UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES ON THE WAVE SUPPRESSOR DEVICE AT THE FRIANT-KERN CANAL HEADWORKS

Hydraulic Laboratory Report No. Hyd-395

ENGINEERING LABORATORIES

COMMISSIONER'S OFFICE
DENVER, COLORADO

August 1, 1955

FOREWORD

The model investigations on the wave suppressor device for the Friant-Kern outlet works were conducted in the Engineering Laboratories of the Bureau of Reclamation at Denver, Colorado, during August and September 1954.

The recommended device evolved from this study was developed through the cooperation of the staffs of the Concrete Dams Design Section and the Hydraulic Laboratory.

During the course of the model studies, Messrs. L. G. Puls, Max Ford, Abe Olshansky, N. W. Cash, and others of the Concrete Dams Design Section frequently visited the laboratory to observe the model tests and discuss the results.

The studies were conducted by Mr. T. J. Rhone under the direct supervision of Mr. A. J. Peterka.

CONTENTS

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Commissioner's Office
Engineering Laboratories
Denver, Colorado
August 1, 1955

Laboratory Report No. Hyd-395
Hydraulic Laboratory
Compiled by: T. J. Rhone
Checked by: A. J. Peterka
Reviewed by: A. J. Peterka

Subject: Hydraulic model studies on the wave suppressor device at the Friant-Kern Canal Headworks

SUMMARY

The hydraulic model studies described in this report were performed to develop a structure which would reduce the height of waves generated in the outlet works stilling basin and which overtopped the concrete lining in the Friant-Kern Canal. Operation of the prototype structure at high canal discharges had shown a very rough water surface that produced waves in the canal about 8 feet high at 4,000 cfs.

A 1:32 scale hydraulic model (Figure 2) was used to develop a wave suppressor which could be added to the existing structure without the necessity of revising the structure itself. Several types of wave suppressor devices were investigated during the tests, including floating rafts, an increased number of baffle piers in the stilling basin, and a series of curtain walls downstream from the stilling basin.

The most effective wave suppressor consisted of a culvert or short tube underpass placed so that all flow passed through the suppressor and dampened the water surface fluctuations for all flows greater than 2,000 cfs, Figure 5. The tests showed that the effectiveness of the underpass was directly proportional to the length; longer culverts resulting in greater wave height reduction, Figure 6, Test No. 3. Considering hydraulic as well as cost considerations, the laboratory engineers and the designers agreed that an underpass 20 feet in length provided sufficient overtopping protection to the canal. Waves entering the canal were reduced to about 2 feet in height, and it would have been necessary to double the suppressor length to further reduce the 2-foot waves to 1-foot waves.

Piezometric measurements were made on the upstream headwall and the roof of the 20-foot-long underpass, Figure 8. These measurements showed that the forces acting on the structure were small.

Wave measurements made at Station 6+30 showed that the underpass was effective as a wave dampener for all discharges above 2,000 cfs; reducing the magnitude of the waves from 8 feet to 2 feet at 5,000 cfs and from 3 feet to 1 foot at 2,000 cfs, Figures 6 and 7.

INTRODUCTION

Friant Dam, located on the upper San Joaquin River about 20 miles north of Fresno, California, is a part of the Central Valley Project. The Friant-Kern Canal furnishes part of the water for the San Joaquin Valley. The canal distributes water between Friant Dam and the Kern River near Bakersfield, California, about 160 miles south of Friant Dam.

Flow into the Friant-Kern Canal is regulated through four 96-inch hollow-jet valves which have a combined capacity of about 5,000 cfs at reservoir elevation 570. The water discharges horizontally from the valves onto a long trajectory apron and into a short stilling basin. A vertical center wall divides the stilling basin into two sections, Figure 1. From the stilling basin the flow passes through a 50-foot-wide rectangular channel about 300 feet long before entering the canal. The bottom width of the canal is 36 feet with 1-1/4:1 side slopes.

Although the Friant-Kern Canal has been in operation for several years, recent water demands have made it necessary to increase the discharge by about 20 percent. With these larger flow quantities severe wave action made it imperative that corrective measures be taken. At 4,000 cfs the waves in the transition and canal overtopped the canal banks; at times the waves reached 8 feet in height and were sufficiently strong to carry off sand bags that had been placed along the top of the canal banks to reduce the overtopping effects. Since it was also contemplated that future water demands might increase the discharge to even greater quantities, it was requested that model studies be made to develop a device to still the wave action.

THE MODEL

The model was built to a scale of 1:32 so that 3-inch hollow-jet valves readily available in the laboratory could be used to represent the 96-inch prototype valves. The model included the four hollow-jet valves, the trajectory curve and stilling basin, the rectangular channel, and about 600 feet of canal, Figure 2.

The stilling basin and rectangular channel were constructed of plywood, the trajectory curve was formed from lightweight galvanized sheet metal fastened to wood templates, and the canal section was represented by concrete screeded to sheet metal templates. The baffle piers and dividing pier were modeled in wood.

Water was supplied to the model through the permanent laboratory supply system and measured with a 4-inch Venturi meter. The head on the valves was measured by water manometers connected to piezometers located one diameter upstream from each valve, Figure 3. Wave measurements were obtained both visually and by an electronic measuring and recording device. All wave measurements were made on the left bank of the canal at the downstream edge of the transition, Figure 2 The height of the waves was taken as the difference between the maximum peak and lowest trough as observed during a period of about 3 minutes.

The flow depth in the canal was controlled by a tail gate at the downstream end of the model. The flow depths were set from information obtained from two sources, one was the calculated depths furnished by the designers and the second was actual measured depths from field observations at the prototype structure. The field observations were taken at three discharges, 3,225, 3,500, and 4,000 cfs. For the first two discharges the design depths and the observed depths were the same, however, at 4,000 cfs the observed depth was 0.52 foot less than the design depth. Two flow versus depth curves were prepared for the model studies in order to be sure that the suppressor would be adequate under either operating condition; one curve was prepared from the computed values and the second by extrapolating the observed values.

THE INVESTIGATION

Operation of the prototype structure had been observed and photographed at discharges of 3,225, 3,500, and 4,000 cfs. The motion pictures made it possible to view prototype operation in the laboratory and to compare the magnitude of the water surface fluctuations with those in the model. The model waves in shape and magnitude appeared to be similar to those in the motion pictures, particularly those above 3,500 cfs. Since future water demands will exceed this figure, it was decided that a wave dampener be developed that was satisfactory for flows up to 5,000 cfs.

Figure 3 shows the flow appearance in the model for discharges of 3,500 and 5,000 cfs. The water surface fluctuation, measured at the downstream end of the transition on the sloping canal bank showed a vertical variation of about 8 feet at 5,000 cfs and almost 5 feet at 3,500 cfs.

