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San Luis Surge Studies Hyd 546

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES OF SURGES DEVELOPED BY REJECTION OF FLOW AT THE FOREBAY PUMPING PLANT, SAN LUIS UNIT CENTRAL VALLEY PROJECT, CALIFORNIA

Report No. Hyd-546

Hydraulics Branch
DIVISION OF RESEARCH

OFFICE OF CHIEF ENGINEER
DENVER, COLORADO

November 1, 1965

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ABSTRACT

Data from a 1:48 scale model supplied the magnitudes and velocities of surges developed in the canal system following rejection of flow at the pumping plant, San Luis Forebay, California, and showed that a side weir was effective in reducing the surges. Data were obtained with capacitance wave probes for partial and complete rejection of flow with and without backflow from the pump discharge lines. Maximum surge peak heights were 5.4 ft for complete rejection of the maximum discharge plus 200% backflow, 4.5 ft with 150% backflow, and 1.9 ft without backflow. Velocities of propagation were 20.7, 20.7, and 19.1 fps, respectively, for the 3 conditions. A 1,500 ft-long weir on the canal sideslope reduced the maximum surge height to 1.0 ft without backflow and 1.3 ft with either 150 or 200% backflow. The reflecting and attenuating characteristics of canal structures were observed and steady-state conditions after flow rejection with the entire flow discharging over the weir were measured. The undular form of the surge wave was analyzed and several comparisons were made with theory. A 1:10 scale sectional model was used to develop the weir crest shape.

DESCRIPTORS-- *pumping plants/ *canals/ *model tests/ *surges/ *trapezoidal channels/ *weirs/ hydraulic transients/ freeboard// bore/wave// discharge coefficients/ viscosity/ Reynolds number/ Froude number/ surface tension/ translatory waves/ unsteady flow/ weir crests/ calibrations/ instrumentation/ laboratory equipment/ measuring instruments/ recording systems/ capacitance/ dielectrics/ electronic equipment/ oscillographs/ research and development
IDENTIFIERS-- wave probes/ Weber number/ Central Valley Project, California/ San Luis Forebay Pumping Plant

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Report No. Hyd-546
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PURPOSE

These studies were conducted to determine the magnitudes and velocities of surges developed in the canal system following rejection of flow at the pumping plant, and to investigate a proposed method of alleviating the surges.

CONCLUSIONS

1. Surges developed in the Forebay Canal were found to have a maximum peak height of 5.4 feet for rejection of the maximum pumped discharge (4,200 cfs (cubic feet per second)) plus an assumed backflow of 200 percent of the pumped discharge, 4.5 feet with an assumed backflow of 150 percent, and 1.9 feet for rejection of the pumped discharge with no backflow, Figure 11. Corresponding average surge velocities were 20.7, 20.7, and 19.1 fps (feet per second), respectively. The average height of the bore following the maximum peaks was 1.5 feet in all three cases.

2. Surface tension and viscosity affected the flow over the weir for heads less than 0.016 foot, corresponding to a prototype head of 0.77 foot. Clinging of the nappe to the downstream face of the weir resulted in an increased discharge coefficient. A residual surge height of 1.0 foot, following attenuation by a 2,073-foot-long weir, was corrected to 1.1 feet, Figure 7.

3. A 1,500-foot-long side weir located between Stations 3+50 and 18+50 on the Forebay Canal was effective in reducing the maximum height of the surge to approximately 1.0 foot without backflow and 1.3 feet with backflow, Figure 11. Average surge velocities were 20.1, 20.4, and 18.3 fps, for the conditions of the preceding paragraph.

4. After attenuation by the side weir, the positive wave split at the turnout; a positive wave with approximately 70 percent of the height of the residual surge traveled upstream in the Delta-Mendota Canal, and a positive wave with a height of approximately 55 percent of the residual surge height traveled downstream. With backflow, these values were approximately 60 and 35 percent, respectively. A small negative surge was reflected back down the Forebay Canal toward the pumping plant.

5. The inverted siphon at Station 3002+50, Delta-Mendota Canal, eliminated the peaks of the surge wave but had no effect on the average bore height. Check 13, at Station 3023, Delta-Mendota Canal, Figure 2, reflected the wave at approximately double its previous height.

6. Friction effects in the model could not be accurately determined. It is recommended that equations developed by other experimenters be used to estimate the attenuation of the wave due to friction. (See footnotes 17/ and 18/.)

7. The maximum water surface in the system (measured at the upstream end of the siphon) was 1.6 feet above the normal pooled water surface approximately 8 minutes (prototype) after initiation of the surge (including backflow).

8. Steady-state conditions occurred with the entire discharge flowing over the side weir about 45 minutes (prototype) after initiation of the surge. The steady-state water surface was 1.1 feet above the normal water surface elevation near the downstream end of the 1,500-foot-long weir, 1.2 feet above normal at the upstream end of the Forebay Canal, and 1.3 feet above normal in the pooled Delta-Mendota Canal.

9. A 1:10 model was used to develop a weir crest shape which will provide adequate discharge capacity and reflect wind-generated waves during normal canal operation, Figures 19 through 22.

DEFINITION OF TERMS

a --area of orifice in backflow tank

A --reciprocal of Froude number of initial flow

A_t --area of backflow tank

C --discharge coefficient of backflow tank orifice

c --wave celerity

C_d --weir discharge coefficient

C_w --dimensionless weir discharge coefficient $(C_d/\sqrt{2g})$

c^* --ratio of height of spillway crest above canal bottom to initial depth of flow

γ --specific weight of water

F_o --Froude number of initial flow, $V_o/\sqrt{gH_o}$

F_w --Froude number of surge wave, $V_w/\sqrt{gH_o}$

g --acceleration of gravity

H_o --initial depth of flow

h --average surge height

h_o --head on backflow tank orifice at time, $t = 0$

h_1 --head on backflow tank orifice at later time

h_{max}--maximum surge height

h_w --head on weir

L --length of weir

l --channel width at elevation of weir crest

λ --wave length, distance between wave crests

μ --dimensionless weir discharge coefficient (same as C_w)

π --pi, 3.1416

Q --discharge over weir

R --Reynolds number of flow over weir

ρ --mass density of water

σ --surface tension of water

t --time

V_o --velocity of initial flow

V_w --velocity of surge wave

W --Weber number of flow over weir

y_i --ratio of maximum surge depth to initial flow depth, before attenuation by weir

y_f --ratio of maximum surge depth to initial flow depth, after attenuation by weir

ACKNOWLEDGMENT

The studies described in this report were accomplished through cooperation between the Canals Branch, Division of Design, and the Hydraulics Branch, Division of Research. Photography was by W. M. Batts, Office Services Branch.

INTRODUCTION

The San Luis Unit of the Central Valley Project in California, Figure 1, includes a system to store surplus water for later release. The Forebay Canal and Forebay Pumping Plant, Figure 2, as parts of this system, will divert water from the existing Delta-Mendota Canal at the maximum rate of 4,200 cfs 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 irrigation flows. This report is concerned with the investigation of surges which would be developed in the Forebay Canal and a portion of the Delta-Mendota Canal if the flow to the Forebay Pumping Plant was rejected due to pump stoppage caused by malfunction or power failure.

Description of Problem

Although safeguards have been included in the design, the possibility exists that power failure might occur at the Forebay Pumping Plant, resulting in stoppage of the pumps. Should such a power failure occur, a surge would be propagated in the Forebay Canal due to rejection of the canal flow and backflow drainage of the pump discharge lines. This surge could not be allowed to travel unreduced into the Delta-Mendota Canal.

When flow in a channel is suddenly halted, due to closing a gate or stopping a pump, a surge wave develops and moves upstream in the channel. The height and velocity of the wave are dependent upon the depth and velocity of the initial incoming flow and on the shape of the channel cross section. Following complete rejection of the inflow, the stream comes to rest after passage of the wave and the discharge in the wave is equal to the discharge of the initial incoming flow. If only a portion of the flow is rejected, the velocity in the channel following passage of the wave is proportional to the unrejected portion of the discharge and the wave discharge is again equal to the rejected discharge. By applying the equations of continuity and momentum (as in development of the hydraulic jump formula) the theoretical height and velocity of the surge wave are obtained. The maximum height of oscillations which occur above the main body of the surge wave can also be estimated through theoretical considerations. All of the aforementioned relationships have been investigaged by many experimenters.

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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 the subject of this model investigation, consisted of a side weir along the 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.

Citrini1/ developed a theoretical approach to the action of a lateral spillway in reducing the height of a positive surge. The development is beyond the scope of this report and will not be presented. The validity and limitations of the theory have been proven by other Italian experimenters, 2/, 3/.

Experimental data from this study are compared with the theoretical derivation in the Investigation section of this report.

It should be noted that the theoretical equations for development and propagation of a surge are, in most cases, applied to prismatic channels with symmetrical alinement and involve complete rejection of flow by rapid closing of a downstream control gate. In reality, as in the case of the Forebay Canal and Pumping Plant, conditions differ from the usual case to such an extent as to warrant hydraulic model studies to insure accurate prediction of prototype behavior.

1/ "Attenuation of a Positive Wave by Means of a Lateral Spillway" by Duilio Citrini, L'Energia Elettrica, Vol 26, No. 10, pp 589-599, 1949. (Translated from Italian by Language Service Bureau.)
2/ "The Action of a Side Weir on the Positive Wave Moving Upstream in an Open Channel," by Bruno Gentilini, Memorie e Studi Dell' Istituto Di Idraulica e Costruzioni Idraulica Del Politecnico Di Milano, No. 78, 1950. (Translated from Italian by Language Service Bureau.)
3/ "Action of Side Weirs and Tilting Gates on Translation Waves in Canals," by Guilio De Marchi, Proceedings of the Minnesota International Hydraulics Conference, August 1953.

THE MODEL

The 1:48 scale model, Figures 3 and 4, included the Forebay Canal and the canal transition to the pumping plant intakes, the turnout from the Delta-Mendota Canal to the Forebay Canal, a section of the Delta-Mendota Canal downstream from the turnout to Station 3023 (Check 13) including the inverted siphon at Station 3002+50, and a section of the Delta-Mendota Canal upstream to Station 2978+70. The Forebay Canal model alinement was on the opposite side of the Delta-Mendota Canal from that of the prototype alinement, Figure 3, because of laboratory space limitations. Most of the model was fabricated from plywood with the exception of warped transition sections formed in concrete. The siphon barrels were made of sheet metal and a slide gate was installed at Check 13. Backflow devices in the model were formed of sheet metal. The overflow side weir was built to elevation tolerances of plus or minus 0.002 foot and consisted of sheet metal formed over wood templates.

Basic model instrumentation consisted of six capacitance-type wave probes with plasticized-enamel coated wire, connected to a six-channel, direct-writing oscillograph, Figure 5. Each wire was 6.25 inches long, mounted in a U-frame. The frames were attached to modified point gage staffs in rack and pinion devices with verniers reading to 0.001 foot. Calibration was accomplished by raising and lowering the probes known distances in a stable pool of water.

Early in the model study some difficulty was experienced in calibrating the probes. Nonlinearity occurred because of wetting and drying characteristics of the plastic dielectric4/, and a careful calibration routine was necessary to obtain linearity. Separate calibrations were made for each test run to ensure accurate data. After establishing the zero datum, the wire was immersed by lowering the probe a known distance. (By lowering the probe more than the required amount, waiting for several seconds, then raising the probe to the correct position, the wetting effect was partially suppressed by prewetting the wire.)

Fifteen to thirty minutes were required for the wire to reach a stable condition. The probe was then raised in increments to the initial zero position to check the linearity. It was also necessary to carefully insulate the impedance bridge circuit of each probe because of zero datum drift caused by room temperature variations. According to other experimenters, 5/ meniscus effects result in an error of approximately plus or minus 0.015 inch (plus or minus 0.06 foot (prototype) for this model), which is not considered significant in measurement of the crest height. The errors in the cited study were found to be greatest at the troughs, (minus 0.01 to plus 0.02 inch) which were not of primary importance in this study.

4/ "Dynamic Calibration of Wave Probes" by Michael D. Pearlman, MIT Department of Naval Architecture and Marine Engineering, July 1963.
5/ "Experiments on Surge Waves" by J. A. Sandover and O. C. Zienkiewicz, Water Power, November 1957.

Water was supplied to the model by a centrifugal pump with discharge rate measured by a volumetrically calibrated orifice meter. The recirculated water caused some difficulty by allowing waterborne materials to be deposited on the wave probes.

Backflow drainage from the discharge lines was simulated by flow from head tanks located above the pumping plant intake bays, Figure 6. A pressure transducer was used to record the head-time characteristics of orifices in the bottoms of the tanks for determination of discharge coefficients and rates of discharge. Initial rates of backflow were controlled by filling the tanks to a predetermined level, then allowing them to drain immediately after rejection of the canal flow.

Depth of flow in the canal was maintained by adjusting the slide gates downstream from the intake bays, Figure 6. Water surface elevations during normal operation of the canal were measured with a point gage at Forebay Canal Station 23+23.

THE INVESTIGATION

2, 073-foot Side Weir

The initial phase of the investigation was concerned with determining the minimum length of side weir necessary to reduce the surge height to an allowable value of approximately 1.5 feet. To obtain general information on the attenuating effect, a weir was installed along the entire length (2, 073 feet) of the Forebay Canal to determine at what point along this length the surge wave was reduced to an allowable height. These data were then used to determine the next trial weir length. Initial tests were made to observe the attenuation of a surge caused by rejection of the maximum canal discharge of 4, 200 cfs, without backflow. Later, similar measurements were made with various backflow rates.

Initial operating conditions in the canal, before generation of the surge, required a maximum flow depth of 15.09 feet at the upstream end of the weir, so that the water surface was 6 inches (prototype) below the crest of the weir. (The weir crest was at elevation 173.7 in the preliminary design.) Friction head loss in the model canal resulted in a water surface elevation difference of 1.15 inches (prototype) so that the water surface was about 7.15 inches below the crest at the downstream end of the weir. This head loss corresponds to a prototype value of Manning's "n" coefficient of approximately 0.018, which is close to the suggested coefficient of 0.017 for large canals.

The surge was initiated by rapid closure of the downstream control gates. Figure 7 shows a reduction from an initial height of 1.8 feet to a final value of 1.0 foot at the upstream end of the 2,073-foot weir. The curve is based on the maximum peaks recorded on the weir side of the canal above the intersection of the canal invert and the 1-1/2:1 side slope. The surge was propagated through the canal at an average velocity of 18.3 feet per second. Upon reaching the turnout, a positive surge with a height of 0.7 foot was propagated upstream in the Delta-Mendota Canal, a positive surge of 0.6-foot height traveled downstream, and a small negative surge was reflected back toward the pumping plant in the Forebay Canal. The surge heights in the Delta-Mendota Canal were measured at the canal centerline.

Effects of Surface Tension and Viscosity

The effects of surface tension and viscosity on formation of the surge and the efficiency of the side weir in the relatively small model were investigated. Experiments on V-notch and sharp-crested weirs[6/, 7/] indicate a marked increase in the discharge coefficient at very low heads due to the nappe clinging to the downstream face of the weir. The clinging effect is caused by surface tension and viscosity of the fluid. Similar tests on round-crested weirs[8/] showed a decrease in the discharge coefficient for low heads. Assuming that similar effects existed in the 1:48 model, tests were made to determine the heads above which the effects of surface tension and viscosity were negligible. Figure 8, which illustrates the variation in a dimensionless coefficient of discharge for a range of values of the Weber and Reynolds numbers, indicates that surface tension and viscosity cause an increased coefficient below a head of approximately 0.016 foot (measured upstream from the crest where velocity head is negligible). A model head of 0.016 foot corresponds to a prototype head of 0.77 foot. In other words, for prototype heads less than 0.77 foot the model will indicate a weir efficiency greater than that which will actually exist in the prototype. The weir profile in the model was terminated immediately downstream from the crest, allowing the overflow to spill down a vertical face. The vertical face corresponded to the downstream face of a sharp-crested weir, thus causing the increased coefficient at low heads.

6/"Precise Weir Measurements" by E. W. Schoder and K. B. Turner, Transactions, ASCE, 1929, Vol 93.
7/Engineering Hydraulics, edited by Hunter Rouse, John Wiley and Sons, Inc., New York, 1958, p 214.
8/"On the Influences of Curvature, Surface Tension and Viscosity on Flow Over Round-Crested Weirs," by G. D. Matthew, Proceedings of the Institute of Civil Engineers (England), Vol 25 May-Aug 1963.

The residual surge height of 1.0 foot following rejection of 4,200 cfs with no backflow corresponds to a head on the weir crest of approximately 0.5 foot, which is less than the critical value. This head results in a Reynolds number of 500 and a Weber number of 1.35 with corresponding values for C of 0.29 on the lower line and 0.44 on the upper line in Figures 8A and B.

$$C_{w1} : C_{w2} = \frac{1}{(h_{w1})^{3/2}} : \frac{1}{(h_{w2})^{3/2}},$$ with the subscripts

referring to the upper and lower lines respectively. As computed above, $h_{w2} = 0.5$ foot (0.0104 foot, model).
Therefore,

$$0.29 : 0.44 = \frac{1}{(h_{w1})^{3/2}} : \frac{1}{(0.0104)^{3/2}} = \frac{1}{(h_{w1})^{3/2}} : 1000$$

or $$(h_{w1})^{3/2} = \frac{0.44}{0.29(1000)} = 0.00152$$

$$h_{w1} = 0.0132 \text{ foot (model)} = 0.6 \text{ foot (prototype)}$$

Therefore, the true head on the weir is 0.6 foot and the true residual surge height is

$$0.5 + 0.6 = 1.1 \text{ feet}$$

The dashed line in Figure 7 shows the attenuation by the weir with correction for surface tension and viscosity.

Subsequent tests with larger initial surge heights resulted in residual heights large enough to warrant neglect of the surface tension and viscosity effects. Tests made with a detergent wetting agent added to the model water supply substantiated the above conclusions, as shown in Figure 9. The initial difference between the curves is probably due to errors in discharge or calibration rather than the wetting agent. The figure shows that, at a surge height of approximately 1.4 feet (0.8 foot above the weir crest at this station), the curves begin to diverge, with the weir being less effective with the wetting agent. At the upstream end of the weir the residual surge height is 1.3 feet with the wetting agent, which is a larger correction than that obtained from Figure 8. The amount of data is limited, and additional tests should be made before definite conclusions are drawn.

