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SURGES IN A TRAPEZOIDAL CANAL DUE TO PUMP FLOW REJECTION

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SYNOPSIS

This paper describes hydraulic model studies of surge formation and propagation in a large trapezoidal canal supplying water to a pumping plant, following rejection of the canal flow due to pump stoppage caused by power failure. The studies were conducted in the Hydraulic Laboratory of the Division of Research, Bureau of Reclamation, Denver, Colorado.

The forms of the surge waves resulting from complete canal flow rejection with and without the superimposed surge due to discharge line backflow were investigated. Characteristics such as average bore heights, heights of maximum oscillation peaks, wave velocities, and wave lengths of the surge were determined for complete rejection of the inflow.

The attenuating effect of a 1,500-foot- (457.2-meter-) long weir on the canal side slope was determined for various values of the Froude number of the canal flow.

RESUMÉ

Ce rapport décrit les études-modèles hydrauliques sur la formation de lame et sa propagation dans un grand canal en forme de trapèze fournissant l'eau à une usine de pompage, sur le rejet de l'écoulement du canal, dû à l'arrêt de pompage, causé par une panne d'électricité. Les études furent dirigées dans le Laboratoire Hydraulique du Département de Recherche, Bureau of Reclamation, Denver, Colorado.

Des enquêtes très précises furent poursuivies en vue de déterminer les formes des lames de fond résultant du rejet complet de l'écoulement du canal avec ou sans lame de fond superposée dûe au courant inverse. Les hauteurs moyennes de la barre d'eau, les hauteurs des pointes maximum d'oscillations, les différentes vitesses de lame et les longueurs de lame constituèrent des éléments caractéristiques, qui furent déterminés pour le rejet complet du courant.

L'effet atténuant d'un long barrage de 1500 pieds (soit 457,2 mètres) sur les rives d'un canal fut déterminé pour différentes valeurs du Nombre Froude de l'écoulement d'arrivée initial.

INTRODUCTION

The San Luis Unit of the Bureau of Reclamation's Central Valley Project in California includes a system to store surplus water during the winter for release during the summer months. The Forebay Canal and Forebay Pumping Plant, as part of this system, will divert water from the existing Delta-Mendota Canal at a maximum rate of 4,200 cfs (118.9 cms) into the Forebay Reservoir. The water will then be lifted by pump-generator units into the San Luis Reservoir. Subsequent releases back into the Forebay Reservoir will generate power and provide required irrigation flows.

Although safeguards have been included in the design, a remote possibility exists that power failure might occur at the Forebay Pumping Plant, resulting in a stoppage of the pumps. Should such a power failure occur, a surge wave would be propagated upstream in the Forebay Canal due to rejection of the canal flow and backflow drainage from the pump discharge lines. This surge, if allowed to travel unreduced through the Delta-Mendota Canal, would overtop the concrete lining and necessarily result in costly additions to the freeboard.

Alternative methods of reducing the surge to an allowable height were considered. The first alternative consisted of radial gates located in the bifurcation from the Delta-Mendota Canal to the Forebay Canal. These gates would open automatically upon power failure at the pumping plant, draw down the water surface to accommodate the initial surge wave, and remain open to divert the rejected canal discharge. The second alternative, which was included in the model investigation, consisted of a weir along the side of the trapezoidal Forebay Canal which would reduce the surge to an allowable value before reaching the bifurcation. The side weir has the advantages of essentially maintenance-free operation and freedom from reliance on mechanical devices.

Citrini[1] has developed a theoretical approach to the action of a lateral spillway in reducing the height of a positive surge in a rectangular channel, and DeMarchi[2] and Gentilini[3] have presented supporting experimental data.

THE EXPERIMENTAL MODEL

The 1:48 scale model included the Forebay Canal and the transition to the pumping plant intakes, the bifurcation from the Delta-Mendota Canal to the Forebay Canal, and several hundred feet of the Delta-Mendota Canal as shown in Figure 1. The discussion presented in this paper will be limited to the formation, propagation, and attenuation of the surge in the Forebay Canal. The model canal section had a bottom width of 20 inches (50.8 cm) with 1-1/2:1 side slopes, an average flow depth of approximately 3.75 inches (9.6 cm), and a length of approximately 48 feet (15 m). The maximum model discharge was 0.263 cfs (7.45 l/s) corresponding to a prototype discharge of 4,200 cfs (118.9 cms).

