Surges in Natural Stream Channels

By S. E. RANTZ

RIVER HYDRAULICS

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ABSTRACT

This report presents the results of an investigation of the travel of surges in a natural stream channel of irregular cross section, slope, and alinement. For the purpose of this study, a surge is defined as a rapid change in discharge from one condition of steady flow to another. The channel studied is a 12.7-mile reach of Mokelumne River below Pardee Reservoir in central California, and in the reach there are 22 rated river gages.

The surges studied were both positive (rising) and negative (falling), and both types had extremely flat wave fronts. Results of the investigation indicate that the time of travel of the initial element of a surge, or the initial disturbance to steady flow, can be computed reliably in accordance with Seddon's principle. Channel properties such as shape, dimension, roughness, and slope affect the velocity of the initial disturbance, and the variation of these factors produces changes in the velocity of the initial element of the surge as it traverses the reach. Given a sufficient number of rated cross sections that are favorably situated, it is possible to compute these changes in wave velocity, because the effect of channel geometry and roughness in a subreach is integrated in the stage-discharge relation for that subreach.

The wave form flattens progressively downstream as a result of channel storage. An actual measure of the varying channel storage between the profiles of steady flow for the initial and final discharges is provided by the rating curves for the cross sections in the reach. Because these rating curves represent the integrated effect of channel geometry and roughness, it follows that the volume of channel storage and consequently the degree of flattening of the wave form are affected by the hydraulic characteristics of the reach. Given the upstream hydrograph and sufficient intermediate rated cross sections, the downstream hydrograph, following the arrival of the initial disturbance, may be computed in accordance with the principle of conservation of mass; that is, the net change in volume of discharge equals the change in storage. In this study the upstream hydrograph was routed downstream by use of a multiple-phase reservoir-type routing technique. Very close agreement was obtained between the computed and recorded downstream hydrographs.

INTRODUCTION

PURPOSE AND SCOPE

The theoretical considerations governing wave travel in prismatic channels of mild slope are well understood and have been corroborated by laboratory analysis. Efforts to obtain supporting data from observations of natural streams have been less successful. This study was made, therefore, in an attempt to fill the need for a rigorous test of the applicability of classic wave theory to natural channels.

The last study of note of this nature, with which the author is familiar, is that of Wilkinson (1945) on the Clinch and lower Tennessee Rivers. Wilkinson obtained reasonably close agreement between observed and computed wave velocities, but he had too few rated cross sections available to him to permit a complete test. For a truly satisfactory test of theory, one should be able to reproduce the hydrograph at the downstream end of a reach of river from a knowledge of the upstream hydrograph and the hydraulic properties of the stream. Because of the nonuniformity of these properties in a natural channel, a stream can rarely be found for which sufficient data are available to permit such a test. Fortunately for this study, a suitable channel is found in a 12.7-mile reach of Mokelumne River below Pardee Reservoir in central California. Reservoir operations there afforded an excellent opportunity for studying the travel of surges, which are defined in this report as rapid changes in discharge from one condition of steady flow to another.

ACKNOWLEDGMENT

The study described in this report was done under a cooperative agreement between the U.S. Geological Survey and the California Department of Water Resources.

The cooperation of the East Bay Municipal Utility District in furnishing data for the 20 staff-gage sites in the Lancha Plana-Clements reach of Mokelumne River is gratefully acknowledged.

HYDRAULIC CHARACTERISTICS OF THE MOKELUMNE RIVER

The reach of the Mokelumne River under investigation extends from the upstream gaging station at Lancha Plana to the downstream gaging station near Clements. The river is controlled 3 miles upstream from Lancha Plana by Pardee Reservoir, which is operated for municipal water supply and power by the East Bay Municipal Utility District. Between the two gaging stations the river drains 46 square miles of low foothill area, and at Clements the river debouches onto the rich agricultural valley floor. The fall in the 12.7-mile reach of river is 92 feet, but the streambed gradient of the upper half of the reach is approximately twice as steep as that of the lower half. Like most natural streams, the cross-sectional shape of the channel is irregular and varies considerably in the reach, and the channel alinement is marked by numerous bends.

