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	Engineering and Design HYDRAULIC DESIGN - SURGES IN CANALS - CHANGE 1	
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PART CXVI
HYDRAULIC DESIGN

CHAPTER 6
SURGES IN CANALS

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PART CXVI
HYDRAULIC DESIGN
CHAPTER 6
SURGES IN CANALS

6-01. SCOPE

This chapter is confined to the problem of surges which occur in navigation canals as a result of lock operations, since a full discussion of other unsteady flow phenomena which may occur in open channels would be feasible only in a large treatise.

a. **Surge Problems.** In canal design, the study of surges is necessary because of the following three problems which are given in their probable order of importance: (1) to an extent governed largely by traffic density; (2) surges which depress the water level reduce the effective channel depth, those which raise the water level encroach upon the freeboard; and (3) surges may impose sudden large loads upon miter gate operating machines. It should be emphasized that in most canals, there is no serious difficulty with surges. Unless traffic density is great, an existing surge problem can quite easily be alleviated. Nevertheless, some consideration of the surge problem is warranted in design, particularly for long, excavated canals controlled by high head locks.

b. **Limitations of Theoretical Analysis.** The theoretical laws of unsteady flow in open channels are well established but, except in a few fields such as flood wave transmission and tidal hydraulics, the techniques of applying the laws to practical problems are not yet well developed. Certain simplifying assumptions which are permissible in the study of canal surges are entirely inappropriate for the analysis of unsteady flow phenomena which may occur in channels used for other purposes. For example, surges of large amplitude cannot be tolerated in practical navigation canals nor can the occurrence of high velocities. Therefore, methods applicable exclusively to surges of small amplitude may be used. In navigation canals, the velocities of flow are so low that they may, for some purposes, be neglected. Limitations exist as to the accuracy which can be realized, regardless of the computation procedures adopted. This is true for several reasons; minor changes in canal cross section produce partial reflections which cannot be evaluated; traffic in the canal has a similar effect; interference between newly generated and reflected surges occurs. The latter problem would not be a serious obstacle if the strength of reflected surges were known, because the effects of two or more waves can readily be combined. However, basic data as to the rate of energy dissipation in unsteady flow are unavailable. For these reasons, and others discussed in detail in several of the references listed at the end of this text, the results of any analysis of canal surges should be treated as an index, comparable with similarly determined indices for canals in successful operation. To avoid presentation of ideas which cannot be supported by observed data, this chapter is confined to the case of level canals which are reasonably regular in form. Surges in canalized rivers are less important than those in canals because the river widths are generally much greater. This is fortunate because the analysis of the effects of major channel irregularities is very speculative. Traffic-induced surges in navigation canals are significant mainly from the standpoint of vessel power requirements and are not considered in this chapter. Surges which occur in lock chambers are likewise excluded as a special problem in lock design and are discussed in chapter 4 of this part.

6-02. TERMINOLOGY

Since the text involves the repeated use of terms which are not in common use, it appears advisable to introduce a few basic definitions at this point.

Surge is generally used to describe the monoclinical gravity wave of either elevation or depression which is generated by a sudden or rapid change in the discharge of an open channel. A surge profile is governed by the rate of change of discharge. In equalizing a navigation lock with an adjacent canal level, the initial head is great with the maximum discharge rate occurring as soon as the valves are fully opened. However, the head falls as the equalization proceeds and the discharge gradually decreases. Thus, the equalization produces not just one monoclinical wave but two, which are of opposite sign; one with a rather steep face which corresponds to the rapid initiation of discharge, and another with a very gentle slope, corresponding to the gradual reduction of discharge. The steep front causes a departure from the initial state of rest in the channel and the gentle front of opposite sign causes a restoration to the initial level and also to the original state of zero flow. Generally speaking, somewhat contrary to the usual meaning of the term, the entire solitary wave of either elevation or depression which is formed in a canal as a result of a lock operation may be called a surge. However, this solitary wave is really not one surge but two of opposite sign.

Positive surges elevate the water surface.

Negative surges depress the water level.

Celerity is the velocity of a wave or element thereof, relative to the velocity of flow. When a wave is propagated through still water, its absolute velocity is equal to the celerity but in moving water the absolute velocity may be either greater or less than the celerity.

Amplitude is the displacement of the water level due to a surge, measured vertically from the original water level.

6–03. GENERATION OF SURGES

a. **Small Surges of Constant Amplitude.**[1,3,4,5,13 *] Although the profiles of the surges generated by lock operations are of varying amplitude, it is instructive to consider first the generation of small surges of constant amplitude. Plate No. 1 presents the derivation of all formulas necessary in the analysis of small surges of constant amplitude. With the formulas presented, it is possible to determine the celerity, amplitude, and absolute velocity of all surges, whether positive or negative, propagated either upstream or downstream. The four possible cases are defined and illustrated on Plate No. 1. When the water is initially at rest, the appropriate case may be selected by making V_1 equal to zero, in which case the celerity and the absolute velocity of the surge are identical. Three equations for the celerity are presented; the method which involves no avoidable approximations and, therefore, called the exact equation, an approximate equation named after St. Venant, and another approximate equation named after Lagrange. The exact equation should be used in theoretical studies and for surges of great amplitude. St. Venant's equation is sufficiently accurate for surges of moderate amplitude. Lagrange's equation may be used for small surges. In a practical case, the three equations can easily be tested by trial computation.

Notation: The symbols used throughout the text and on the plates are listed at the end of the chapter.

Sign Conventions: In the derivations, shown on Plate No. 1, all depths, velocities, and discharges are considered positive; all differences have been written by deducting the lesser quantity from the greater so the differences also remain positive. Careful study of the definition sketches will prevent error in sign.

