UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

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Memorandum to Chief Designing Engineer

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REDESIGN OF CHECK DROPS
SUNNYSIDE MAIN CANAL
YAKIMA PROJECT - WASHINGTON

by

H. G. DEWEY, JR., JUNIOR ENGINEER

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Denver, Colorado,

February 28, 1939.

Denver, Colorado, February 28, 1939.

MEMORANDUM TO CHIEF DESIGNING ENGINEER
(H. G. Dewey, Jr.)

Subject: Redesign of check drops - Sunnyside Main canal - Yakima project, Washington.

1. Introduction. This report deals with the model studies made on check drop no. 4. In the report of an inspection trip to the Yakima project from May 25 to June 6, 1938 (memorandum to Chief Engineer, August 8, 1938, by Engineer J. E. Warnock), mention was made of the excessive scour and poor flow conditions existing at most of the check drops in the Sunnyside Main canal. In that report (page 8), it was stated: "The original designed capacity of the canal was much less than that now being carried. As the demand increased, larger quantities were handled. The resulting increase of velocity scoured the canal to such an extent that a series of 23 or 24 check drops were constructed in the canal. The scour downstream from at least 18 of these drops has been a source of continuous trouble since the drops were constructed. The first two were designed for a capacity of 1,076 second-feet and, for many years, have been handling 1,300 second-feet. Attempts to stop this scour have been primarily by riprapping, but little or no improvement has been accomplished. At drop no. 2, the width of the canal a short distance downstream from the structure is at least 50 percent greater than the normal width, and Superintendent Moore says that the hole in the center is about 15 feet deep. The size of the hole scoured below these structures is excessive considering the very small amount of drop. The stream of water through the control is concentrated in the center of the canal, and the high velocity prevails for several hundred feet downstream with little dissipation. As the high-velocity water leaves the drop, there is a difference in water level due to the velocity head. This causes an inflow of water on each side near the structure. This inflow is harmful; (1) as it causes a heavy flow upstream to replace that carried away by the high-velocity stream; and (2) as it disrupts all tendency toward the formation of a hydraulic jump. To completely solve this problem, it will be necessary to make a model study of a typical structure. It is believed that the majority of the faults can be remedied by extending the abutment wall sufficiently far downstream so that the adjacent water will not be drawn into the stream, and a hydraulic jump is permitted to form. The intermediate training walls were included in the Marshall Ford stilling pool for this same purpose. The flow conditions through the structure itself can be greatly improved by streamlining the steel brackets which support the flashboards."

On July 16, 1938, the Superintendent of the Yakima project submitted a letter to the Chief Engineer, requesting that a study be made of this problem. In paragraph (1) of this letter, it was stated: "From time to time during the past 15 to 20 years, various members of the organization have made suggestions and plans for remodeling some 20 or more drops or checks of standard design on the Sunnyside canal to prevent or reduce excessive scouring and erosion in the light volcanic ash soils below these structures." In paragraph (6), it was stated: "You will recall, no doubt, that this matter was discussed with you at the time of your visit to the project in March. I have also conferred with Mr. McBirney and Mr. Warnock relative to the problem. It occurs to me that a test on a model of this design would be enlightening in reaching a solution." In paragraph (9), it was stated: "It is requested that your office make a study of this problem, looking toward a suitable modification of these structures, and furnish designs for our use in connection with a C.C.C. work program to be undertaken this fall on at least one drop, such as no. 4, illustrated herewith," figure 1 of this report, "which is typical of the worst conditions."

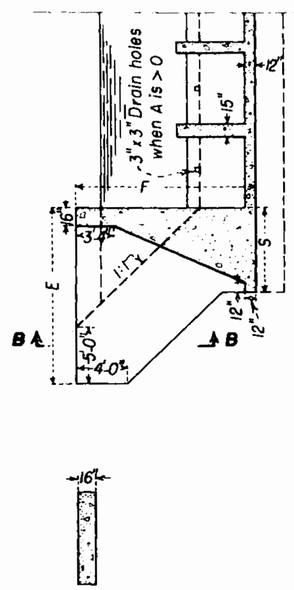
As a result of these recommendations, the problem of the redesign of the check drops was assigned on July 22, 1938, to the hydraulic laboratory of the United States Bureau of Reclamation. At that time, it was decided to make model studies on check drop no. 4. The recommended design determined from this study may readily be adapted to the other check drop structures since the drops in the Main canal are of a standard design (figure 2).

2. The Sunnyside Main canal. The Sunnyside division of the Yakima project includes about 106,000 acres. Nearly 90,000 acres are now being irrigated by the Sunnyside Main canal, which has its diversion dam and headworks near Parker and terminates about 75 miles to the southeast, near Acton, Washington (figure 3). At the time of its purchase by the Reclamation Service, the canal had a capacity of 650 second-feet at the headworks. After its purchase, plans were made to enlarge the canal and increase its capacity to 1,076 second-feet at the headworks. This work was completed in 1912. From 1917 to 1922, additional improvements were made, allowing the capacity at the headworks to be increased to 1,300 second-feet, which is maintained at the present time. Due to increasing the capacity of the canal, the hydraulic gradient was changed, resulting in increased velocities and lower water surface. To meet this condition, 23 check drops (figure 2) were built in the canal about two miles apart. These structures raised the water surface so that the desired amount of water could be diverted through the adjoining laterals and, at the same time, reduced the