Except for periods immediately after rains or during protracted dry spells, this reach of river is virtually in static equilibrium with the ground-water table; that is, there is no measurable difference in discharge at the Lancha Plana and Clements gaging stations while steady flow is maintained at the Pardee Reservoir outlet works. This is consistent with the known characteristics of the channel, the Mokelumne River being an effluent or gaining stream upstream from the reach and an influent or losing stream downstream from Clements. However, there are significant losses in discharge between Lancha Plana and Clements during protracted dry periods. Conversely, during and immediately after rains there is an appreciable amount of local inflow into the reach, but this local inflow is generally of short duration after the cessation of rainfall. The surges selected for study occurred during periods when local inflow or seepage loss was negligible.

Much is known of the hydraulic properties of the stream channel. In 1930, as part of a study in connection with a water-rights litigation, the East Bay Municipal Utility District established staff gages at 20 sites on the river between the Lancha Plana and Clements recording gages. These staff gages were located at obvious changes in channel geometry of the river and their location is shown on figure 10. Cross sections were obtained and the stage-discharge relation determined at each of the staff-gage sites during periods of steady outflow from Pardee Reservoir. The rating curves for the staff-gage sites were revised in 1938 and again in 1940, and a few observations were obtained in 1955 and 1956 for the purpose of checking the revised

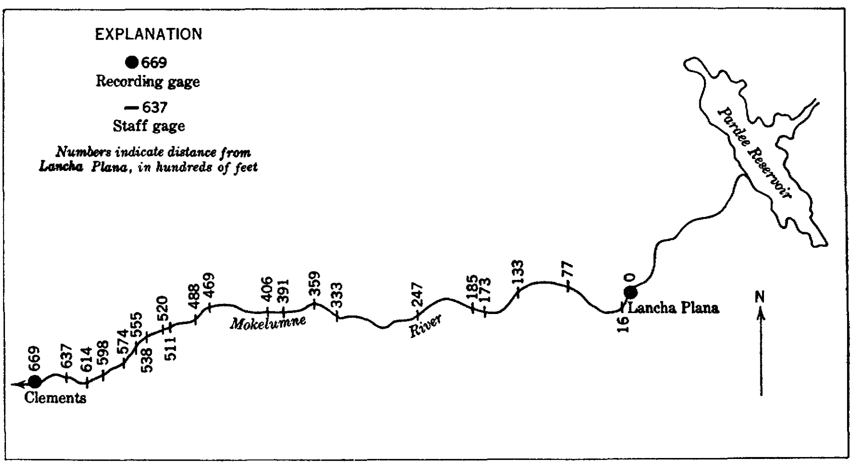

FIGURE 10.—Map of Mokelumne River showing location of staff gages between Lancha Plana and Clements.

stage-discharge relations. It was found that all ratings were prone to shift when the channel carried unusually high flows, but the shifts were essentially parallel. For example, consider two discharges, Q_1 and Q_2, at a given gaging site, that differ by a relatively small amount, ΔQ. On several occasions during the years, the corresponding water-surface elevations, h_1 and h_2, at the site will have changed, but at all times the difference in water-surface elevation, Δh, corresponding to ΔQ, will remain essentially unchanged. Therefore, the slope of the curve that relates the discharge to cross-sectional area is virtually constant for any particular discharge at any gaging site in the reach.

The method of operating Pardee Reservoir makes the downstream reach of river ideal for studying the travel of surges in natural channels. Flows of less than 5,000 cfs (cubic feet per second) are completely controlled and pass through either the turbines at the base of the dam or through the sluice valves of the dam. The actual operating procedures of the reservoir are somewhat complicated by varying demands for hydroelectric generation and by the necessity of complying with the legal rights of downstream water users. Large abrupt changes in water release must be avoided in order to prevent damage to the levees downstream from Clements. Usually the outlet works are operated so that the changes produced in water-surface elevation in the Lancha Plana-Clements reach are less than 1 foot. When it is necessary to change the reservoir release by large amounts, several successive small changes in outflow are made, with at least an hour elapsing between successive gate operations. Figure 11 is a typical stage hydrograph for the upstream gage at Lancha Plana.