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[1]Citrini, Giulio. "Sull' attenuazione di un'onda positiva ad opera di uno sfioratore laterale (attenuation of a positive wave by means of a lateral spillway)." L'Energia Elettrica, Milano, Volume XXVI, No. 10, 1949.

[2]DeMarchi, Giulio. "Action of side weirs and tilting gates on translation waves in canals." Proceedings of the Minnesota International Hydraulics Conference, August 1953.

[3]Gentilini, Bruno. "L'Azione Di Uno Sfioratore Laterale Sull'onda Positiva Ascendente In Un Canale (The action of a side weir on the positive wave moving upstream in an open channel)." Memorie e Studi Dell'Istituto Di Idraulica e Costruzioni Idrauliche Del Politecnico Di Milano. Centro Lombardo Di Ricerche Idrauliche Del Consiglio Nazionale Delle Ricerche, No. 78, 1950.

Rejection of the canal inflow was accomplished by rapid closure of slide gates located at the pump intakes. Backflow from the pump discharge lines was simulated by head tanks which were allowed to drain through orifices sized to produce the required maximum backflow rate.

Tests showed that flow depths of less than approximately 0.2 inch (5 mm) over the weir were affected by forces of surface tension and viscosity, indicating a weir efficiency greater than that of the corresponding prototype. Similar findings were presented by Schoder and Turner[4] in tests on sharp-crested weirs.

INSTRUMENTATION

Basic model instrumentation consisted of six capacitance-type wave probes connected to a 6-channel direct-writing oscillograph. The probes proved to be very successful in measuring the size and form of the surge wave. Some nonlinearity occurred because of the plasticized-enamel dielectric wire coating[5]. A careful calibration routine was necessary to obtain linearity and separate calibrations were made for each test run to ensure accurate data. It was also necessary to insulate carefully the impedance bridge circuit of each probe to prevent zero datum drift caused by room temperature variations.

According to other experimenters[6], meniscus effects result in an error of approximately ±0.015 inch (0.38 mm) which was not considered significant. The errors were found to be greatest at the wave troughs, which were not of primary importance in this study.

CHARACTERISTICS OF THE SURGE WAVE

The size and form of the surge wave were first recorded following complete rejection of the inflow, without backflow from the pump discharge lines. Three wave probes were placed at each measuring station to determine both the longitudinal and transverse form of the wave. Data were recorded at a section approximately 13 feet (4 m) (model) upstream from the pumping plant at Station 18+66 (the weir, which ended at Station 18+50, was in place for this test but data would be identical without the weir), and a section approximately 25 feet (7.6 m) upstream from the pumping plant at Station 12+90 (without the weir). Three conditions of initial inflow were imposed: maximum discharge (6-pump operation), one-half maximum discharge (3 pumps), and one-sixth maximum discharge (one pump). The depth of flow in the canal remained constant at an average depth of 3.75 inches (9.6 cm) for all test runs. Flow was stopped by rapid closure of the downstream slide gates.

Figure 2 illustrates the variation of average surge height, following complete flow rejection, with the Froude number of the canal flow. The linear relationship indicated by the limited data is supported by the accompanying theoretical curve, which was derived from the equations of continuity and momentum, using an electronic digital computer. The scatter in the data is probably due to slight variations in the initial inflow conditions, since each section was recorded at a different time. Although agreement is quite good, the experimental curve lies above the theoretical curve. No conclusions should be drawn from this until more data are available to more carefully define the experimental curve. Experiments in rectangular channels have shown good agreement with theory.

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[4]Schoder, E. W., and Turner, K. B., "Precise weir measurements." Transactions, American Society of Civil Engineers. Volume 93, 1929.