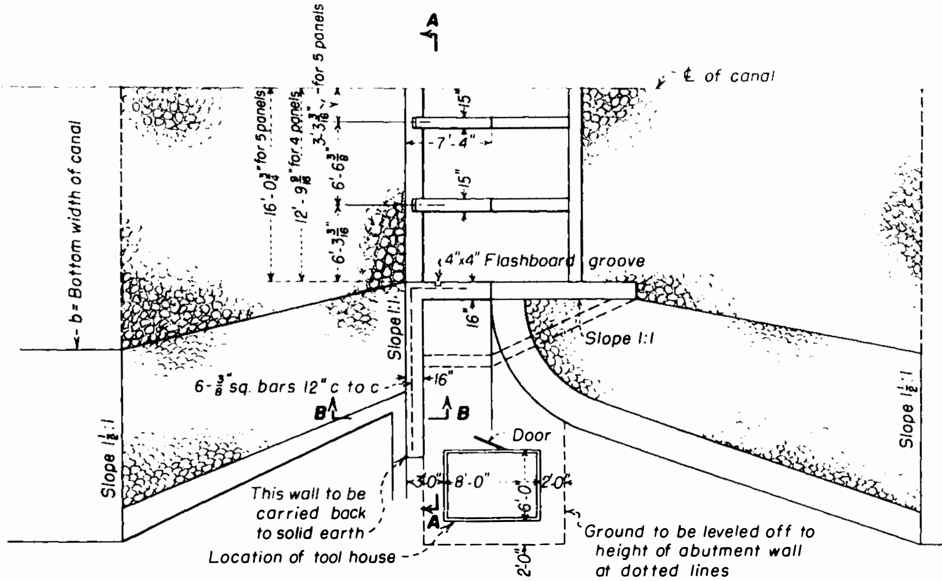
A. LOOKING UPSTREAM- DISCHARGE 1275 SECOND-FEET

B. LOOKING UPSTREAM

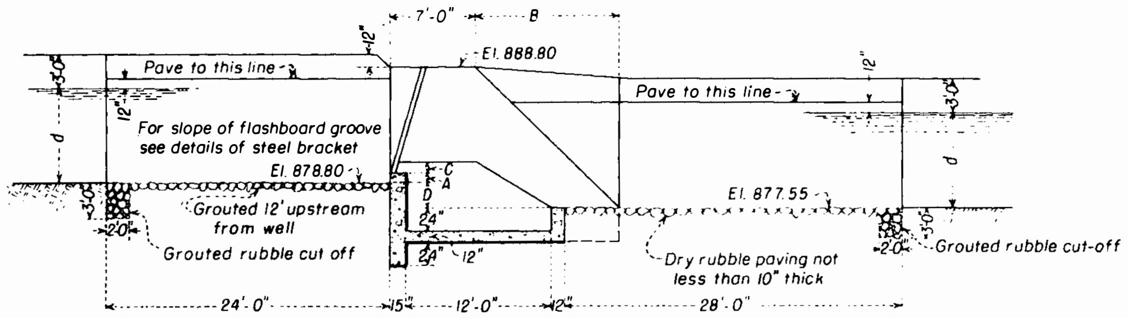
SCOUR AT CHECK DROP 4 - OCTOBER 1938



SECTION B-B



HALF PLAN

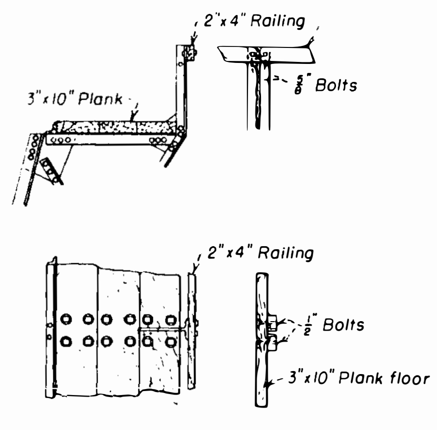


SECTIONAL ELEVATION

SCALE $\frac{1}{8}"=1'$

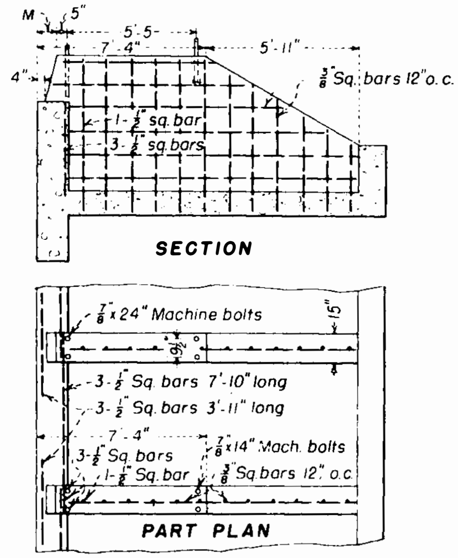
NOTE

Make last 8' of paving rough with projections above the normal surface (to impede velocity)