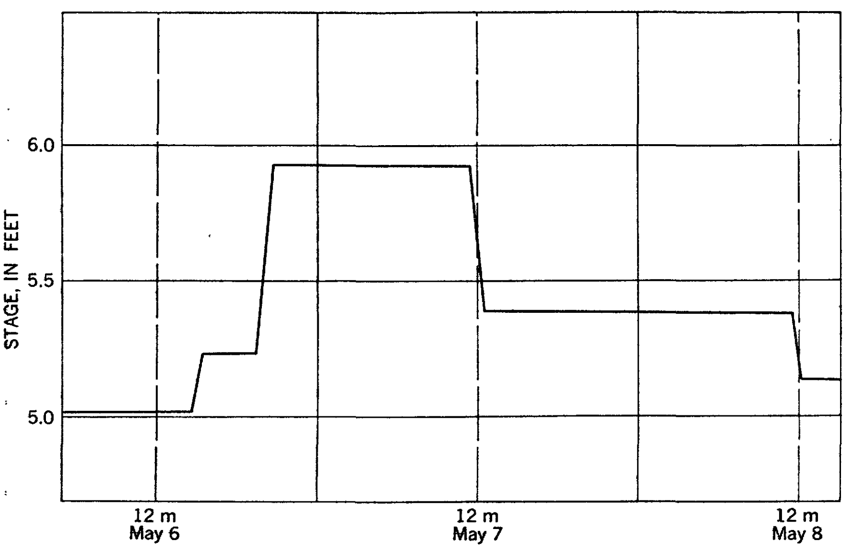

FIGURE 11.—Typical stage hydrograph at Lancha Plana, May 1958.

The surges shown in figure 11 flatten in their travel downstream from Lancha Plana and this is evident from the discharge hydrographs shown in figure 12 for the gages at Lancha Plana and Clements. The

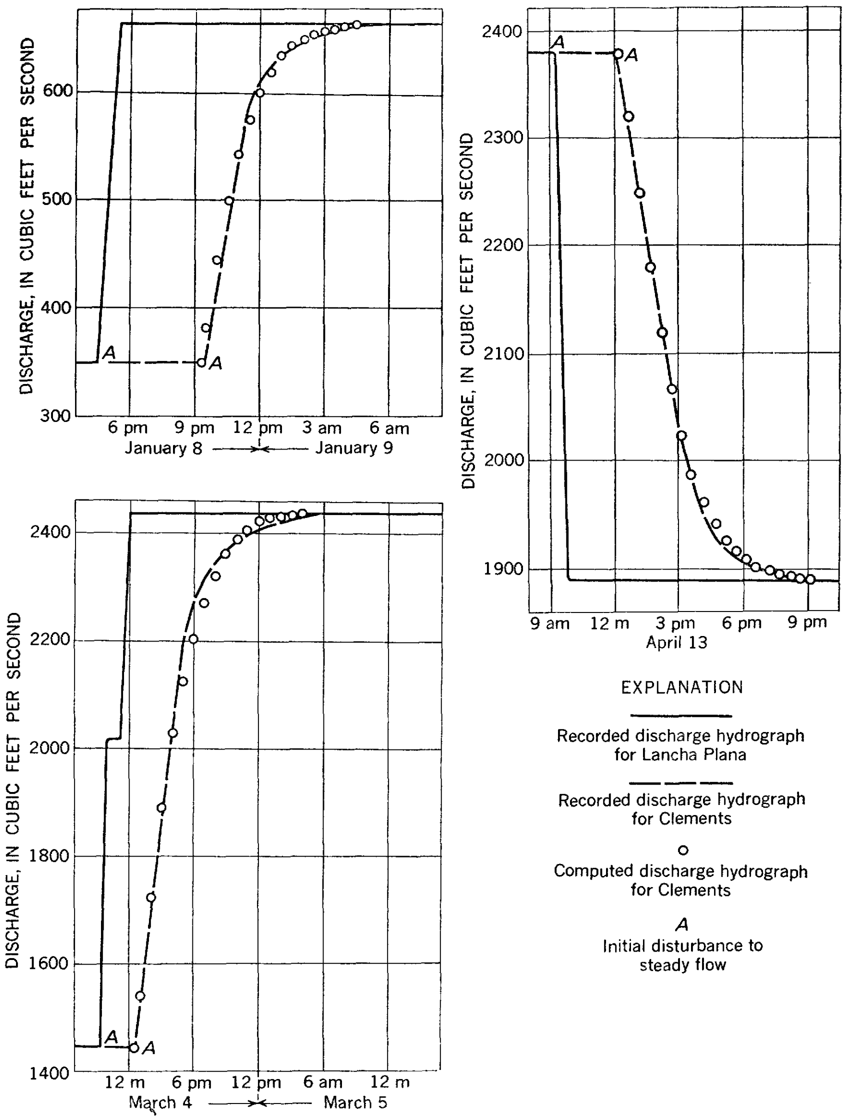

FIGURE 12.—Discharge hydrographs for Lancha Plana and Clements, 1958.