Continuity Law: The continuity law is used in the form applicable to unsteady flow problems. The shaded area of length S and y represents the change in cubic content of the channel in one

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*Numbers shown thus refer to references listed at end of this chapter. Wave mechanics is a problem in hydrodynamics which is not well adapted to elementary treatment. The text has been limited to readily comprehensible material. The references given compensate to some extent for the loss of exactitude incurred in simplifying the text to the greatest possible extent.

second. It is evident that the change in cubic content of the channel which occurs in one second must equal the difference between the discharges into and out of the channel.

Momentum Law: In applying the momentum law, a period of one second is considered. The applicable force is the difference in hydrostatic pressures upstream and downstream from the surge front. The velocity change is merely the difference between the velocities prior to passage and in the wake of the surge front. The mass to be used in the momentum law must be the mass which is subject to the velocity change in a period of one second. It might appear that the mass to be used in applying Newton's Second Law corresponds to the volume SD_1, but this is erroneous. A surge traveling downstream overtakes the flow at a rate $(S-V_1)$ and one traveling upstream encounters the flow at a rate $(S+V_1)$. Thus, the mass to be used in the momentum equation is $wD_1/g(S-V_1)$ for surges traveling downstream, and $wD_1/g(S+V_1)$ for surges traveling upstream.

Form of Solution: The momentum and continuity laws yield two simultaneous equations which may be solved in any desired manner. Generally, it is convenient to eliminate V_2 and to solve for C as done on Plate No. 1. However, other solutions may be more useful in some cases. The expression for C in terms of D_1 and D_2 cannot be solved directly, since D_2 is unknown; successive approximations provide the most convenient solution. Owing to the notation adopted, the expression for C is identical in form for Cases I, II, III, and IV. It should be noted carefully that in Cases I and III which deal with the positive surge, D_2 exceeds D_1, whereas in Cases II and IV, dealing with the negative surge, D_1 exceeds D_2.

Equations for Height of Surge: For all four cases, an expression is given for y, in terms of Q_1, Q_2, V_1 and C. It should be noted that it is not necessary to use the exact expression for C. Whenever St. Venant's and Lagrange's equations for celerity provide a sufficiently accurate value of C, that value of C may be used in the equations for surge height.

b. **Small Surges of Varying Amplitude.** The generation of small monoclinical wavelets by small discharge has been illustrated by Plate No. 1. Any gradual variation of discharge can be represented with any desired degree of precision as a succession of small instantaneous discharge changes. It is evident, therefore, that gradually varying discharge is no obstacle to the determination of resulting surge profiles. It is only necessary to compute the celerity of each of a series of wavelets and to take account of the varying velocity of flow. This method of determining a surge profile may be called the step method.[1,3,4,6] The step method may be used not only in determining surge profiles in navigation canals but also other surges. For a large and fairly rapid discharge change it may not be necessary to establish the surge profile. The exact equation for celerity may then be used directly to find the surge amplitude. The stationary hydraulic jump is merely a stopped finite surge, a phenomenon which can occur only when the velocity of the oncoming flow in Case III equals the celerity of the stopped surge.[4,5] S. M. Woodward and C. J. Pasey[14] discuss the moving hydraulic jump and show that the absolute velocity of the jump is governed by the celerity and the velocity of flow. The jump may move either upstream or downstream. The case of the jump moving downstream is not covered on Plate No. 1, but is governed by principles identical to those used in that illustration. While these problems are not directly pertinent to the study of surges in navigation canals they do present some interesting aspects which clarify the over-all problem and the concept of celerity and its relationship to absolute velocity. For the gradually varying discharges which generate surges in navigation canals, either the step method or a quick approximate method, as described below, should be used in obtaining surge profiles. In the step method, each of a series of small discharge changes is treated by the principles developed for surges of constant amplitude. However, because the surges which occur in practical navigation canals are small, the effect of neglecting variations in the celerity with changes in depth is also rather small. Furthermore, the celerity varies with the square root of the depth so the percentage variation in celerity is about one-half as great as the percentage variation in depth. The Lagrange equation, which is adopted herein for all celerity computations, gives the following variation of celerity with depth:

Depth (ft.)	*Celerity (ft. per sec.)*
8	16.1
10	18.0
12	19.7
15	22.0
20	25.4
25	28.4
30	31.1
40	35.9
50	40.2

It is clear that celerities are very high in comparison with the velocities which can be permitted to occur in a navigation canal. However, the maximum velocity of flow which accompanies the passage of a surge caused by a lock operation need not be very high to cause some trouble in navigation. The velocity has a very sudden onset which a pilot cannot anticipate. Thus the velocity of flow due to a surge, despite its small magnitude and its relatively short duration, is much more dangerous than a comparable velocity which is constant or which arises gradually like a tidal current. Sometimes, and this is particularly true of deep canals, the velocity of flow is so low in comparison with the celerity that a reasonably correct surge profile can be obtained by use of the assumptions that:

The effect of depth variation upon the celerity of all elements of the surge profile is negligible. On the basis of this assumption, all elements of the profile are assumed to be transmitted at the celerity corresponding to the original depth, using the Lagrange equation.

The effect of flow velocity upon the absolute velocity of all elements of the surge profile is negligible. On the basis of this assumption, the celerity is treated as the absolute velocity of all elements of the surge.