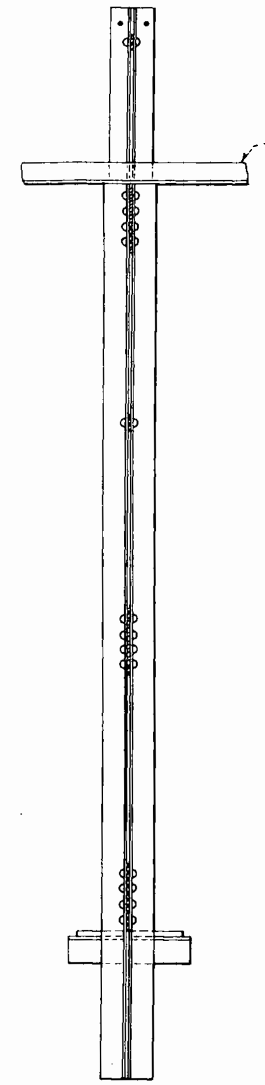
DETAIL OF FLOOR AND RAILING FASTENINGS

SCALE $\frac{1}{2}"=1'$

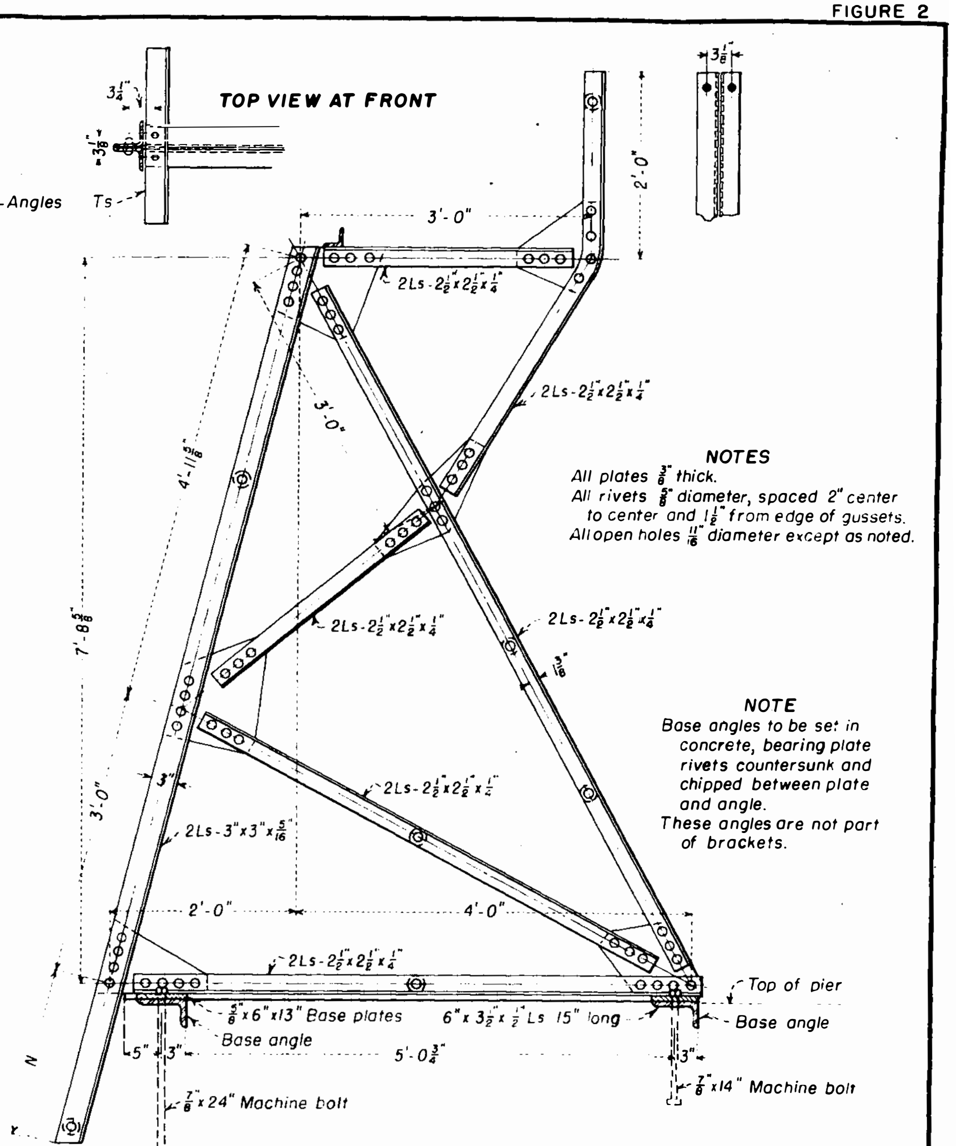


DETAIL OF REINFORCEMENT AND BOLT CENTERS

SCALE $\frac{1}{4}"=1'$



FRONT VIEW



DETAILS OF STEEL BRACKET

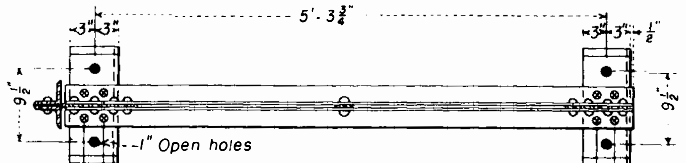
SCALE $1"=1'$

NOTES

All plates ⅜" thick.
All rivets ⅝" diameter, spaced 2" center to center and 1⅛" from edge of gussets.
All open holes $\frac{11}{16}$" diameter except as noted.

NOTE

Base angles to be set in concrete, bearing plate rivets countersunk and chipped between plate and angle.
These angles are not part of brackets.