Waves in the stilling basin itself were not excessively large, but were magnified by the transition before they entered the canal. Since the stilling basin performance was satisfactory and the only problem in operation was the high waves, it was felt that a wave dampener of a type that could be added to the existing structure without other modification was called for; also field construction had to be completed in the relatively short time between irrigation seasons.

Preliminary tests showed that additional baffle piers placed in the stilling basin had a tendency to increase rather than reduce the magnitude of the water surface fluctuations. Floating rafts also failed to materially reduce the wave action in the canal so no extensive investigations were performed on either of these methods. Other devices tested are discussed below.

Curtain Walls

The use of curtain walls had proved effective in solving other wave problems where the flow quantities involved had been considerably less than those involved in the Friant-Kern tests. However, it was felt that two or more curtains might prove adequate in reducing the waves in the current problem.

The initial tests were made with two curtain walls; one wall was placed at Station 4+18.02 in the channel, Figure 4. The lower edge of the wall was set 8-1/2 feet above the channel floor in order to be effective at lower discharges. A second curtain wall was set over the baffle blocks about 46 feet upstream from the first wall with the lower edge resting on top of the baffle blocks, 11 feet above the channel floor, Figure 4. Both walls were vertical and extended the full width of the channel and to the full height of the side walls.

The curves in Figure 4 show the vertical water surface variation at various discharges. At 5,000 cfs the two curtain walls reduced the magnitude of the waves from about 8 feet to 3.3 feet; although this was a considerable improvement the waves still had a tendency to overtop the canal banks.

A third curtain wall was next installed about 55 feet downstream from the first wall, or at Station 4+73, Figure 4. The bottom of this wall was placed 10 feet above the channel floor. The wave heights for various discharges are also shown by the curves in Figure 4. The third wall was effective in reducing the vertical variation to about 2.2 feet at 5,000 cfs.

Although the three curtain walls were effective in reducing the water surface fluctuation to reasonable limits, there was still a tendency for long-period surges and swells to pass through or under all three of the walls. Although these surges were not as extensive as the original water surface fluctuations, they were still unsightly, and combined with the general complexity of the structure it was decided to investigate the use of a short tube underpass type of wave suppressor.

Short Tube Underpass

A short tube underpass consisting of a horizontal roof set a specific distance under the water surface and vertical upstream and downstream headwalls was investigated, Figure 5. Preliminary trials showed great promise in reducing the wave action in the canal so tests were performed to determine the most favorable location and dimensions. The governing criteria in all the tests were the visual appearance of the flow and the wave heights as measured at the downstream end of the transition section.

Underpass Location

The initial tests, performed with a roof 21 feet in length, were made to determine the best longitudinal positioning for the structure. The most advantageous location was with the downstream end of the underpass placed just upstream from the P.C. or approximately at Station 4+18, Figure 2. When the underpass was placed a short distance upstream from this location it was in the turbulent zone of the hydraulic jump and there was a tendency for some of the turbulence to pass under the roof and to cause waves on the downstream side. When the underpass was placed downstream from the P.C , or in the curve, its effectiveness was reduced, probably due to the asymmetry of the flow leaving the underpass in the curve. When the underpass was placed downstream from the curve it was too close to the canal. There was a slight amount of turbulence on the surface of the flow leaving the underpass, and it was desirable to have a short distance of channel for this turbulence to quiet before entering the transition between the channel and the canal, since the transition greatly magnified any surface fluctuations. For all of the subsequent tests the downstream end of the underpass was placed at Station 4+18.

Underpass Height

In determining the optimum opening between the underpass roof and the channel floor, the 21-foot-long roof was located at Station 4+18 and the model was operated at a discharge of 5,000 cfs. The vertical position of the roof was varied and wave height measurements obtained for six different locations. The wave height measurements were obtained by measuring the distance between minimum and maximum water surface fluctuations on the sloped canal banks just downstream from the transition, Figure 2. This variation was converted to a vertical fluctuation for later comparison with the electronic wave measurements. The results of the optimum opening tests are shown as Test No. 1 on Figure 6. As shown by the curve on this figure, the optimum position of the roof is about 11 feet above the canal floor; however, the roof could be placed from 10 to 12 feet above the floor with negligible variation in the results. The dotted portion of the curve is the estimated effect for openings greater than 14 feet.

Underpass Length

The next test was performed to determine the effect of roof length on wave reduction. For this test the downstream end of the underpass was placed at Station 4+18 with the roof set 11 feet above the channel floor. Both the discharge and length of roof were varied and wave measurements obtained at the downstream end of the transition. Roof lengths of 10, 21, and 40 feet were tested at discharges of 2,000 to 5,000 cfs in 500 cfs intervals. The results of these tests were plotted on the curves labeled Test No. 3 on Figure 6. (Test No. 2

is discussed later.) These curves show that a roof 20 feet in length reduces the water surface fluctuation about 75 percent or from 8 feet to 2 feet in magnitude while a further increase in underpass length from 20 to 40 feet accomplished only an additional 1-foot decrease in wave height.

On the basis of these results an underpass 20 feet in length was selected for installation in the prototype structure. Although the water surface fluctuation was still 2 feet in magnitude it was thought that a further reduction in wave height would not be worth the greatly increased cost of a longer underpass.

Performance Tests, Recommended Underpass

The final test on the wave suppressor was made with the recommended underpass. Water surface fluctuations for the full discharge range were obtained, Figure 7. Pressure readings along the roof and upstream headwall for the maximum discharge were also obtained, Figure 8.