These assumptions and the approximate method based thereon should not be used unless the limitations of the method are clearly appreciated. However, the approximate method is very convenient whenever its use is permissible. The deeper the canal and the lower the velocities generated, the smaller becomes the error in use of the approximate method. Presentation of the approximate method in this chapter is justified in part by the fact that the variations in the surge profile, subsequent to generation, cannot be analyzed with precision.

c. **Illustrative Example of Surge Generation.** To aid in the use of the step method and the approximate method, illustrative examples have been prepared.

The following basic data are assumed:

Lock Chamber Dimensions:
- Length .... 600 ft. between gates.
- Width .... 110 ft. between walls.
- Lift .... 30 ft.

Canal Dimensions:
- Mean Depth .... 14 ft.
- Surface Width .... 200 ft.

Timing Factors:
- Equalization Period .... 8 minutes.
- Valve Operation Period .... 1 minute.

Since a canal of relatively shallow depth; a lock of rather high lift, and short equalization and valve operation periods have been selected for the example, the comparison is unfavorable to the approximate method. That is, the difference between the surge profiles indicated by the comparison is about as great as will occur in any practical problem. Plate No. 2 shows the principal curves for the example. Figure *a* of that plate shows the water level in the chamber as a function of time for the operation of equalization with the lower pool. This curve was obtained by use of differential equations of discharge under falling head. In a practical case, a comparable curve might be obtained from model tests. For simplicity, inertial effects have been ignored in deriving the curve; therefore, the typical "overtravel" of levels is absent. Plate No. 2, Figure *b* shows the discharge out of the chamber as a function of time. Plate No. 2, Figure *c* shows the surge profile in the canal downstream from the lock at the instant when equalization is completed. Amplitudes obtained by both the approximate method and the step method are shown. The approximate method is illustrated in Table 1 and the step method in Table 2.

Approximate Method: Recalling that the approximate method is based on the assumptions of *constant* celerity and treatment of the celerity as an absolute velocity, the surge profile may be fully determined by the application of Lagrange's equation and the law of continuity to a series of arbitrarily selected discharges sufficient to define the discharge hydrograph.

The celerity given by Lagrange's equation is

* $$C=\sqrt{gD_1}=21.2 \text{ ft. per sec.}$$

Since, in the approximate method, the absolute velocity of all elements of the surge is constant and equal to the celerity, the amplitude of any element of the surge is given by the equation for surge amplitude (Case I):

* $$y=\frac{Q_2-Q_1}{C+V_1}$$

Since all data in Plate No. 1 are based on unit width, the discharge must be divided by the surface width of the canal before using the equations. In the present example, the width is 200 ft. and Q_1 and V_1 are zero so:

$$y=\frac{Q_2}{(200)(21.2)}=\frac{Q_2}{4240}$$

The station of each amplitude at the end of the equalization period is simply the product of the celerity and the travel time (in seconds), that is, the time which has elapsed since generation of the amplitude in question. For a negative surge, generated when the lock is equalized with the upper pool, the calculations are identical, in the approximate method, but the surge amplitude is measured below the original level. The surge profile determined by the approximate method is always identical in shape to the discharge hydrograph.

Step Method: In the step method, as in the approximate method, a number of discharges are arbitrarily chosen from the hydrograph. However, the discharges should be quite closely spaced on the hydrograph so that the heights of the individual wavelets are small. In the illustrative computations, the celerity and velocity of each wavelet are based upon the preexisting velocity, and depth, using the Lagrange equation. It would be more accurate to use the St. Venant equation, particularly in the portion of the computation in which rather large depth and velocity changes occur. However, the St. Venant equation requires repeated trial computations because the celerity is dependent upon the amplitude which is unknown. The error due to use of the Lagrange equation is tolerable for the study of navigation canal surges, but it should be recognized that an approximate method is being used. The columns of Table 2 are filled in on the basis of the following instructions:

Col. 1—Generation Time: Enter the time from the inception of the occurrence of each of the arbitrarily assumed sudden discharge changes.

Col. 2—Discharge: Enter the discharges corresponding to the times shown in col. 1.

Col. 3—Discharge Increments: Enter the discharge changes, using positive signs for increased discharge and negative signs for decreased discharge. (All discharge changes were considered positive in Plate No. 1, but in tabular computations it is convenient to use plus and minus signs to denote respectively increments and decrements of discharge.)

Col. 4—Initial Depth: Enter the depth which obtains prior to the discharge change, D_1. Note that the initial depth, D_1 of any step corresponds to the changed depth D_2 of the preceding step.

Col. 5—Celerity: Enter the celerity corresponding to D_1 using Lagrange's equation.

Col. 6—Initial Velocity: Enter the initial velocity, V_1, Corresponding to Q_1 and D_1. (All discharges shown on Plate No. 1 are expressed in cubic feet per second per foot of width. In this example, the width is 200 ft., so the discharges per unit width are determined by inspection).

Col. 7—Surge Velocity: Enter the absolute surge velocity, which is the sum of C and V_1 for Cases I and II, the difference between C and V_1 for Cases III and IV.

Col. 8—Amplitude: Enter the amplitude of each element of the surge, which is equal to the discharge change divided by S, in all cases. (In Plate No. 1, the amplitude, y, was always considered positive but, in tabular computations it is convenient to use plus and minus signs to denote, respectively, increments and decrements of depth).

Col. 9—Changed Depth: Enter the depth subsequent to the depth change, D_2, which is the algebraic sum of D_1 and y.

Col. 10—Accumulated Amplitude: Enter the accumulated amplitude which is the algebraic sum of all previously occuring values of y.

Col. 11—Travel Time: Enter the time which remains until the end of the discharge changes.

Col. 12—Station: Enter the product of the absolute surge velocity and the travel time.