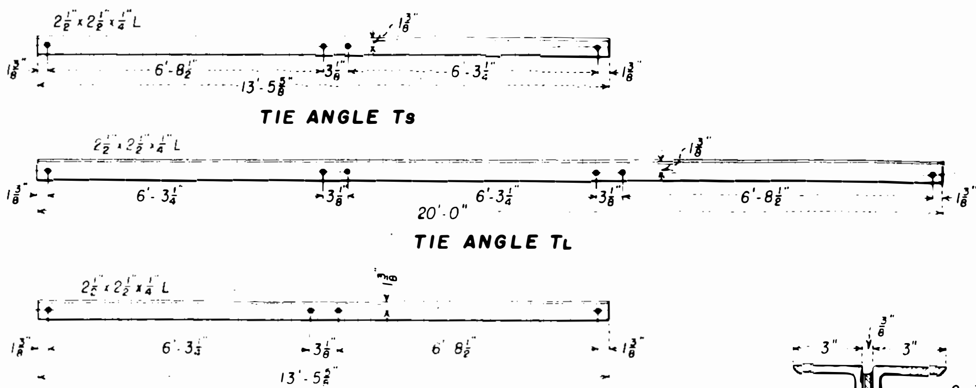


TOP VIEW OF BASE

TABLE OF DIMENSIONS AND QUANTITIES

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
REVISED NO. OF DROP	b	d	Q*	D	NO. OF PANELS	A	C	B	E	F	S	M	N	CONCRETE CU. YDS.	REINF. STEEL #	PAVING SQ. YDS.	NO. OF BRACKETS	STR. STEEL #	NO. OF FLASHBDS	FLOOR, ETC FT. B. M.	BRACKETS MARKED	ENGINEERS STATION
1	46	8.0	1076	1'-6"	5	0'-0"	2'-0"	11'-6"	15'-0"	14'-6"	5'-8¾"	0'-10"	2'-3"	92.6	441	413	4	2280	85	322	B-2	156
2	45	8.0	1055	2'-0"	5	0'-6"	1'-6"	12'-0"	15'-0"	15'-0"	5'-10¾"	0'-8½"	1'-9"	97.7	441	410	4	2280	80	322	B-3	239
3	45	8.0	1055	2'-0"	5	0'-6"	1'-6"	12'-0"	15'-0"	15'-0"	5'-10¾"	0'-8½"	1'-9"	97.7	441	410	4	2280	80	322	B-4	310
4	45	8.0	1055	1'-3"	5	0'-0"	2'-0"	11'-3"	15'-0"	14'-3"	5'-7¾"	0'-10"	2'-3"	89.6	441	410	4	2280	85	322	B-5	386
5	44	8.0	1039	1'-7¼"	5	0'-0"	2'-0"	11'-7¼"	15'-0"	14'-7¼"	5'-9"	0'-10"	2'-3"	96.5	441	407	4	2280	85	322	B-6	475
6	44	8.0	1039	2'-3"	5	1'-0"	1'-0"	12'-3"	15'-0"	15'-3"	5'-11¾"	0'-7"	1'-2¾"	102.5	441	407	4	2280	75	322	B-7	546
7	43	8.0	1010	1'-0"	5	0'-0"	2'-0"	11'-0"	15'-0"	14'-0"	5'-6¾"	0'-10"	2'-3"	88.2	441	404	4	2280	85	322	B-8	635
8	42	8.0	982	1'-9"	5	0'-6"	1'-6"	11'-9"	15'-0"	14'-9"	5'-9¾"	0'-8½"	1'-9"	96.2	441	401	4	2280	80	322	B-9	755
9	42	8.0	982	1'-0"	5	0'-0"	2'-0"	11'-0"	15'-0"	14'-0"	5'-6¾"	0'-10"	2'-3"	88.2	441	401	4	2280	85	322	B 10	812
10	41	8.0	982	2'-0"		Zillah Wasteway																893
11	40	8.0	966	1'-0"	5	0'-0"	2'-0"	11'-0"	15'-0"	14'-0"	5'-6¾"	0'-10"	2'-3"	88.2	441	394	4	2280	85	322	B-12	963
12	40	8.0	952	1'-0"	5	0'-0"	2'-0"	11'-0"	15'-0"	14'-0"	5'-6¾"	0'-10"	2'-3"	88.2	441	394	4	2280	85	322	B-13	1049
13	40	8.0	938	1'-0"	5	0'-0"	2'-0"	11'-0"	15'-0"	14'-0"	5'-6¾"	0'-10"	2'-3"	88.2	441	394	4	2280	85	322	B-14	1125
14	38	8.0	915	2'-6"	5	2'-0"	0'-0"	12'-6"	15'-0"	15'-6"	6'-0¾"	0'-4"	0'-2¼"	106.2	441	390	4	2280	65	322	B-15	1225
15	36	8.0	892	2'-0"	5	1'-0"	1'-0"	12'-0"	15'-0"	15'-0"	5'-10¾"	0'-7"	1'-2¾"	97	441	382	4	2280	75	322	B-16	1313
16	36	8.0	892	2'-0"	5	1'-0"	1'-0"	12'-0"	15'-0"	15'-0"	5'-10¾"	0'-7"	1'-2¾"	97	441	382	4	2280	75	322	B-17	1364
17	34	8.0	840	1'-0"	5	0'-0"	2'-0"	11'-0"	15'-0"	14'-0"	5'-6¾"	0'-10"	2'-3"	88.2	441	374	4	2280	85	322	B-18	1488
18	32	8.0	808	1'-9"		Snipes Mountain Lateral																1601
19	26	7.1	595	0'-6"	5	0'-0"	1'-1¼"	9'-7¼"	14'-0"	12'-7¼"	5'-1"	0'-7¼"	1'-4"	75.3	441	316	4	2280	75	322	B-20	1660
20	24	7.0	556	0'-6"	5	0'-0"	1'-0"	9'-6"	14'-0"	12'-6"	5'-0¾"	0'-7"	1'-2¾"	71.3	441	310	4	2280	75	322	B-21	1790
21	24	6.6	515	1'-3"	4	S.C.W.W. Headworks						0'-5¼"	0'-7¼"		See S P 677				56		B-24	1965
22	22	6.7	490	0'-9"	4	0'-1¼"	0'-7¼"	9'-5½"	13'-8½"	12'-5½"	5'-0½"	0'-5¾"	0'-9¾"	58.6	349	346	3	1710	56	240	B-23	2141
23	22	6.0	430	0'-7¼"	4	0'-0"	0'-0"	8'-7¼"	13'-0"	11'-7¼"	4'-9"	0'-4"	0'-2¼"	54.3	349	332	3	1710	48	240	B-26	2420

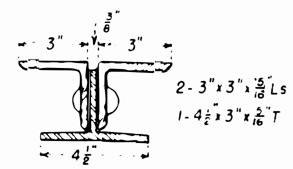
* To obtain present discharge, multiply tabular value by $\frac{1300}{1076}$.



TIE ANGLE Ts

TIE ANGLE TL

TIE ANGLE TR



CROSS SECTION OF FACE OF BRACKETS AS RECONSTRUCTED IN WINTER OF 1908

SCALE $3"=1'$ SEE S. P. 600 FOR DETAILS

NOTES

This drawing traced from Dwg. SP628, dated January 1909.
Elevations shown are for Check Drop 4 only.
For detail of model see Figure 5.