circles shown on the hydrographs for Clements represent computed values of outflow from the reach, and they indicate the close agreement between the recorded flow and that computed by routing the Lancha Plana hydrograph through the reach. A knowledge of Seddon's principle (Thomas, 1935) is essential to an understanding of the methodology used and the results obtained in the routing process. Although the general aspects of this principle are familiar to most hydraulic engineers, some of the details will bear repeating and are discussed in the paragraphs that follow.

SEDDON'S PRINCIPLE OF WAVE TRAVEL

Seddon's principle is best explained by reference to figure 13, where for a given gaging site, discharge is plotted as the ordinate, and the corresponding cross-sectional area as determined from the rating curve for steady flow is plotted as the abscissa. The curve of figure 13 is concave upward, a shape typical of all ordinary channel sections where the velocity increases as the area increases. In accordance with Seddon's principle, $V_w=dQ/dA$, where V_w is the absolute wave velocity of a disturbance to steady flow. Thus, for either a rising (positive) or a falling (negative) surge, preceded by a steady flow Q_2, the velocity of the initial disturbance, or initial element of the surge, is represented by the slope of the tangent to the curve at P_2, or tan θ_{2_t}. It should be emphasized that Seddon's law is strictly applicable only to extremely flat wave fronts, a condition that permits

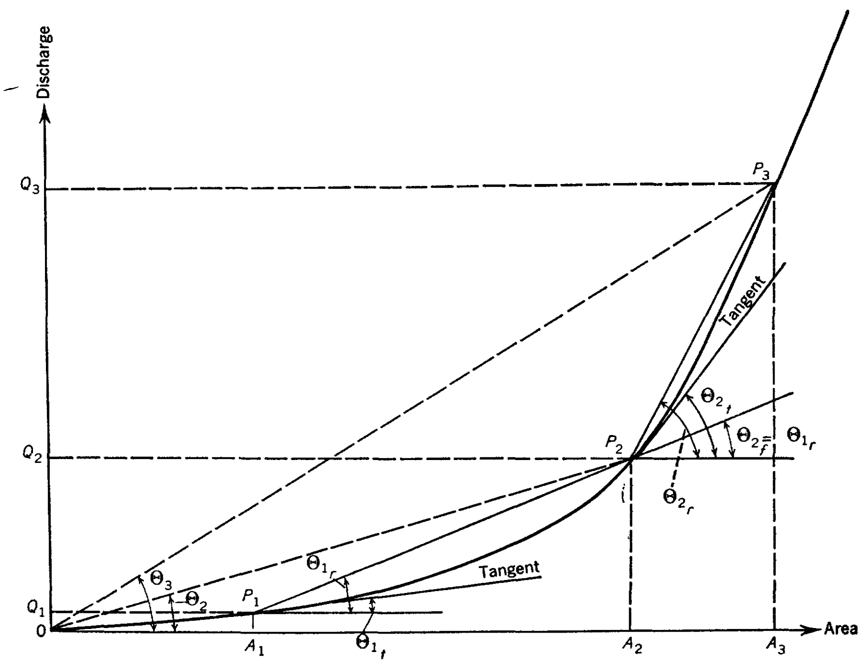

FIGURE 13.—Graphical representation of discharge, velocity, and area relations at a gaging station for a rising or falling surge. Subscripts: *f*, falling (negative) surge; *r*, rising (positive) surge; *t*, tangent to curve.

the term dQ/dA to be determined from the rating curve for steady flow. Figure 13 illustrates graphically the fact that the wave velocity corresponding to any discharge is greater than the velocity of flow for that discharge. With a discharge Q_2, for example, the wave velocity equals tan θ_{2_t}, whereas the flow velocity equals the smaller value, tan θ_2.