Computations showed that capillary effects were negligible in the formation of the surge wave. The equation for the celerity of a surface wave is

$$c = \sqrt{\frac{\lambda}{2\pi}\frac{\gamma}{\rho} + \frac{2\pi}{\lambda}\frac{\sigma}{\rho}} \qquad 9/$$

where the first term under the radical is governed by gravity, $\left(\frac{\gamma}{\rho}\right)$ and the second term by capillarity, $\left(\frac{\sigma}{\rho}\right)$

λ = distance between wave crests (wave length), feet;

γ = specific weight, pounds per cubic foot;

ρ = mass density, slugs per cubic foot;

σ = surface tension, pounds per foot.

For λ = 3.0 feet (within range of model wave lengths),

γ = 62.4, ρ = 1.94, and σ = 0.005:

$$\frac{\lambda}{2\pi}\frac{\gamma}{\rho} = \frac{3.0}{2\,(3.1416)}\;\frac{62.4}{1.94} = 15.358$$

$$\frac{2\pi}{\lambda}\frac{\sigma}{\rho} = \frac{2\,(3.1416)}{3.0}\;\frac{0.005}{1.94} = 0.0054$$

Thus, the influence of capillarity is considerably less than one percent of the gravity influence.

9/Elementary Mechanics of Fluids, by Hunter Rouse, John Wiley and Sons, Inc., New York, 1959, p. 324.

Backflow from the Pump Discharge Lines

The volume of backflow from the pump discharge lines depends on the position of the pump impeller vanes at the time of power failure and the time required for the vanes to be feathered following the power failure. The exact backflow characteristics of the pumps were unknown at the time of the model study; however, it was possible to describe the operation in general terms and to estimate the characteristics.

Immediately after power failure, a short period of time is required to overcome the forward inertia of the impellers and allow for acceleration of the backflow to the maximum rate. Upon power failure, it is assumed that the impeller vanes will begin to move to a feathered position. Under a head of 50 feet, the corresponding total backflow volume for six units was assumed to be about 90, 000 cubic feet and the maximum backflow rate approximately 6, 300 cfs (150 percent of the maximum pumping discharge). If the vanes became stuck in the most adverse position due to a control unit malfunction, the backflow rate could be as high as 8, 400 cfs (200 percent of the maximum pumping discharge). Although the latter condition was considered improbable, tests were conducted for both 150 and 200 percent backflow to span the range of possible conditions.

Independent measurements showed that the velocity of propagation of the backflow surge was about 24 fps as compared to 19 fps for the rejection surge, indicating that in the 150-foot distance between the intakes and the end of the weir, the backflow surge would overtake the rejection surge if initiated about 1.6 seconds (prototype) after rejection. It was, therefore, desirable to determine the attenuating effect of the weir on the combined rejection and backflow surges, the most adverse condition that could occur in the prototype.

The size of each backflow tank was determined according to the required volume and head for 150 percent backflow. The required size of the orifice in the bottom of each tank was estimated by assuming a discharge coefficient and computing the required area by using $Q = Ca\sqrt{2g\,h_o}$ The assumption was then checked by recording the time-discharge relationships for the tanks with the calculated orifices in place. The equation for the discharge coefficient of an orifice discharging under a falling head is

$$C = \frac{1}{t}\,\frac{A_t}{a}\,\frac{2}{\sqrt{2g}}\,(h_o^{1/2} - h_1^{1/2})$$

where

h_o = initial head in tank at t = 0,

h_1 = head in tank at later time t,

A_t = area of tank,

a = area of orifice, and

g = acceleration of gravity.

The variables, t, h_o, and h_1 were recorded, thus allowing the computation of C and the calculation of the discharge rate. It was found that the original assumption of the orifice size was too small. The head was increased to produce the required initial rate of discharge, resulting in a total simulated backflow volume greater than that of the prototype. However, the backflow surge height is affected only by the maximum rate of backflow discharge. The total volume affects the length and shape of the backflow surge wave which were, for purposes of this study, relatively unimportant.

After completion of the model study, additional information was received regarding the backflow characteristics of the pumps. Manufacturer's model tests indicated that at 50-foot head and 24° vane angle (wide open), the maximum backflow rate would be 3,720 cfs for six units, or about 89 percent of the maximum pumping discharge. At the feathered position (minus 5° vane angle) the maximum backflow rate would be 630 cfs for six units, which is only about 15 percent of the maximum pumping discharge.

The original estimates of 200 and 150 percent backflow for the described conditions were therefore not supported by the manufacturer's test data. However, since the model studies described in this report included data for surge formation without backflow, it is possible to interpolate for the correct conditions.

Another condition which was not included in the model investigation was that of sustained backflow with the pumps operating as turbines. Occurrence of this type of operation is relatively rare. With the siphon retaining its prime, the sustained backflow would be about 3,700 cfs or about 88 percent of the pumping capacity. The surge formed by this sustained backflow would be nearly identical in form to that initiated by rejection of an equal amount of inflow.

If power interruption occurred during turbine operation, the reverse speed of the pumps would be controlled by the siphon and the backflow rate of 3,700 cfs would continue to prevail. If the siphon breaker actuated and the discharge lines were allowed to drain, the maximum rate of backflow would again be about 3,700 cfs according to the manufacturer's tests.

1,500-foot Weir between Stations 3+50 and 18+50

The initial tests had indicated that the weir could be reduced in length while still maintaining adequate attenuation. Since Figure 7 indicates that a 1,500-foot-long weir will produce a residual surge height of about 1.2 feet, it was decided to determine the effect of a 1,500-foot-long weir between Stations 3+50 and 18+50.

A series of tests determined surge characteristics and weir attenuation following (1) complete rejection of maximum discharge with and without backflow, (2) complete rejection of partial discharge with and without backflow, and (3) partial rejection of maximum discharge with and without backflow. The initial flow conditions at Station 23+23 were maintained the same as in the tests on the 2,073-foot weir; thus, the normal water surface was about 7.2 inches (prototype) below the crest at the downstream end of the weir. Surge heights, peak heights, and wave velocities for the 1,500-foot-long weir are summarized in Tables 1 through 5, along with data for no weir which will be described later. Comparison of the residual surge heights at Station 2+85 for rejection of 4,200 cfs with no backflow indicates that the 1,500-foot-long weir is nearly as effective as the 2,073-foot weir for this condition. The largest residual surge at Station 2+85 was 1.3 feet, with either 150 or 200 percent backflow; this surge height is considered to be within allowable limits. The 1,500-foot-long weir was, therefore, recommended for inclusion in the final design.

Figure 10 illustrates the weir attenuation for various values of the Froude number of the canal flow and Figure 11 shows the variation of maximum peak height along the weir following rejection of the maximum inflow, with and without backflow. Figures 10 and 11 actually show both the combined attenuating effect of the side weir and the decay of the maximum backflow peak due to instability (illustrated by the solid lines in Figure 11). The unfortunate scatter of data points in Figure 10 is at least partially due to the inability to duplicate the backflow from the manually operated head tanks for all test runs. The experimental data points are compared to corresponding theoretical curves obtained from the solution of Citrini's[10] equation:

10/Op. Cit.

$$y_f(y_1-1)\left[1+\frac{3}{4}(y_1-1)\right]+(y_1^2-y_f^2)\sqrt{\frac{1}{8}(y_f+y_1)}-y_1(y_f^2-1)\sqrt{\frac{1}{8}(y_f+1)}$$

$$-\frac{\frac{\mu}{2}\frac{L}{\ell}A(y_f+y_1)^2(y_f+y_1-2c^*)\sqrt{\frac{1}{8}(y_f+y_1)(y_f+y_1-2c^*)}}{y_f+y_1+A(y_f^2-1)\sqrt{\frac{1}{8}(y_f+1)}+A(y_1-1)\left[1+\frac{3}{4}(y_1-1)\right]+A(y_f+y_1)\sqrt{\frac{1}{2}(y_f+y_1)}}=0$$

in which

y_1 = ratio of surge depth to initial depth at downstream end of weir;

y_f = ratio of surge depth to initial depth at upstream end of weir;

μ = dimensionless weir discharge coefficient, $\frac{C_d}{\sqrt{2g}}$

L = weir length;

ℓ = channel top width;

A = reciprocal of Froude number of initial flow;

c^* = ratio of height of spillway crest above channel floor to initial depth of flow.

The channel top width was taken as the channel width at the elevation of the weir crest. Although the equation was developed for rectangular channels, no attempt was made to modify it for trapezoidal channels because of the length of the equation. Solution of Citrini's equation is extremely complicated and subject to errors in calculation. A computer program, presented in the appendix to this report, was prepared to facilitate rapid calculation and to obtain a high degree of reliability in the solution.

Figure 10A shows that the experimental data points lie below the theoretical curve. As mentioned above, this was partially due to the decay of the backflow peaks which would have occurred in the absence of the weir. Also, Citrini's relationship was intended for use in determining the attenuation of the average surge height, which was not measurable at the upstream end of the model weir because of reflections from the canal turnout. The equation was therefore applied to the maximum peaks, which could be determined in the model.

Figures 10B and C show that as the initial canal velocity decreases, the weir becomes less efficient than indicated by theory. Several points, for probe sections 1 and 2, lie to the left of the limiting asymptote and show an increase in the surge height as the wave travels upstream.

Citrini states that the accuracy of the equation deteriorates as $\frac{L}{\ell}$ increases, with a maximum error of about 15 percent for $\frac{L}{\ell} = 10$. In the present study $\frac{L}{\ell} = 11.84$ (based on a symmetrical section) The comparisons of Figure 10 demonstrate that Citrini's relationship allows an estimate of the attenuating effect of the weir, but that the model study was necessary to accurately evaluate the weir performance.

Attenuation and Reflection Characteristics of Canal Structures

Surge waves are partially reflected by changes in shape or cross-sectional area. Data and observations are presented with reference to specific structures in the Forebay Canal and in the reach of the Delta-Mendota Canal included in the model study. Because of the complicated configuration of the system, reflections were not followed beyond the initial reflection. Combining of negative (lower than the original water surface) and positive (higher than the original water surface) waves of small amplitude resulted in loss of identification of specific waves. A theoretical treatment (with graphical solutions) of the reflection characteristics of channel discontinuities can be found in Favre's classical paper11/. The solutions are relatively complicated and lengthy and will not be further discussed in this report.

Effect of the Angled Transition to the Pumping Plant

The change in cross-sectional area at the upstream end of the transition undoubtedly caused a positive reflection of the initial surge wave resulting in a smaller positive wave traveling back toward the pumping plant. However, the undular form of the wave, with a long train of

11/ "Etude Theorique et Experimentale des Ondes de Translation dans les Canaux De'couverts (Theoretical and Experimental Study of Translatory Waves in Open Canals)" by Henry Favre, Dunod, Paris, 1935. Translated from French by the Language Service Bureau.

oscillations, made this reflection indistinguishable. As the wave was not fully developed upon reaching the upstream end of the transition, the reflection should have been of minor consequence. The angle of the transition had no apparent effect on the angle of propagation of the initial wave through the Forebay Canal. That is, the wave front was perpendicular to the canal centerline, which is contrary to an oblique form which might be expected. The wave seemed to "follow" the centerline through the transition. However, the transition influenced the form by causing a slightly higher wave on the side of the canal opposite the weir. This condition became less pronounced as the wave traveled away from the pumping plant and was barely noticeable by the time the wave reached the weir.

Effect of the Turnout to the Delta-Mendota Canal

As previously described, the initial wave, upon reaching the turnout, was split into three component waves. Surges were propagated both upstream and downstream in the Delta-Mendota Canal and a negative wave was reflected back toward the pumping plant. A series of measurements indicated that for rejection of the inflow without backflow a positive surge with a height of approximately 70 percent of the initial peak surge height was propagated upstream in the Delta-Mendota Canal and a positive surge with a height of about 55 percent of the initial surge traveled downstream; the size of the negative wave was indistinguishable because of the undulations following the initial positive wave. With 150- or 200-percent backflow, waves with heights of 60 to 65 percent and 35 to 40 percent of the initial surge height traveled upstream and downstream, respectively.

Effect of the Siphon

The inverted siphon at Delta-Mendota Canal Station 3002+50 removed the peaks of the undulatory wave and flattened the wave front as the wave passed through the siphon barrels. The average height of the wave remained unchanged. Reflections from the transition leading to the siphon and from the siphon entrance headwall were indistinguishable.

Effect of the Dead End at Check 13

As predicted by theory (as in Favre's work), the surge height was approximately doubled upon reflecting off the dead end. This condition is applicable equally to positive and negative surges and will continue until the waves are attenuated to a negligible size by friction.

Other Observations

It was noted that although the siphon removed the oscillation peaks of the wave, the peaks reformed as the wave continued along the canal beyond the siphon. This observation was also true for small residual waves following splitting at the bifurcation. Curves in the canal alinement had no apparent effect on the wave form. The wave front remained perpendicular to the canal centerline.

Surge Propagation in the Forebay Canal without the Side Weir

A series of tests was made with no side weir to more effectively evaluate the effect of the weir. The characteristics of the surge wave as it traveled through the Forebay Canal unattenuated by artificial means were determined. The rejection surge wave height will be reduced by friction; however, in the length of channel under consideration the maximum oscillation peak increased along the channel as the surge approached full development, Figure 11. The backflow surge, superimposed upon the rejection surge, was attenuated by energy loss due to friction and by a tendency for the initial peak of the wave to deteriorate due to instability; the latter influence was predominant. The backflow surge initially demonstrated an increase in size during development, which was followed by a fairly rapid decrease in size, Figure 11. The wave tended to become more stable as it traversed the canal.

In general, test conditions for the 1,500-foot-long weir were duplicated. The data are summarized in Tables 1 through 5. Without the side weir, data for partial rejection were taken for only 150 percent backflow, which is the assumed operating condition. Data for conditions of no backflow and 200 percent backflow can be estimated from the available data. The tables should be adequate to estimate surge heights and velocities for design purposes.

Observations on the Longitudinal Form of the Surge Wave

The general form of the rejection surge, without backflow from the discharge lines, was undular, as observed by many experimenters. It is often assumed that the surge is always of direct form with a level water surface behind the initial front. This premise is, in general, incorrect except for values of h/H greater than approximately 0.28 12/, for which the wave front becomes unstable and eventually breaks (h is the average surge height, H is the initial channel depth). The undular form of the surge wave has been explained by Jones 13/ as an oscillatory movement

12/ "Mathematical Theory of Irrotational Translation Waves" by G. H. Keulegan and G. W. Patterson, Research Paper RP 1272, Journal of Research, National Bureau of Standards, Vol 24, January 1940.
13/ "Some Observations on the Undular Jump," by L. E. Jones, Journal of the Hydraulics Division, American Society of Civil Engineers, May 1964.

caused by the transition between the maximum and average surge heights. Relationships among average surge height, peak height, wave length, and surge velocity are presented in Figures 12 through 16, for the data in Table 1.

Figure 12 illustrates the variation of average surge height, following complete flow rejection, with the Froude number of the canal flow. The experimental data are supported by the accompanying theoretical curve, which was derived from the equations of continuity and momentum. The scatter in the data is probably due to slight variations in the initial inflow conditions, since the wave heights at each section were recorded at a different time.

Figure 13 shows the variation of average wave velocity through the canal reach with the Froude number of the canal flow. The accompanying theoretical curve, also derived from continuity and momentum principles, shows theoretical velocities up to 10 percent higher than the measured velocities without the weir, and up to 14 percent higher velocities than those measured with the weir. Measurements of the velocity distribution showed that near the upstream end of the canal transition to the pumping plant, the surface velocity was approximately 25 to 30 percent higher than the average velocity. The theoretical curve is based on the average velocity; therefore, the higher surface velocity could explain the apparent retardation of the surge wave. Figure 13 also demonstrates the effect of the side weir in reducing the velocity of the wave. The effect grows less as the wave velocity increases. The curves tend to a value of $F_W = 1.00$, which corresponds to the celerity of a gravity wave in still water ($F_O = 0$).

Figure 14 shows variation in wave length (L in Figure 12) with wave velocity and illustrates the difference in wave length at two sections in the canal. For any given wave velocity, the wave length apparently increases as the wave is propagated upstream. The difference becomes negligible below a wave Froude number of approximately 0.87. Sandover and Zienkiewicz14/ observed a decreasing wave length with an increase in wave velocity contrary to Figure 14, but they 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's15/ 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 14 might also show a decreasing wave length for an increasing wave velocity.

14/Op. cit.
15/Op. cit.

Perhaps the most important relationship in the study of surges in open channels is demonstrated in Figure 15. Knowledge of the height of the peaks which form above the average surge height is essential to the proper design of canal freebroad requirements. Technical literature shows a wide variation in this relationship, 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 15, this study indicated an essentially linear relationship with the maximum oscillation peak being approximately 1.18 times the average surge height. Other investigators have found this ratio to vary from 1.1 to 2.0, in rectangular channels.

Backflow from the discharge lines resulted in superimposing a wave of a modified solitary form on the average height of the rejection surge. The height of the backflow surge is a direct function of the maximum rate of backflow discharge. The measured relationship between backflow surge height and maximum backflow discharge rate is shown in Figure 16. To find the total surge height due to rejection and backflow, the maximum backflow surge height from Figure 16 should be added to the average rejection surge height, h, (exclusive of the oscillatory peaks) from Figure 12. The theoretical curve for the average backflow wave height, developed by continuity and momentum principles, substantiates the experimental data. The experimental maximum curve indicates that the oscillation peaks of the backflow wave are approximately 1.6 times the average backflow surge height at this particular measuring station.

Longitudinal wave forms following rejection of the maximum discharge with and without backflow are shown in Figure 17. 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 exhibited a concave form (lower in the center). The model wave, following rejection of the maximum discharge with 150 percent backflow, is shown in Figure 18. The breaking edges of the wave are caused by instability due to the lesser depth over the canal side slopes.

The data presented should be applicable to other trapezoidal channels of this relative size and shape. It should be noted, however, that factors such as velocity distribution in the channel, friction, and the ratio of wave height to channel depth affect the formation and propagation of the surge waves.

Attenuation of the Rejection Surge by Friction

Friction effects could not be accurately evaluated in the model because of (1) the relatively small scale model which resulted in extremely small changes in wave height, and (2) the uncertainty that the wave had become fully developed at the measuring stations. Sandover and Zienkiewicz 16/ state that friction has little effect on the height of the initial peak, affecting primarily the troughs and distance between peaks (wave length). A straight prismatic channel, longer than that available for this study, would be necessary to properly evaluate the attenuating effects of friction. Sandover and Zienkiewicz present equations for the change in the undular profile and the attenuation of the oscillation peaks. After the oscillation peaks have been dissipated, the viscous damping of the stable wave form can be described by relationships presented by Keulegan 17/, 18/.