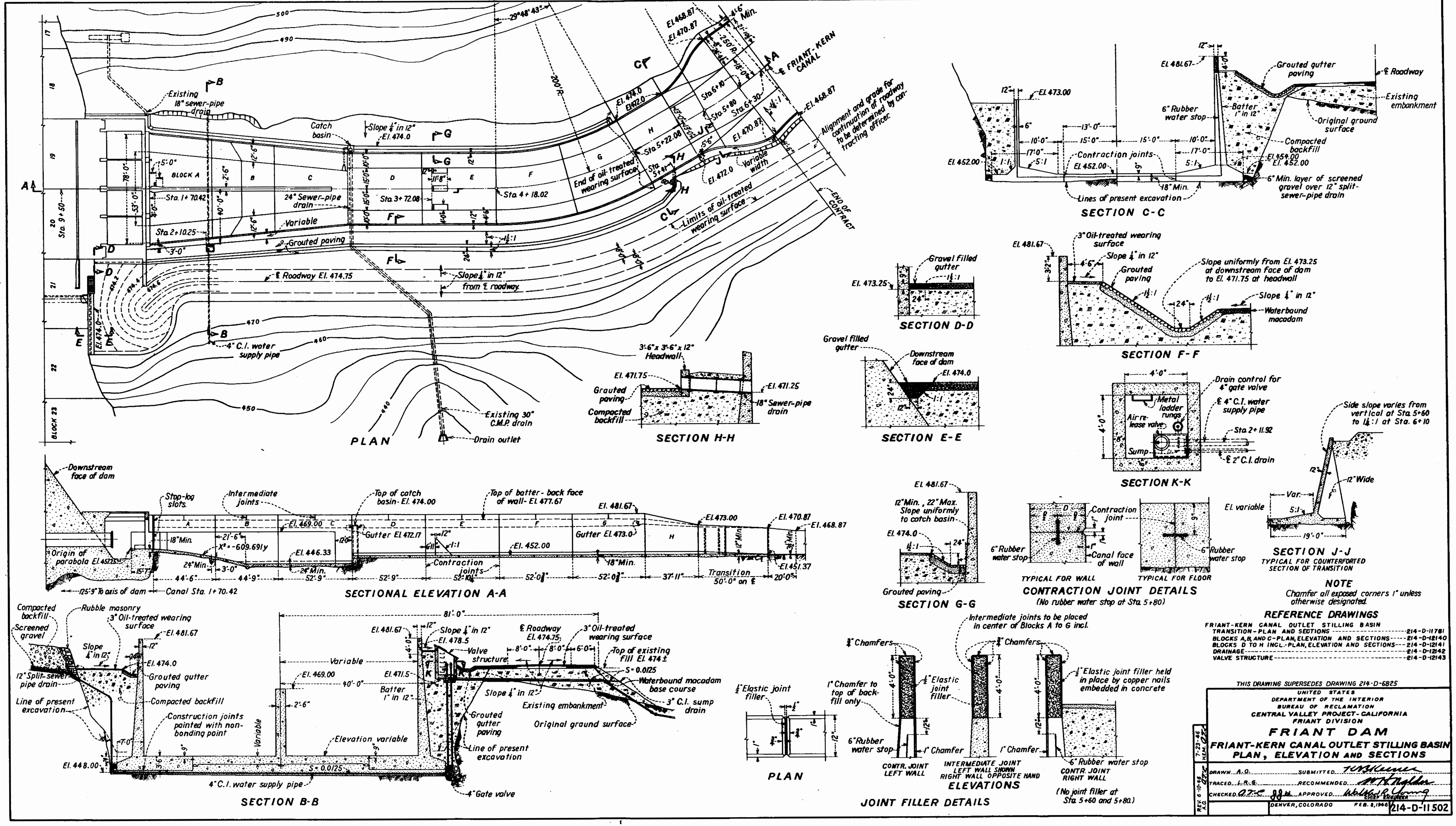
The pressures on the upstream headwall were at all times either equivalent to the hydrostatic pressure or slightly less, indicating that the average force on the headwall was negligible. Since the pressures were measured with piezometers, the inertia of the water column in the indicating manometers prevented reading the momentary impact effect of the waves, however, allowance for wave impact should be made during design. The pressures along the roof of the underpass were less than atmospheric at 5,000 cfs, Figure 8, showing that there was no uplift on the slab, but rather a tendency to hold the structure down. The reduced pressures along the roof are comparable to those occurring in an outlet or short tube with a sharp-edged entrance. The flow contraction at the entrance produces the slightly below atmospheric pressure.

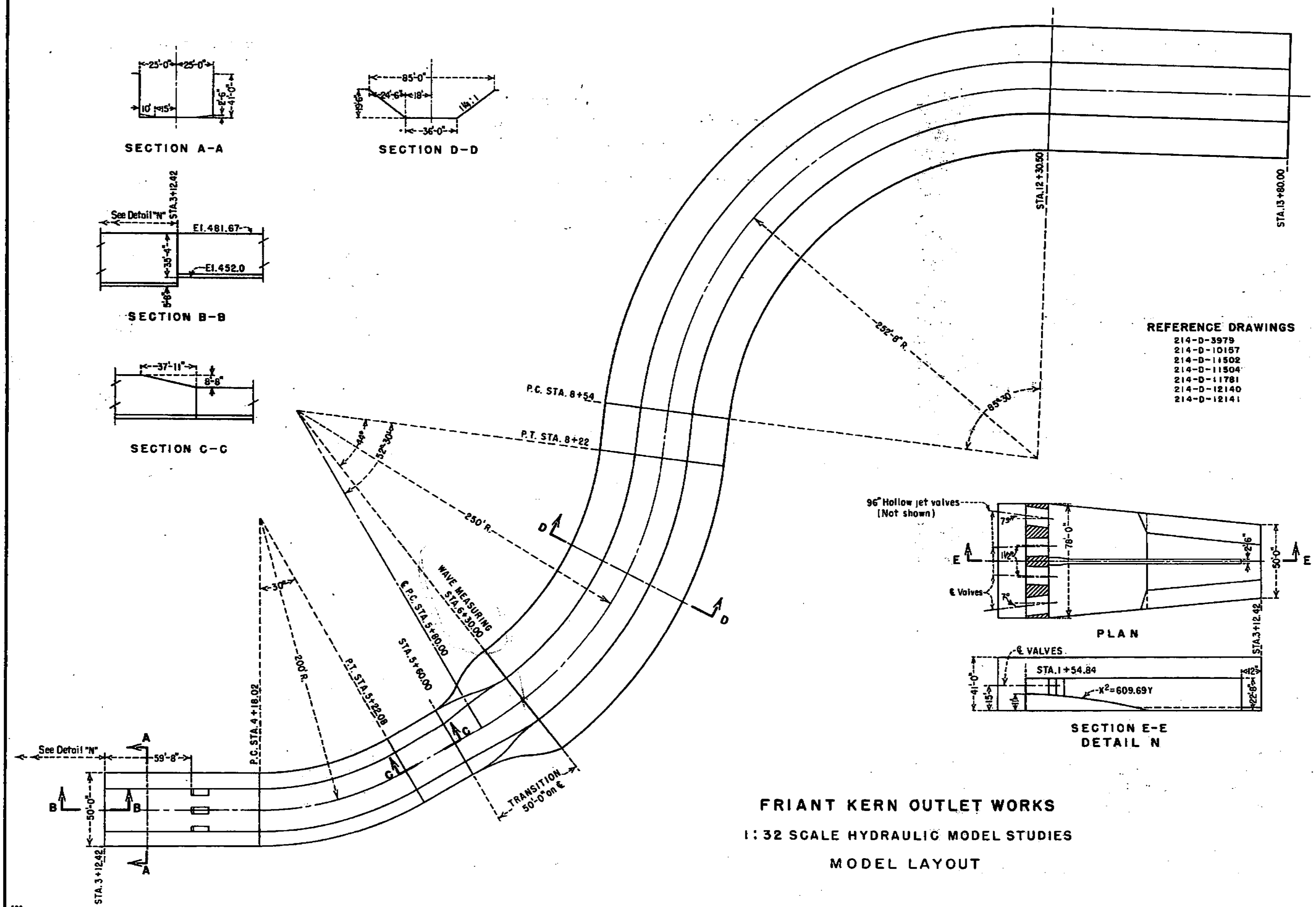
The wave heights or water surface fluctuations were measured for the full range of discharges by an electronic wave recorder placed at the downstream end of the transition. The measurements were made both with and without the wave suppressor installed so that the effectiveness of the suppressor for all discharges could be shown. The results of these measurements are plotted as Test No. 2 on Figure 6. These curves show that the suppressor has some effect at all discharges where the water surface is in contact with the culvert roof. The device becomes ineffective for discharges less than about 2,000 cfs, depending on the depth in the canal. Since the water surface fluctuation or waves are less than 1 foot in height at this discharge and the normal depth is well below the top of the canal lining, the suppressor should be adequate in this respect.

