1, p. 43–52.

The integration of the storage differential equation at present is usually done mostly by graphical or numerical procedures. An approach to the analytical integration of that equation is the subject of this paper. A new method of fitting the given background curves by mathematically tractable expressions is introduced. The storage-outflow discharge relation is expressed in the form of a power function. A general differential equation for water storage $y'+cPy^2-cy^k=o$ is derived, with c and k constant for the given reservoir, outflow shape and type of flow, and P being the inflow hydrograph. The analytical solutions of this equation for $P=oP=$constant, and certain $P=f(t)$ are given for the integrable cases [tables 1 to 3, eqs. (12) to (29)]. The application of the results obtained is discussed at the end of the paper.

[Author's abstract.]

790. Yevdjevich, V. M., 1959, Effect of sudden water release on the reservoir free-outflow hydrograph: U.S. Natl. Bur. Standards, Jour. Research, B. Math. and Math. Phys., 63 B, no. 2, p. 117–129.

The free-outflow hydrograph is studied in the case of a sudden water release from a reservoir. The outflow hydrograph is called free when it is not affected by the tailwater levels. Both the effect of the steep negative wave, created by sudden water release, and the effect of the flow resistance are analyzed through the use of a fictitious inflow hydrograph. The water accelerated by the steep negative wave movement, whether or not the flow resistance is taken into account, gives this fictitious hydrograph.

The procedures are given for the computation of the wave celerities and the new water velocities along the reservoir, for the computation of wave-front heights and the maximum outflow discharge, and for the determination of the fictitious and total inflow hydrograph. The examples are given for the pyramidal reservoirs of rectangular and parabolic cross sections. A general procedure for the determination of the reservoir free-outflow hydrograph in an approximate form is derived. The example of the Savage River Reservoir (Maryland) is given.

[Author's abstract.]

791. Yuy, Vi-Chzhun', 1959, Krivaya ruslovogo dobeganiya i vliyanie na nee izmeneniya chisla kharakternykh uchastkov [The curve of channel travel time, and the influence on it by the change of number of characteristic reaches]: Meteorlogiya i Gidrologiya [U.S.S.R.], no. 11, p. 27–34.

The author starts from the Kalinin and Milyukov paper (1958), which introduces the curve of travel time and the length of characteristic reach. This length is small for small rivers. After discussing the properties of the approach by Kalinin and Malyukov, the author proves that for the interval of time Δt it is permissible to use average discharges in computations. Δt is used in functional relationship to discharge change in the given reach. The analysis of influence of the number of characteristic reaches on the curve of travel time is given in tables, thus representing (as a parameter) the number of computational time-units required. Results of the study have been applied in some sections of the Amur River.

792. Zheleznyak, I. A., 1959, Transformation of floods by a system of reservoirs: Hydrologic Convention, 3d, [U.S.S.R.], v. 4: Section for the water resources development, Leningrad, p. 107–114.

793. Honda, M., 1959–60, A theoretical investigation of the interaction between shock waves and boundary layers: Tohoku Univ. [Japan], Inst. High Speed Mech. Repts., v. 11, p. 23–53.

794. Annemueller, H., 1960, Der Einfluss des Turbinenschnellschlusses auf die Schiffahrt in Kraftwasserstrassen. [The effect of rapid shutoffs of turbines on the navigation along canals of water power stations]: Der Bauingenieur [Germany], no. 9, p. 338–342.

For a sudden shutoff of turbines, the height of the surge is

$$h=(V^2+V\sqrt{V^2+4gH})/2g$$

and the celerity of the surge is

$$C=-\frac{V}{2}+\sqrt{g\left(4+\frac{h}{2}\right)+\left(\frac{V}{2}\right)^2}.$$

795. Ball, F. K., 1960, Finite tidal waves propagated without change of shape: Jour. Fluid Mech. [Great Britain], v. 9, pt. 4, p. 506–512.

Coriolis terms are introduced into the equations governing the motion of a finite tidal wave. Various types of solution are found for waves which travel without change of shape, and which are periodic with sharp crests and broad troughs. The classical result that such waves cannot be propagated without change of shape is therefore untrue in these circumstances.

[Author's abstract.]

796. Bezanov, K. A., 1960, K teorii difraktsii Udarnykh voln. [Contribution to the theory of diffraction of shock waves]: Prikladnaya Matematika i Mekhanika [U.S.S.R.], v. 24, no. 4, p. 718–722.