TABLE 1.—*Determination of surge profile by the approximate method*

(1) T_g Generation time (sec.)	(2) Q_1 Discharge (cu. ft. per sec.)	(3) y Amplitude (ft.)	(4) T_t Travel time (sec.)	(5) L Station (ft.)
0	0	0	480	10, 200
30	4, 400	1. 04	450	9, 550
60	8, 220	1. 94	420	8, 920
120	7, 060	1. 66	360	7, 650
180	5, 870	1. 38	300	6, 370
240	4, 600	1. 11	240	5, 100
300	3, 520	. 83	180	3, 820
360	2, 350	. 55	120	2, 550
420	1, 180	. 28	60	1, 270
480	0	0	0	0

TABLE 2.—*Determination of surge profile by the step method*

(1) T_g Generation time (sec.)	(2) Q Discharge (cu. ft per sec.)	(3) Q_2-Q_1 Discharge increments (cu. ft. per sec.)	(4) D_1 Initial depth (ft.)	(5) C Celerity (ft. per sec.)	(6) V_1 Initial velocity (ft. per sec.)	(7) S Surge velocity (ft. per sec.)	(8) y Amplitude (ft.)	(9) D_2 Changed depth (ft.)	(10)* Σy Accumulated amplitude (ft.)	(11) T_t Travel time (sec.)	(12) L Station (ft.)
0	0	-----	14. 00	21. 2	0	21. 2	----	14. 00	0	480	10, 200
20	2, 900	2, 900	14. 00	21. 2	0	21. 2	0. 68	14. 68	. 68	460	9, 750
40	5, 700	2, 800	14. 68	21. 7	1. 0	22. 7	. 62	15. 30	1. 30	440	10, 000
60	8, 200	2, 500	15. 30	22. 2	1. 9	24. 1	. 52	15. 82	1. 82	420	10, 100
90	7, 600	−600	15. 82	22. 6	2. 7	25. 3	−. 12	15. 70	1. 70	390	9, 820
120	7, 100	−500	15. 70	22. 5	2. 4	24. 9	−. 10	15. 60	1. 60	360	8, 970
150	6, 500	−600	15. 60	22. 4	2. 3	24. 7	−. 12	15. 48	1. 48	330	8, 150
180	5, 900	−600	15. 48	22. 4	2. 1	24. 5	−. 12	15. 36	1. 36	300	7, 360
240	4, 700	−1, 200	15. 36	22. 2	1. 9	24. 1	−. 25	15. 11	1. 11	240	5, 790
300	3, 500	−1, 200	15. 09	22. 1	1. 5	23. 6	−. 25	14. 86	. 86	180	4, 270
360	2, 300	−1, 200	14. 84	21. 9	1. 2	23. 1	−. 26	14. 60	. 60	120	2, 780
420	1, 200	−1, 100	14. 58	21. 7	. 8	22. 5	−. 24	14. 36	. 36	60	1, 350
480	0	−1, 200	14. 34	21. 4	. 4	21. 8	−. 28	14. 08	. 08	0	0

*With reference to Figure c on Plate No. 3 and column 10 above, it will be noted that in this method the "accumulated amplitude" is used in the development of the Surge Amplitude Curve.

In Plate No. 2, Figure *c* it is shown that the difference in profiles obtained by the two methods is quite small. As previously mentioned, the difference will be smaller in a deeper canal or when the lock lift is smaller in comparison with the canal depth. This is true because the amplitude-depth ratio and the velocity-celerity ratio will be smaller. Case I and Case II in Plate No. 1 formulas are applicable to the surges generated downstream from the lock; Case III and Case IV formulas should be used for the surge generated upstream from the lock.

6-04. REFLECTION CONDITIONS

Surges may be reflected positively, negatively, or partially. In the latter case, the reflection may have either a positive or negative sign.

a. **Positive Reflection.** When a surge or any other wave form encounters a barrier, such as a dead end, the wave reverses its direction and travels back upon itself without change in sign. This is termed positive reflection. Thus, a positive wave is reflected as a positive wave, a negative wave as a negative wave. This phenomenon is known as positive reflection. The resultant profile and the velocity conditions in the neighborhood of the reflecting surface may be obtained by algebraic combination of the effects of the reflected and the oncoming waves.[5] At the reflecting surface, the amplitudes of oncoming and reflected waves correspond to the same point on the profile. Therefore, a graph of the stage at the reflecting surface, plotted as a function of time, is proportional to a stage-time graph at a remote point in the channel but the ordinates at the reflecting surface are twice as great.

b. **Negative Reflection.** When a surge or other wave form encounters a very large increase in the channel width, such as a canal entrance to a sea or large lake, the surge is reflected negatively. Thus, a positive wave is reflected as a negative wave and vice versa. There is no simple mechanical analogue for negative reflection, making it difficult to visualize; but H. Rouse[6] offers a good explanation of the phenomenon and E. I. Brown[8] provides an example of application in a practical problem. As in the case of positive reflection, the resultant profile in the neighborhood of the reflecting surface may be obtained by algebraic combination of the effects of oncoming and reflected profiles. A curious phenomenon occurs when a wave form symmetrical about its midpoint is reflected negatively. When the midordinate of the profile is being reflected, the resultant profile throughout its entire length is momentarily of zero amplitude. An observed imperfect example of this phenomenon is shown in the papers of F. W. Edwards and E. Soucek.[10] In complete negative reflection, which requires a very large expansion, no change in the elevation of the water surface occurs at the reflecting section.

c. **Partial Reflection.** Complete positive reflection corresponds to the dead end and complete negative reflection corresponds to the very large sudden expansion. These cases are the defining limits of infinite contraction and infinite expansion respectively. It is apparent that finite expansions and contractions must correspond to intermediate reflection conditions. It may be further noted that no reflection occurs in a channel of uniform cross section so such a channel represents a borderline case between partial positive and partial negative reflection. C_r, which may be called a reflection coefficient, is defined as the ratio of the reflected amplitude to the original oncoming amplitude of any wave form. Certain conclusions may at once be drawn as to the effect of width changes upon the reflection coefficient. Let n represent the ratio of width of the reflecting channel to the width in which the oncoming wave is propagated. Thus, if b represents the width of channel in which the surge approaches the reflecting section, (nb) is the width of the reflecting section. The same depth is assumed in both channels.[8, 15] The relationship between n and C_r is shown below.