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

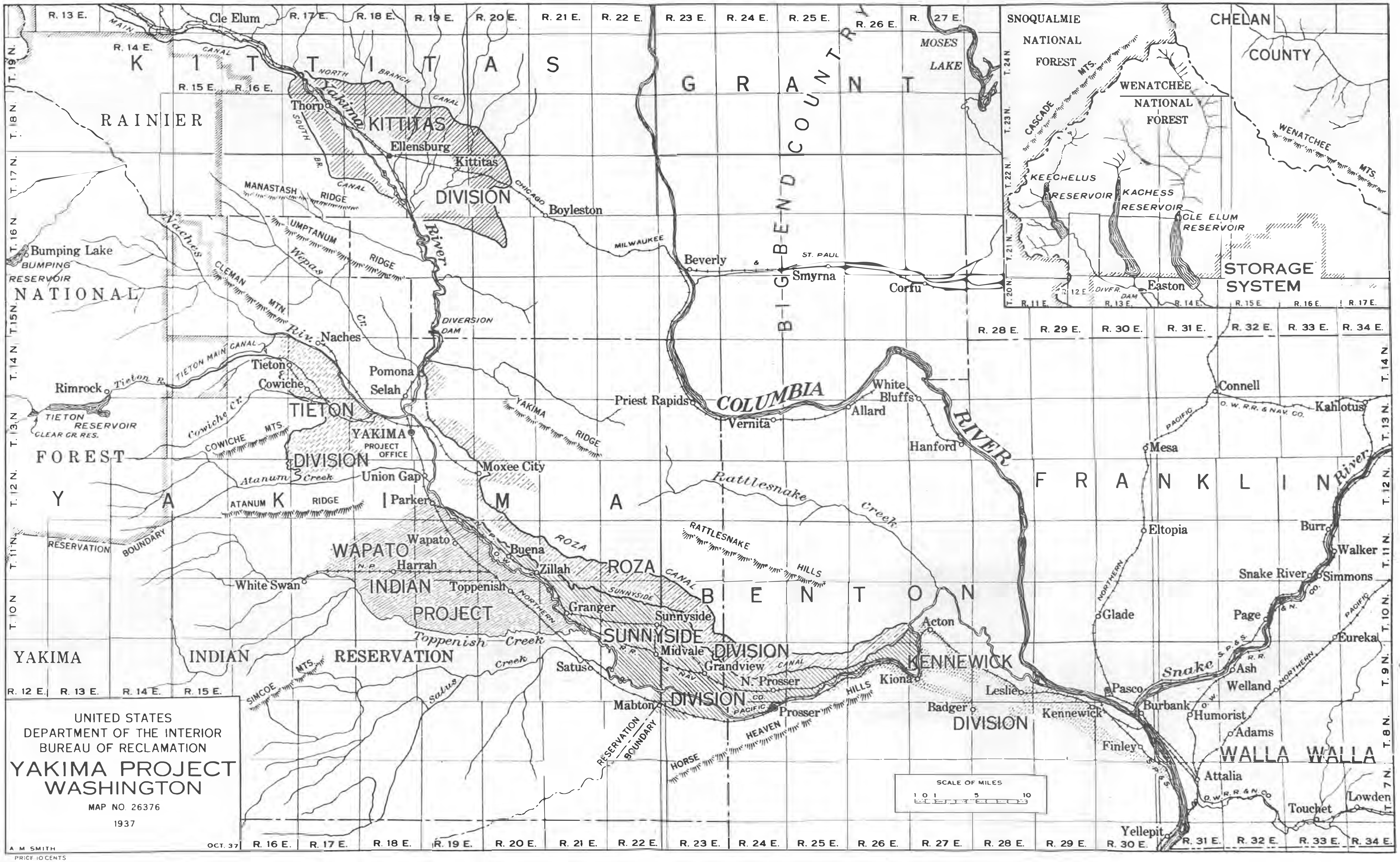
YAKIMA PROJECT - WASHINGTON

MAIN CANAL STANDARD DROPS

DRAWN E.A.N.	SUBMITTED
TRACED C.A.B.	RECOMMENDED
CHECKED E.H.E.	APPROVED

33-D-1772

FIGURE 3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DEPARTMENT OF THE INTERIOR
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YAKIMA PROJECT
WASHINGTON
MAP NO. 26376
1937
A. M. SMITH
PRICE 10 CENTS
OCT. 37
KITTITAS
GRANT
BIG BEND COUNTRY
RAINIER
NATIONAL
FOREST
YAKIMA
BENTON
FRANKLIN
WALLA WALLA
YAKIMA INDIAN RESERVATION
KITTITAS DIVISION
TIETON DIVISION
WAPATO INDIAN PROJECT
ROZA
SUNNYSIDE DIVISION
KENNEWICK DIVISION
YAKIMA PROJECT OFFICE
COLUMBIA
RIVER
Snake River
Yakima River
Cle Elum
Thorp
Ellensburg
Kittitas
Boyleston
Beverly
Smyrna
Corfu
Bumping Lake
BUMPING RESERVOIR
Naches
Tieton
Cowiche
Rimrock
TIETON RESERVOIR
CLEAR CR. RES.
Pomona
Selah
YAKIMA
Union Gap
Moxee City
Parker
Wapato
Harrah
Buena
Zillah
Toppenish
White Swan
Granger
Sunnyside
Midvale
Grandview
N. Prosser
Prosser
Satus
Mabton
Kiona
Acton
Badger
Leslie
Kennewick
Finley
Pasco
Burbank
Humorist
Adams
Attalia
Yellepit
Touchet
Lowden
Priest Rapids
Vernita
White Bluffs
Allard
Hanford
Connell
Kahlotus
Mesa
Eltopia
Glade
Burr
Walker
Snake River
Simmons
Page
Eureka
Ash
Welland
MANASTASH RIDGE
UMPTANUM RIDGE
CLEMAN MTN.
COWICHE MTS.
ATANUM RIDGE
YAKIMA RIDGE
RATTLESNAKE HILLS
Rattlesnake Creek
HORSE HEAVEN HILLS
SIMCOE MTS.
Toppenish Creek
Satus Creek
Atanum Creek
Cowiche Cr.
Wenas Cr.
Naches Riv.
Tieton R.
DIVERSION DAM
MAIN CANAL
NORTH BRANCH CANAL
SOUTH BR. CANAL
TIETON MAIN CANAL
ROZA CANAL
SUNNYSIDE CANAL
RESERVATION BOUNDARY
CHICAGO MILWAUKEE & ST. PAUL
NORTHERN PACIFIC
O. W. R.R. & NAV. CO.
S. P. & S.
MOSES LAKE
SNOQUALMIE NATIONAL FOREST
CASCADE MTS.
WENATCHEE NATIONAL FOREST
WENATCHEE MTS.
CHELAN COUNTY
KEECHELUS RESERVOIR
KACHESS RESERVOIR
CLE ELUM RESERVOIR
Easton
DIVER. DAM
STORAGE SYSTEM
SCALE OF MILES
R. 13 E.
R. 14 E.
R. 15 E.
R. 16 E.
R. 17 E.
R. 18 E.
R. 19 E.
R. 20 E.
R. 21 E.
R. 22 E.
R. 23 E.
R. 24 E.
R. 25 E.
R. 26 E.
R. 27 E.
R. 28 E.
R. 29 E.
R. 30 E.
R. 31 E.
R. 32 E.
R. 33 E.
R. 34 E.
T. 19 N.
T. 18 N.
T. 17 N.
T. 16 N.
T. 15 N.
T. 14 N.
T. 13 N.
T. 12 N.
T. 11 N.
T. 10 N.
T. 9 N.
T. 8 N.
T. 7 N.

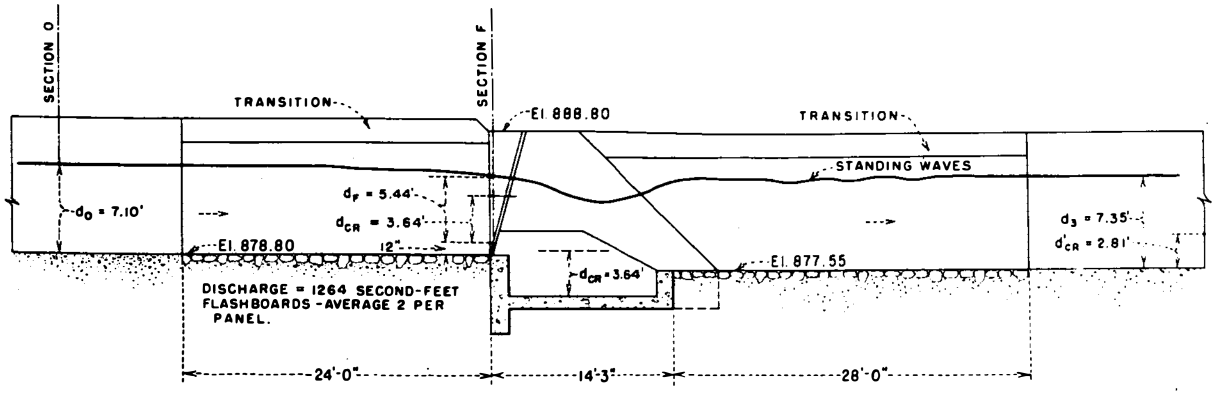