[5]Pearlman, Michael D. "Dynamic calibration of wave probes." MIT Department of Naval Architecture and Marine Engineering. July 1963.

[6]Sandover, J. A., and Zienkiewicz, O. C. "Experiments on surge waves." Water Power. November 1957.

Figure 3 illustrates the variation of average wave velocity through the canal reach with the Froude number of the canal flow and indicates the effect of the side weir in reducing the wave velocity. Figure 4 shows variation in wave length with wave velocity and illustrates the change in wave length as the wave travels through the canal. For any given wave velocity, the wave length increases as the wave is propagated upstream. The difference becomes negligible below a wave Froude number of approximately 0.87. Sandover and Zienkiewicz[6] observed a decreasing wave length with an increase in wave velocity, contrary to Figure 4, but hinted that this relationship was a function of the distance from the point of initiation of the surge by stating that "Along the length of the channel, however, for one run the wave length increases at first then steadily decreases." Gentilini's[3] data also indicate that the wave length-wave velocity relationship is dependent upon the location of the measuring section. At a section more distant from the origin of the surge, therefore, a plot similar to Figure 4 might also show a decreasing wave length for an increasing wave velocity. Additional data will be necessary to prove or disprove this premise.

Perhaps the most important relationship in the study of surges in open channels is demonstrated in Figure 5. The formation of peaks above the average surge height is sometimes neglected and is essential to the proper design of canal freeboard requirements. Technical literature shows a wide variation in this relationship. This variation is due to the effects of several variables such as (1) distance of the measuring station from the point of initiation of the surge, (2) methods of experimental measurement, and (3) velocity distribution in the channel before surge propagation. As shown in Figure 5, this study indicated an essentially linear relationship with the maximum oscillation peak being approximately 1.18 times the average surge height. For design purposes, the average surge height, h, can be obtained from Figure 2, and the maximum oscillation peak, h_{max}, is subsequently available from Figure 5. The undular form of the surge wave has been explained by Jones[7] as an oscillatory movement caused by the transition between parallel and curvilinear flow. In the stable range (before the peaks begin to break) the ratio between the surge height and the maximum oscillatory peak has been found to vary from approximately 1.1 to 2.0 for rectangular channels.

EFFECT OF BACKFLOW FROM THE PUMP DISCHARGE LINES

In the case under study, the backflow from the pump discharge lines resulted in a substantial addition to the rejection surge. The volume of backflow is dependent on the position of the pump impeller vanes at the instant of power failure and the time required for the vanes to close following the power failure. The rate of backflow with the vanes feathered was estimated at 150 percent of the pump capacity. If, however, the vanes became stuck in the most adverse position due to a control unit malfunction, the backflow could be as high as 200 percent of the pump capacity. Although the latter condition was considered highly improbable, tests were conducted for backflows of 150- and 200-percent pump capacity to span the range of possible conditions.

The form of the surge wave consisted of a combination of the rejection surge and the backflow surge, the latter being superimposed upon the former. The downstream slide gates were closed instantaneously to initiate the rejection surge and the backflow head tanks were allowed to drain immediately thereafter, adding the surge due to backflow. The fastest required time of closure of the prototype impeller vanes was estimated to be approximately 15 seconds. A short time is required to overcome the forward inertia of the impellers and

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[7]Jones, L. E. "Some observations on the undular jump." Proceedings, American Society of Civil Engineers (Journal of the Hydraulics Division). Volume 90, No. HY3, May 1964.

provide for acceleration of the backflow. Velocities of the rejection surge and backflow surge, measured independently in the model, indicated that the backflow surge would overtake the rejection surge at the downstream end of the side weir if initiated 1.5 seconds after rejection. It was, therefore, desirable to determine the attenuating effect of the weir with the backflow surge superimposed on the rejection surge, which would be the worst possible condition that could occur in the prototype.

ATTENUATING EFFECT OF THE SIDE WEIR

Preliminary tests on a 2,073-foot-(631.8-m) long side weir indicated that a 1,500-foot- (457.2-m) long weir would attenuate the surge to an allowable height. The weir was installed between Station 3+50 and Station 18+50 and data were collected for the three inflow conditions previously described. Surges were initiated for complete rejection without backflow, with 150-percent backflow, and with 200-percent backflow. The crest of the weir was an average of 0.5 foot (prototype) above the initial water surface.