It is customary in computing wave velocity by the Seddon formula to substitute for dQ the difference between initial and final discharges, and to substitute for dA the difference between initial and final cross-sectional areas. The computed wave velocity for the initial disturbance is then too large for rising surges and too small for falling surges. The larger the increment ΔQ, the greater the error. All this is evident from a further inspection of figure 13. For an initial steady flow Q_2, any change in discharge—whether an increase or a decrease—will create a wave whose initial element travels with a velocity equal to tan θ_{2_t}. If the discharge rises to Q_3, the computed wave velocity will be $(Q_3-Q_2)/(A_3-A_2)$, or tan θ_{2_r}. The computed velocity will then be too large, since $\theta_{2_r}>\theta_{2_t}$. However, should the discharge fall from Q_2 to Q_1, the computed wave velocity will be $(Q_2-Q_1)/(A_2-A_1)$, or tan θ_{2_f}. The computed wave velocity is now too small, since $\theta_{2_f}<\theta_{2_t}$.

Identical wave velocities will be computed for a rise from Q_1 to Q_2 or a fall from Q_2 to Q_1, because both ΔQ and ΔA are identical in the two computations. Nevertheless, it is apparent from θ_{1_t} and θ_{2_t} that the initial disturbance to steady flow Q_2 will travel faster than the initial disturbance to steady flow Q_1. These aspects of Seddon's principle have been pointed out because of their direct bearing on the interpretation of the results of this investigation.

OBSERVED AND COMPUTED WAVE VELOCITY OF INITIAL DISTURBANCE

The wave velocity of the initial disturbance to steady flow was computed by the Seddon formula for 10 surges that occurred in 1957 and 1958. The term "initial disturbance", as previously explained, refers to the beginning of the change from steady flow and is illustrated by points labeled A on the discharge hydrographs of figure 12. The method of computing the mean wave velocity is shown in the sample computation of table 1. The wave velocity at each of the 22 gage sites was computed from the formula $V_w=\Delta Q/\Delta A$, where ΔQ represents the difference in discharge between the two steady flow conditions and ΔA is the increment of the cross-sectional area corresponding to ΔQ. The mean wave velocity in each subreach between two gages was obtained by averaging the computed wave velocities at the ends of the subreach. The length of each subreach was then divided by the subreach wave velocity to obtain the traveltime. Finally, the total

TABLE 1.—*Sample computation of velocity of initial disturbance in Lancha Plana—Clements reach*

[Date of determination, Jan. 8, 1958. Q_1, 349 cfs; Q_2, 665 cfs; ΔQ, +316 cfs]

Station (See fig. 10)	Length of subreach (ft)[1]	Difference in cross-sectional area, ΔA (sq ft)	Wave velocity Vw (fps)	Mean wave velocity in subreach (fps)	Travel-time in subreach (sec)	Sum of travel-times (sec)
0	1,600	83	3.81	4.54	352	0
16		60	5.27			352
77 [2]	6,100	87	3.63	4.45	1,371	1,723
614	2,300	146	2.16	2.66	865	15,145
637		100	3.16			16,010
669	3,200	98	3.22	3.19	1,003	17,013

$$\text{Average wave velocity in reach}=\frac{66{,}900 \text{ ft}}{17{,}013 \text{ sec}}=3.93 \text{ fps}$$

[1] Distance from preceding station.
[2] Stations 133 through 598 omitted.

length of the reach, 12.7 miles, was divided by the total traveltime to give the average wave velocity between Lancha Plana and Clements.

Pertinent data concerning the surges selected for study are found in table 2, where the surges are listed in order of magnitude of the initial discharge. It is apparent from columns 8 and 9 that for the range of discharges investigated, the velocity of the surge increases as the initial discharge increases. It was not feasible to study higher discharges, as this would have involved uncontrolled releases over the dam spillway. The hydrographs of spillway discharges show well-rounded peaks and troughs, whose time of occurrence cannot be pinpointed with sufficient accuracy for use in a study involving traveltimes of 3 hours or less. The surges studied are amenable to treatment in accordance with Seddon's principle because the wave fronts are extremely flat and actually are not discernible to the eye. The flatness of the wave front is evident from an inspection of columns 5 and 6 of table 2 which show, respectively, the time elapsed during the change in discharge at Lancha Plana and the range in increments of stage at the 22 gages for each surge. The rate of change of stage for a flat wave front has no effect on the velocity with which the initial disturbance travels, but does affect the degree to which the surge profile is flattened by channel storage. This effect is discussed later in the report.