This paper investigates the diffraction of impacting waves at the straight bank, which has a small angle in comparison with the direction of movement of the front of impact wave.

797. Carter, R. W., and Godfrey, R. G., 1960, Storage and Flood-Routing: U.S. Geol. Survey Water-Supply Paper 1543–B, 104 p.

The basic equations used in flood routing are developed from the law of continuity. In each method the assumptions are discussed to enable the user to select an appropriate technique.

In the stage-storage method the storage is related to the mean gage height in the reach under consideration. In the discharge-storage method storage is determined from weighted values of inflow and outflow discharge. In the reservoir-storage method storage is considered as a function of outflow discharge alone.

A detailed example is given for each method to illustrate that particular technique.

[Authors' abstract.]

798. Gessner, Walter, 1960, Schwall, Sunk and Wassersprung. Berechnung auf der Grundlage dimensionloser Kenngroessen. [The positive and negative bores, and hydraulic jump. Computation with dimensionless values]: Der Bauingenieur [Germany], no. 8, p. 286–291.

The formulas for surges are derived for a prismatic channel and a horizontal bottom, and for the conditions when: the front of surge does not change by progressing; the friction resistance is replaced; the discharge is changed suddenly; and the velocities are uniformly distributed in a cross section. The formulas, tables, and graphs for the computation of characteristic surges are expressed in dimensionless values ($\xi = \Delta A/A$) and in Froude number.

779. Goroshnikov, E. A., 1960, Transformatsiya makjimal nykh urovney vesennego polovodya v ustevom uchastke Severnoy Dviny [Transformation of maximum level of spring flood in the estuarine portion of the North Dvina]: Meteorologiya i Gidrologiya [U.S.S.R.], no. 8, p. 38–41.

The attenuation of peak levels of floods is studied along 140 km long mouth reach of North Dvina River. An exponential function is fitted to the relation of peak level and position x of cross sections along this reach. From the error of a sine type function of x, the other fitting is a sum of exponential and sine functions, and the third order fitting adds also a third linear function.

800. Guyot, M. Th., Nougaro, Jean, and Thirriot, Claude, 1960, Méthode de calcul numérique des phénomènes transitoires dans un canal ouvert [Method of numerical computation of transient phenomena in an open channel]: Acad. sci. [Paris] Comptes rendus, v. 250, p. 55–57.

This paper presents the principle and the realization of a more rigorous method for numerical computation of level variation due to the passage of an intumescence than was presented in two preceding papers by the same authors (Comptes rendus, v. 248, p. 2950 and v. 249, p. 1858; see items 757 and 758). The flow chart for the digital computer is given.

801. Guyot, M. Th., Nougaro, Jean, and Thirriot, Claude, 1960, Étude numérique des régimés transitoires dans les canaux [Numerical study of transient regimes in channels]: Houille Blanche [France], no. B., p. 814–832.

Numerical calculation on an IBM 650 ordinator of the propagation of solitary waves in free flow canals, considering transient conditions. The three assumptions used are:

(1) Breakdown of the movement into successive steps;

(2) Negligible vertical accelerations;

(3) Small wave heights above the initial water level.

Starting from Saint Venant's equations continuity and dynamic equations and with the above assumptions, the two following methods are developed:

(a) Numerical transposition of a graphical method put forward by M. Nougaro at the 6th General Meeting of the IAHR (Paper D–5, the Hague, 1955);

(b) A method directly based on the theory of characteristics which though more complex, is also more rigorous.

[Abstract from Houille Blanche.]

802. Hampel, R., 1960, Bruchversuch an einer Bogensperre der Wildbachverbauung [Dam breach experiment on the arch dam for sediment control]: Oesterreichische Wasserwirtschaft [Austria], nos. 8 and 9, p. 187–193.

803. Kalinin, G. P., and Levin, A. G., 1960, Ispolzovanie electronnoy modeliruyushchey ustanovki dlya prognoza dozhdevikh povodkov [The use of electronic model device for the forecast of rainfall floods]: Meteorologiya i Gidrologiya [U.S.S.R.], no. 12.