velocities in the canal. Most of the check drops were built in the winter of 1907-1908, and the remaining ones were completed during the period 1909-1916.

3. Solution by model study. Model studies are generally adapted to give a solution to a hydraulic problem when there is no analytical method that can be applied with reasonable accuracy. The scour effect of flow through existing small drop structures is a problem of this type. It is entirely possible, however, to make an analytical study of the type of flow, for example, hydraulic jump or standing wave, that exists at the drop structures. When this is done, it is possible to give the reasons for their unsatisfactory operation in addition to the reasons given as a result of observation made on the existing structure and on the model. In order to classify the particular type of flow that exists at check drop no. 4, it is first necessary to discuss the hydraulic jump and the standing wave, referring to a specific-energy diagram.

4. The specific-energy diagram. For a given discharge, a specific-energy diagram is obtained by plotting, as abscissae, specific energy, which is the sum of the velocity head, $\frac{V^2}{2g}$, and the depth of flow, d, and plotting, as ordinates, d, the depth of flow. Figure 4C shows the specific-energy diagram for check drop no. 4. It has been plotted for a discharge of 1,264 second-feet through the rectangular section having a width of 32.13 feet. It must be understood that the specific energy varies with the depth of flow in a section and is referred to the bottom as a datum which may change from section to section. At one particular depth of flow for the given discharge, the specific energy is a minimum. This depth at which a given discharge flows with a minimum specific energy is called "critical depth." It is now possible, by referring to the specific-energy diagram, to trace the change of specific energy in the flow with a change of depth in a section.

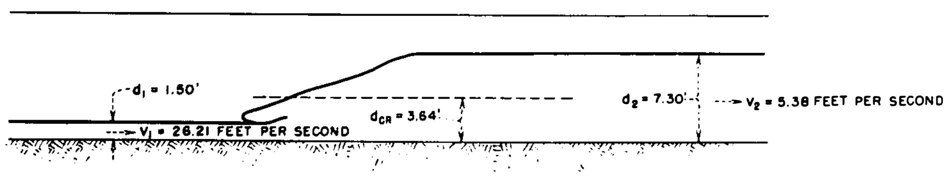
5. The hydraulic jump. Referring to figures 4B and 4C, it is seen that a hydraulic jump is the phenomenon by which rapid flow at a depth, d_1, below critical depth passes in an abrupt manner to tranquil flow at a depth, d_2, above critical depth. The amount of specific-energy head lost in the jump is represented by $E_1 - E_2$. Now, for a constant cross section, the velocity, V_1, above the jump has been reduced to a velocity, V_2, below the jump.