When

$n=1.0$	$C_r=$ zero.
$n=$ zero	$C_r=+1.0$.
n is infinite	$C_r=-1.0$.
$n>1.0$	C_r is negative.
$n<1.0$	C_r is positive.

To satisfy the five known conditions concerning the variation in the reflection coefficient with the channel width ratio, the following expression may be written by inspection:

$$C_r = \frac{1-n}{1+n}$$

This equation agrees with derivations arrived at by E. I. Brown[8] and H. Lamb.[15] It should be noted that Brown's equation gives the resultant amplitude, that is the sum of oncoming and reflecting amplitude, not the reflected amplitude directly. The general problem of reflection and transmission coefficients for section changes other than simple width changes is beyond the scope of this chapter.

6–05. STABILITY AND LONGEVITY OF SURGES

A freshly generated surge profile can be determined with sufficient accuracy for practical needs. The manner in which a surge is reflected by a width change can be analyzed without difficulty. However, the apparently simpler problem of determining the changes which occur in the profile of a surge as it traverses a canal is not on a satisfactory basis for practical applications. Since problems in stability and longevity of surges are too complex for treatment in this chapter, study of the references is suggested. Boussinesq, [5,8] has investigated the stable solitary, positive, gravitational wave, his equation involves not only amplitude and depth but also the surface curvature. From this study and a similar one by Rayleigh,[5] it is possible to obtain the profile of the positive solitary wave of stable form, that is, one whose elements all have the same celerity. It has been suggested,[5] that the equations of Boussinesq and Rayleigh be used for predicting, at least qualitatively, the trend of profile variation for profiles which do not correspond to the stable form. This possibility, no doubt, has some application. Essentially stable profiles have been observed to travel many miles with inappreciable change, yet the elementary expressions for celerity, based only upon depth and amplitude, require a positive solitary wave to develop an abrupt front face and an ever-flattening back face. This characteristic of the elementary expressions for celerity was exhibited by the step method calculation of a surge profile in the illustrative example of surge generation. Instead of quickly developing an abrupt front as the computation indicates, it appears that the surge develops a gently rounded profile of fairly stable form.[10] Abrupt fronts, which should be clearly visible, have not been observed in navigation canals. Furthermore, it has been shown [5, 13] that an abrupt breaking surge of the shock type can occur only when the surge amplitude exceeds the depth. For the amplitude-depth ratios which occur in navigation canals, surges assume the smooth undular form. For these reasons, it may be concluded that a solitary wave generated by lock operations develops a fairly stable form, particularly in the case of the positive wave. It is known that a solitary wave of depression is less stable than a solitary wave of elevation. As an aid to the judgment in estimating the stability of wave forms, the observations of J. Scott Russell,[6] although made more than 100 years ago, are of considerable value. The problems of wave stability are too complex for complete treatment in this chapter, being within the province of hydrodynamics.[15] Even if a stable form is assumed, losses of energy, however small, must occur because water in motion is involved. Ultimately, the expenditure of energy must reduce the concentration of a solitary wave about its mid-point. No satisfactory theoretical basis is available for determining energy losses and the corresponding profile changes. Some of Russell's observations showed the decrease in maximum amplitude of a solitary wave as a function of distance traversed.[5, 6] Additional data are needed to place the problem of energy loss in solitary waves on a sound basis. If the rate of energy loss were determinable, the corresponding profile changes could be determined because the potential and kinetic energies of waves can be evaluated.[6, 8, 15] The questions of stability and longevity of surges are of considerable practical importance in the case of navigation canals. A surge problem of greater magnitude will exist if the surges generated tend to retain their strength and integrity over long periods of time than if the surges tend to become undular, break up, and lose their strength. This is true because the synchronism of the effects of newly generated and reflected surges may create severe conditions.[10] It is, therefore, unfortunate that a practical basis for estimating the longevity of surges cannot be suggested at this time. Observation of surge conditions, perhaps purposefully created, in existing canals is at present the best basis for estimating not only the stability of surge profiles but the longevity of surges as well. Only in this manner can the effects of the minor partial reflections, which occur in unlined canals and numerous other uncertainties, be reliably evaluated.