804. Kalinin, G. P., and Milyukov, P. I., 1960, Raschet uravney vody v nizhnikh byefakh GES [Computation of Water levels in tail-race canals of water power plants]: Meteorologiya i Gidrologiya [U.S.S.R.], no. 2, p. 19–23.

The computation of water levels below a hydroelectric powerplant is carried out by using the cross section of one half of characteristic river reach, or

$$l=(Q/2S)(dH/dQ),$$

and the flow rating curve $Q=f(H)$. Good agreement between the computed and observed level fluctuation at that cross section is shown.

805. Kalinin, G. P., Milyukov, P. I., and Nechayeva, N. S., 1960, Prostaya elektronnaya modeliruyushchaya ustanovka dlya prognoza pavodkov [The simple electronic model equipment for the flood forecast]: Meteorologiya i Gidrologiya [U.S.S.R.], no. 8, p. 20–28.

The basic formulas and principles underlining the electric analog integrating device are given. The device is described and the electric schematics are given. Discussion of application of device ends the paper.

806. Klemens, Vit, 1960, Zjednodusene reseni transformace povodnove vlyn [Simplified solution of flood wave routings in reservoirs]: Vodohospodarsky Casopis [Czeckoslovakia], v. 8, no. 4, p. 317–326.

807. Laitone, E. V., 1960, The second approximation to cnoidal and solitary waves: Jour. Fluid Mech. [Great Britain], v. 9, pt. 3, p. 430–444.

The expansion method introduced by Friedrichs (1948) for the systematic development of shallow-water theory for water waves of large wave length was used by Keller (1948) to obtain the first approximation for the finite-amplitude solitary wave of Boussinesq (1872) and Rayleigh (1876), as well as for periodic waves of permanent type, corresponding to the cnoidal waves of Korteweg and de Vries (1895).

The present investigation extends Friedrich's method as so to include terms up to the fourth order from shallow-water theory for a flat horizontal bottom, and thereby obtains the complete second approximations to both cnoidal and solitary waves. These second approximations show that unlike the first approximation, the vertical motions cannot be considered as negligible, and that the pressure variation is no longer hydrostatic.

[Author's abstract.]

808. LeMehaute, Bernard, 1960, Periodical gravity wave on a discontinuity: Soc. Civil Engineers Am. Proc., v. 86, no. HY 9, Paper 2646, p. 11–41; Am. Soc. Civil Engineers Trans., v. 126, Paper 3212, p. 1006–1036.

A general theory is presented to study the transmission and reflection of gravity waves on a discontinuity to the first order of approximation. Particular attention is given to the cases of an obstruction, a change of width, and a change of depth. In the latter case only, the gravity wave arriving at an angle is studied.

[Author's synopsis.]

809. Nash, J. E., 1960, A unit hydrograph study with particular reference to British catchments: Inst. Civil Engineers, Proc., v. 17, p. 249–282.

The moments of the instantaneous unit hydrograph are correlated with the topographical characteristics for a large number of British catchments, and a general equation for the instantaneous unit hydrograph chosen. The use of the correlation to predict the hydrograph for catchments where sufficient data on rainfall and streamflow are not available is explained, and examples given.

[Author's synopsis.]

810. Nechaeva, N. S., 1960, Opredelenie kriteriya rasplastyvaniya pavodochnykh voln i voln popuskov [Determination of the attenuation criterion of flood waves and of release waves]: Tsentralniy Inst. Prognozov [U.S.S.R.] Trudy (Proc.), no. 105, p. 53–66.

As an analogy to reservoir effects, the attenuation of schematic regular flood hydrographs along a river channel is studied. For a river reach, the ratio $k=Q_d/Q_u$ (Q_d=downstream peak discharge, Q_u=upstream peak discharge) is expressed in different simplified ways: $k_1=1-W/\Sigma Q$, W= storage in river reach, ΣQ=hydrograph volume; or $k_2=(1-W/2\Sigma Q)(1-aW/2\Sigma Q)$, where a=correction coefficient; or $k_3=(1-W/2\Sigma Q)(1-a_1W/2\Sigma Q)(1-a_2W/2\Sigma Q)$; and so on, in which final $k_n=(1-W/N\Sigma Q)^N$, where N=1–5. The next formula is given as $k=f(N, t/\tau)$, where $\tau=dW/dQ$, and t=duration of unit flood release.