Figure 7 is composed from the actual electrically recorded traces of the water surface fluctuations in the model. The time and magnitude scales have been converted to prototype dimensions, however. In this figure the effectiveness of the wave suppressor at three discharges is shown. The figure also shows that although the waves that are present with the suppressor in place are about 2 feet in height at 5,000 cfs, this is the extreme between high and low points in the fluctuation. Actually, any single wave is seldom more than 1-1/2 feet in height or about 9 inches above the average water surface.

In all of the above tests the depth of flow in the canal was based on depths obtained during operation of the prototype structure and measured about one-fourth mile downstream from the outlet works. At 4,000 cfs the depth was about 1/2-foot less than the design depth of 15.22 feet. The design depth for 5,000 cfs is 17.22 feet. When the flow depth in the model was increased to correspond to the design depth, water surface fluctuations were reduced slightly below those shown in the data and curves of Figure 7.

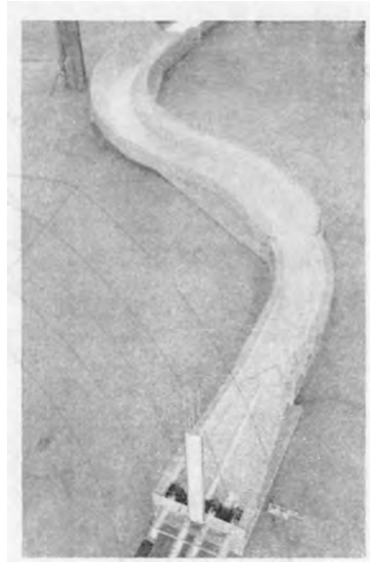
FIGURE 1
REPORT HYD. 395



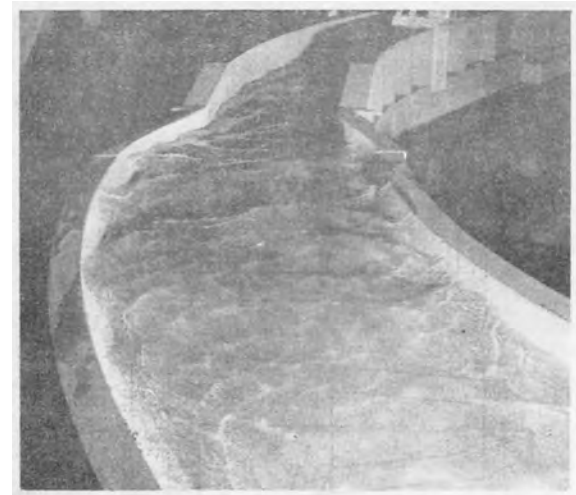


FRIANT KERN OUTLET WORKS
1:32 SCALE HYDRAULIC MODEL STUDIES
MODEL LAYOUT

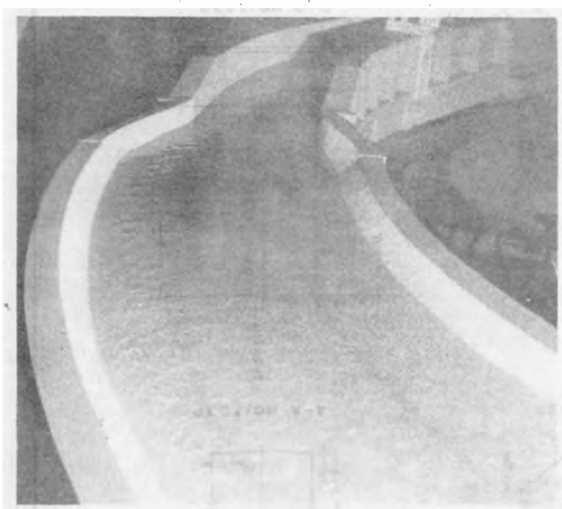
Without wave suppressor



1:32 Scale Model



Without wave suppressor



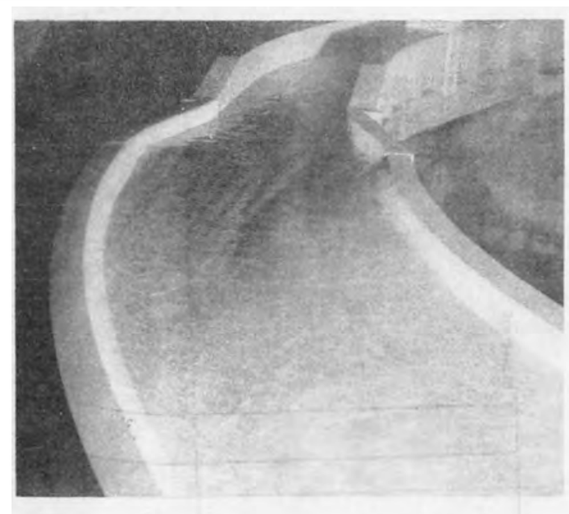
With wave suppressor

Discharge - 3500 cfs

FRIANT-KERN OUTLET WORKS

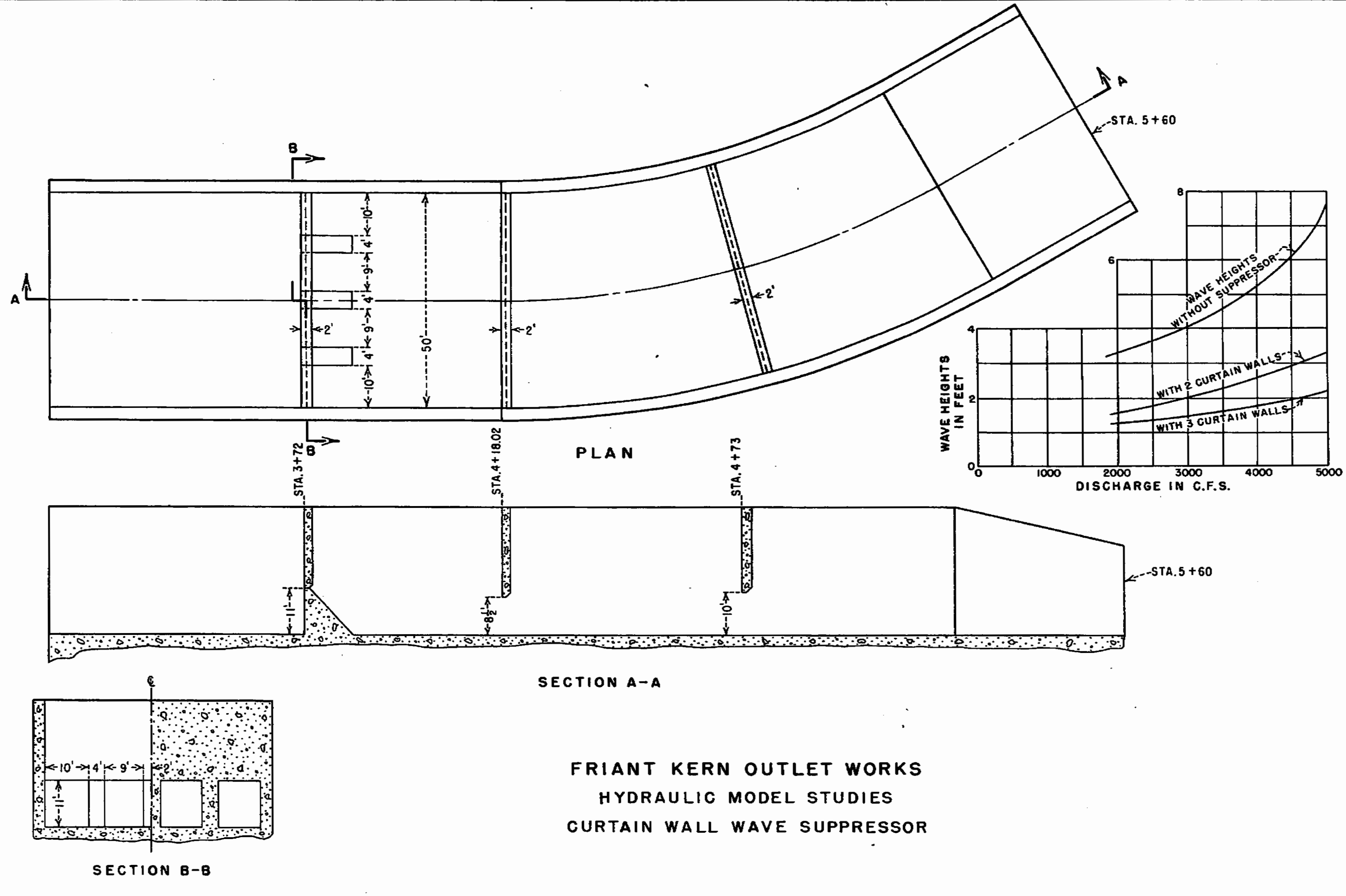
Hydraulic Model Studies

Flow appearance with and without short tube underpass type of wave suppressor.