Summary of Operation of the System Following Rejection of the Maximum Discharge of 4,200 cfs

The water surface variation at the upstream end of the siphon was measured to determine the maximum depth in the pooled Delta-Mendota Canal downstream from the bifurcation following rejection of the maximum discharge. The maximum water surface was about 1.6 feet above the pooled water surface approximately 8 minutes (prototype) after initiation of the surge. The difference between 150 and 200 percent backflow was indistinguishable.

Steady conditions, with the entire discharge flowing over the weir, occurred about 45 minutes (prototype) after initiation of the surge. At this time, the water surface rise above the normal water surface elevation was approximately 1.1 feet near the downstream end of the 1,500-foot weir, approximately 1.2 feet at the upstream end of the Forebay Canal, and about 1.3 feet in the pooled Delta-Mendota Canal. The dimensions are referred to the normal water surface datum at elevation 173.2 (Forebay Canal datum), and have been corrected for the effects of surface tension and viscosity.

Development of the Side Weir Crest Shape

Because of the relatively small scale model, the true discharge characteristics of the prototype weir were uncertain, i.e., the model weir might be either more or less efficient than the prototype weir (excluding the range in which viscosity and surface tension are known to be important). Also, it was desired to develop a weir shape that would inhibit

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16/ Op. cit.

17/ "Characteristics of the Solitary Wave" by J. W. Daily and S. C. Stephan, Jr. Proceedings, American Society of Civil Engineers, Vol 77, Separate No. 107, December 1951.

18/ "Gradual Damping of Solitary Waves" by G. H. Keulegan, Journal of Research, National Bureau of Standards, Vol 40, 1948.

spilling due to waves formed by wind during normal operation of the canal and still maintain a satisfactory discharge capacity during emergency operation.

A 1:10 scale model of a 25-foot-long section of the weir was installed in a glass-sided flume. Observations of wave reflecting characteristics and measurements of the discharge coefficient were made for various configurations. Tests were first made on the original profile as installed in the 1:48 model. This profile and its approximate discharge coefficient are shown in Figure 19A. The original shape was modified by extending the crest horizontally upstream to provide a 6-inch vertical wall for reflection of wind waves, Figure 19B. This change resulted in a lower discharge coefficient and inadequate discharge capacity.

The profile was further modified in an attempt to increase the discharge coefficient by including a 12-inch-wide notch at the normal canal water surface, which is 6 inches below the weir crest. This modification provided a vertical face for reflection of waves but did not significantly alter the original crest shape. The profile and its coefficients are shown in Figure 20. Impingement of the flow on the upper portion of the vertical face was observed which could result in increased eddy losses and a reduced coefficient. The notch was therefore widened to 15 inches in an effort to aleviate this condition. The corresponding coefficients, Figure 20, show an improvement for heads below about 2 feet. Flow over the weir is shown in Figure 21 and the wave reflecting capabilities are demonstrated in Figure 22. This profile was recommended for inclusion in the final design. It must be noted that the discharge coefficients were measured under steady-state conditions, with stable heads. When the weir operates during passage of a surge wave, the vertical component of velocity should result in higher coefficients than those presented.

The coefficient of 3.2 for the preliminary profile of Figure 18 corresponds to a dimensionless coefficient ($C_w = C_d/\sqrt{2g}$) of approximately 0.4. The head of 1 foot (prototype) represents a Reynolds number of about 1,420 and a Weber number of about 5.4. From Figure 8, these values correspond to a dimensionless coefficient of approximately 0.38. These calculations show that for a head of 1 foot (prototype), the 1:48 weir and the 1:10 weir exhibit essentially the same discharge coefficient, suggesting that data from the 1:48 weir are reliable above the critical head of about 0.77 foot (prototype).

Details of the Recommended Design

The construction details of the recommended design are shown in Figures 23 through 26. Figure 23 exhibits the general configuration of the overflow weir and accompanying structures. The weir shape is shown in Figure 24. The elevation of the weir crest was raised 0.1 foot (prototype) above the crest elevation used in the model tests. This difference corresponds to only 0.002 foot in the model and should have no significant effect on the test results, except for the condition of sustained weir overflow. It can be assumed that water surface elevations for this condition would be 0.1 foot (prototype) higher than those indicated by the model. During discharge over the weir, the flow is accumulated in an unlined basin, then discharged down a baffled apron drop, Figure 25, into a wasteway. The wasteway passes under the Delta-Mendota Canal, as shown in the general plan, Figure 2. The alinement of the Delta-Mendota Canal with sections showing the height of concrete lining is also indicated in Figure 26. After the model studies were completed, the alinement of the Delta-Mendota Canal was revised to permit the abandonment of San Luis siphon and allow Forebay wasteway to be passed under the canal.

METRIC EQUIVALENTS

Metric equivalents of important quantities referred to in this report are listed in Table 6.

Table 1

COMPLETE REJECTION OF FLOW TO SIX PUMPS, THREE PUMPS, AND ONE PUMP, NO BACKFLOW

			No weir					1,500-ft side weir				
Inflow cfs	Rejection cfs	Backflow cfs	Probe Station	Probe Section	Ave surge height	Peak height	Ave surge velocity	Probe Station	Probe Section	Ave surge height	Peak height	Ave surge velocity
				1	1.63	1.92			1	1.54	1.72	
			12+90	2	1.54	1.54		18+66	2	1.54	1.87	
				3	1.54	1.73			3	1.54	1.87	
4,200	4,200	0		1	-	1.92	19.1		1	-	1.08	18.3
			2+85	2	-	1.54		2+85	2	-	0.87	
				3	-	1.92			3	-	1.01	
				1	0.72	0.72			1	0.96	1.01	
			12+90	2	0.62	0.62		18+66	2	-	0.86	
				3	0.77	0.77			3	0.96	1.10	
2,100	2,100	0		1	-	0.77	19.4		1	-	0.96	18.9
			2+85	2	-	0.48		2+85	2	-	1.15	
				3	-	0.67			3	-	0.96	
				1	0.34	0.34			1	-	0.19	
			12+90	2	0.29	0.34		18+66	2	-	0.38	
				3	0.34	0.34			3	-	0.19	
700	700	0		1	-	0.29	19.6		1	-	0.38	19.5
			2+85	2	-	0.19		2+85	2	-	0.34	
				3	-	0.19			3	-	0.19	

Surge heights are in prototype feet.
Surge velocities are in prototype feet per second.
Blank spaces (-) indicate that surge height could not be accurately determined because of small amplitude or interference by reflections.
Initial depth was 15.08 feet at Sta. 2+50.
See Figure 17 for locations of probe sections.

Table 2

COMPLETE REJECTION OF FLOW TO SIX PUMPS, THREE PUMPS, AND ONE PUMP, 150 PERCENT BACKFLOW

Inflow cfs	Rejection cfs	Backflow cfs	No weir					1,500-ft side weir				
			Probe Station	Probe Section	Ave surge height	Peak height	Ave surge velocity	Probe Station	Probe Section	Ave surge height	Peak height	Ave surge velocity
				1	1.63	3.50			1	1.54	4.48	
			12+90	2	1.54	3.65		18+66	2	1.54	4.02	
				3	1.54	3.55			3	1.54	4.61	
4,200	4,200	6,300		1	-	3.15	20.7		1	-	1.34	20.4
			2+85	2	-	3.10		2+85	2	-	1.20	
				3	-	2.90			3	-	1.34	
				1	0.72	2.11			1	0.96	2.26	
			12+90	2	0.62	1.92		18+66	2	-	2.26	
				3	0.77	2.11			3	0.96	2.40	
2,100	2,100	3,150		1	-	2.02	20.0		1	-	1.30	20.1
			2+85	2	-	1.78		2+85	2	-	1.44	
				3	-	1.92			3	-	1.34	
				1	0.34	0.77			1	-	0.77	
			12+90	2	0.29	0.63		18+66	2	-	0.67	
				3	0.34	0.77			3	-	0.91	
700	700	1,050		1	-	0.72	19.8		1	-	0.67	20.0
			2+85	2	-	0.63		2+85	2	-	0.86	
				3	-	0.72			3	-	0.86	

See notes on Table 1.

Table 3

COMPLETE REJECTION OF FLOW TO SIX PUMPS, THREE PUMPS, AND ONE PUMP, 200 PERCENT BACKFLOW

			No weir					1,500-ft side weir				
Inflow cfs	Rejection cfs	Backflow cfs	Probe Station	Probe Section	Ave surge height	Peak height	Ave surge velocity	Probe Station	Probe Section	Ave surge height	Peak height	Ave surge velocity
				1	1.63	4.50			1	1.54	5.47	
			12+90	2	1.54	5.20		18+66	2	1.54	4.90	
				3	1.54	5.00			3	1.54	5.57	
4,200	4,200	8,400		1	-	4.03	20.7		1	-	1.34	20.1
			2+85	2	-	3.80		2+85	2	-	1.20	
				3	-	4.03			3	-	1.34	
				1					1	0.96	2.83	
			12+90	2				18+66	2	-	2.54	
				3		No			3	0.96	2.98	
2,100	2,100	4,200		1					1	-	1.34	20.5
			2+85	2				2+85	2	-	1.15	
				3		data			3	-	1.44	
				1					1	-	0.96	
			12+90	2		taken		18+66	2	-	0.77	
				3					3	-	0.96	
700	700	1,400		1					1	-	0.86	20.2
			2+85	2				2+85	2	-	1.15	
				3					3	-	1.01	

See notes on Table 1.

Table 4

REJECTION OF FLOW TO THREE PUMPS WITH SIX PUMPS OPERATING
NO BACKFLOW, 150 PERCENT BACKFLOW, AND
200 PERCENT BACKFLOW

			No weir					1,500-ft side weir				
Inflow cfs	Rejection cfs	Backflow cfs	Probe Station	Probe Section	Ave surge height	Peak height	Ave surge velocity	Probe Station	Probe Section	Ave surge height	Peak height	Ave surge velocity
				1					1	0.82	0.96	
			12+96	2		No		18+66	2	0.91	1.15	
				3					3	0.86	1.01	
4,200	2,100	0		1		data			1	-	0.77	17.7
			2+23	2				2+85	2	-	0.62	
				3		taken			3	-	0.72	
				1					1	0.82	2.50	
			12+96	2	*0.96	*2.63		18+66	2	0.91	2.16	
				3					3	0.86	2.64	
4,200	2,100	3,150		1			19.9		1	-	1.06	19.0
			2+23	2	-	*2.27		2+85	2	-	0.82	
				3					3	-	1.01	
				1					1	0.82	3.02	
			12+96	2				18+66	2	0.91	2.83	
				3		No			3	0.86	3.22	
4,200	2,100	4,200		1					1	-	1.25	19.0
			2+23	2		data		2+85	2	-	1.06	
				3					3	-	1.30	
						taken						

*Probes at centerline only.
See notes on Table 1.

Table 5

REJECTION OF FLOW TO ONE PUMP WITH SIX PUMPS OPERATING
NO BACKFLOW, 150 PERCENT BACKFLOW, AND
200 PERCENT BACKFLOW

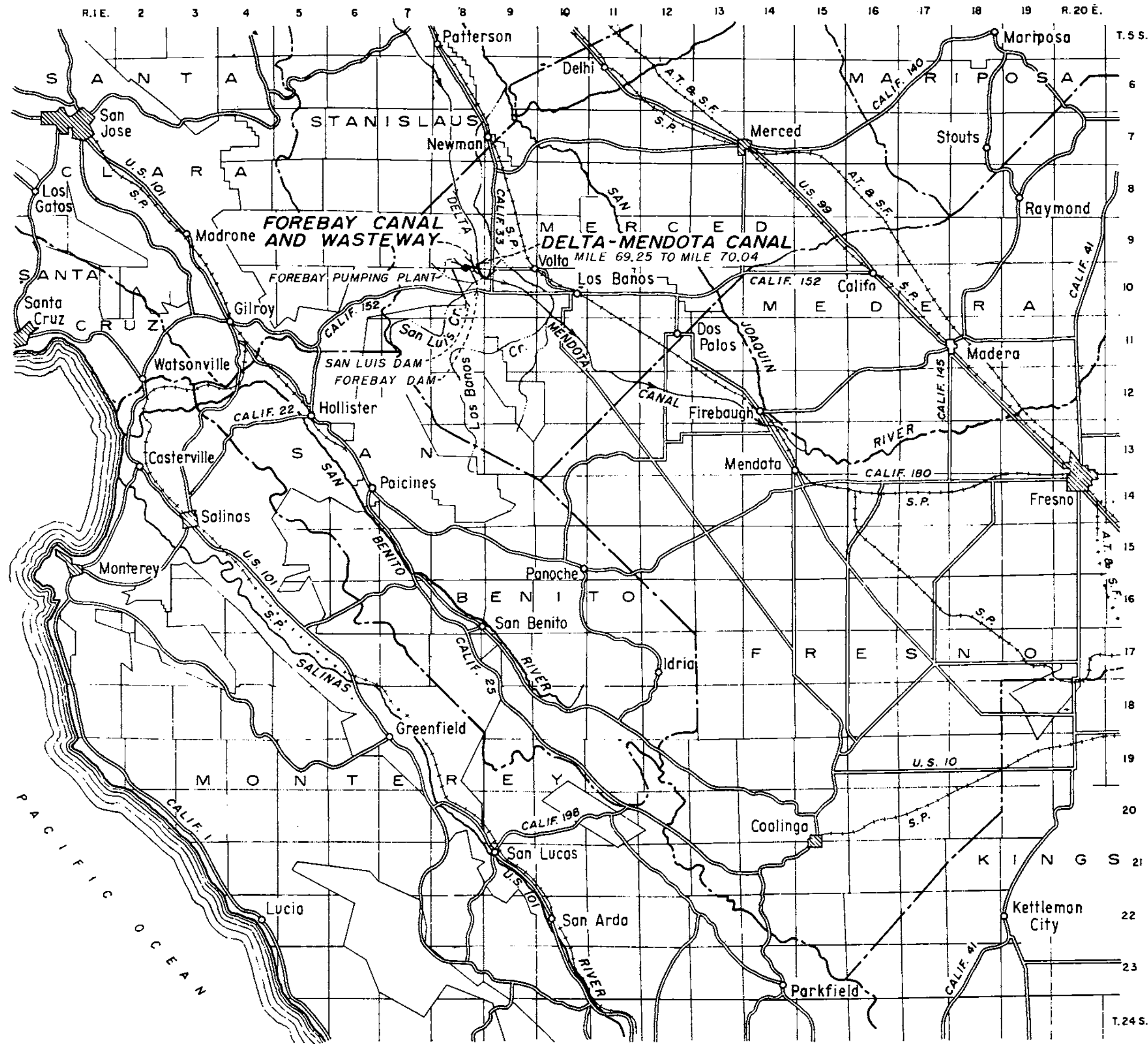
			No weir					1,500-ft side weir				
Inflow cfs	Rejection cfs	Backflow cfs	Probe Station	Probe Section	Ave surge height	Peak height	Ave surge velocity	Probe Station	Probe Section	Ave surge height	Peak height	Ave surge velocity
				1					1	-	0.38	
			12+96	2		No		18+66	2	-	0.34	
				3		data			3	-	0.29	
4,200	700	0		1		taken**			1	-	0.43	17.0
			2+23	2				2+85	2	-	0.34	
				3					3	-	0.24	
				1					1	-	0.96	
			12+96	2	-	*1.18		18+66	2	-	0.72	
				3					3	-	0.91	
4,200	700	1,050		1			18.0		1	-	0.77	17.8
			2+23	2	-	*0.73		2+85	2	-	0.77	
				3					3	-	0.77	
				1					1	-	0.96	
			12+96	2		No		18+66	2	-	0.82	
				3		data			3	-	1.06	
4,200	700	1,400		1		taken			1	-	0.77	17.7
			2+23	2				2+85	2	-	0.91	
				3					3	-	0.77	

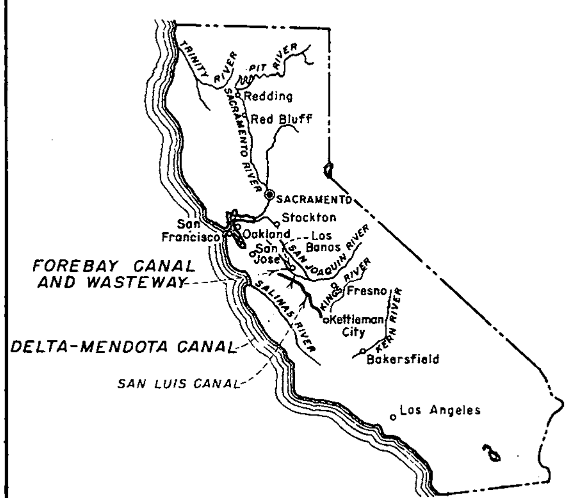
*Probes at centerline only.
**Should be identical to data for 1,500-ft weir since peaks are below weir crest.
See notes on Table 1.