An inspection of column 11 of table 2 reveals how closely the computed wave velocities agree with the recorded values. Even more important is the fact that the 5 positive (rising) surges have computed velocities that are slightly high and the 5 negative (falling) surges have computed velocities that are somewhat low, which is strikingly demonstrated in the 2 groups of surges with initial discharges of approximately 700 and 2400 cfs, respectively. As explained in the previous discussion of Seddon's principle, these departures of the

TABLE 2.—*Travel of surges on Mokelumne River from Lancha Plana to Clements*

Date	Discharge (cfs)			Time for change in discharge at Lancha Plana (hours)	Range in increments between Q_1 and Q_2		Recorded travel time of initial disturbance (hours)	Velocity of initial disturbance			
								Mean (fps)		Error of computed mean velocity (percent)	Range of computed mean velocity in subreaches (fps)
	Initial Q_1	Final Q_2	Change ΔQ		Stage Δh (feet)	Cross-sectional area ΔA (sq ft)		Recorded	Computed		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Jan. 8, 1958	349	665	+316	1.00	0.45–0.89	52–146	5.00	3.72	3.93	+5.6	2.25–5.58
Jan. 13, 1958	665	337	−328	1.50	.51–.95	52–154	4.00	4.65	3.86	−16.8	2.23–5.14
Mar. 13, 1958	690	1,080	+390	.75	.34–.98	43–142	3.67	5.06	5.33	+5.3	3.12–8.08
May 29, 1957	700	1,440	+740	1.00	.66–1.82	87–258	3.58	5.19	5.46	+4.8	3.37–8.15
Mar. 4, 1958	1,445	2,020	+575	.67	.44–1.07	61–173	3.33	5.58	6.18	+10.8	4.24–8.20
Apr. 13, 1958	2,380	1,890	−490	.67	.37–.89	51–146	2.92	6.36	6.00	−6.0	4.20–8.26
Mar. 10, 1958	2,400	1,875	−525	.67	.42–.93	58–162	3.00	6.19	5.85	−5.5	4.16–8.02
May 31, 1957	2,460	2,920	+460	.75	.31–.84	44–134	3.17	5.86	5.89	+0.5	4.00–9.12
June 6, 1957	3,270	2,900	−370	.75	.26–.65	37–99	3.00	6.19	5.93	−4.2	4.04–8.86
June 21, 1958	4,210	3,720	−490	.50	.31–.82	46–133	3.00	6.19	5.86	−5.3	3.97–9.48

computed from the observed velocities are precisely the result to be expected in a valid demonstration of the theory.

Particularly interesting is the group of surges whose initial discharge is approximately 700 cfs. It will be noted that the computed velocities of the two positive surges agree closely with the recorded velocities, whereas the computed velocity of the negative surge shows a much greater departure from the recorded value. The situation here is similar to that depicted in figure 13, where Q_2 on the figure may be said to represent 700 cfs, and P_2 is located at a change in the curvature of the discharge-area relationship. Figure 13 shows that the wave velocity computed for a positive surge ($\tan \theta_{2_r}$) will agree closely with the true wave velocity ($\tan \theta_{2_r}$), but that the wave velocity computed for a negative surge ($\tan \theta_{2_f}$) will be considerably less than the true wave velocity. To go a step further, if Q_1 on figure 13 is assumed to represent a discharge of 350 cfs, then figure 13 depicts the situation involving the first two surges listed in table 2. These two surges, one positive and the other negative, have approximately the same absolute ΔQ (320 cfs) and the final discharge of the rising surge is equal to the initial discharge of the falling surge. The computed wave velocities for both surges will be identical, since $\theta_{1_r}=\theta_{2_f}$. However, the true wave velocity of the rising surge ($\tan \theta_{1_t}$) is slower than the true wave velocity of the falling surge ($\tan \theta_{2_t}$), since $\theta_{1_t}<\theta_{2_t}$. This agrees with the values shown in column 9 of table 2. Also, from the position of points P_1 and P_2 on the discharge-area relation curve of figure 13, it is expected that the computed wave velocity of the rising surge will agree more closely with its true velocity, than will the velocity of the falling surge. This too, is borne out by the percentage error shown in column 11 of table 2.