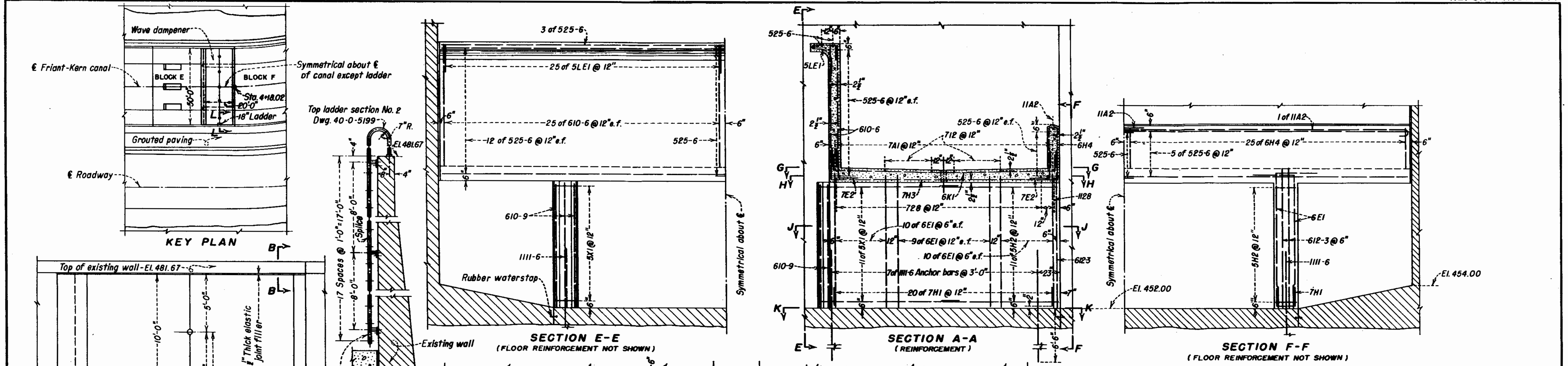


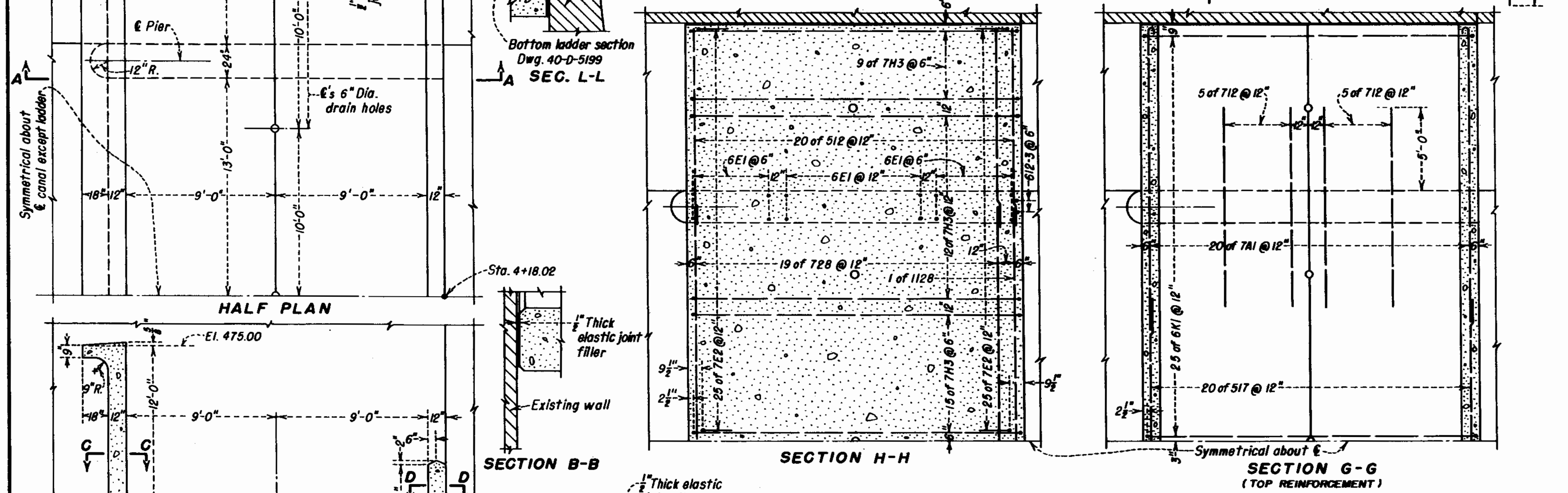
With wave suppressor

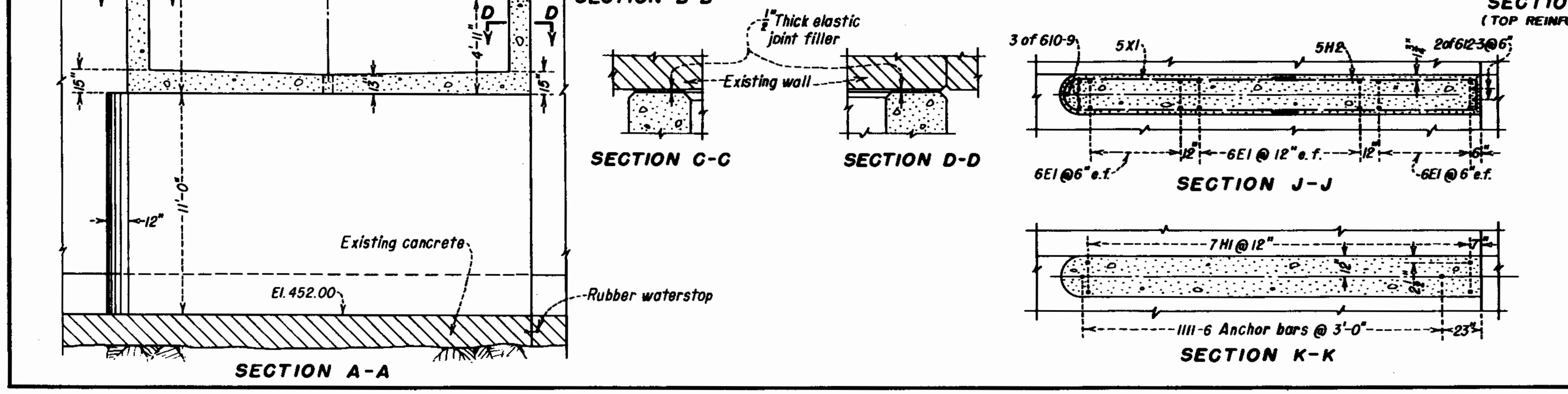
Discharge - 5000 cfs.



FRIANT KERN OUTLET WORKS
HYDRAULIC MODEL STUDIES
CURTAIN WALL WAVE SUPPRESSOR



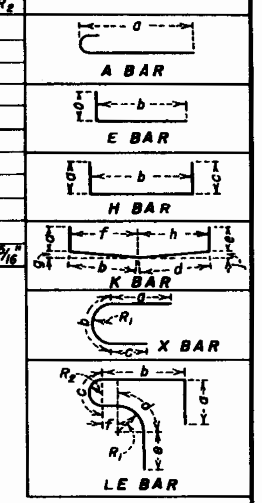




BENT BAR LIST

MARK	BAR NO.	TOTAL LENGTH	NO. REQ'D.	a	b	c	d	e	f	g	h	j	R_1	R_2
7A1	7	19'-3"	40	18'-4"										
11A2	11	28'-0"	2	26'-5"										
6E1	6	15'-3"	116	3'-7"	11'-9"									
7E2	7	4'-3"	100	21½"	2'-7"									
7H1	7	4'-6"	40	19"	19"	19"								
5H2	5	21'-6"	22	9'-11½"	20½"	10'-0"								
7H3	7	24'-6"	72	2'-7"	19'-7"	2'-7"								
6H4	6	7'-6"	50	3'-7"	7"	3'-6½"								
6K1	6	22'-6"	50	19"	9'-9½"		9'-9½"	18½"	9'-9½"	2"	9'-9½"	2"		
5X1	5	25'-6"	22	11'-5"	2'-8"	11'-5"							10¼"	