Table 6

METRIC EQUIVALENTS OF IMPORTANT QUANTITIES

Bottom width of Forebay Canal	80 feet	24.4 meters
Flow depth in Forebay Canal	15 feet	4.6 meters
Maximum pumping plant capacity	4,200 cubic feet per second	118.9 cubic meters per second
Length of preliminary side weir	2,073 feet	631.9 meters
Length of recommended side weir	1,500 feet	457.2 meters
Peak surge height following rejection of maximum discharge (1)	1.9 feet	0.58 meters
Peak surge height with 150 percent backflow (2)	4.5 feet	1.37 meters
Peak surge height with 200 percent backflow (3)	5.4 feet	1.65 meters
Average surge velocity for (1)	19.1 feet per second	5.82 meters per second
Average surge velocity for (2) and (3)	20.7 feet per second	6.31 meters per second
Average surge height for (1), (2), and (3)	1.5 feet	0.46 meters



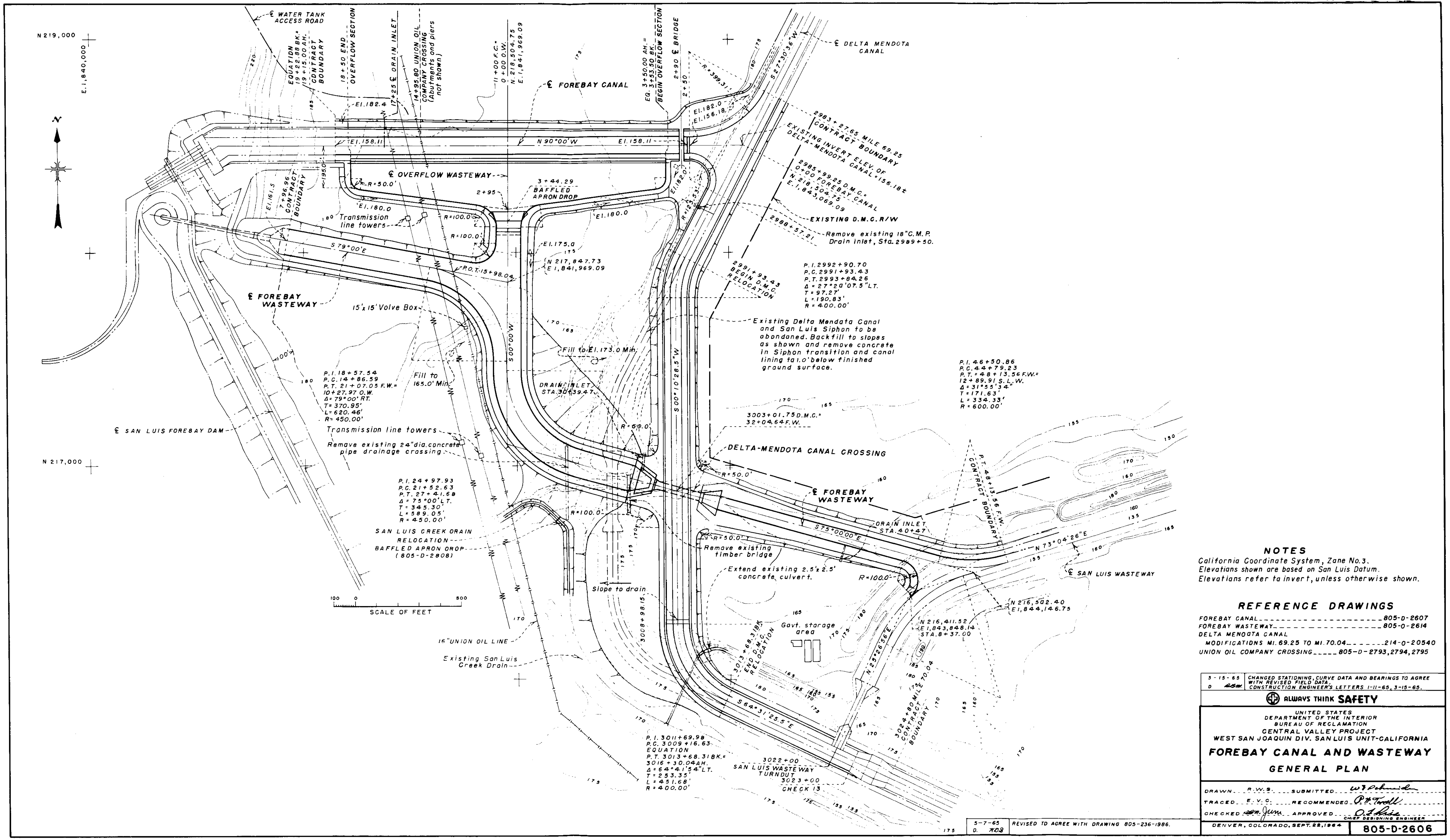


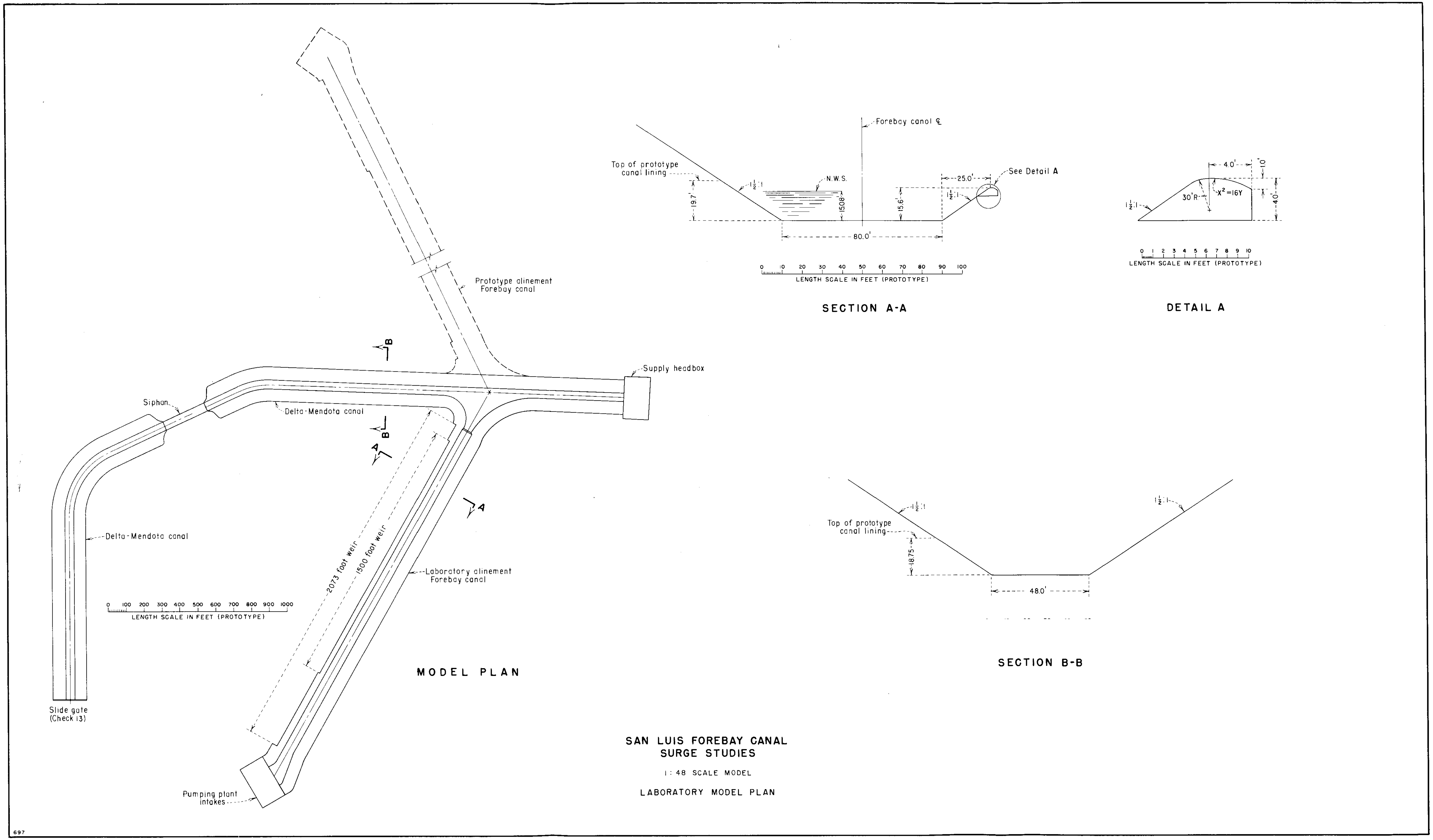
KEY MAP

ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT
WEST SAN JOAQUIN AND DELTA DIVISIONS-CALIFORNIA
FOREBAY CANAL AND WASTEWAY
AND
DELTA-MENDOTA CANAL MODIFICATIONS
MILE 69.25 TO MILE 70.04
LOCATION MAP

DRAWN D.B.M. SUBMITTED P. W. Terrell
TRACED RECOMMENDED O. L. Rice
CHECKED APPROVED
CHIEF ENGINEER
DENVER, COLORADO SEPT. 22, 1964 805-D-2605

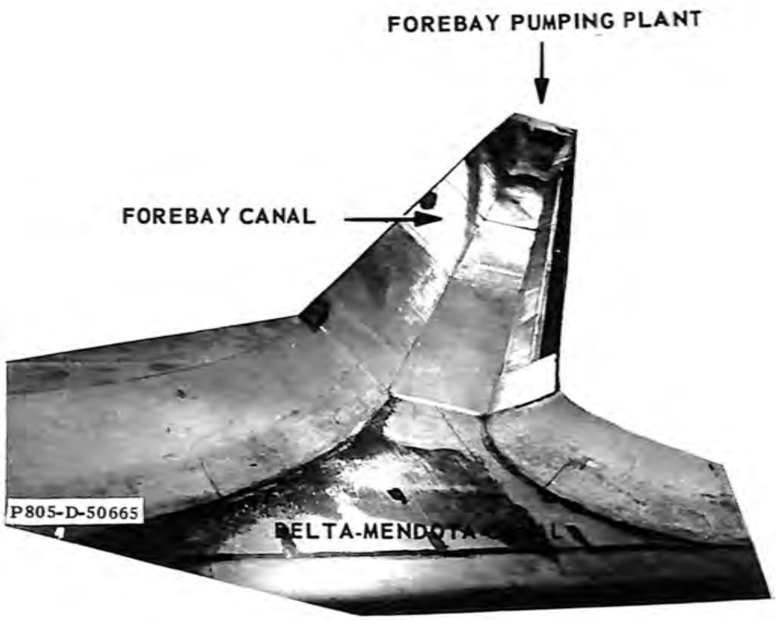

℄ FOREBAY CANAL
℄ OVERFLOW WASTEWAY
℄ FOREBAY WASTEWAY
℄ SAN LUIS FOREBAY DAM
℄ DELTA MENDOTA CANAL
DELTA-MENDOTA CANAL CROSSING
℄ SAN LUIS WASTEWAY
SCALE OF FEET
NOTES
California Coordinate System, Zone No. 3.
Elevations shown are based on San Luis Datum.
Elevations refer to invert, unless otherwise shown.
REFERENCE DRAWINGS
FOREBAY CANAL 805-D-2607
FOREBAY WASTEWAY 805-D-2614
DELTA MENDOTA CANAL
MODIFICATIONS MI. 69.25 TO MI. 70.04 214-D-20540
UNION OIL COMPANY CROSSING 805-D-2793, 2794, 2795
ALWAYS THINK SAFETY
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT
WEST SAN JOAQUIN DIV. SAN LUIS UNIT-CALIFORNIA
FOREBAY CANAL AND WASTEWAY
GENERAL PLAN
DENVER, COLORADO, SEPT. 25, 1964
805-D-2606

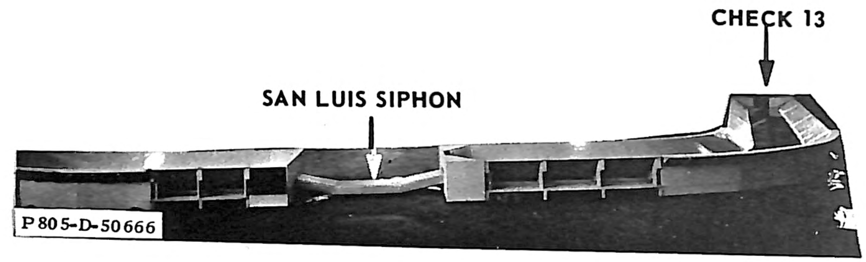


SAN LUIS FOREBAY CANAL
SURGE STUDIES

1 : 48 SCALE MODEL

LABORATORY MODEL PLAN



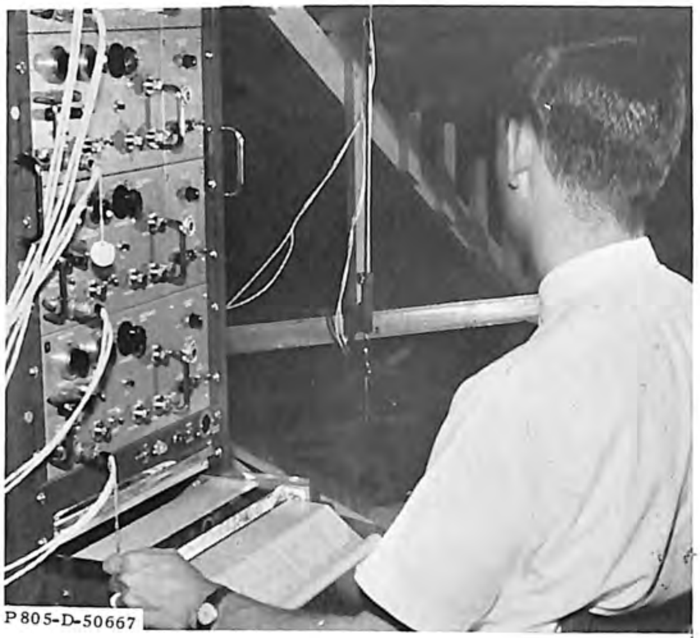


SAN LUIS FOREBAY CANAL
SURGE STUDIES

1:48 Scale Model

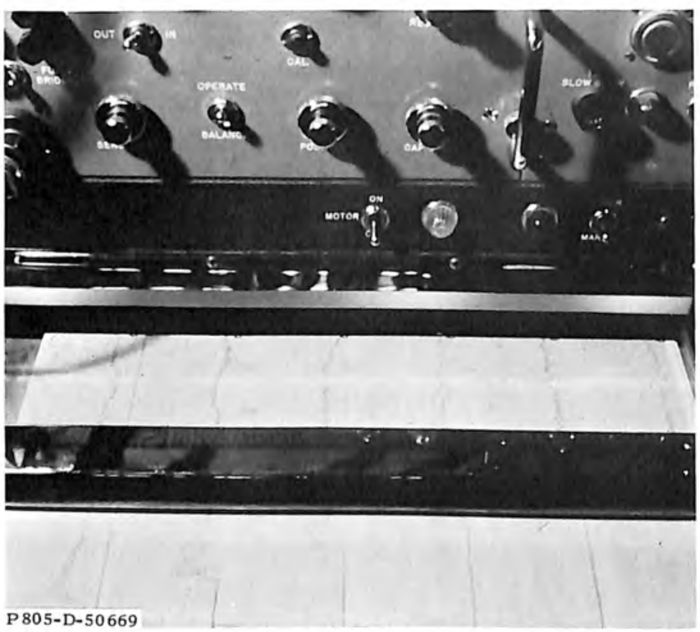
Model Configuration

Figure 5
Report Hyd-546



A. Wave probe and recorder

B. Capacitance wave probe



C. Oscillograph recording of
wave forms

SAN LUIS FOREBAY CANAL
SURGE STUDIES

1:48 Scale Model

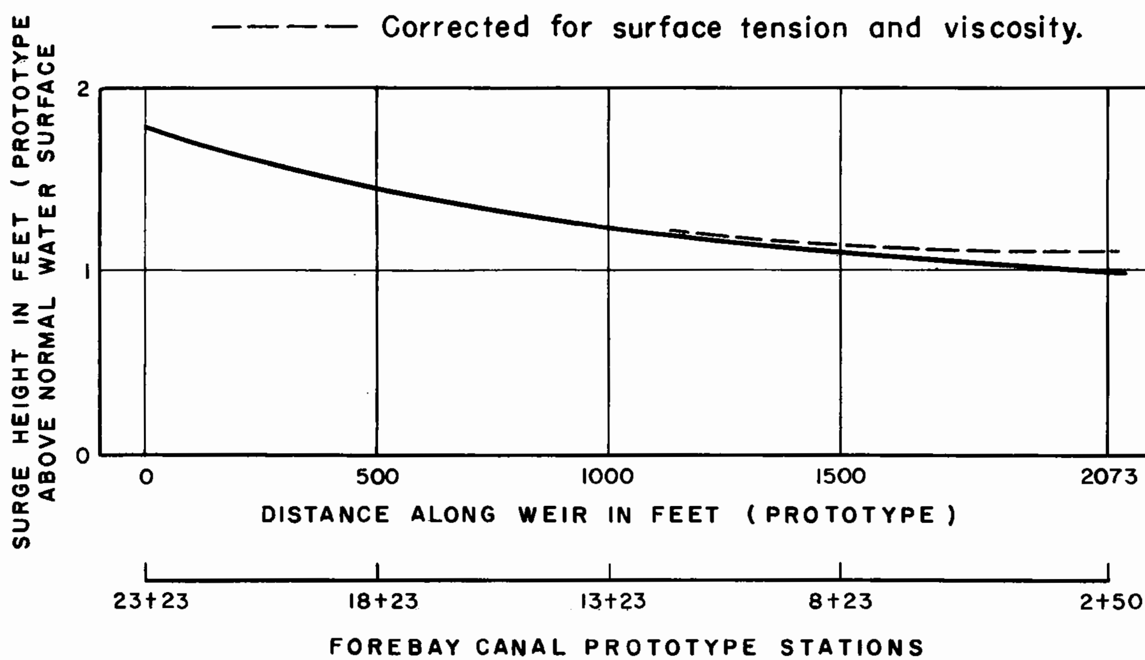
Model Instrumentation

Figure 6
Report Hyd-546

SAN LUIS FOREBAY CANAL
SURGE STUDIES

1:48 Scale Model

Backflow Device



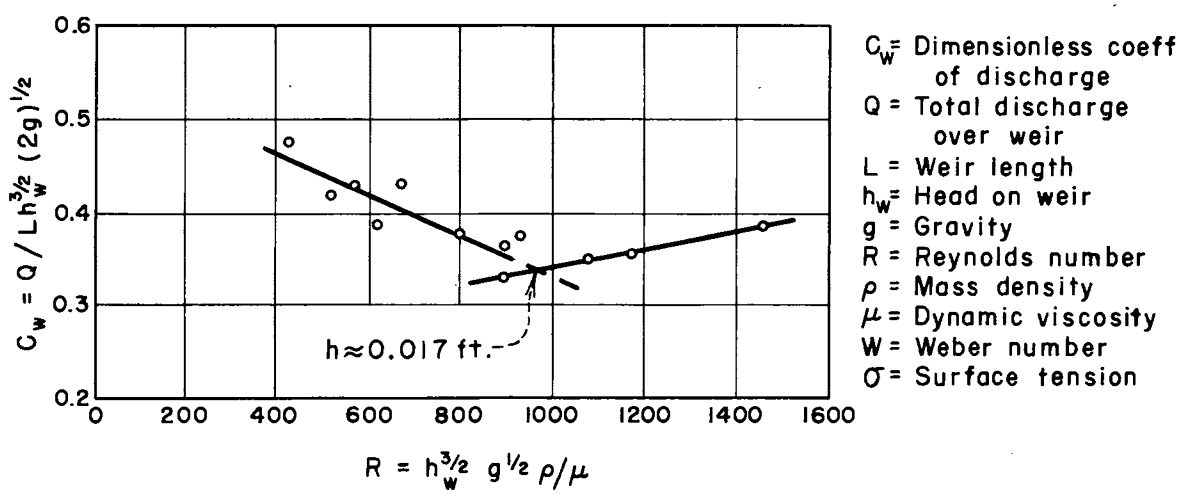
SAN LUIS FOREBAY CANAL
SURGE STUDIES

1:48 SCALE MODEL

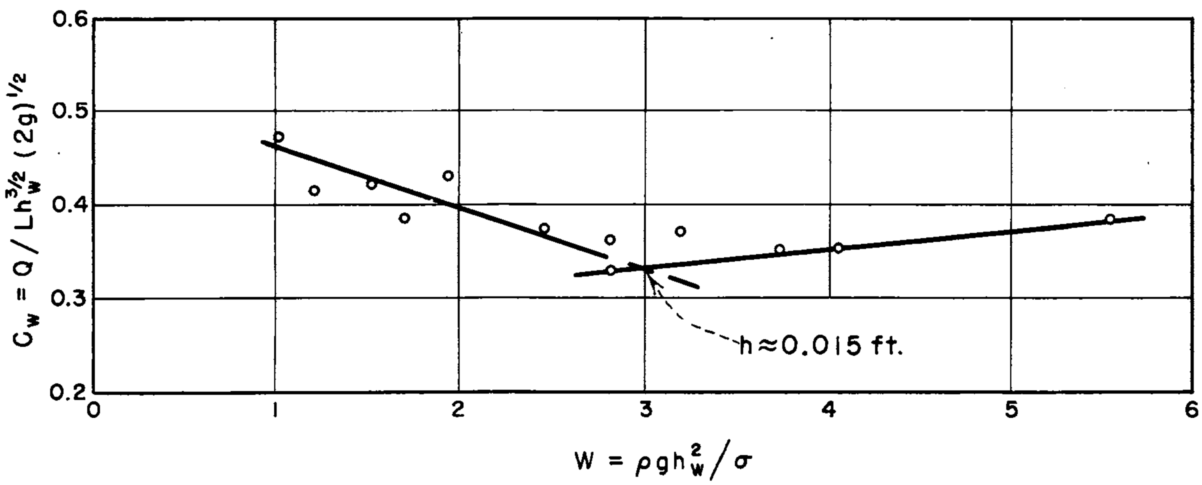
ATTENUATING EFFECT OF 2073-FOOT-LONG WEIR ON SURGE CAUSED BY COMPLETE REJECTION OF 4200 CFS