Figure 6 shows the reduction in peak surge height after traversing the weir section. The curves actually demonstrate the combined attenuating effect of the side weir and the decay of the maximum backflow peak due to instability. The unfortunate scatter of data is at least partially due to the inability to duplicate repeatedly and exactly the backflow from the manually operated head tanks for all test runs. The theoretical curve for a Froude number of 0.250 in a rectangular channel is extracted from Citrini's paper[1]. The curves compare favorably. Citrini's curve is for a weir length-channel width ratio of 10, whereas the weir under study corresponds to a ratio of approximately 11.8.

Figure 7 summarizes wave forms with and without the weir for the described conditions of inflow and backflow. Some general observations on the form of the surge wave should be noted. First, the records illustrate errors encountered in relying on data from a single measuring section, such as the canal centerline. Differences in surge heights from one side of the channel to the other were nearly indistinguishable for small surges, either with or without the weir. As surge heights increased, particularly with the superimposed backflow surge, the initial peaks of the surge exhibited a concave form (lower in the center).

The attenuation of the surge without the effect of the side weir can be determined by comparing the left and right columns of Figure 7. The rejection wave is reduced by friction and the heights of the oscillation peaks vary along the channel as the surge approaches full development (the latter premise was observed but is not distinguishable in Figure 7). Friction has negligible effect on the maximum peak of the rejection surge without backflow. However, the friction effect is readily apparent in the average surge height and in the depths of the troughs. Sandover and Zienkiewicz[6] observed similar results. The backflow surge is attenuated by friction and, to a greater extent, by a tendency for the trailing slope of the wave to flatten out.

Figure 8 shows the form of the surge following rejection of the maximum discharge of 4,200 cfs (118.9 cms) with a backflow rate of 200 percent of pump capacity. The breaking edge noted on either side of the channel is due to instability caused by the lesser flow depth over the side slopes.

CONCLUSIONS

A hydraulic model was used successfully to determine the mechanics of surge formation and propagation in a large trapezoidal canal, and to study the attenuating effect of a side weir.

The average wave height increased linearly with the Froude number of the canal flow and agreed quite favorably with theory. Contrary to results observed by other experimenters, the surge wave length increased with wave velocity, which supports the premise that the wave length-wave velocity relationship is a function of the distance from the surge source. Generally, the studies showed that the maximum surge peak was about 1.18 times the average surge height.

The side weir significantly reduced the average wave heights and wave velocities. The attenuating effect of the 1,500-foot-long weir on the maximum peaks compared favorably with theory derived for rectangular channels.

The data presented should be generally applicable to other trapezoidal channels of this relative size and shape. Factors such as velocity distribution, friction, and the ratio of wave height to channel depth, however, affect the formation and propagation of the surge. Where unusual accuracy is necessary, a model study of the particular channel may be warranted.

FIGURE CAPTIONS

Figure 1. The 1:48 scale model

Figure 1. Example à l'échelle 1:48

Figure 2. Variation of average wave height with Froude number of initial flow

Figure 2. Variation de la hauteur moyenne de la lame avec un Nombre Froude de l'écoulement initial

Figure 3. Variation of wave velocity with Froude number of initial flow

Figure 3. Variation de la vitesse de la lame avec un Nombre Froude de l'écoulement initial

Figure 4. Variation of first wave length with wave velocity

Figure 4. Variation de la première hauteur de la lame avec sa vitesse

Figure 5. Average wave height-maximum peak relationship

Figure 5. Rapport entre la hauteur moyenne de la lame et sa pointe maximum

Figure 6. Attenuation of maximum peaks by 1,500-foot side weir

Figure 6. Atténuation des points maximum par un barrage de 1500 pieds

Figure 7. Effects of weir and superimposed backflow on longitudinal wave forms

Figure 7. Effets du barrage et du courant inverse superposé sur les formes de la lame