5LE1	5	6'-9"	50	15"	23"	7½"	18"	14"	4½"				11½"	2 5/16"

SHAPE



STRAIGHT BAR LIST

MARK	BAR NO.	LENGTH	NO. REQ'D.
1128	11	28'-0"	1
1111-6	11	11'-6"	14
728	7	28'-0"	19
712	7	12'-0"	20
612-3	6	12'-3"	4
610-9	6	10'-9"	6
610-6	6	10'-6"	100
525-6	5	25'-6"	74
517	5	17'-0"	20
512	5	12'-0"	40

BAR No.	5	6	7	11
STANDARD PIN DIA.	4"	5"	6"	11"

NOTES
All dimensions are to ℄ of bar.
All bends are to be standard unless otherwise designated.

QUANTITIES
Approximate quantities of concrete and reinforcement bars are 106 cu. yds. and 20,000 lbs., respectively.

CONCRETE FINISHES
All surfaces ---------------- F2 or U2

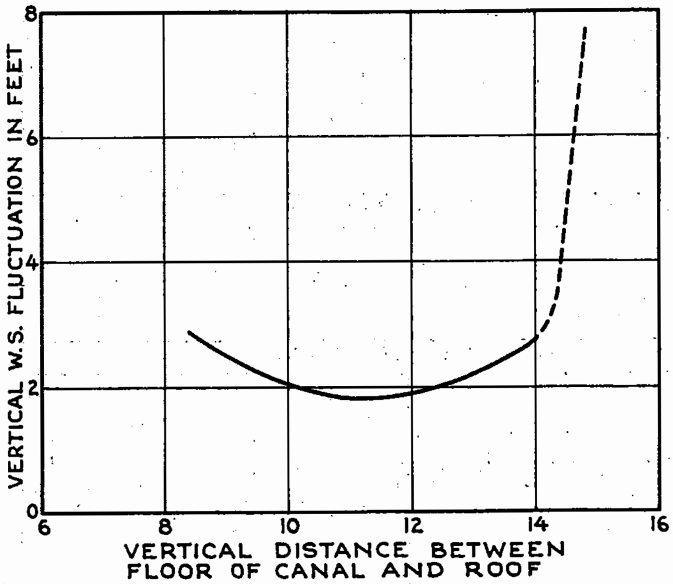
NOTES
Chamfer 1" or tool all exposed corners unless otherwise specified
e.f. = each face