FIGURE 7
REPORT HYD-546

697



A. EFFECT OF VISCOSITY



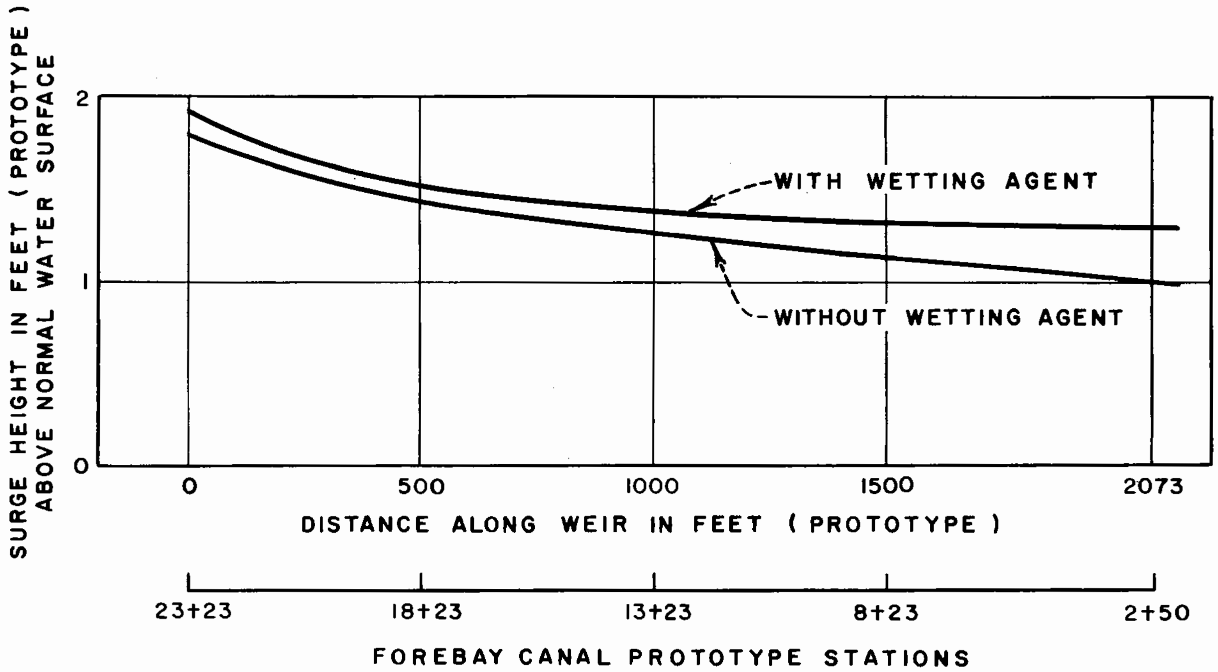
B. EFFECT OF SURFACE TENSION

SAN LUIS FOREBAY CANAL
SURGE STUDIES

1:48 SCALE MODEL

EFFECTS OF VISCOSITY AND SURFACE TENSION
ON WEIR DISCHARGE COEFFICIENT

FIGURE 9
REPORT HYD-546



SAN LUIS FOREBAY CANAL
SURGE STUDIES

1:48 SCALE MODEL

EFFECT OF WETTING AGENT ON ATTENUATING ABILITY OF WEIR FOR SURGE CAUSED BY COMPLETE REJECTION OF 4200 CFS

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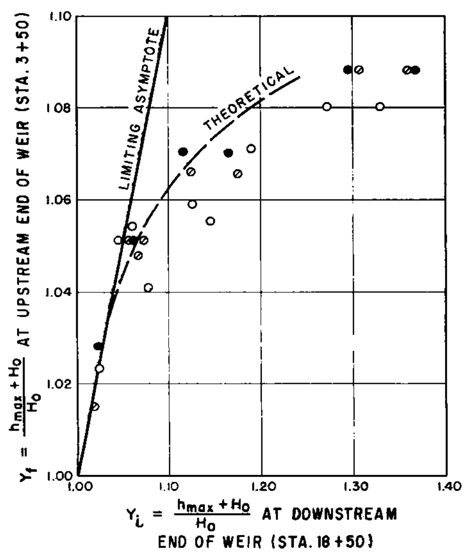
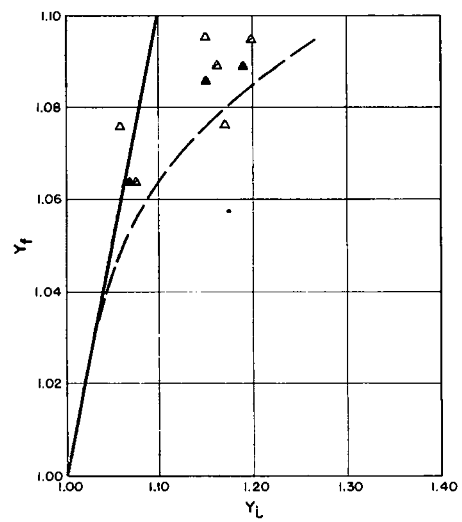
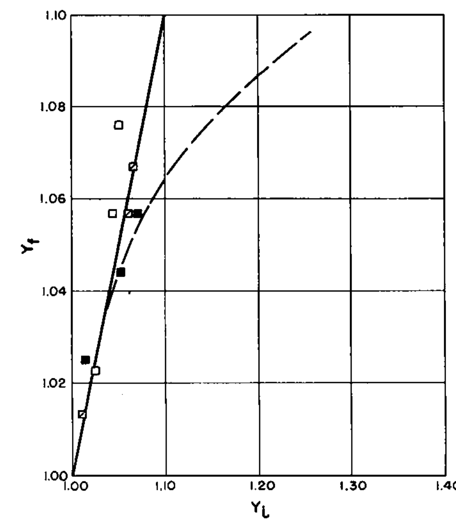


A. INFLOW OF 4200 C.F.S.
F_o = 0.123



B. INFLOW OF 2100 C.F.S.
F_o = 0.062



C. INFLOW OF 700 C.F.S.
F_o = 0.021

NOTES

Theoretical curves are derived from Citrinis equation (See Appendix to this report)., for a weir discharge coefficient of 3.5.

●, ▲, ■ Probe Section 1 (See figure 17)
○, △, □ Probe Section 2
⊘, ◬, ⊠ Probe Section 3

H_o = Initial canal depth (Figure 12)
h_{max} = Height of maximum surge peak above initial canal depth.
F_o = Froude number of initial flow.

SAN LUIS FOREBAY CANAL
SURGE STUDIES

1 : 48 SCALE MODEL

ATTENUATION OF MAXIMUM PEAKS BY
1500-FOOT LONG SIDE WEIR

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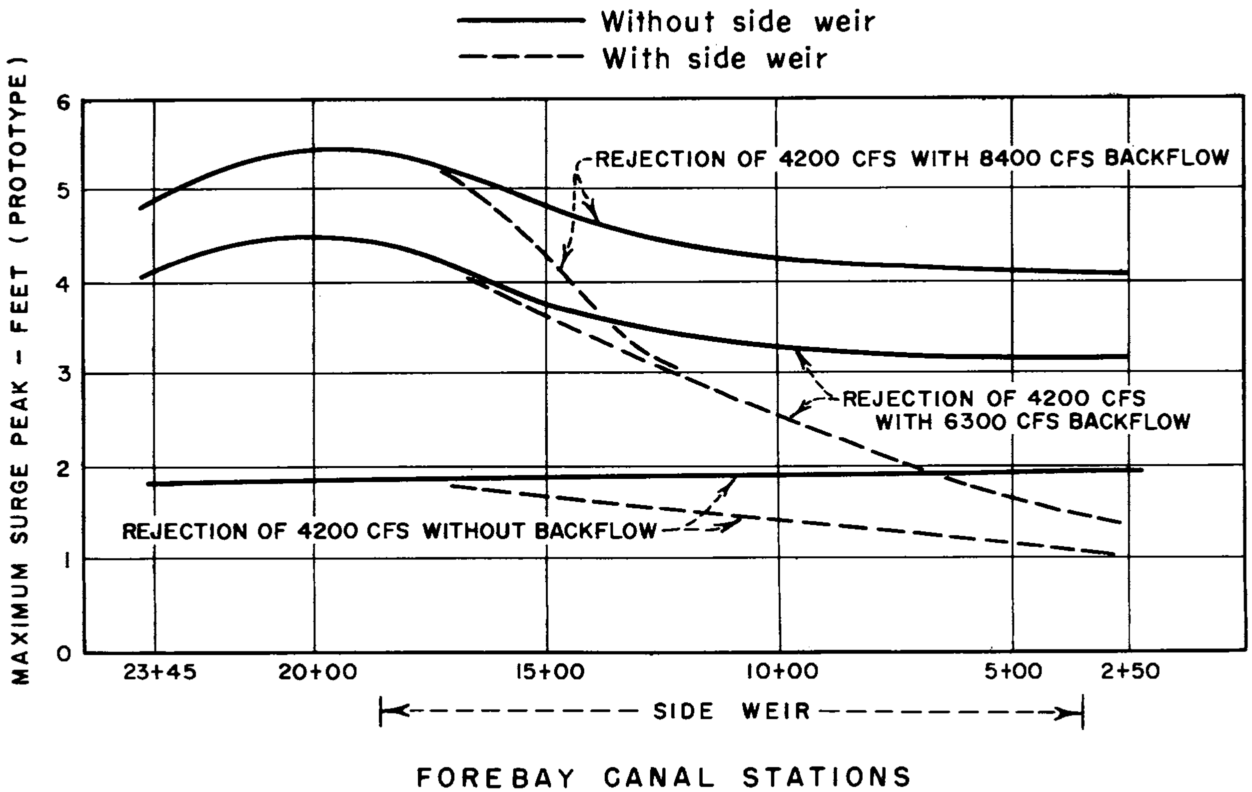
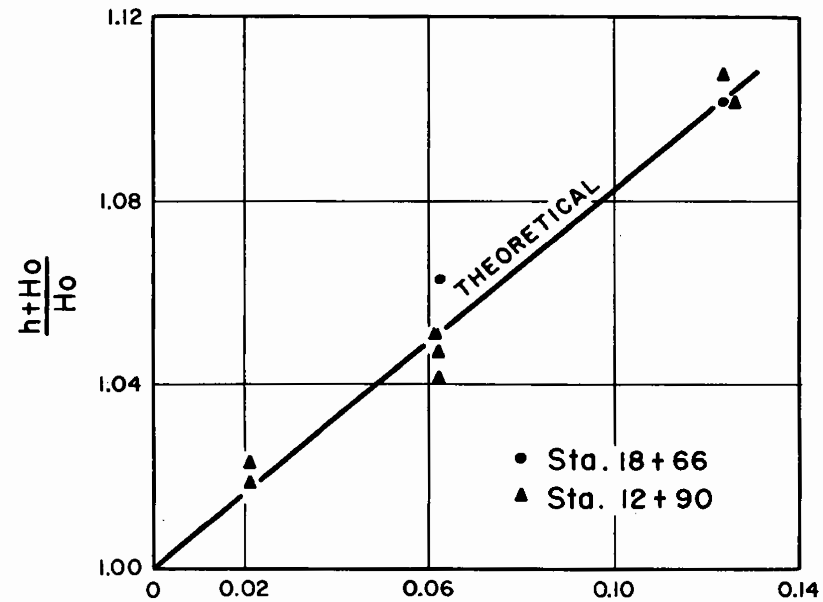
FIGURE 10
REPORT HYD-546



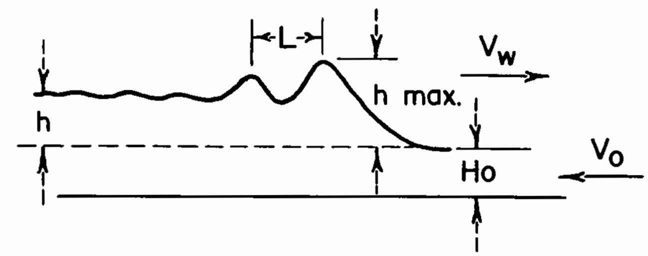
SAN LUIS FOREBAY CANAL SURGE STUDIES

1:48 SCALE MODEL

VARIATION OF MAXIMUM PEAK ALONG CHANNEL WITH AND WITHOUT SIDE WEIR



$$F_o = \frac{V_o}{\sqrt{gH_o}}$$



DEFINITION SKETCH

SAN LUIS FOREBAY CANAL
SURGE STUDIES

1 : 48 SCALE MODEL

VARIATION OF AVERAGE SURGE HEIGHT
WITH FROUDE NUMBER OF INITIAL FLOW

FIGURE 12
REPORT HYD-546

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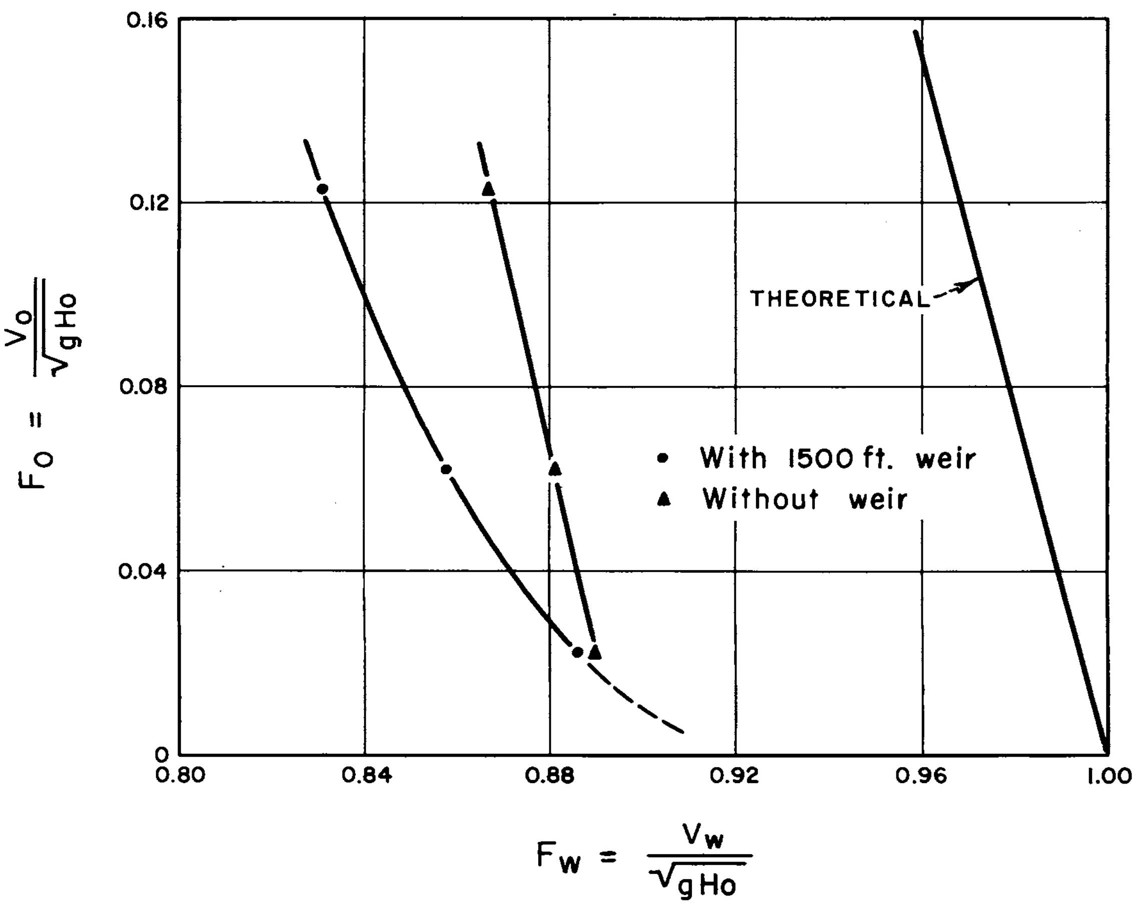


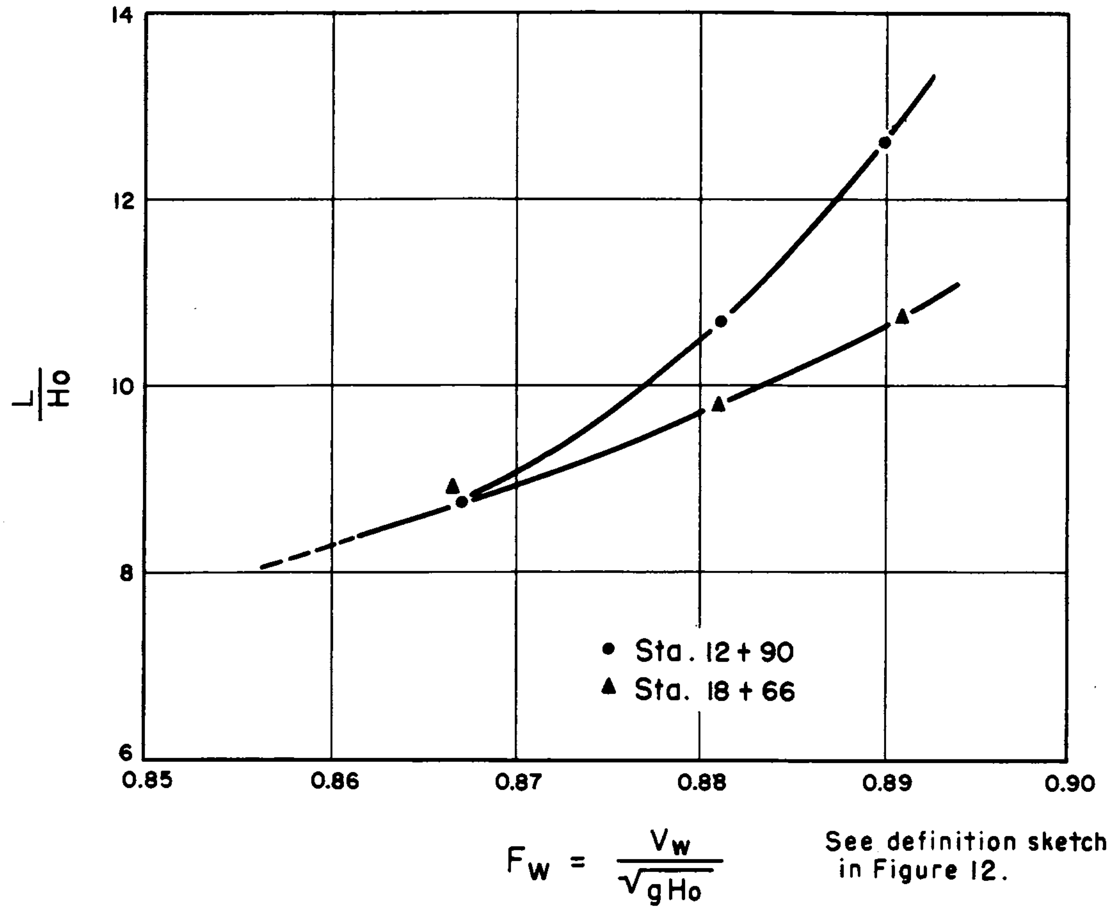
See Definition Sketch in Figure 12.