Figure 8. Form of surge wave at downstream end of weir following rejection and backflow

Figure 8. Forme de la lame de fond à l'extrémité en aval du barrage, après rejet et courant inverse

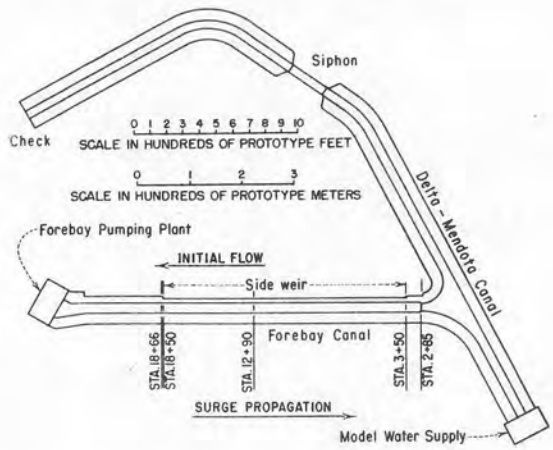

Figure 1. The 1:48 scale model

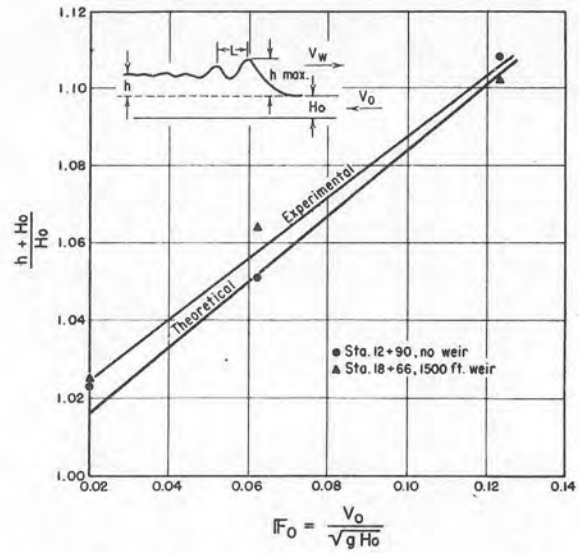

Figure 2. Variation of average wave height with Froude number of initial flow