811. Nougaro, Jean, Thirriot, Claude, and Guyot, M. T., 1960, Étude numérique des régimes transitoires dans les canaux déconuverts [Numerical study of transient régimens in the open channels]: Conf. on Hydraulics, Budapest [Hungary], 1960, rept. 6, p. 1–10.

This paper summarizes some of the works already published by the three authors. The method of characteristics is applied for the intumescences

traveling along a canal. The integration is carried out by finite differences, programmed for a digital computer. The principles and examples of computation flow charts are described, and examples of computations are given.

812. Partenscky, H. W., 1960, Waves in navigation canals due to lock filling: Am. Soc. Civil Engineers Proc., v. 86, no. WW 1, p. 85–122. Discussion by Escoffier, F. F., v. 86, no. WW 3, p. 163–171.

Wave celerities and wave deformations along canals, created by navigation lock operations, are studied. The special emphasis is on the wave height, wave celerity, wave deformations caused by changes in cross section, theoretical investigations of the fillings phenomena, formation of front of negative waves, profiles of waves due to lock filling, deformation of trailing part of waves, and so forth. Model tests are used to verify the wave theory, the analysis of the surge being measured in the prototype.

813. Rose, D., 1960, Ueber die quantitative Ermittlung der Gezeiten und Gezeitenstroemie in Flachwassergebieten mit dem Differenzverfahren [On the quantitative determination of tides and tidal flows in shallow water regions by finite-differences method]: Francius-Inst. Mitt. [Hanover, Germany], no. 18, p. 1–159.

814. Sinha, G., 1960, Methods of mathematical computation for tidal rivers and their application as practiced in the Netherlands: Inst. Engineers Jour. [India], v. 40n., no. 9, pt. 1, p. 556–570.

815. Supino, Giulio, 1960, Sopra le onde di traslazione nei canali, La equazione linearizzata [On the translation waves in channels, a linearized equation]: Accad. Nazionale dei Lincei [Rome] Rend. (Class of Physical, Math., and Natural Sciences), ser. 8, v. 29, fasc. 5 and 6, Note I, Nov., p. 239–243 and Note II, Dec., p. 472–476.

In the first note a particularly simple procedure is developed for the limitations and approximations related to the attenuation of waves and to their propagation celerity under linearized conditions. The solution of the two De Saint-Venant linearized partial differential equations has the form

$$\varphi=\frac{A}{2}e^{\eta x}\sin\nu\left(t-\frac{x}{\omega}\right),$$

which does not have a general character. The solutions represent only the waves propagating with constant celerity (equal for discharges and levels), called the "regime waves." In the second note, some limitations and approximations for η (coefficient of attenuation) and ω (wave celerity) are analyzed.

816. Ter-Krikorov, A. M., 1960, Sushchestvovanie periodicheskikh voln vyrozhdayushskikhsya v uedinennuyu [The existence of periodic waves degenerating into a solitary wave]: Prikladnaya Matematika i Mekhanika [U.S.S.R.], v. 24, no. 4, p. 622–636.

The long wave on the water surface degenates into a solitary wave, when the wavelength tends to be infinite (cnoidal waves). The paper gives proof of the existence of cnoidal waves, which is valid for the total range of cnoidal waves.

817. Urban, Jaroslav, 1960, Transformace povodonve vlny v nadrzi za soucasne manipulace se stavidly [Transformation of flood wave in reservoir

with simultaneous maneuver of control gates]: Vodohospodarsky Casopis [Czeckoslovakia], v. 8, no. 4, p. 299–316.

818. Ursell, F., 1960, Steady wave patterns on a nonuniform steady fluid flow: Jour. Fluid Mech. [Great Britain], v. 9, pt. 3, p. 333–346.

A steady slightly nonuniform flow with a free surface is subject to a concentrated surface pressure which gives rise to a pattern of surface waves. (For gravity waves on deep water this is the well-known Kelvin ship-wave pattern.) The motion is assumed inviscid, and the waves are assumed small. A theory is developed for the wave pattern, based on the following assumptions:

(1) The stream velocity component normal to a wave crest is equal to the phase velocity based on the local wavelength.

(2) The separation between consecutive crests is equal to the local wavelength.

These assumptions are expressed in mathematical form, and the existence of a set of characteristic curves (associated with the group velocity) is deduced from them. These characteristics are not identical with the crests. Let the additional assumption be made that:

(3) The characteristics all pass through the point disturbance; the characteristics are then completely defined and may be constructed by a step-by-step process starting at the point disturbance. The same construction gives the direction of the wave crests at all points. The wave crests can then be deduced.