SAN LUIS FOREBAY CANAL
SURGE STUDIES

1 : 48 SCALE MODEL

VARIATION OF AVERAGE SURGE VELOCITY WITH
FROUDE NUMBER OF INITIAL FLOW

697

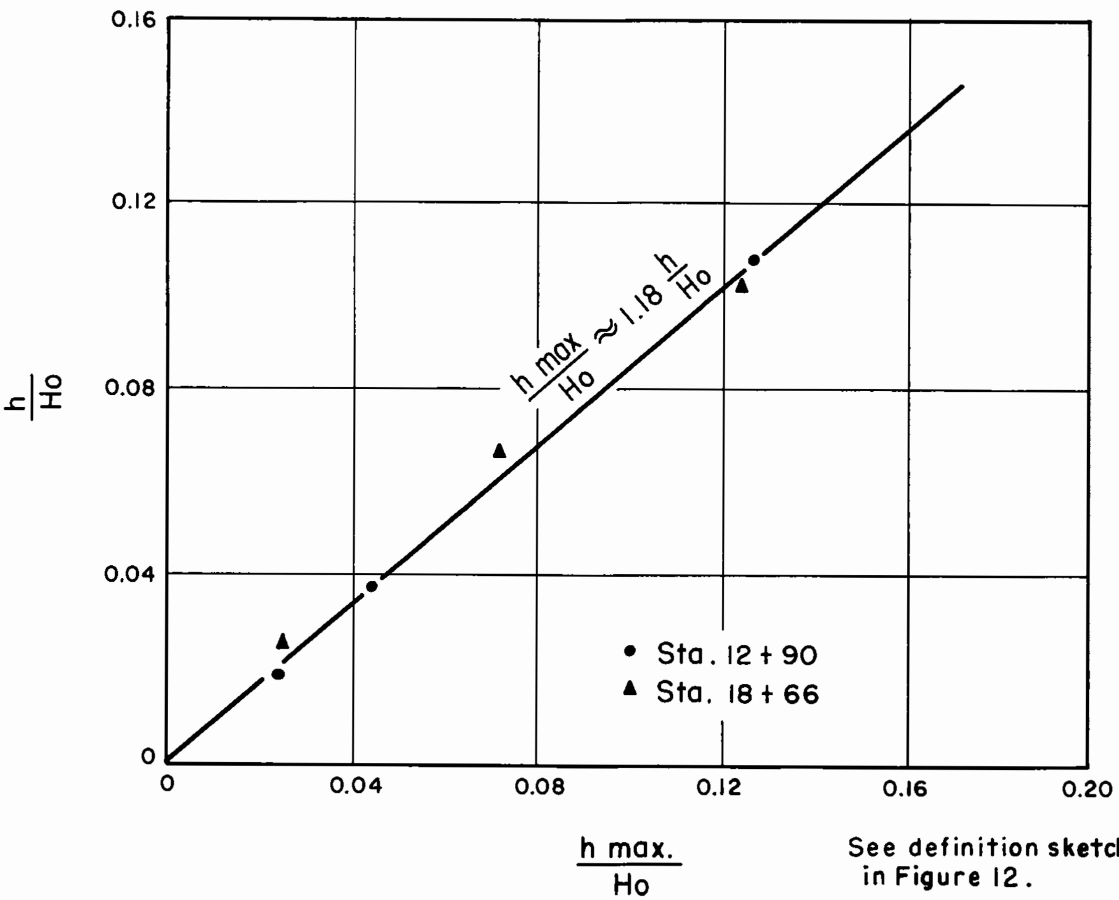


See definition sketch in Figure 12.

SAN LUIS FOREBAY CANAL
SURGE STUDIES

1 : 48 SCALE MODEL

VARIATION OF FIRST WAVE LENGTH WITH WAVE VELOCITY

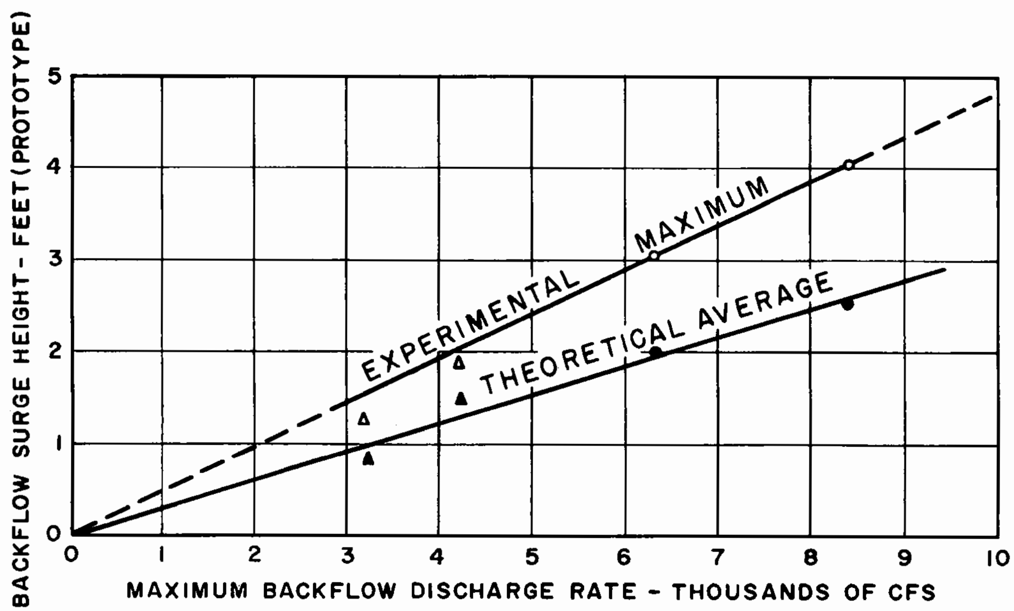


See definition sketch in Figure 12.

SAN LUIS FOREBAY CANAL
SURGE STUDIES

1:48 SCALE MODEL

AVERAGE SURGE HEIGHT–
MAXIMUM PEAK RELATIONSHIP



NOTES

Backflow surge heights measured above average height of rejection surge.

○ ● Full rejection of 4200 cfs inflow

△ ▲ Full rejection of 2100 cfs inflow

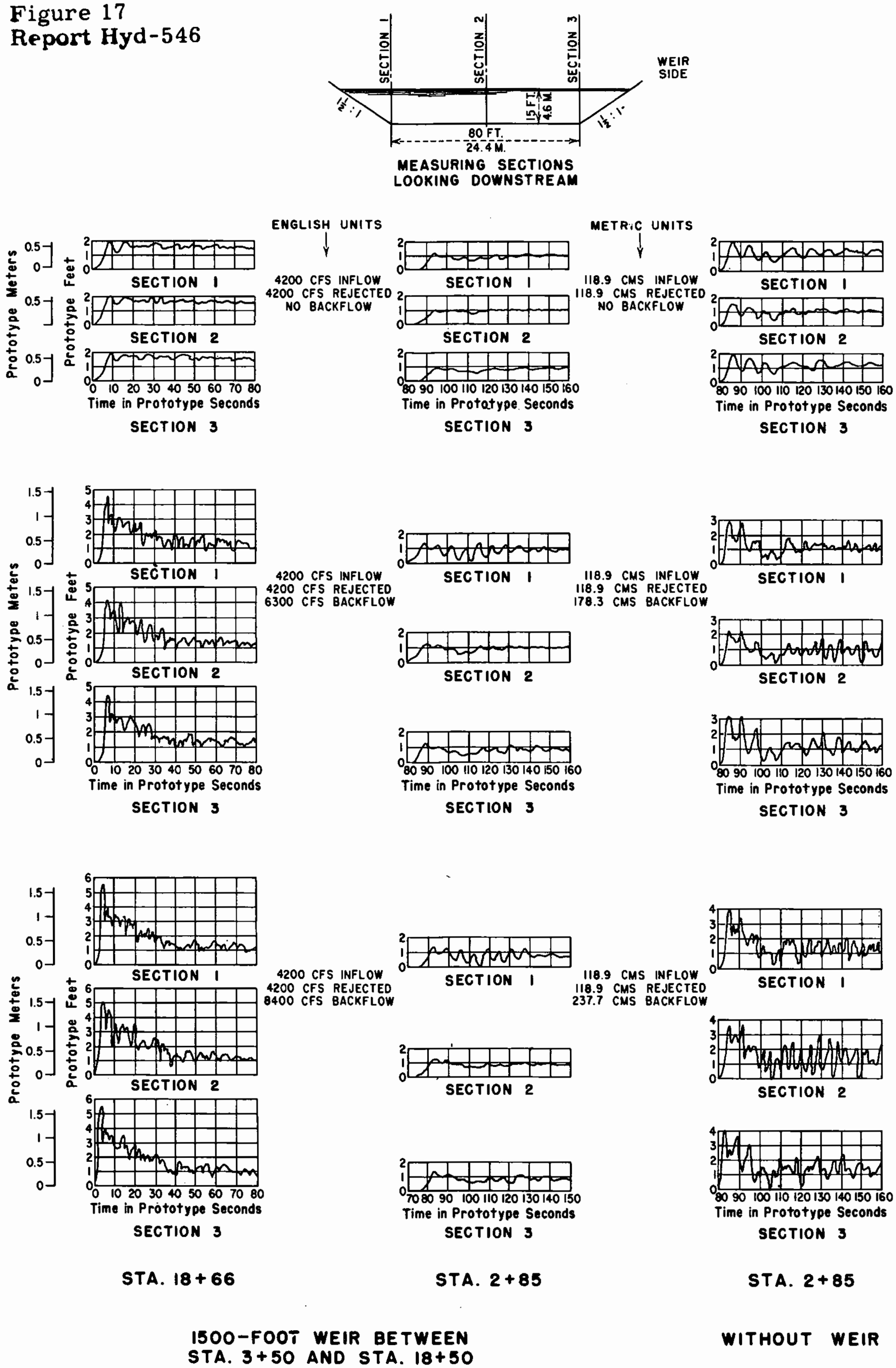
Measured at Sta. 18 + 66 on left side looking downstream.

SAN LUIS FOREBAY CANAL
SURGE STUDIES

1 : 48 SCALE MODEL

VARIATION OF BACKFLOW SURGE HEIGHT WITH BACKFLOW DISCHARGE RATE

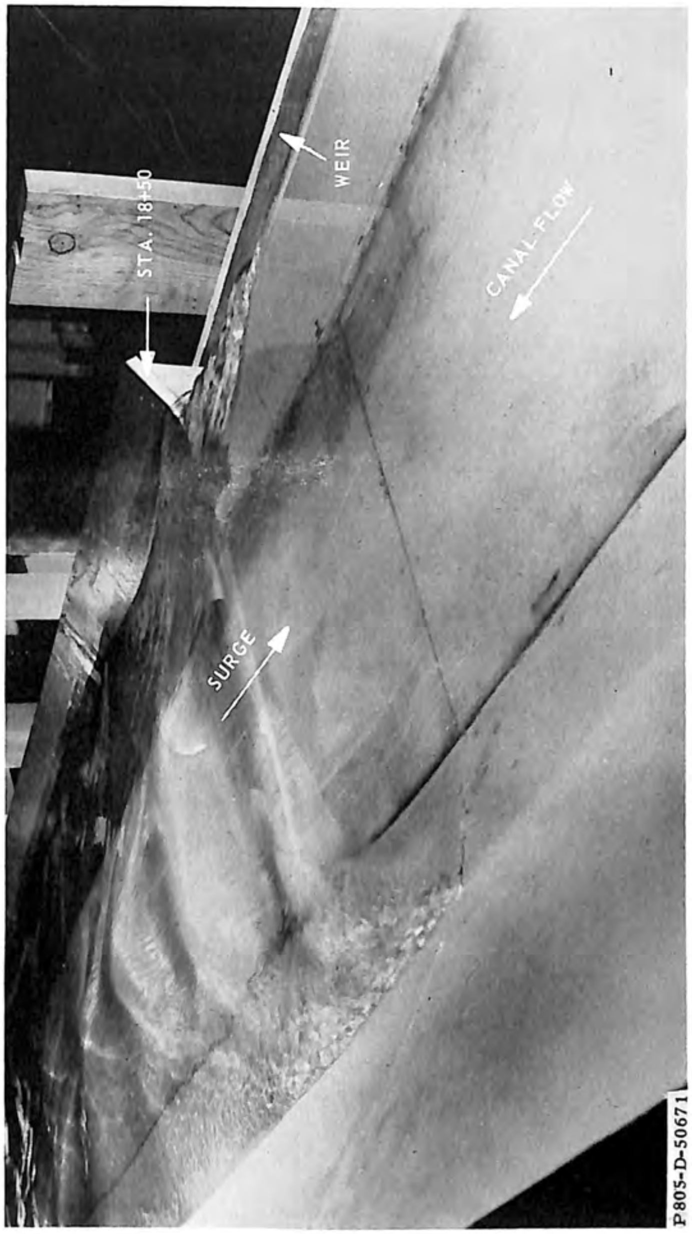
Figure 17
Report Hyd-546



SAN LUIS FOREBAY CANAL
SURGE STUDIES
1:48 Scale Model

Longitudinal Wave Forms

Figure 18
Report Hyd-546

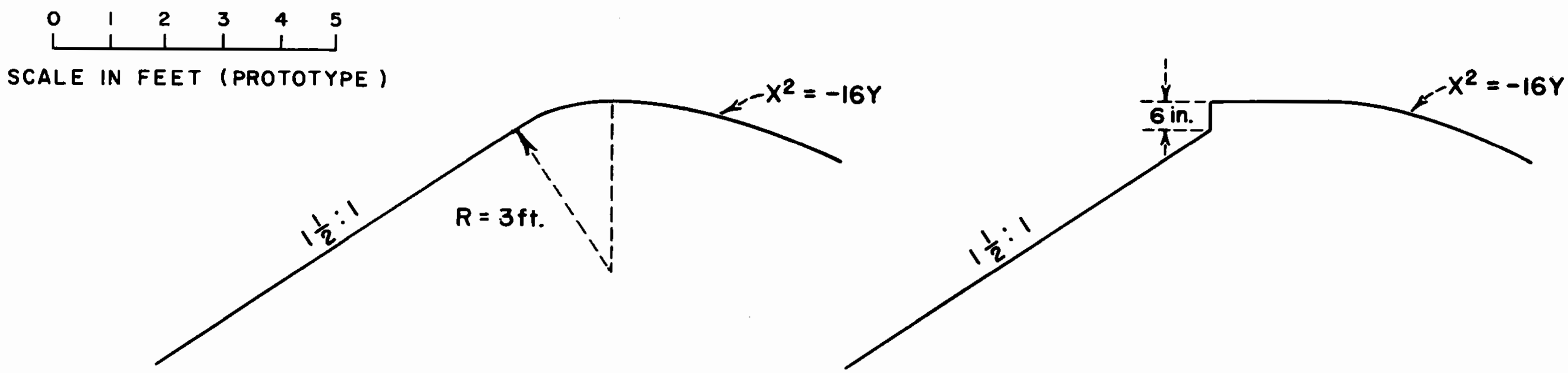


SAN LUIS FOREBAY CANAL
SURGE STUDIES

1:48 Scale Model

Surge Wave for Rejection of 4,200 cfs
Plus Backflow of 6,300 cfs

Note breaking leading edges



PRELIMINARY WEIR CREST
$C_D \approx 3.2$ at H = 1.0 ft.

PRELIMINARY CREST WITH 6-INCH WAVE SCUPPER
$C_D \approx 2.9$ at H = 1.0 ft.