6-06. SURGE CONTROL

If the costs of surge control are prohibitive or if control is otherwise impracticable, the only solution is to anticipate, evaluate, and make the best design possible for the conditions discussed in paragraph 6-01 *a*. If, however, some positive action is warranted to prevent or alleviate a surge problem, the following specific methods should be considered.

a. **Reduction in Equalization Rate.** It has been shown that surge amplitudes are, in general, proportional to discharge rates. Therefore, a surge can be limited to any desired degree by increasing the lock equalization period. In a new design, this would be done by providing culverts of low capacity; in an existing canal, the culverts would be throttled in any suitable manner. Delay to traffic is the only objection to the use of this method. Since this solution is the only one which can be adopted at very small cost, it should always be considered very carefully.

b. **Increase in Channel Dimensions.** An increase in channel depth increases the celerity and reduces the surge amplitude; an increase in width decreases the surge amplitude without change in celerity. In either case, the velocity of flow due to a surge is reduced as is also the collision hazard. The cost is usually an important objection to the use of generous channel dimensions for surge control. Of course, if large channel dimensions are needed for any other purpose, the surge problem is benefited incidentally.

c. **Surge Control Reservoirs.** The following discussion of a surge control reservoir covers only the control of surges in an upper pool; it is evident that a similar application is possible for equalization of the lower pool. The surge control reservoir has small conduits or other restricted inlets joining the reservoir and the upper pool. The lock has culverts leading to both the reservoir and to the upper pool directly. In raising the level in the chamber, the water is first drawn to the greatest feasible extent from the reservoir but the final equalization is accomplished with water taken from the upper pool. The relative amounts drawn from the two sources depend upon the reservoir area. To prevent excessive equalization periods, the inflow to the lock can be switched from the reservoir to the upper pool before the lock and reservoir has been fully equalized. The small outlets connecting the reservoir and upper pool should be proportioned or controlled so as to just refill the reservoir prior to the next lockage. Thus, the rate of draft of the reservoir upon the upper pool is very low and the surges due to refilling of the reservoir are negligible. The principal objections to the use of a surge control reservoir are the costs of the reservoir itself and the cost of the additional culverts, valves, and controls. However, in an important canal, the use of a surge control reservoir may be feasible, particularly if favorable topographic conditions exist. A surge control reservoir was considered in 1940 for the Panama Canal Third Locks project. The model tests indicated that the reservoir would have been very effective.

d. **Channel Expansions.** The phenomenon of partial negative reflection at a channel expansion suggests the possibility that such expansions could be used to limit surge amplitudes. When the surge encounters the expansion, which should be located very near the lock, partial negative reflections will occur. Since the sign of the negatively reflected wave is opposite to that of the oncoming wave, some cancellation will occur. It is necessary to consider the transmitted component which enters the expansion and its positive partial reflection from the contraction following the expansion in a complete analysis. The effect of a channel expansion can be determined only by carefully combining the effects of all components in accordance with the theory of partial reflections. It is evident, however, that some beneficial effects can be obtained in this manner.

In a favorable topographic situation the use of channel expansions might be a feasible method of surge control, although no purposeful application of this method is known.

6-07. MODEL TESTS

Since the generation, propagation, and reflection of surges are essentially dynamic and gravitational phenomena, model tests based on Froude's Law can be used in the study of surges. Until the laws of energy loss in waves are better known, some question will exist as to the accuracy with which the longevity of surges is represented by a model test. However, the results are probably as good or better than those obtainable by any other method. Distortion is permissible for some purposes, for example studies of the effects of channel expansions. For tests which must represent the behavior of model vessels at junctions and bends, the model should be undistorted. Distortion is also inadvisable in any study in which the longevity of surges is an important factor. Observational sensitivity is the most important criterion governing the model size. The wave forms generated by lock operations are so long that surface tension probably has little effect, even if the model is quite small. However, if a model surge assumes an undular form, surface tension might affect the celerity. Stilling wells or other devices which involve time lag cannot be used. Direct observation of water levels on streamlined staff gages is feasible; good results were obtained in a model prototype comparison.[10] The very best sensitivity which can be expected from directly observed gates when the stage changes rapidly is probably about 0.002 ft. Thus, if the prototype surges are to be represented to the nearest 0.1 ft., the scale ratio, model to prototype, should not be less than about 1:50. In determining the model scale, the characteristics of the phenomena should be considered carefully. Waves which have a length of only several inches in the model are affected by surface tension and do not, therefore, behave as do corresponding prototype waves. While the number of available model-prototype comparisons is small, available data [10] and theoretical considerations indicate that model studies of surge problems should be quite reliable and trustworthy.

LIST OF SYMBOLS

b =width of approach channel
C =celerity, defined as relative velocity of surge measured with respect to V_1
C_r =surge or wave reflection coefficient
D =depth of water in channel, with subscripts denoting
  $_1$=prior to passage of surge
  $_2$=in wake of surge
g =acceleration of gravity
L =distance
n =ratio of width of reflecting channel to width in which oncoming wave is propagated
Q =discharge in channel, with subscripts denoting
  $_1$=prior to passage of surge
  $_2$=in wake of surge
S =absolute velocity of surge
T =time, with subscripts denoting
  g=of generation
  t=of travel
V =velocity in channel, with subscriptions denoting
  $_1$=prior to passage of surge
  $_2$=in wake of surge
w =unit weight of water
y =difference between depths upstream and downstream from surge
$\sum$ =algebraic summation
$\approx$ =is approximately equal to

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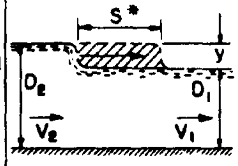
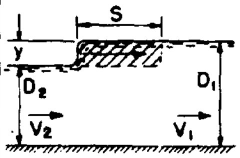
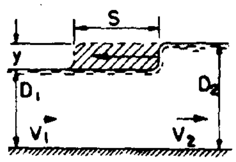
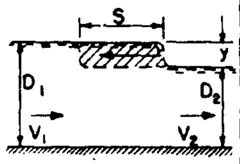
LIST OF TABLES