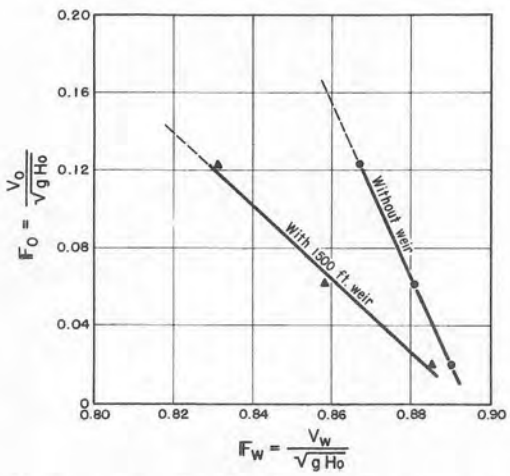

Figure 3. Variation of wave velocity with Froude number of initial flow

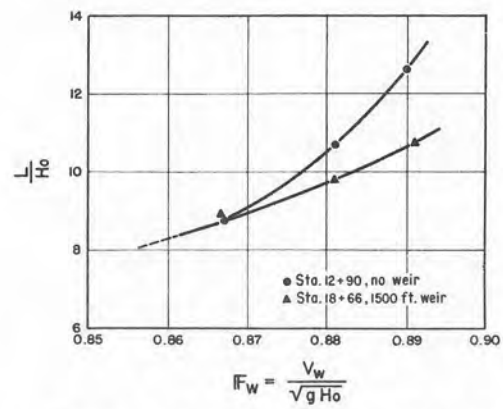

Figure 4. Variation of first wave length with wave velocity

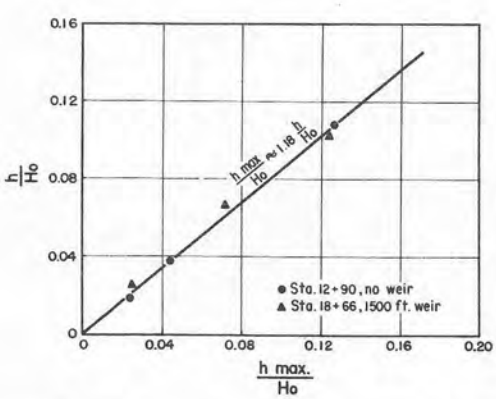

Figure 5. Average wave height-maximum peak relationship