SAN LUIS FOREBAY CANAL
SURGE STUDIES

1:10 SCALE MODEL

DEVELOPMENT OF SIDE WEIR PROFILE SHAPE
PRELIMINARY DESIGNS

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FIGURE 19
REPORT HYD-546

Figure 21
Report Hyd-546

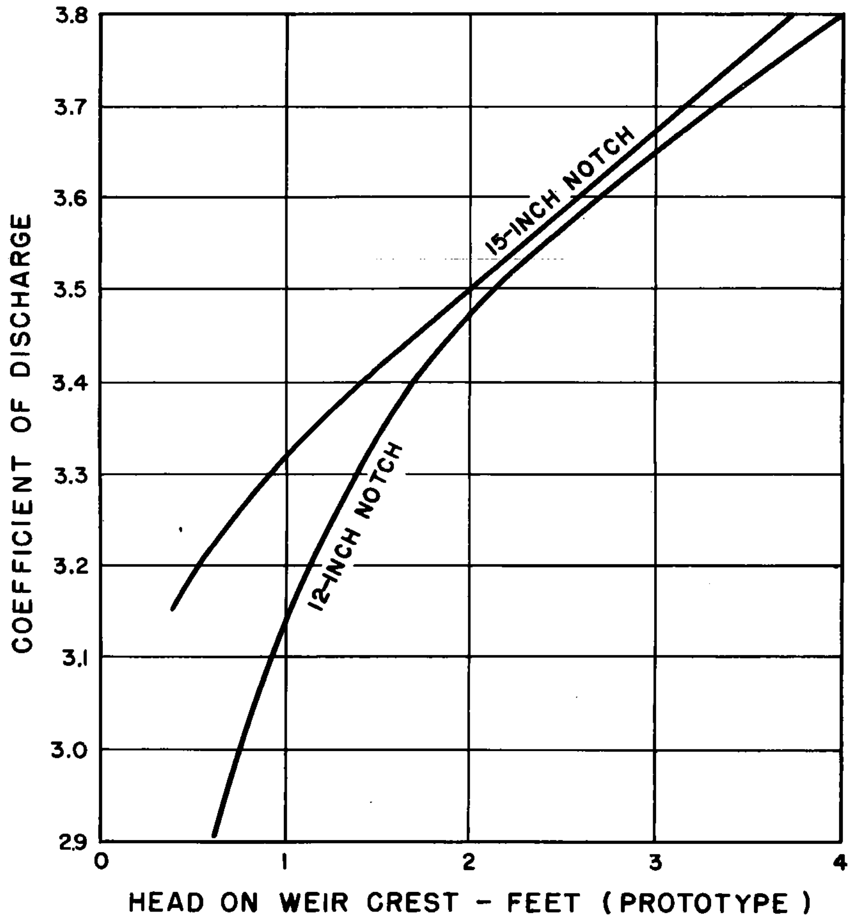
A. $h_w = 1.0$ foot.

B. $h_w = 3.0$ feet.

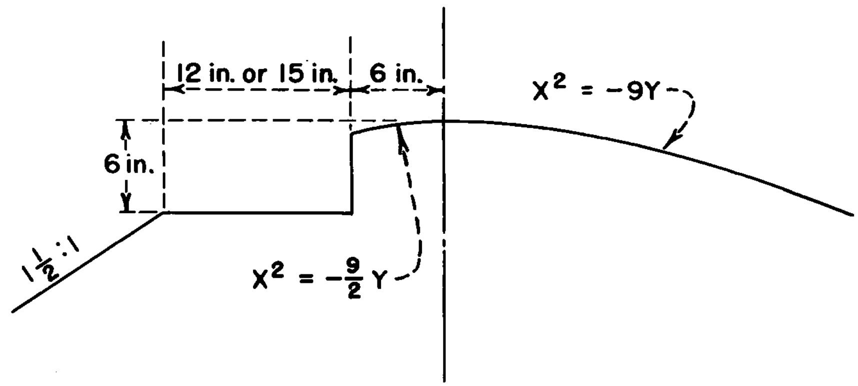
SAN LUIS FOREBAY CANAL
SURGE STUDIES

1:10 Scale Model

Flow Over Recommended Weir Profile



Note: 15-inch notch is included in the final design.



RECOMMENDED WEIR CREST

SAN LUIS FOREBAY CANAL
SURGE STUDIES

1:10 SCALE MODEL

DEVELOPMENT OF SIDE WEIR PROFILE SHAPE
RECOMMENDED DESIGN

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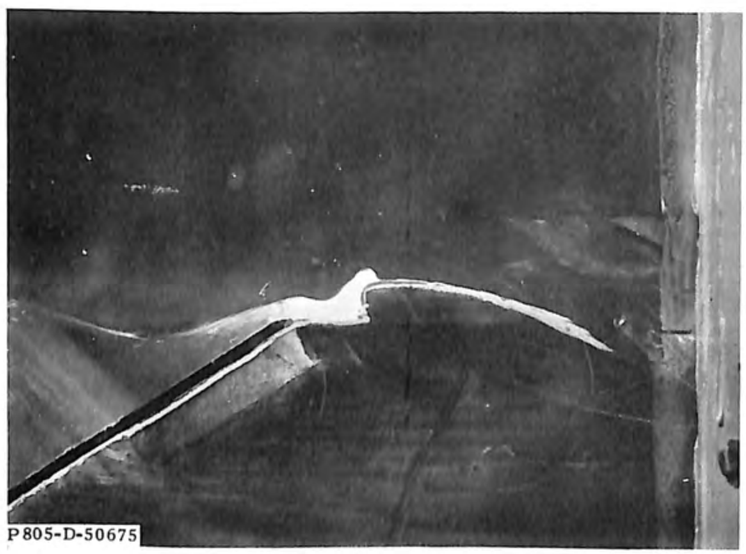
FIGURE 20
REPORT HYD-546

P805-D-50674

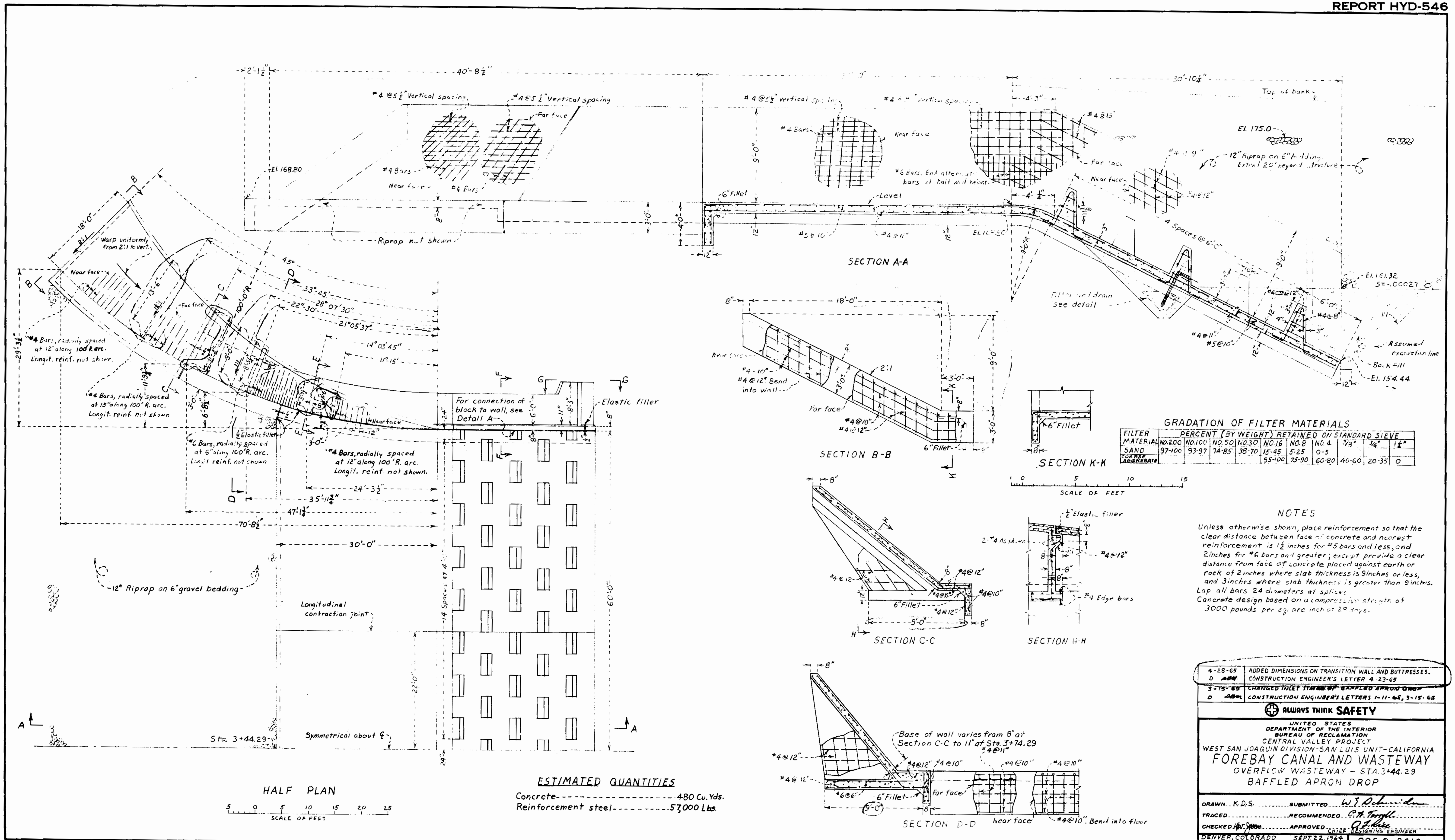


P805-D-50675

SAN LUIS FOREBAY CANAL
SURGE STUDIES

1:10 Scale Model

Waves Impinging on
Recommended Weir Profile



ESTIMATED QUANTITIES

Concrete ---------------------- 480 Cu. Yds.
Reinforcement steel ---------- 57,000 Lbs.

GRADATION OF FILTER MATERIALS

FILTER MATERIAL	PERCENT (BY WEIGHT) RETAINED ON STANDARD SIEVE									
	NO.200	NO.100	NO.50	NO.30	NO.16	NO.8	NO.4	3/8"	3/4"	1½"
SAND	97-100	93-97	74-85	38-70	15-45	5-25	0-5			
COARSE AGGREGATE					95-100	75-90	60-80	40-60	20-35	0

NOTES

Unless otherwise shown, place reinforcement so that the clear distance between face of concrete and nearest reinforcement is 1½ inches for #5 bars and less, and 2 inches for #6 bars and greater; except provide a clear distance from face of concrete placed against earth or rock of 2 inches where slab thickness is 9 inches or less, and 3 inches where slab thickness is greater than 9 inches.
Lap all bars 24 diameters at splices.
Concrete design based on a compressive strength of 3000 pounds per square inch at 28 days.

4-28-65 D	ADDED DIMENSIONS ON TRANSITION WALL AND BUTTRESSES. CONSTRUCTION ENGINEER'S LETTER 4-23-65
3-15-65 D	CHANGED INLET STATION OF BAFFLED APRON DROP CONSTRUCTION ENGINEER'S LETTERS 1-11-65, 3-15-65

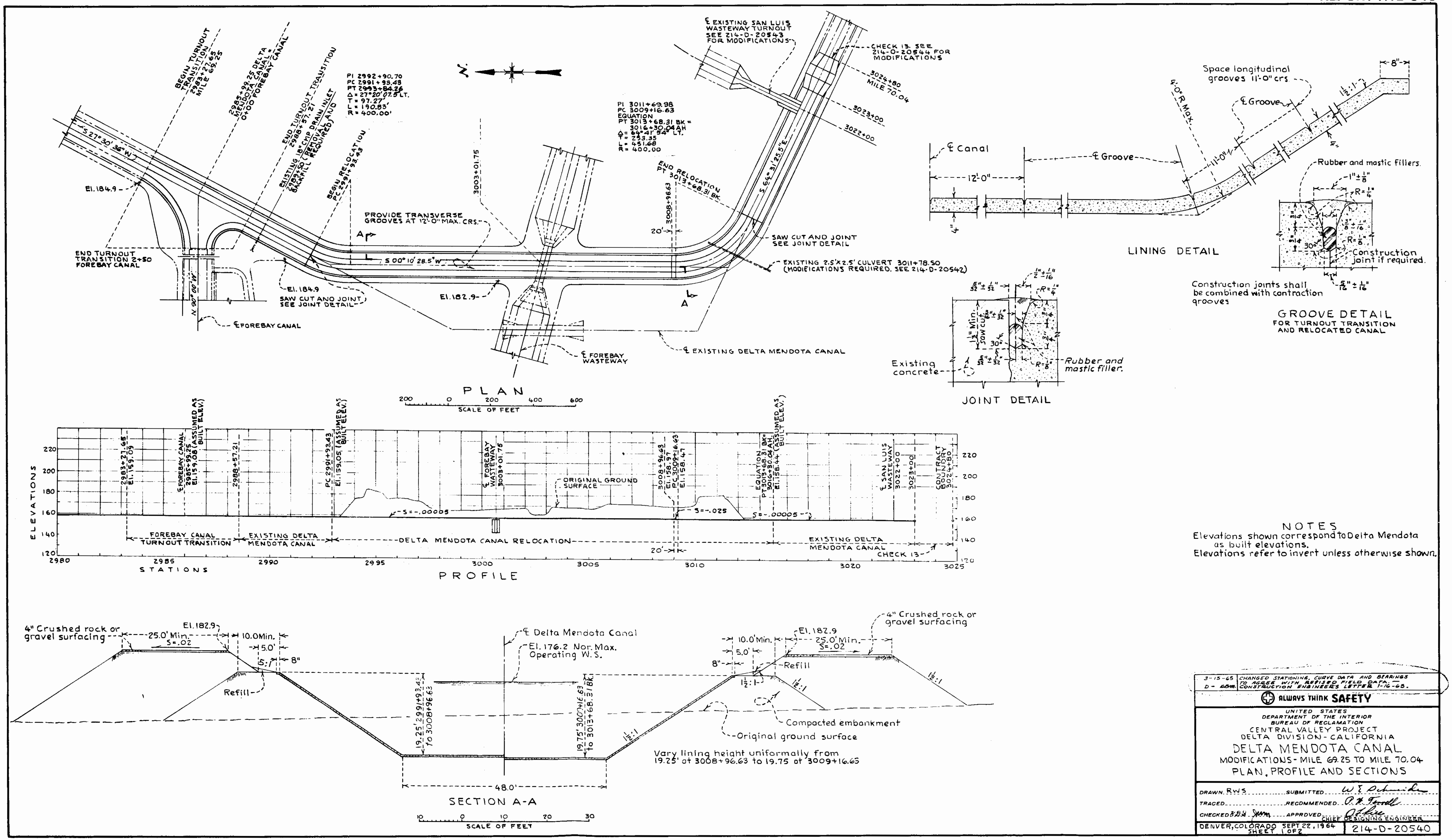
ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT
WEST SAN JOAQUIN DIVISION-SAN LUIS UNIT-CALIFORNIA

FOREBAY CANAL AND WASTEWAY

OVERFLOW WASTEWAY - STA. 3+44.29
BAFFLED APRON DROP

DRAWN K.D.S. SUBMITTED W. E. Schneider
TRACED RECOMMENDED C. H. Terrell
CHECKED H.W. J. APPROVED O. J. Rice
CHIEF DESIGNING ENGINEER
DENVER, COLORADO SEPT. 22, 1964
SHEET 1 OF 2
805-D-2612



ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT
DELTA DIVISION - CALIFORNIA