LIST OF ILLUSTRATIONS

Notes on Derivations	CASE I Positive Surge Moving Downstream	CASE II Negative Surge Moving Downstream	CASE III Positive Surge Moving Upstream	CASE IV Negative Surge Moving Upstream
ILLUSTRATION AND DEFINITION SKETCH				
CONTINUITY LAW Do. (IN TERMS OF V_2)	$S(D_2-D_1)=V_2D_2-V_1D_1$ $V_2=\frac{1}{D_2}(SD_2-SD_1+V_1D_1)$	$S(D_1-D_2)=V_1D_1-V_2D_2$ $V_2=\frac{1}{D_2}(V_1D_1-SD_1+SD_2)$	$S(D_2-D_1)=V_1D_1-V_2D_2$ $V_2=\frac{1}{D_2}(V_1D_1-SD_2+SD_1)$	$S(D_1-D_2)=V_2D_2-V_1D_1$ $V_2=\frac{1}{D_2}(SD_1-SD_2+V_1D_1)$
FORCE ($w=62.5$) MASS ($g=32.2$)	$\frac{w}{2}\left[(D_2)^2-(D_1)^2\right]$ $w\frac{D_1}{g}(S-V_1)$	$\frac{w}{2}\left[(D_1)^2-(D_2)^2\right]$ $w\frac{D_1}{g}(S-V_1)$	$\frac{w}{2}\left[(D_2)^2-(D_1)^2\right]$ $w\frac{D_1}{g}(S+V_1)$	$\frac{w}{2}\left[(D_1)^2-(D_2)^2\right]$ $w\frac{D_1}{g}(S+V_1)$
VELOCITY CHANGE Do. (WITH V_2 ELIMINATED)	(V_2-V_1) $(S-V_1)\left(\frac{D_2-D_1}{D_2}\right)$	(V_1-V_2) $(S-V_1)\left(\frac{D_1-D_2}{D_2}\right)$	(V_1-V_2) $(S+V_1)\left(\frac{D_2-D_1}{D_2}\right)$	(V_2-V_1) $(S+V_1)\left(\frac{D_1-D_2}{D_2}\right)$
MOMENTUM LAW (FORCE EQUALS MASS x ΔV) Do. (SOLVED FOR CELERITY)	$\frac{1}{2}(D_2-D_1)(D_2+D_1)$ $=\frac{D_1}{g}(S-V_1)^2\left(\frac{D_2-D_1}{D_2}\right)$ $C=(S-V_1)$ $C=\sqrt{\frac{gD_2}{2D_1}(D_1+D_2)}$	$\frac{1}{2}(D_1-D_2)(D_1+D_2)$ $=\frac{D_1}{g}(S-V_1)^2\left(\frac{D_1-D_2}{D_2}\right)$ $C=(S-V_1)$ $C=\sqrt{\frac{gD_2}{2D_1}(D_1+D_2)}$	$\frac{1}{2}(D_2-D_1)(D_2+D_1)$ $=\frac{D_1}{g}(S+V_1)^2\left(\frac{D_2-D_1}{D_2}\right)$ $C=(S+V_1)$ $C=\sqrt{\frac{gD_2}{2D_1}(D_1+D_2)}$	$\frac{1}{2}(D_1-D_2)(D_1+D_2)$ $=\frac{D_1}{g}(S+V_1)^2\left(\frac{D_1-D_2}{D_2}\right)$ $C=(S+V_1)$ $C=\sqrt{\frac{gD_2}{2D_1}(D_1+D_2)}$
AMPLITUDE (BASED ON CONTINUITY LAW)	$y=\frac{Q_2-Q_1}{C+V_1}=\frac{\Delta Q}{S}$	$y=\frac{Q_1-Q_2}{C+V_1}=\frac{\Delta Q}{S}$	$y=\frac{Q_1-Q_2}{C-V_1}=\frac{\Delta Q}{S}$	$y=\frac{Q_2-Q_1}{C-V_1}=\frac{\Delta Q}{S}$

* The shaded area of length S and height y represents the change in cubic content of the channel in one second.

ST. VENANT AND LAGRANGE CELERITY EQUATIONS

Both derived from exact equation for celerity

$$C=\sqrt{\frac{gD_2}{2D_1}(D_1+D_2)}$$

CASES I AND III — Positive Surges

$$D_2=D_1+y$$

$$C=\sqrt{\frac{g(D_1+y)(2D_1+y)}{2D_1}}$$

$$C=\sqrt{\frac{g\left[2(D_1)^2+3D_1y+y^2\right]}{2D_1}}$$

$$C=\sqrt{g\left(D_1+\frac{3}{2}y+\frac{1}{2}\frac{y^2}{D_1}\right)}$$

$$C=\sqrt{gD_1}\sqrt{1+\frac{3y}{2D_1}+\frac{1}{2}\left(\frac{y}{D_1}\right)^2}$$

$$C\approx\sqrt{gD_1}\sqrt{1+\frac{3}{2}\frac{y}{D_1}}$$

$$C\approx\sqrt{gD_1}\left(1+\frac{3}{4}\frac{y}{D_1}\right)$$

St. Venant's Equation

CASES II AND IV — Negative Surges

$$D_2=D_1-y$$

$$C=\sqrt{\frac{g(D_1-y)(2D_1-y)}{2D_1}}$$

$$C=\sqrt{\frac{g\left[2(D_1)^2-3D_1y+y^2\right]}{2D_1}}$$

$$C=\sqrt{g\left(D_1-\frac{3}{2}y+\frac{1}{2}\frac{y^2}{D_1}\right)}$$

$$C=\sqrt{gD_1}\sqrt{1-\frac{3y}{2D_1}+\frac{1}{2}\left(\frac{y}{D_1}\right)^2}$$

$$C\approx\sqrt{gD_1}\sqrt{1-\frac{3}{2}\frac{y}{D_1}}$$

$$C\approx\sqrt{gD_1}\left(1-\frac{3}{4}\frac{y}{D_1}\right)$$

St. Venant's Equation

Whenever $\frac{3}{4}\frac{y}{D_1}$ is negligible in comparison with unity, for either positive or negative surge, it is permissible to use Lagrange's Equation $C=\sqrt{gD_1}$