The depths d_1 and d_2, upstream and downstream from the jump, respectively, are called conjugate depths. A definite relation between them may be obtained by applying the momentum



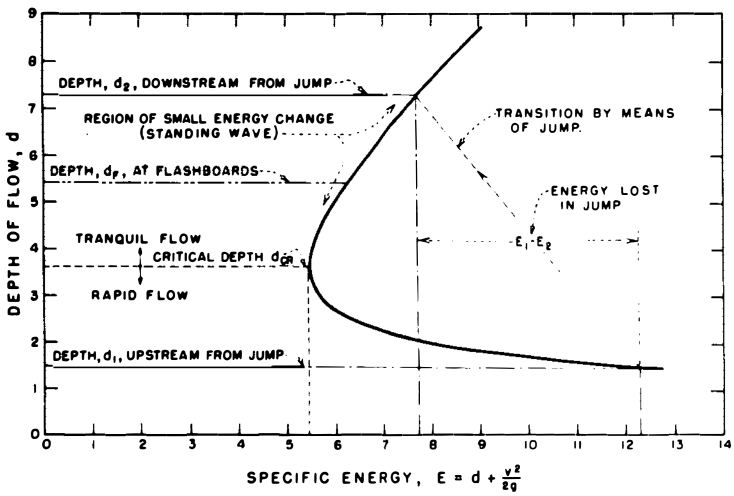
A

DEPTH RELATION THROUGH DROP STRUCTURE



B

HYDRAULIC JUMP



C

SPECIFIC ENERGY DIAGRAM

DISCHARGE 1264 SECOND-FEET - WIDTH AT DROP 32.13 FEET

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YAKIMA PROJECT - WASHINGTON

REDESIGN OF CHECK DROPS - SUNNYSIDE MAIN CANAL

HYDRAULIC MODEL STUDIES - 1:15 SCALE

HYDRAULIC ANALYSIS AT CHECK DROP 4

DRAWN	H G D. JR.	SUBMITTED	
TRACED	B B - F B.	RECOMMENDED	
CHECKED	H G D JR	APPROVED	
	DENVER, COLORADO - JAN 3, 1939	33-D-1773	

principle. This relation is

$$d_2 = -\frac{d_1}{2} + \sqrt{\frac{2V_1^2 d_1}{g} + \frac{d_1^2}{4}}$$

From this relation, if either depth and corresponding velocity are known for a given discharge, the other depth and corresponding velocity are readily determined. It is important to remember that this relation applies only to a hydraulic jump.

Further study of the specific-energy diagram reveals that, if, in a constant cross section, the depth of flow, d_2, be less than d_1, there will be a gain in specific energy. But, if the depth of flow, d_2, be greater than d_1, there will be a loss in specific energy. In the particular case where the increase in depth above d_1 passes through critical depth, then, as has been seen, a hydraulic jump occurs with a large loss in energy. It will also be observed that, in the case of a hydraulic jump, the ratio, $\frac{d_1}{d_2}$, is much less than unity. This is evident since large energy losses are accompanied by great changes in depth.

6. <u>The standing wave.</u> If sufficient energy is not present in the flow to cause d_1 to be below critical depth, then the flow is said to be in the tranquil state, usually characterized by standing waves or surface undulations and very small energy loss. This phenomenon may also occur even with d_1 just slightly below critical depth. Referring to the specific-energy diagram in the region above critical depth, if the depth of flow, d_2, is less than d_1, there will be a loss in specific energy. But, if the depth of flow, d_2, be greater than d_1, there will be a gain in specific energy. These energy changes are directly opposite to those encountered in the hydraulic jump, and standing waves may form with either an increase or decrease in d_2. There is no abrupt change of depth when standing waves exist, so that the ratio, $\frac{d_1}{d_2}$, is nearly unity. Applying this to the specific-energy diagram in the region above critical depth, it is seen that the rate of change of energy is very small.

7. <u>Classification of check drop no. 4.</u> To determine analytically the type of flow that exists at check drop no. 4 is difficult due to the fact that the losses inherent in the flow through the structure are not readily determined. It is possible, however, to compute the depth of flow at the flashboards by applying Bernoulli's equation from the canal section above the drop to

the flashboards. Bernoulli's equation is referred to a constant datum instead of a variable one, as provided in the specific-energy relation.

From observed field data:

Q = 1,264 cubic feet per second.
Depth of flow upstream, d_o, = 7.10 feet.
Depth of flow downstream, d_3, = 7.35 feet.
Flashboards average two per panel.

Writing Bernoulli's equation from section O to section F (figure 4A):

(1) $d_o + \frac{V_o^2}{2g} = d_F + \frac{V_F^2}{2g} + 1.00 +$ losses. (The losses are small in this reach and may be neglected.)

with $d_o = 7.10$ feet, $\frac{V_o^2}{2g} = 0.16$.

Also, $V_F d_F = q$, discharge per foot of width. The width is 32.13 feet at rectangular section.

Now, $V_F d_F = \frac{1,264}{32.13} = 39.33$

$\therefore V_F = \frac{39.33}{d_F}$, substituting in (1),

$$7.10 + 0.16 = d_F + \frac{24.05}{d_F^2} + 1.00.$$

(2) $6.26 = d_F + \frac{24.05}{d_F^2}$.