DELTA MENDOTA CANAL
MODIFICATIONS - MILE 69.25 TO MILE 70.04
PLAN, PROFILE AND SECTIONS

DENVER, COLORADO SEPT 22, 1964 SHEET 1 OF 2
214-D-20540

FIGURE 23
REPORT HYD-546

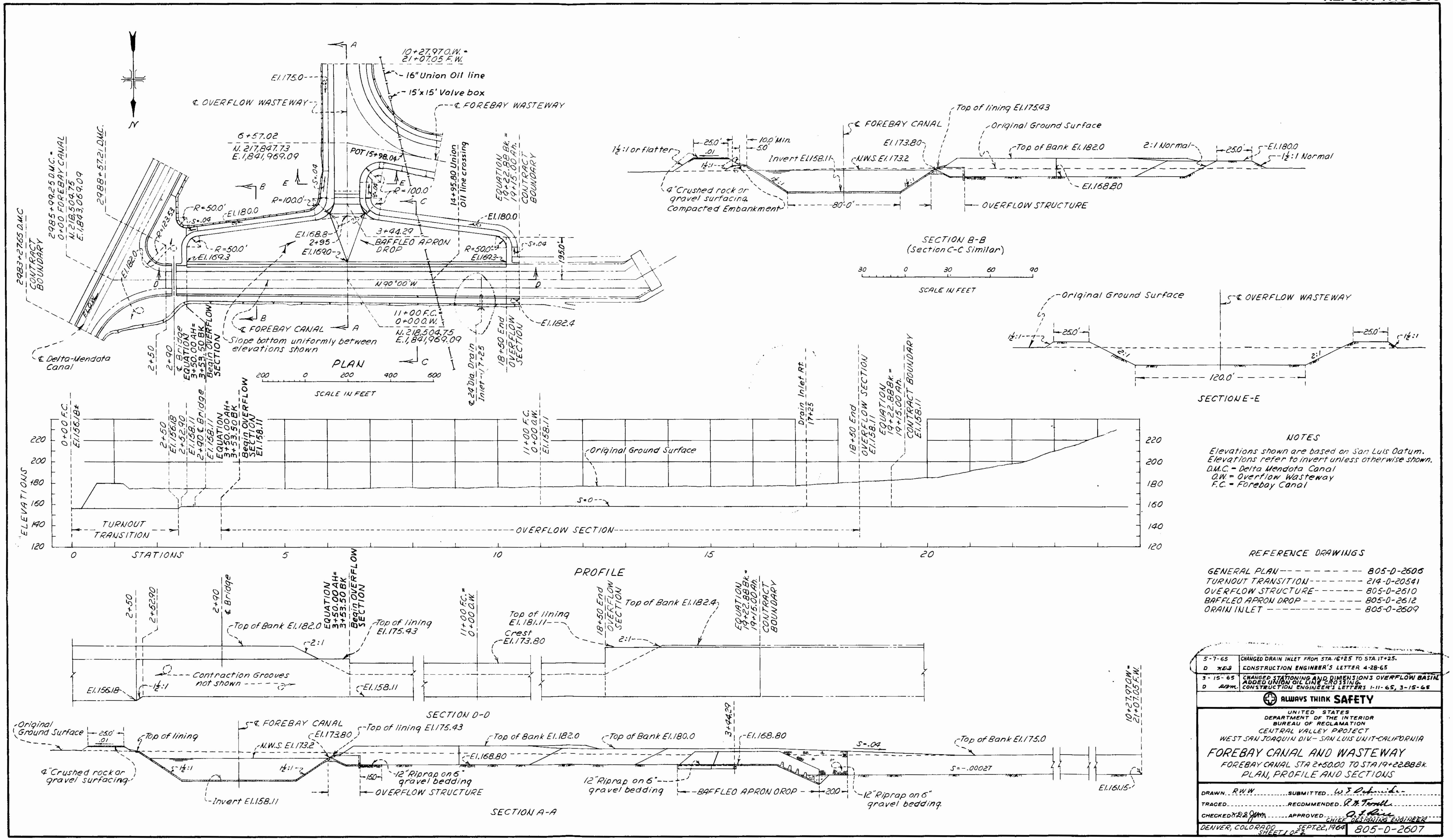
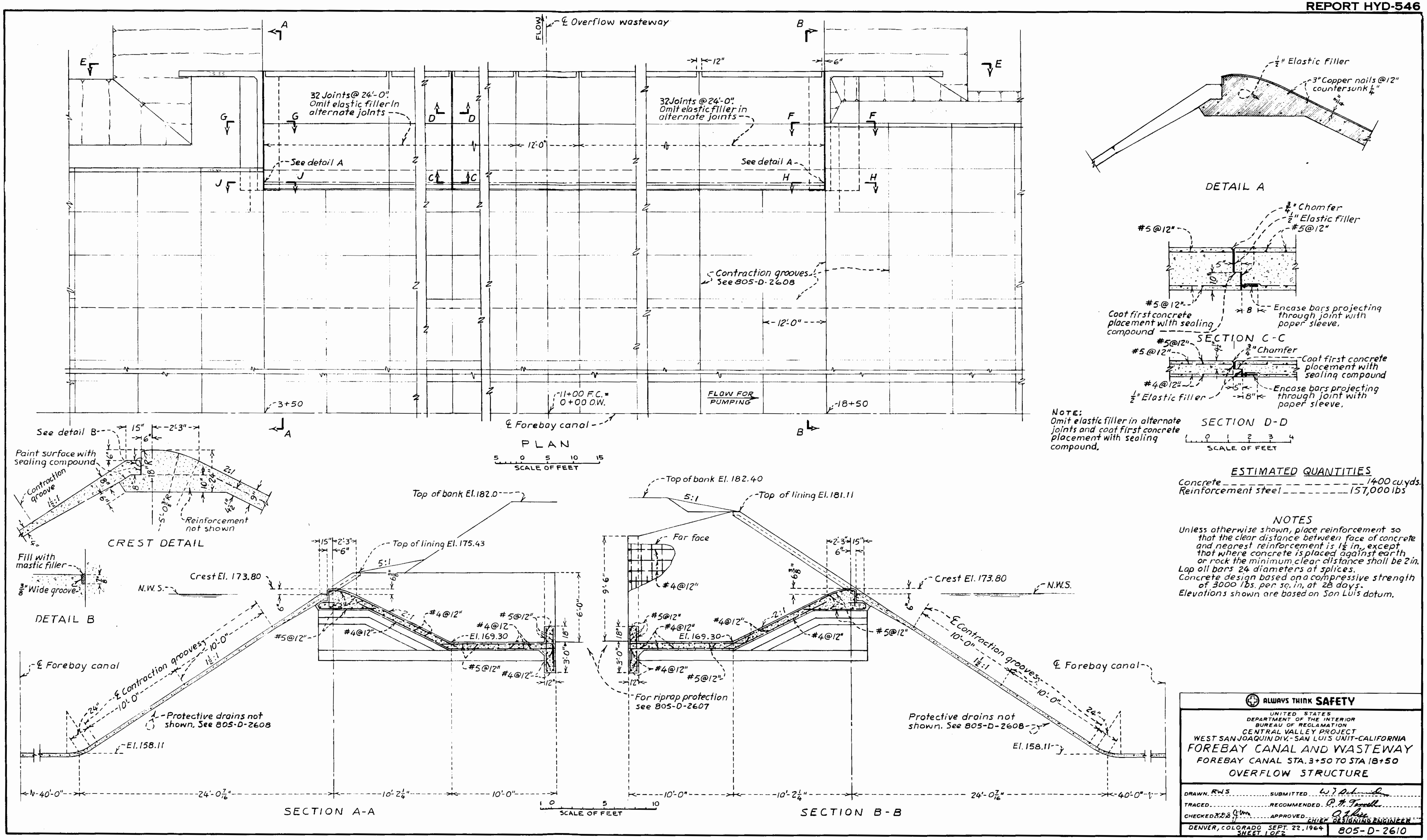


FIGURE 24
REPORT HYD-546



APPENDIX

Electronic Digital Computer Program to Solve Citrini's Equation for Attenuation of a Rejection Surge by a Lateral Spillway

PROGRAM DESCRIPTION

The program was developed to solve an equation derived by Citrini19/ for the attenuation of an open channel surge by a lateral spillway. The lengthy equation is presented in the Investigation Section of this report and will not be repeated here.

The program was written in the FORTRAN IV (FORmula TRANslation) language and can be used on most electronic digital computers. No special operating procedures are required.

Solution of the equation was accomplished by the bisection method. The required result was the surge height following attenuation by the lateral spillway (or side weir). Dimensionless forms, YI and YF, were used in the computations. YI was the ratio of the surge depth (Y2) to the normal water depth (Y1) before attenuation by the weir, and YF was the corresponding ratio after attenuation by the weir. In the bisection method, upper and lower limits are chosen which are expected to bracket the correct solution. In this case, the upper limit (YF1) was the ratio of the surge depth before attenuation to the normal canal water depth (YI) and the lower limit (YF2) was assigned the value 1.0, which corresponds to complete destruction of the surge wave by the side weir. The trial value of YF is taken at the midpoint between the limits and substituted in the equation, which appears in the form of a function statement (RESID). The correct solution is reached when the value of the function is equal to zero or is within an arbitrarily chosen limit on either side of zero. In this case, the absolute value of the function required for a correct solution was 0.00001. In the bisection method, the limits are adjusted and iterations continue until the solution is obtained. For each iteration, the trial value of YF was taken at the midpoint between the upper and lower limits. If the corresponding value of RESID was found to be positive, the upper limit was assigned the trial value of YF and the lower limit remained the same. If the value of RESID was negative, the lower limit was assigned the trial value of YF and the upper limit remained the same. Then a new trial YF was obtained at the midpoint between the new limits. The procedure was repeated and the range between limits became smaller, until the correct solution was obtained or until 20 iterations (sufficient for the data used) had been accomplished. An error check was included, in case the method did not converge to the correct solution.

Input data consisted of the channel bottom width (B), the side slopes (S), the discharge coefficient of the lateral spillway (COEF), the initial flow depth (Y1), the surge depth (Y2), the distance of the spillway crest above

19/Op. Cit.

the channel floor (SPWYD), the spillway length (XL), and the Froude number of the initial flow (F). Each input variable was allotted an eight-character field, with the decimal point placed as required. A sample input data sheet is included in this appendix.

The output data included the input variables listed above and the ratio of the surge depth to the initial depth before (YI) and after (YF(2)) attenuation by the spillway. Ten eight-character fields were required for the output. Three characters to the right of the decimal point were specified for all variables except the spillway length, which required only two decimal places. A representative output listing is also included in this appendix.

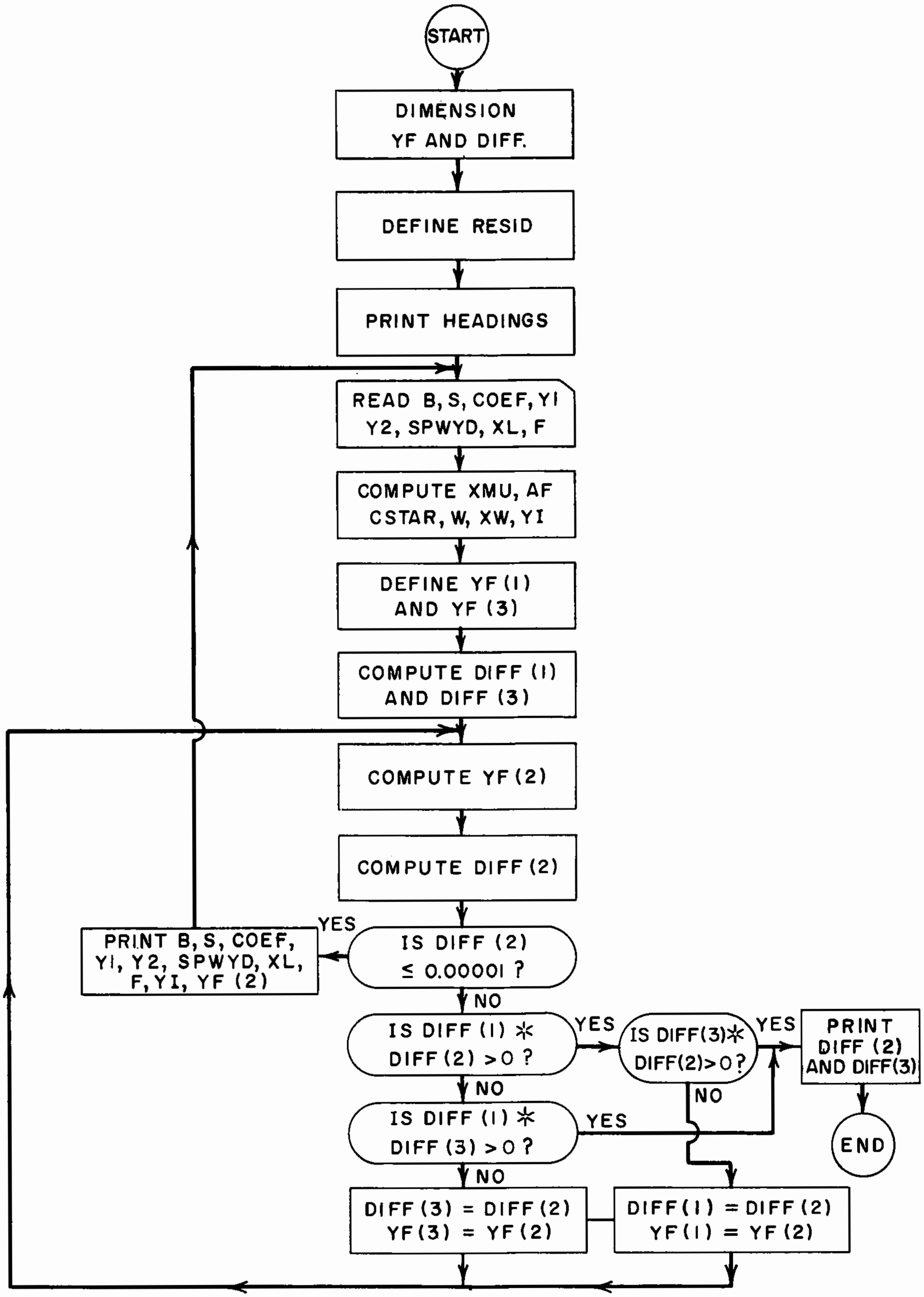
ACKNOWLEDGMENT

The computer program was prepared by Paul W. Merkens from Region 2, Bureau of Reclamation, Sacramento, California, who was a rotation engineer in the Hydraulics Branch during the summer of 1964.

DEFINITION OF VARIABLES USED IN THE PROGRAM

YF and YF ( )	--ratio of surge depth to initial depth following attenuation by the spillway (Y_f).
DIFF ( )	--value of equation for trial value of YF.
RESID	--statement function name.
YI	--ratio of surge depth to initial depth before attenuation by the spillway (Y_i).
XMU	--dimensionless spillway discharge coefficient (μ).
XL	--spillway length (L).
W	--width of channel at elevation of spillway crest (ℓ).
AF	--reciprocal of Froude number of initial flow (A).
CSTAR	--ratio of SPWYD to YI (C*).
B	--channel bottom width.
S	--channel side slopes.
COEF	--spillway discharge coefficient.
Y1	--initial depth.
Y2	--surge depth before attenuation by spillway.
SPWYD	--height of spillway crest above channel floor.
F	--Froude number of initial flow.
XW	--ratio of spillway length to channel top width at spillway crest ($\frac{L}{\ell}$).
I	--subscript, fixed-point variable.

Terms in parentheses are those which appear in original equation.



FLOW CHART FOR DIGITAL COMPUTER SOLUTION OF CITRINI'S EQUATION FOR SURGE ATTENUATION BY A LATERAL SPILLWAY

697

PROGRAM LISTING

FORTRAN IV SOURCE STATEMENTS FOR ELECTRONIC DIGITAL COMPUTER PROGRAM TO SOLVE CITRINI'S EQUATION FOR ATTENUATION OF A REJECTION SURGE BY A LATERAL SPILLWAY

```
 EFN              PROGRAM:  HKATTN                       JOB:     0831HKATTN

C      DETERMINATION OF SURGE ATTENUATION DUE TO CANAL SIDE SPILLWAY
       DIMENSION YF(3),DIFF(3)
 10010 RESID(YF,YI,XMU,XL,W,AF,CSTAR)=
      1((YF*(YI-1.0)*(1.0+0.75*(YI-1.0)))
      2+ ((YI**2)-(YF**2))*(SQRT(0.125*(YF+YI)))
      3-(YI*((YF**2)-1.0)*(SQRT(0.125*(YF+1.0)))))-
      4(((XMU/2.0)*(XL/W)*AF*((YF+YI)**2)*(YF+YI-2.*CSTAR)
      5*(SQRT(0.125*(YF+YI)*(YF+YI-2.0*CSTAR)))) /
      6(YF+YI+AF*((YF**2)-1.0)*(SQRT(0.125*(YF+1.0)))
      7+AF*(YI-1.0)*(1.0+0.75*(YI-1.0))
      8+AF*(YF+YI)*(SQRT(0.5*(YF+YI)))))
 13030FORMAT(1H1,79H      B           S         COEF      Y1        Y2       SPWYD
      1 L           F         YI        YF)
 1305 FORMAT(1X,6F8.3,F8.2,1X,3F8.3)
 1330 FORMAT(8F8.0)
      WRITE (6,1303)
 1331 READ (5,1330) B,S,COEF,Y1,Y2,SPWYD,XL,F
      XMU = COEF/8.0199
      AF=-1.0/F
      CSTAR = SPWYD/Y1
      W = B+2.0*S*SPWYD
      XW=XL/W
      YI=Y2/Y1
      YF(1)=YI
      YF(3) = 1.0
      DO 1003 I=1,3,2
      DIFF(I)=RESID(YF(I),YI,XMU,XL,W,AF,CSTAR)
 1003 CONTINUE
 1005 DO 1950 J=1,20
      YF(2)=(YF(1)+YF(3))/2.0
      DIFF(2) = RESID(YF(2),YI,XMU,XL,W,AF,CSTAR)
      IF((ABS(DIFF(2))) .LE. 0.00001) GO TO 1951
      IF((DIFF(1)*DIFF(2)) .GT. 0.0) GO TO 1004
      IF((DIFF(1)*DIFF(3)) .GT. 0.0) GO TO 1701
      DIFF(3) = DIFF(2)
      YF(3) = YF(2)
      GO TO 1950
 1004 IF((DIFF(3)*DIFF(2)) .GT. 0.0) GO TO 1701
      DIFF(1) = DIFF(2)
      YF(1) = YF(2)
 1950 CONTINUE
 1951 WRITE (6,1305) B,S,COEF,Y1,Y2,SPWYD.XL,F,YI,YF(2)
      GO TO 1331
 1700 FORMAT (9H DIFF(I)=F9.4)
 1701 WRITE (6,1700)(DIFF(I),I=1,3)
      END
```

BUREAU OF RECLAMATION

LABORATORY PUNCH CARD DATA

USE / COLORED CARDS			
PROBLEM	Solution of Citrini's Equation	JOB NO.	#
DETAIL	Attenuation of Rejection Surge by	RETURN TO	
FEATURE	Lateral Spillway		
PROJECT	San Luis Unit, CVP	ROOM / BLDG. / PHONE	

BOTTOM WIDTH	SIDE SLOPES	DISCHARGE COEFFICIENT	INITIAL DEPTH	SURGE HEIGHT	CREST HEIGHT	SPILLWAY LENGTH	FROUDE NUMBER
B	S	COEF	Y1	Y2	SPWYD	XL	F
80.0	1.5	3.5	15.0	16.0	15.5	1500.0	0.123
				16.4			
				16.8			
				17.2			
				17.6			
				18.0			
				18.4			
				18.8			

SAMPLE DATA SHEET

REMARKS

BY | DATE | CHECKED | SHEET OF

GPO 83

EXAMPLE OF PRINTED RESULTS

B	S	COEF	Y1	Y2	SPWYD	L	F	YI	YF
80.000	1.500	3.500	15.000	16.000	15.500	1500.00	.123	1.067	1.051
80.000	1.500	3.500	15.000	16.400	15.500	1500.00	.123	1.093	1.060
80.000	1.500	3.500	15.000	16.800	15.500	1500.00	.123	1.120	1.067
80.000	1.500	3.500	15.000	17.200	15.500	1500.00	.123	1.147	1.073
80.000	1.500	3.500	15.000	17.600	15.500	1500.00	.123	1.173	1.077
80.000	1.500	3.500	15.000	18.000	15.500	1500.00	.123	1.200	1.081
80.000	1.500	3.500	15.000	18.400	15.500	1500.00	.123	1.227	1.085
80.000	1.500	3.500	15.000	18.800	15.500	1500.00	.123	1.253	1.088

GPO 853-728

ABSTRACT

Data from a 1:48 scale model supplied the magnitudes and velocities of surges developed in the canal system following rejection of flow at the pumping plant, San Luis Forebay, California, and showed that a side weir was effective in reducing the surges. Data were obtained with capacitance wave probes for partial and complete rejection of flow with and without backflow from the pump discharge lines. Maximum surge peak heights were 5.4 ft for complete rejection of the maximum discharge plus 200% backflow, 4.5 ft with 150% backflow, and 1.9 ft without backflow. Velocities of propagation were 20.7, 20.7, and 19.1 fps, respectively, for the 3 conditions. A 1,500 ft-long weir on the canal sideslope reduced the maximum surge height to 1.0 ft without backflow and 1.3 ft with either 150 or 200% backflow. The reflecting and attenuating characteristics of canal structures were observed and steady-state conditions after flow rejection with the entire flow discharging over the weir were measured. The undular form of the surge wave was analyzed and several comparisons were made with theory. A 1:10 scale sectional model was used to develop the weir crest shape.

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DESCRIPTORS-- *pumping plants/ *canals/ *model tests/ *surges/ *trapezoidal channels/ *weirs/ hydraulic transients/ freeboard// bore/wave// discharge coefficients/ viscosity/ Reynolds number/ Froude number/ surface tension/ translatory waves/ unsteady flow/ weir crests/ calibrations/ instrumentation/ laboratory equipment/ measuring instruments/ recording systems/ capacitance/ dielectrics/ electronic equipment/ oscillographs/ research and development
IDENTIFIERS-- wave probes/ Weber number/ Central Valley Project, California/ San Luis Forebay Pumping Plant

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