Assumptions of the same type as (1) and (2) have long been familiar in various applications of ray tracing. For uniform flows the present theory gives the same pattern as the method of stationary phase.

[Author's abstract.]

819. Wiegel, R. L., 1960, A presentation of cnoidal wave theory for practical application: Jour. Fluid Mech. [Great Britain], v. 7, pt. 2, p. 273–286.

Cnoidal wave theory is appropriate to periodic waves progressing in water whose depth is less than about one-tenth the wavelength. The leading results of existing theories are modified and given in a more practical form, and the graphs necessary to their use by engineers are presented. As well as results for the wave celerity and shape, expressions and graphs for the water particle velocity and local acceleration fields are given. A few comparisons between theory and laboratory measurements are included.

[Author's abstract.]

820. Zhidkov, A. P., 1960, Raschet neustanovivshegosiya dvizheniya vody po metodu G. P. Kalinina i P. I. Milyukova dlya nizhnego byefe Rybinskoy GES pri sutochnom regulirovanii [Computation of unsteady water flow by the method of G. P. Kalinin and P. I. Mi yukov for the tailrace level of Rybinsk water powerplant during daily regulations]: Tsentralniy Inst. Prognozov [U.S.S.R.] Trudy (Pıoc.), no. 96, p. 85–126.

In this report the concept of characteristic river reach is used. The reach is determined under the condition that the discharge of that reach is uniquely determined, and that the storage effect is the same as that of a reservoir storage. Storage is $L=(Q/S)\ (dH/dQ)$, in which Q, S and H are discharge, slope, and level of water surface, respectively. The simple storage equation is applied for flood routing. The example of the Ribinsk powerplant is used to compare this method, the method of finite differences

applied to two partial differential equations, and the observed fluctuations in flood routing. The precision of anticipated fluctuations was the same for both computational methods. The use of Kalinin-Milyukov method is recommended, and suggestions for further research are given.

821. Benney, D. J., 1961, A non-linear theory for oscillations in a parallel flow: Jour. Fluid Mech. (U.S.), v. 10, pt. 2, p. 209–236.

Three-dimensional periodic oscillations in the shear flow region between two parallel streams are considered up to that second order of the oscillation amplitude. It is shown that, as an integral part of the oscillation, there is a mean secondary flow in the nature of a longitudinal vortex. Despite the dissimilarity of the profile of the basic flows, several of the principal features of the calculated results can be compared with those observed for the Blasius flow by Schubauer and Klebanoff and Tidstrom at the National Bureau of Standards.

[Author's abstract.]

822. Escande, L., Nougaro, Jean, Castex, L., and Barthet, H., 1961, Influence de quelques parametres sur une onde de crue subute à l'aval d'un barrage [The influence of certain parameters on a sudden flood of wave downstream from a dam]: Houille Blanche [France], no. 5, p. 565–575.

Initial experimental results obtained with a 1:300 scale model of the Truyere valley below the Sarrans dam.

Wave celerity, form and amplitude, as governed by various parameters such as impounded water level, initial rate of flow downstream from the dam, and river bed roughness.

[Abstract from Houille Blanche.]

823. Faure, J., and Nahas, N., 1961, Étude numérique et expérimentale d'intumescences à forte courbure du front [A numerical and experimental study of steep-fronted solitary waves]: Houille Blanche [France], no. 5, p. 576–587.

The use of an electronic ordinator to resolve the propagation of a steep-fronted wave on a dry or wet bed, which is governed both by De Saint-Venant's equations and the shock wave (or wavefront) equations.

Consideration of the most general case of a rectangular valley of constant width (dry or wet downstream bed, gradient and friction), and of the case of any prismatic or rectangular valley with a wet downstream bed.

Comparison between theoretical and experimental results.

[Abstract from Houille Blanche.]

824. Gerritt, Abraham, 1961, Some aspects of surface water wave scale effects: Am. Soc. Civil Engineers Proc., v. 87, no. HY 1, Paper 2708, p. 41–56.

A study of the scale effects due to surface tension for model studies of water gravity waves generated by moving pressure disturbance is presented. The region of the wave system, generated by the moving pressure disturbance in which scale effects occur, has been determined by explaining the physical meaning of the asymptotes occurring in the fish-line problem.