From the above analysis it may be concluded that for surges with flat fronts in natural channels, the velocity of the initial disturbance to steady flow may be computed reliably in accordance with Seddon's principle, if a sufficient number of rated cross sections, favorably situated, are available. The reliability of the computed velocity will vary inversely with the magnitude of the ΔQ selected.

Column 7 of table 2 shows the wide range in increments of cross-sectional area, ΔA, at the 22 gages for each surge. This variation in ΔA for any given surge results in a wide range of computed wave velocities for the individual subreaches between Lancha Plana and Clements, as indicated in column 12. Furthermore, the value of ΔA at any gage site is related to the shape of the rating curve for that gage, and the rating curve itself integrates the effect of channel geometry and roughness of the adjacent subreach. It is concluded, therefore, that channel properties, such as shape, dimension, roughness, and slope affect the velocity of the initial disturbance, and the variation of these factors produces changes in the velocity of the surge as it traverses the reach. It is evident, too, that there is a degree of risk involved in computing the average wave velocity for a long reach of natural channel when the computations are based on only a few rated cross sections. The results may be seriously in error if the rated cross sections are not typical of the reach.

FLATTENING OF THE SURGE PROFILE

It is evident from the discharge hydrographs of figure 12 that the flat wave front at Lancha Plana is flattened considerably more by the time the surge reaches Clements. This flattening of the profile is attributable to the effect of channel storage, the mechanics of which are explained in the following example.

Assume that the initial steady flow at Lancha Plana is increased by 300 cfs in 1 hour to a new state of steady flow, and that the initial disturbance travels 4 miles to a site A during this hour, and 3 miles more to a site B during the second hour. At the end of the first hour the initial disturbance will have reached site A, but there will have been no increase in discharge at site A. During this first hour, the storage space available in the 4-mile reach between the profiles of steady flow for the initial and final discharges will have been decreased by the inflow of a volume equivalent to 150 cfs-hrs, 150 cfs being the average value of ΔQ during the first hour. During the second hour, an additional volume of 300 cfs-hrs will have entered the reach between Lancha Plana and site A, but the discharge at site A at the end of the second hour will have increased by something less than 300 cfs, because some of the inflow into the reach remains in the reach as channel storage. In the meantime, the initial disturbance will have reached site B at the end of the second hour, but there will be no increase in discharge at site B. There is no need to carry this example any further; it is already evident that some flattening of the surge profile has occurred at site A, and the flattening becomes progressively more pronounced as the surge moves downstream.

The channel storage effect is associated with the channel geometry and the roughness, because the combined effect of the geometry and roughness establishes the Δh corresponding to the difference in discharge between any two rates of steady flow. In turn, Δh establishes the amount of storage between the profiles of these two rates of steady flow.

The example given in a preceding paragraph may be used to illustrate how the flattening of the surge is affected by the length of time that elapses at the upstream site while the flow changes from steady discharge Q_1 to steady discharge Q_2. It was shown that in the first hour, the original storage space between Lancha Plana and site A decreased by a volume equivalent to 150 cfs-hrs. Had the ΔQ of 300 cfs occurred at Lancha Plana in one-half hour, the original storage available in the reach would have decreased in the first hour by 225 cfs-hrs (an average flow of 150 cfs for the first half hour and the full flow of 300 cfs for the second half hour). Storage differences, such as those discussed above, may have an important effect on the flattening of the surge profile, particularly in short reaches.

COMPUTATION OF THE CLEMENTS HYDROGRAPH

In order to demonstrate the effect of channel storage on the flattening of the wave profile, the hydrographs at Clements were computed for three surges for comparison with the recorded flows. The method used is based on the principle of the conservation of mass for an incompressible fluid; that is, the net change in volume of discharge equals the change in storage. The surges selected for study are those shown in figure 12. A complication arises in the computations, owing to the fact that the difference between recorded inflow at Lancha Plana and recorded outflow at Clements gives values for change in storage that are somewhat greater than the computed channel storage between the profiles of initial and final discharge. The additional increment of storage space required for consistency definitely can be attributed to bank storage.