REFERENCE DRAWING
FRIANT-KERN CANAL OUTLET STILLING BASIN-
BLOCKS D TO H INCLUSIVE-PLAN, ELEVATION AND SECTIONS ---- 214-D-12141

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

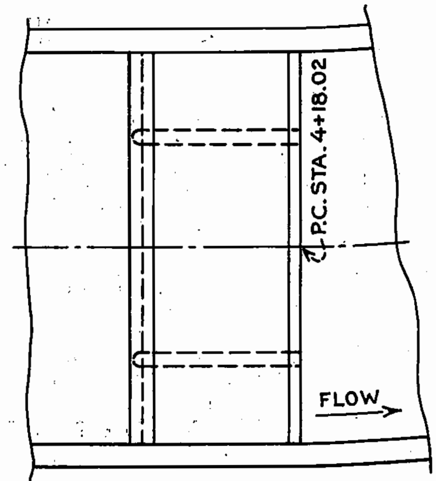
FRIANT DAM
FRIANT-KERN CANAL OUTLET STILLING BASIN WAVE DAMPENER

DRAWN N.W.C. SUBMITTED R. J. Larson
TRACED R.E.G. RECOMMENDED J. J. Hammond
CHECKED A.Z.C. RBD APPROVED J. A. Blodgett
ASSOC. CHIEF ENGINEER
DENVER, COLORADO, SEPT. 27, 1954
214-D-19485

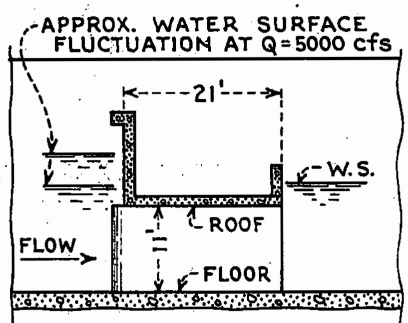


TEST NO. 1

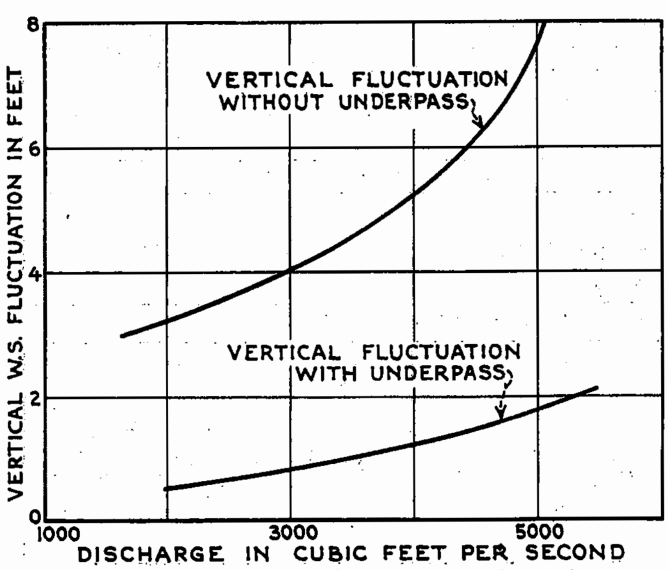
TO DETERMINE MOST EFFECTIVE ELEVATION FOR ROOF - Q=5000 cfs.