[Author's synopsis.]

825. Gherardelli, Luigi, 1961, Contributo alla teoria del mato vario delle correnti a pelo libero, Applicazione del methodo di integrazione di Riemann [Contribution to the theory of variable motion for open water-courses. Application of Riemann integration method]: Energia Elettrica [Italy], v. 38, no. 1, 14 p.

Reference is made herein to the linear equation of variable (unsteady) motion for linear flows. This equation may be easily solved by the Riemann integration method after a suitable change of variables. The process suits the numerical calculations.

[Author's synopsis.]

826. Guelton, M., Weingaertner, P., and Sevin, Ph., 1961, Fonctionnement en éclusées du canal industriel de Basse-Durance [Lock-controlled operation of the industrial diversion works on the Lower Durance]: Houille Blanche [France], no. 5, p. 597–612.

Description of the diversion works, which include five successive canal sections and discharges into the "Etang de Berre" at sea level. Other features are: total length 80 km; total gross head 256 m; flow capacity 250 cu. M per sec; output 500,000 kw.

Description of the "by the lockful" operation of the system, which can take on the total load or be shut down very quickly. Load variations can be distributed simultaneously over all five power stations, so that they act as a single 500,000 kw unit.

Description of a study of the solitary waves resulting from varying conditions due to the upstream and downstream power stations in each canal reach.

Description of the provisions made in the successive canal reaches to absorb such waves and the means provided in the individual power station relief works to ensure the safety of the various canal sections.

[Abstract from Houille Blanche.]

827. Kalinin, G. P., 1961, Vizallások és vízhozamok előrejelzése a nempermanens vízniozgás alapegyenleteinek Közelitő megoladása alapján [Forecasting stages and discharges by the approximate solution of the fundamental equations of unsteady flow]: Vizűgyi Közlemenyek [Hungary], no. 4, p. 420–436.

Using the storage differential equations for flood routing, the author stresses that the basic problem in routing is the selection of the reach Δx. The "characteristic bed reach" is introduced, as in author's earlier papers with hydraulic interpretation, which offers a theoretical explanation for the trend of the numerical constants of the Muskingum routing methods. A comparison of the hydraulic basis of the method with the unit hydrograph method yielded the same results as those obtained by I. E. Nash, using a different approach in 1960.

[English summary, p. 51.]

828. Laboratoire National d'Hydraulique, 1961, Intumescence consécutive à une variation brusque de débit [Intumescence following a sudden discharge change]: Lab. Natl. Hydraulique, Châtou [France], Rept. no. 2, T.359B.

829. Laboratoire National d'Hydraulique, 1961, Calcul sur ordinateur électronique: mouvement non permanent dans les canaux, onde de crue [Computation on the electronic computer; unsteady movement in channels, flood wave]: Lab. Natl. Hydraulique, Châtou, [France], Rept. T.400B.

830. Levin, A. G., and Zhidkov, A. P., 1961, Predvychislenie khoda uravney reki Volgi nizhe Stalingradskoy GES metodom elektromodelirovaniya [Computation of level hydrograph of Volga River below Stalingrad

Hydroelectric Powerplant by the method of electric models]: Meteorologiya i Gidrologiya [U.S.S.R.], no. 8, p. 38–41.

The application of an electric analog device for flood routing, developed by Kalinin, Milyukov, and Nechaeva, is applied to the Volga River below the Stalingrad Hydroelectric Power Station.

831. Montuori, G., 1961, La formazione spontanea dei treni d'onde su canali a pendenza molto forte [The spontaneous formation of wave trains in the canals with very steep slope]: Energia Elettrica [Italy], no. 2, p. 127–141.

832. Northrop, W. L., and Timberman, C. W., 1961, Use of computers for Kansas River flood studies: Am. Soc. Civil Engineers Proc., v. 87, no. HY 4, Paper 2866, p. 113–150.

A general digital computer program is described that produces flow hydrographs for an extensive network of basin subareas using unit hydrographs and routing coefficients. This concept is expanded to determination of releases for a system of reservoirs. Development of an electronic analog for reservoir operation is also described.

[Authors' synopsis.]