Earlier in this report it was mentioned that local inflow into the Lancha Plana-Clements reach ceases very shortly after the cessation of rainfall. The extremely permeable coarse alluvium composing the river banks and underlying the flood plain, which is so effective in draining off subsurface water during wet periods, apparently transmits water rapidly in a lateral direction as the river level fluctuates. Downstream from Clements, diurnal fluctuations in the river level have been observed to cause similar diurnal fluctuation in water levels in wells as far as 700 feet from the river (Piper and others, 1939, p. 162–163). The increment of bank storage required for consistency in this study is relatively small, amounting to the equivalent of a wedge of water with a depth of Δh at the bank, tapering to zero depth at a distance 25 to 75 feet away from the stream, depending on the stage.

Before proceeding with the computation of the Clements hydrographs, the computed channel storage was increased by the amount of bank storage required to make the total volume of storage equal to the difference between recorded inflow and outflow volumes. The volume of bank storage added was distributed uniformly over the 12.7 miles of river channel. It is emphasized that this bank storage does not enter into the computation of wave velocity, because there is little downstream movement of water temporarily stored in the banks.

The procedure used in routing the Lancha Plana hydrograph to Clements can be described as a multiple-phase reservoir-type routing technique. The 12.7-mile reach was first divided into 4 or 5 subreaches of varying length. The lengths used were determined in the following manner. For a given surge, a graph of accumulated distance versus accumulated wave traveltime was prepared. This graph is not shown in the report, but it is merely a plot of the values found in two of the columns of table 1; namely, column 1 versus column 7. The length used for the upstream subreach was that value on the graph that corresponds to a wave traveltime equal to the time that elapsed at Lancha Plana during the change in discharge from Q_1 to Q_2. Lengths of the other subreaches were likewise determined from this graph, using time intervals that were approximately equal to one-fourth or one-fifth of the total traveltime between Lancha Plana and Clements. Thus, the time of arrival of the initial disturbance was known for each site marking the downstream end of a subreach.

For each subreach, a lag-and-route procedure was then used. It will be recalled that any change of inflow into a subreach that occurs while the initial disturbance is traveling to the downstream end of that subreach causes no change in outflow but merely changes the storage in the subreach. The traveltime of the initial disturbance in each subreach, therefore, constitutes the lag in outflow for the subreach. The subsequent routing, using the modified storage, was simplified by making the following two assumptions. First, storage in the subreach is uniquely related to the outflow (reservoir-type routing), and second, the storage-outflow relation for each subreach is linear. These assumptions usually lead to no serious error in computing the final outflow for a reach, if a sufficient number of subreaches are used and the change in stage is small.

The actual routing procedure used for each subreach was the semigraphical Goodrich method. No details of this method are given here, as a description of the Goodrich method is in most standard hydrology texts (for example, Johnstone and Cross, 1949, p. 167–171). Successive applications of the lag-and-route procedure to each subreach resulted in the computed hydrographs for Clements shown in figure 12. The agreement between the recorded and computed discharge is very satisfactory.

SUMMARY

For the purpose of this report, a surge is defined as a rapid change in discharge from one state of steady flow to another. The surges studied were both positive (rising) and negative (falling), and both types had extremely flat wave fronts.

It was found that the time of travel of the initial disturbance to steady flow can be computed reliably in accordance with Seddon's principle. Channel properties such as shape, dimension, roughness, and slope affect the velocity of the initial disturbance, and the variation of these factors produces changes in the velocity of the initial element of the surge as it traverses the reach. Given a sufficient number of rated cross sections that are favorably situated, it is possible to compute these changes in wave velocity, because the effect of channel geometry and roughness in a subreach is integrated in the stage-discharge relation for the subreach.

The wave form flattens progressively downstream as a result of channel storage. An actual measure of the varying channel storage between the profiles of steady flow for the initial and final discharges is provided by the rating curves for the cross sections in the reach. Because these rating curves represent the integrated effect of channel geometry and roughness, it follows that the volume of channel storage and consequently the degree of flattening of the wave form are affected by the hydraulic characteristics of the reach. Given the upstream hydrograph and sufficient intermediate rated cross sections, the downstream hydrograph, following the arrival of the initial disturbance, may be computed in accordance with the principle of conservation of mass; that is, the net change in volume of discharge equals the change in storage.

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