SURGE ANALYSIS

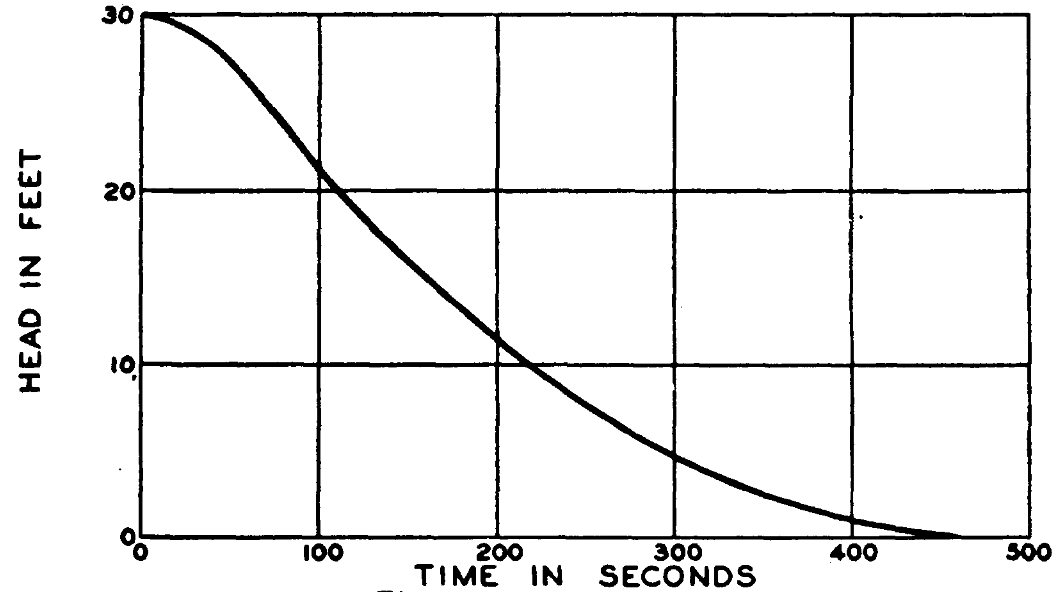


Fig.(a) - WATER LEVEL

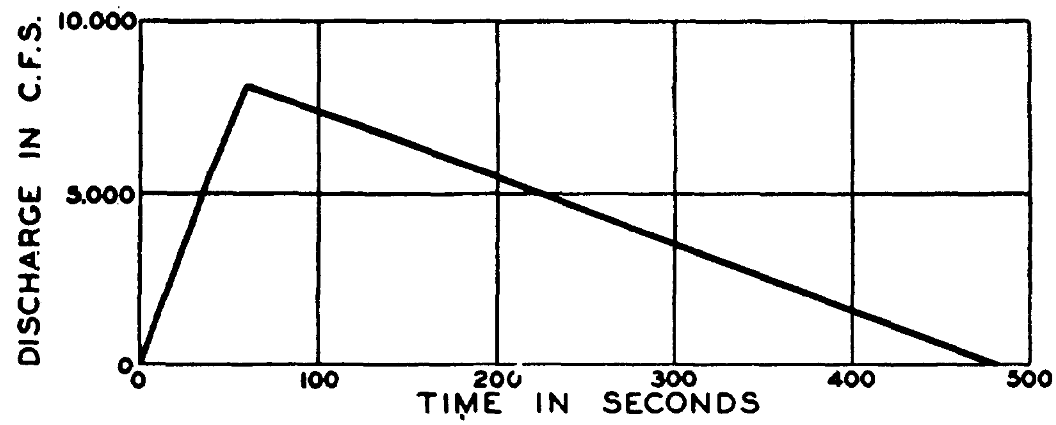


Fig.(b) - HYDROGRAPH

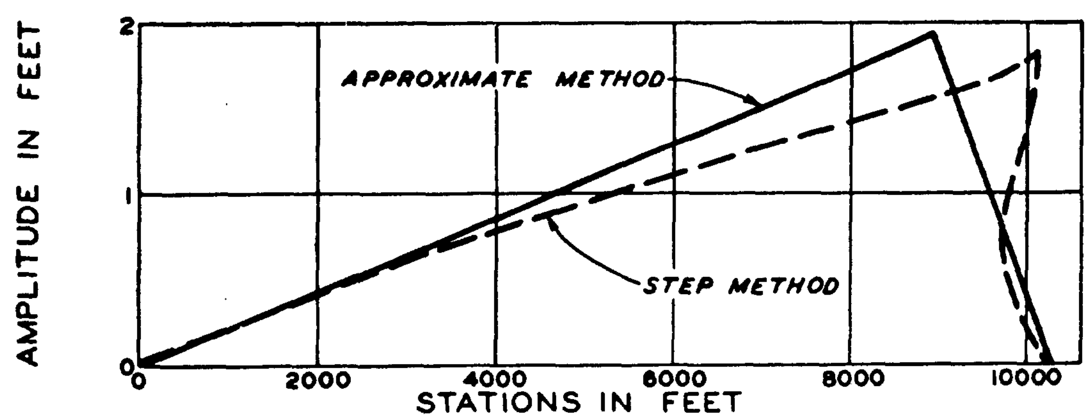


Fig.(c)-SURGE AMPLITUDE

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