833. Preissmann, A., and Cunge, J. A., 1961, Calcul du mascaret sur machine électronique [Tidal bore calculation on an electronic computer]: Houille Blanche [France], no. 5, p. 588–596.

Fundamental difficulties encountered in the numerical resolution of De Saint-Venant's equations by the finite-differences method, in the case of discontinuous flow conditions. Introduction of a "psuedo viscosity" as an approximate representation of the loss of head due to the travelling surge wave—and hence the development of the bore. Preliminary systematic calculations on simplified examples showing up the various factors liable to affect the formation and propagation of the bores.

[Abstract from Houille Blanche.]

834. Preissmann, A., and Werner, G., 1961, Application du calcul des intumescences sur machine électronique à divers cas pratiques [Application of translation wave calculations on an electronic computer to practical cases] Houille Blanche [France], no. 5, p. 613–621.

The three following possible applications of electronic computer to solitary wave calculations are discussed:

1. Systematic trial flood wave calculations, in order to assess the validity of certain semi-empirical rules governing flood propagation;
2. Study of the stability of control afforded by automatic gates;
3. Calculation of flows in a tunnel alternating between pressure flow and free surface conditions.

[Abstract from Houille Blanche.]

835. Rantz, S. E., 1961, Surges in natural channels: U.S. Geol. Survey Water-Supply Paper 1369–C, 90 p.

This report presents the results of an investigation of the travel of surges in a natural channel. It is demonstrated that the initial element of a flat wave front travels with varying velocity in accordance with Seddon's principle. Progressive flattening of the wave form is shown to be related to channel storage. The investigation was made on a 12.7–mile reach of Mokelumne River below Pardee Reservoir in California. Data for 10 surges are tabulated and analyzed.

[Author's abstract.]

836. Sorensen, Torben, and Larsen, Ian, 1961, Hydraulic calculations of storm flood levels in connected shallow basins with special reference to the Limfjord, Denmark: Internat. Assoc. Hydraulic Research, 9th Convention, Dubrovnik (Yugoslavia), 1961, Proc., Paper 3, 41, p. 1–8.

In this paper the calculation of storm flood levels in connected shallow basins is treated as a quasistationary case taking into account the wind set-up in the basins and the combination of wind set-up and flow in the connecting channels.

The calculations were made on a digital computer, the Danish DASK machine being used for this purpose.

The water levels in the Limfjord were calculated for 55 different storms between 1931 and 1959 to establish frequency distributions of the highest high waters at 12 different points.

[Authors' abstract.]

837. Supino, Giulio, 1961, Sopra le onde di traslazione nei candli, B) Il caso non-lineare [On the translation waves in channels. B) the non-linear case]: Accad. Nazionale dei Lincei (Rome), Rend. (Class of Physical, Math., and Natural Sciences), ser. 8, v. 30, fasc. 2, p. 140–148.

The integration of the two De Saint-Venant partial nonlinear differential equations is treated, after they have been reduced to second-order partial differential equations. Different cases for solutions are analyzed and discussed.

838. Thirriot, Claude, 1961, Étude de la convergence d'un procédé numérique de calcul des intumescences [Study of the convergence of a numerical procedure for computation of intumescences]: Acad. sci. [Paris] Comptes rendus, v. 252, p. 1421–1423.

Following the previous notes (Comptes rendus v. 248, p. 2950, 1959; v. 249, p. 1858, 1959, and v. 250, p. 55, 1960) of applying the method of characteristics and IBM 650 digital computer to computation of intumescences, the comparison of formulas of first and second order permits the selection of finite difference ΔQ (discharge). The departure between results given by the two formulas are analyzed, and an approximation of errors involved is developed.

839. Thirriot, Claude, 1961, Étude adimensionnelle des intumescences [Dimensionless study of intumescences]: Acad. sci. [Paris] Comptes rendus, v. 252, p. 3395–3397.

The relative (dimensionless) values are introduced for length, velocity area, and time, and the two De Saint-Venant equations are correspondingly changed. The use of the equations in demensionless form, and their discussion are given.

840. Thirriot, Claude, 1961, Élude des phénomènes pneumatiques provoqués par la propagation d'intumescence dans une galerie de fuite d'usine hydroélectrique souterraine [Study of compressed air phenomena created by a propagation of intumescence (bore) in a tailrace gallery of an underground hydroelectric power station]: Acad. sci. [Paris] Comptes rendus, v. 252, p. 4108–4110.