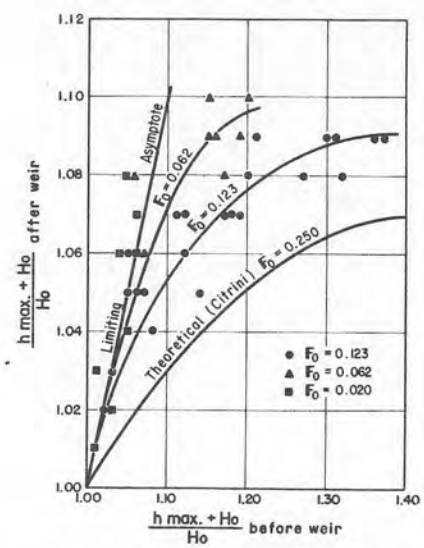

Figure 6. Attenuation of maximum peaks by 1,500-foot side weir

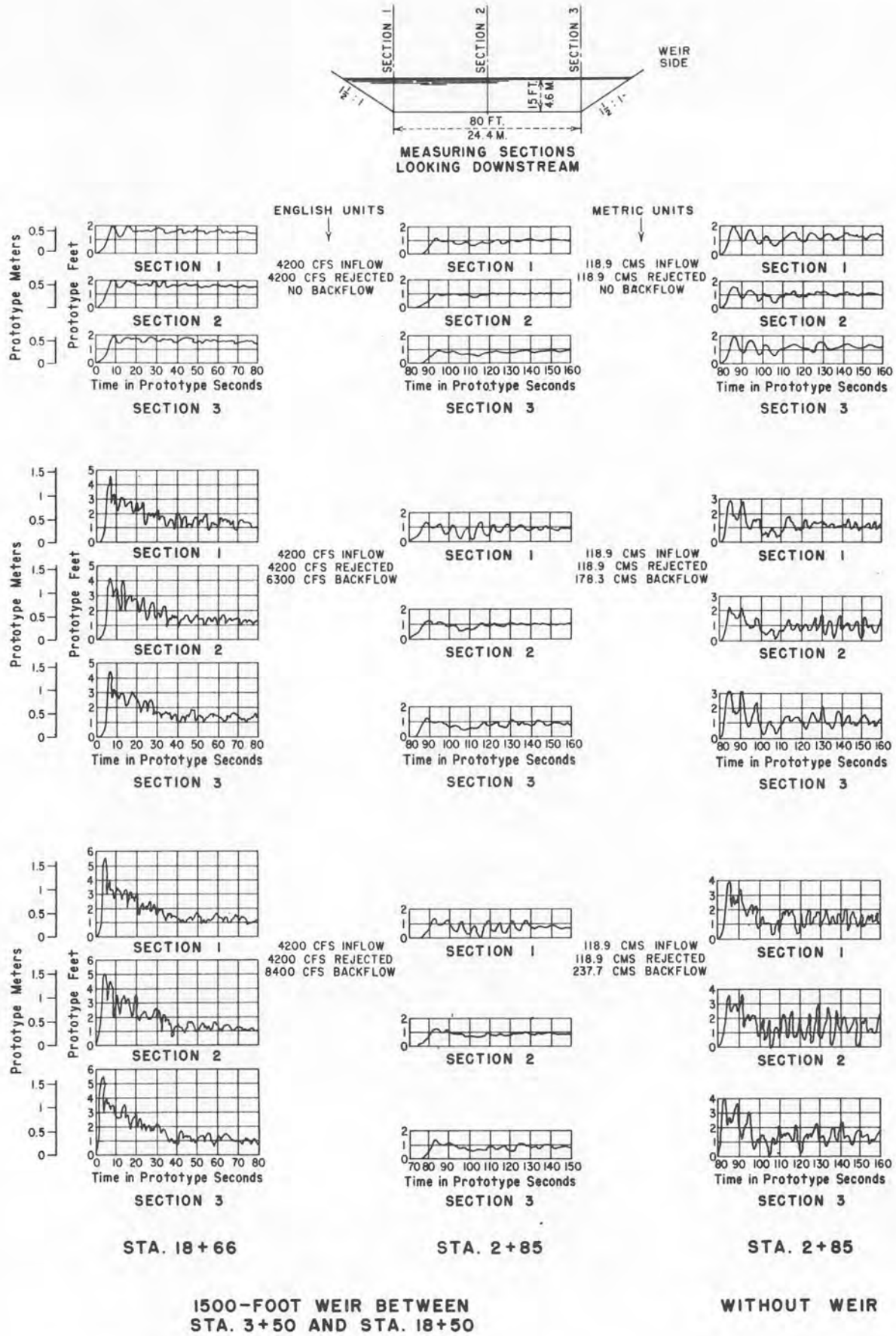

Figure 7. Effects of weir and superimposed backflow on longitudinal wave forms

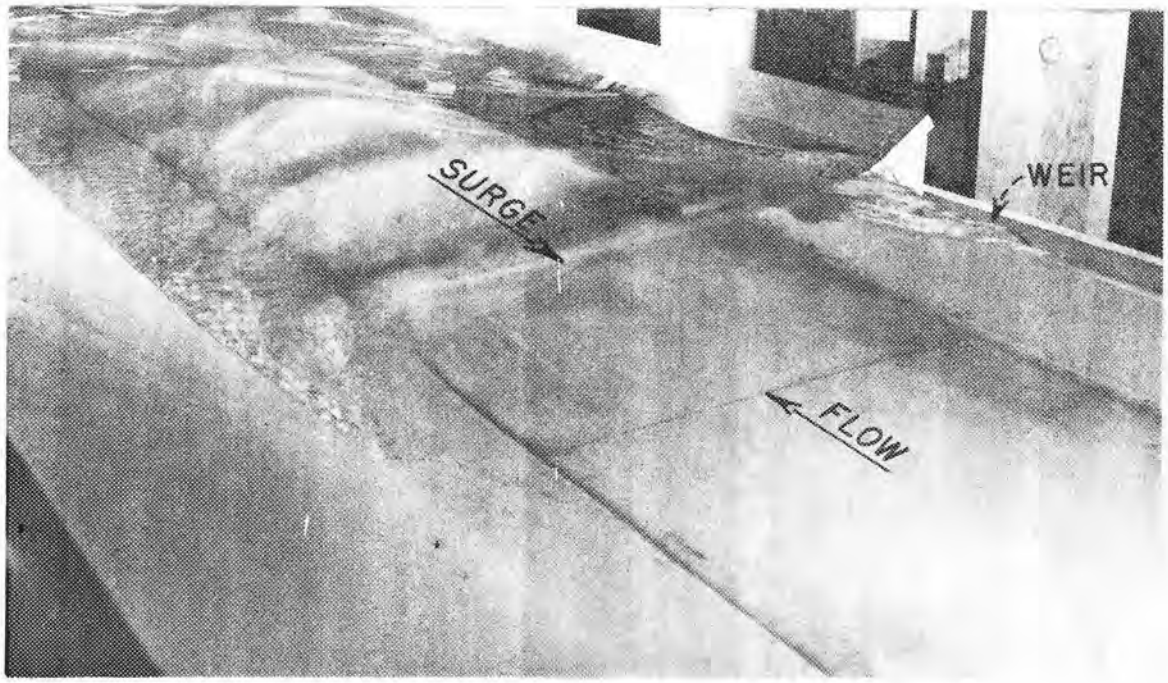

Figure 8. Form of surge wave at downstream end of weir following rejection and backflow