PLAN

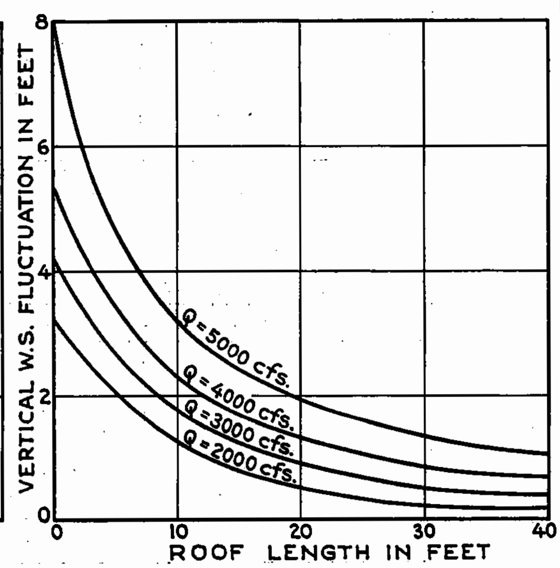


SECTION



TEST NO. 2

TO DETERMINE EFFECTIVENESS OF UNDERPASS AT VARIOUS DISCHARGES



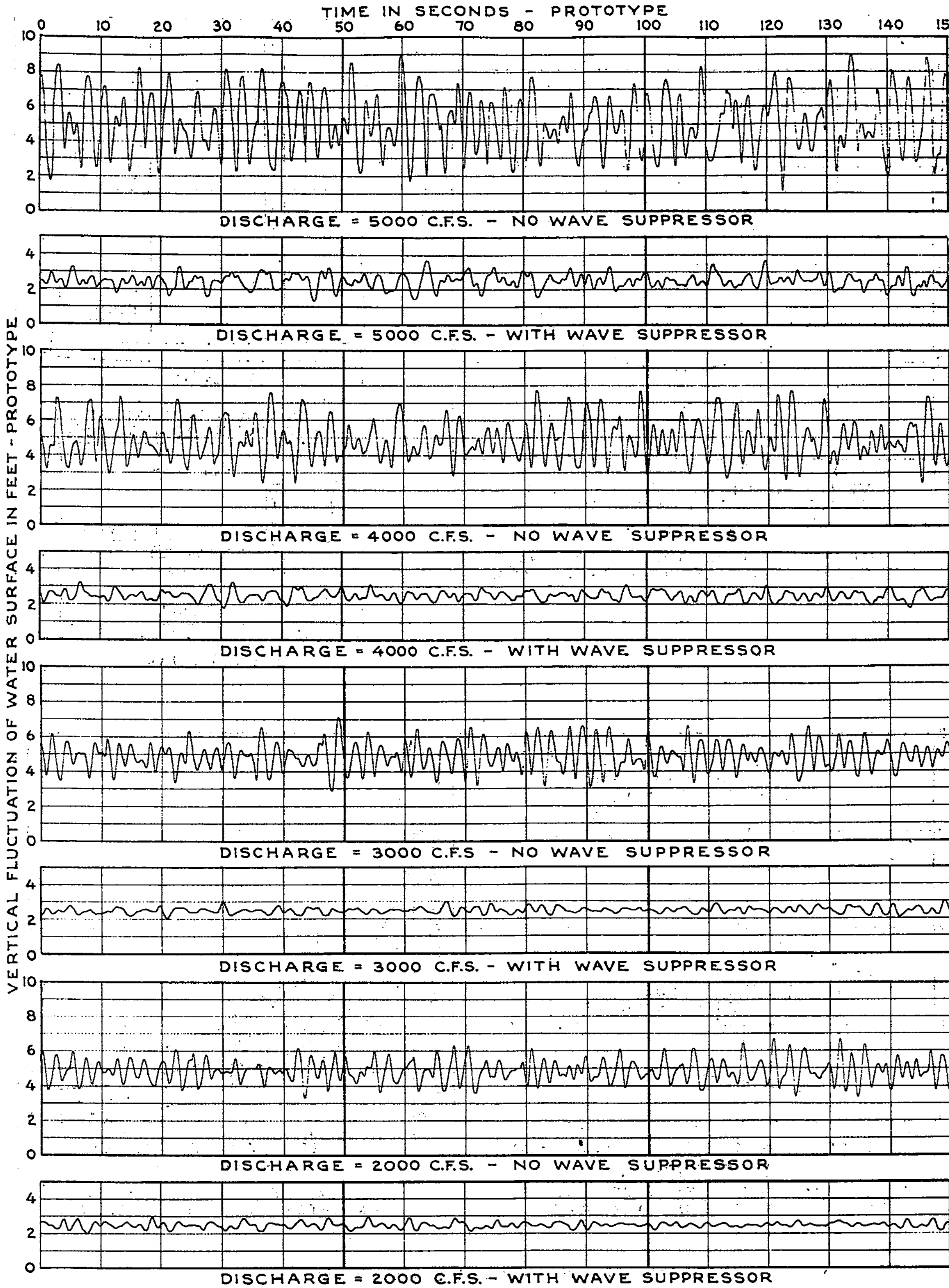
TEST NO. 3

EFFECT OF UNDERPASS LENGTH ON WATER SURFACE FLUCTUATION

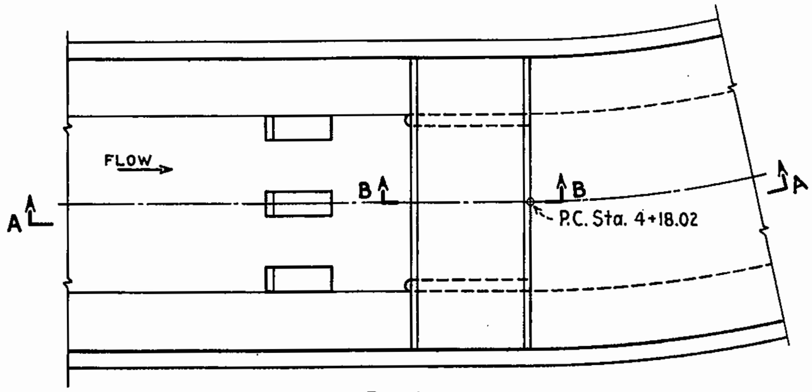
WAVE SUPPRESSOR FOR
FRIANT - KERN CANAL
RESULTS OF HYDRAULIC MODEL TESTS

509

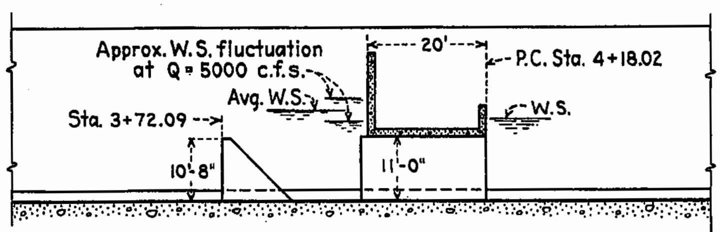
FIGURE 7
REPORT HYD. 395



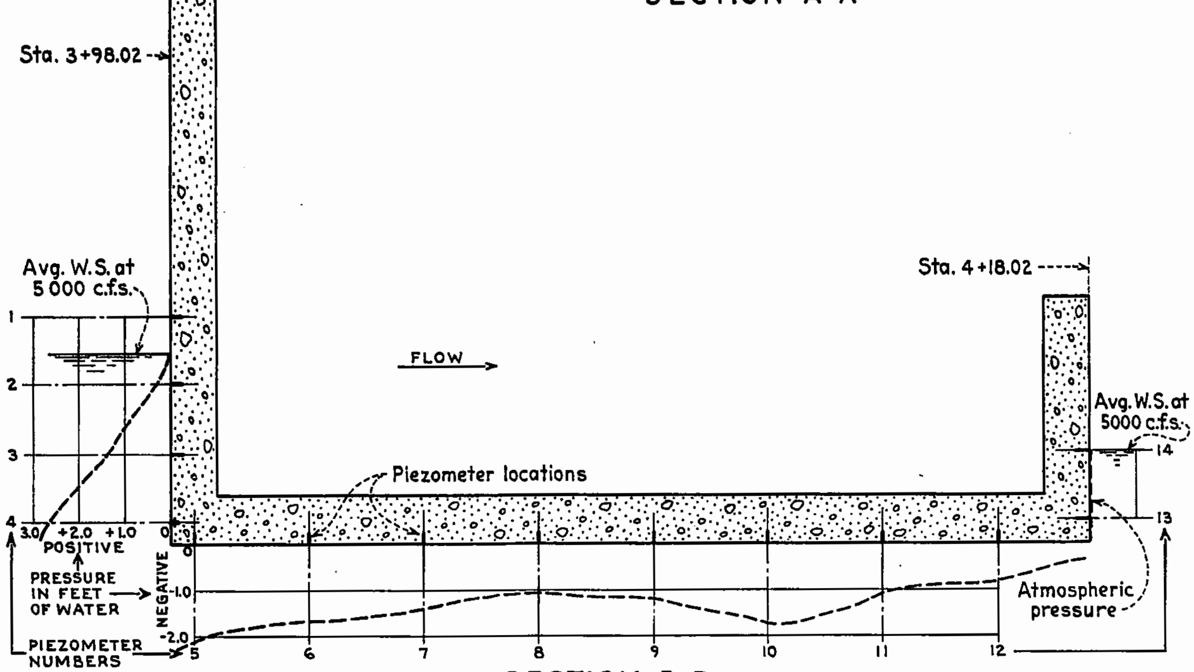
FRIANT-KERN CANAL OUTLET WORKS
HYDRAULIC MODEL STUDIES
WAVE RECORDS



PLAN



SECTION A-A



SECTION B-B
PRESSURE DISTRIBUTION ON UNDERPASS

FRIANT-KERN OUTLET WORKS
HYDRAULIC MODEL STUDIES
PRESSURE DISTRIBUTION ON UNDERPASS