Following the note (Comptes Rendus v. 249, 1959, p. 2718), the shock wave downstream from the bore in a gallery is represented schematically by two partial differential equations. In this note the equations are reduced to dimensionless values. The Mach number is used. The equa-

tions are then linearized, and solutions are obtained by using the operational computation. The computation is also carried out on IBM 650 digital computer, and the velocities of air at the end of gallery are given in function of time.

841. Wolska-Boschenek, J., 1961, Problème aux limites pour un domaine non borné dans la théorie du mouvement non stationnaire d'un liquide visqueux [Boundary problem for a non limited domain in the theory of unsteady movement of a viscous fluid]: Archive for Rational Mechanics and Analysis [Germany], v. 7, no. 3, p. 196–211.

842. Yano, Katsumasa, 1961, Theoretical research on the surging phenomena of the high tide by the typhoon into the rivers and canals: Kyoto Univ. [Japan], Disaster Prevention Research Inst. Pub., p. 194–197.

Basing on the data of the Ide-Bay Typhoon, when the records of the travelling-up phenomena of the high tide into the Kiso-River and others were clearly recognized, the author tried to solve their phenomena theoretically and found some interesting characteristics, by solving the differential equations by means of Rieman integral method.

[Author's English synopsis.]

843. Yevdjevich, V. M., 1961, Unsteady free surface flow in a storm drain (general and analytical study): Colo. State Univ. Eng. Research, Rpt. CER 61 VMY 38, p. 1–76.

As a preliminary general study the broad scope of this report is the outline of problems, the selection of mathematical tools and procedures, and the elaboration of a general approach for further studies by hydraulic model and by digital computer investigations in order to develop a set of routing methods for storm drain floods.

The initial and boundary conditions, applications, and the general approach selected are briefly enumerated in the introduction. The integration of differential equations by method of finite differences is treated in detail, special attention is given to boundary problems.

844. Harder, J. A., 1962, Analog models for flood control systems: Am. Soc. Civil Engineers Proc., v. 88, no. HY 2, Paper 3074, p. 63–74.

The success of applying analog model techniques to tidal flow channels has led to the extension of the technique to the simulation of flood control systems. The similarity relationships for long period flood waves are derived.

Methods for the simulation of the channel sections are described, with particular attention paid to the nonlinear properties of the stage-discharge relation and the stage-storage relation. The introduction of boundary conditions in the form of streamflows and rainfall excesses is programmed for each six hours in the prototype, for a total period of ten days, by special pinboards. Each simulated flood requires five milliseconds, after which initial conditions of flow, at the initial instant of the ten-day period, are reestablished in preparation for the next flood period. Time history of stage or discharge at any two of fifty stations can be displayed simultaneously on an oscilloscope screen. Provision is made for the insertion of up to six reservoirs into the analog, each of which is provided with a programmable output simulating the action of reservoir release gates. It is expected that the analog will be useful both in project planning and during flood emergencies.

[Author's synopsis.]

INDEX BY AUTHORS

[This index by authors refers to the numbers of the references in the Chronological Bibliography. The index is arranged alphabetically according to author's last name. The Russian names are transcribed. For each author's name the number of each reference and the year of publication are shown inside the parentheses. When there are two or more authors, each reference is indexed under the names of all coauthors. Two or more authors are indicated by an asterisk (*) before the identifying number of such a reference: one asterisk if there are two authors, two asterisks if there are three authors, three asterisks if there are four authors, and so forth]

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[Some entries in "The Chronological Bibliography," refer in the abstracts to authors and their works connected with the unsteady flow in channels and flood routing. The following index of these authors is given with the author's name, the year of the published work (which is not abstracted or annotated in this bibliography), the reference in the chronological bibliography which mentions the author and his work, and the year in which the reference was published. Some authors appear in the previous list and in this index. The reference in this index is made only to the works not abstracted or annotated in the previous index. This index can orient the reader in searching for the additional references which are not abstracted or annotated here]

INDEX BY SUBJECTS

[This index is based on the abstracts in the chronological bibliography and not on the original text in the references. The purpose of this index is only to orient the reader toward the main references; there is no intention of showing full coverage of a subject by listing all references. It was the tendency in compiling this index to represent a subject by a listing no more than approximately 10 percent of the total number of references]