It is found, by solving equation (2), that $d_F = 5.44$ feet, or $d_F = 2.55$ feet. Critical depth at section F is obtained from the relation

$$d_{cr} = \left[\frac{q^2}{g}\right]^{1/3} = \left[\frac{(39.33)^2}{32.2}\right]^{1/3} = 3.64 \text{ feet.}$$

(Critical depth in the trapezoidal canal section is $d'_{cr} = 2.81$ feet.) It is noted that one value of d_F is greater than critical depth, but the other value is less than critical depth. It can readily be seen that only one value applies here. If the tail water below the drop could be lowered sufficiently, the depth of flow over the flashboards would be practically critical depth; furthermore, it would be impossible for the flow to pass below critical depth at this point in its natural tendency to fall since critical depth corresponds to the least possible content of energy. Any additional lowering of the water surface at the flashboards below critical depth would require energy to be added from an outside source. Hence, the value of d_F, which is greater than critical depth, applies here, or $d_F = 5.44$ feet.

It has been shown that the following criteria are necessary for the formation of a hydraulic jump: (1) The depth of flow upstream from the jump, d_1, must be below critical depth, and the depth of flow downstream from the jump, d_2, must be above critical depth; (2) d_2 must be greater than d_1; and (3) the conjugate depths, d_1 and d_2, must bear a definite relation to each other. It has also been shown that, when d_1 and d_2 are above critical depth, standing waves will form. The criteria necessary for the formation of a hydraulic jump are not satisfied at check drop no. 4 because: (1) Considering d_F to be similar to d_1, the depth upstream from a jump, it has been shown that d_F is greater than critical depth; (2) if, in the conjugate-depth relation, we let $d_F = d_1 = 5.44$ feet, then $d_2 = 2.28$ feet, which is not only less than d_F, but it is also less than critical depth, which is impossible since no energy has been added from an outside source. The criterion necessary for the existence of standing waves is satisfied at check drop no. 4 because: (1) d_F is greater than critical depth; and (2) the presence of excessive tailwater depth at all times not only results in d_2 being above critical depth, but it prevents the flow from approaching critical depth at the flashboards.

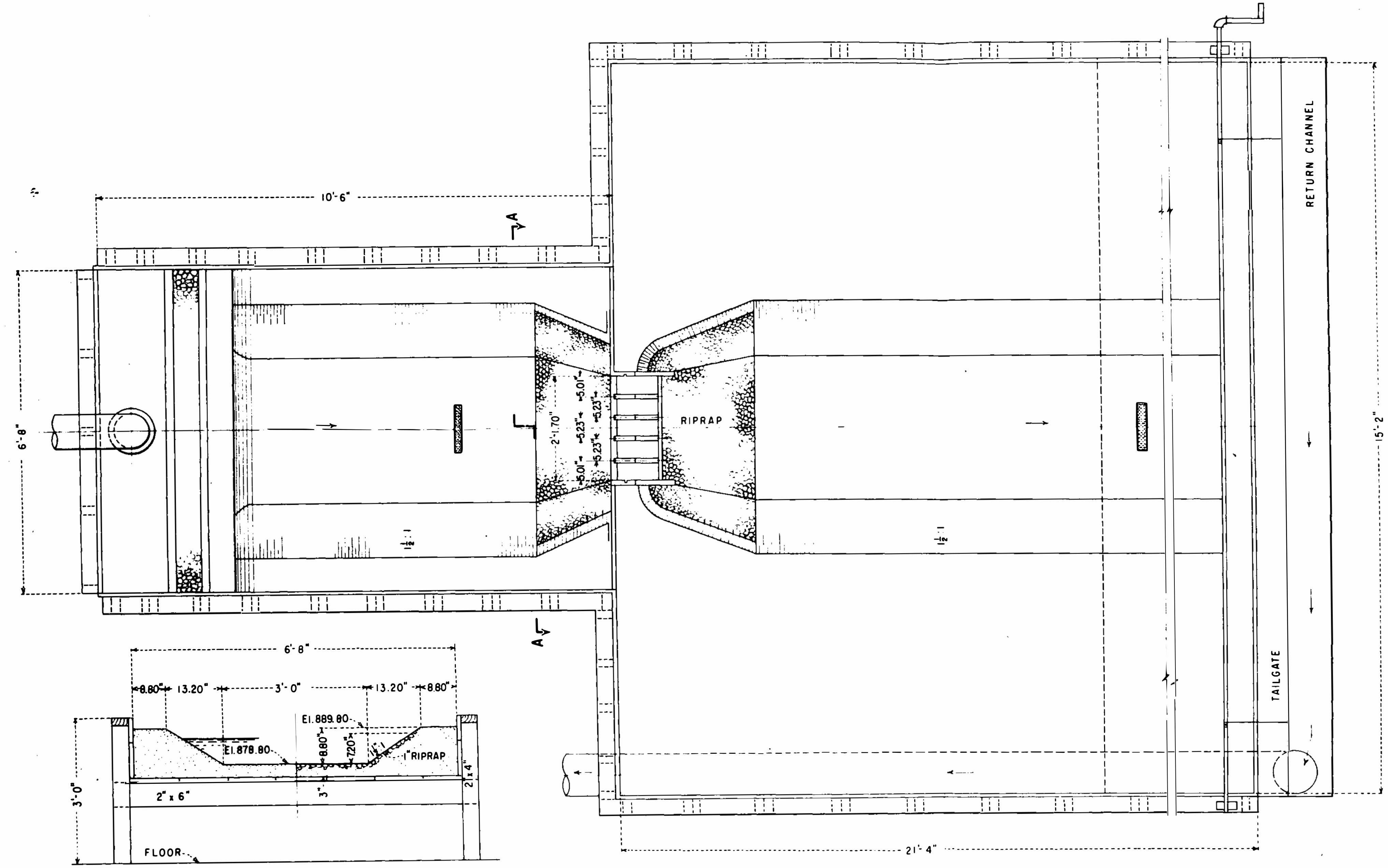
8. Comparison of losses. If the losses inherent in the flow were known from section F to any other points in the check basin, it would be possible to write Bernoulli's equation to determine the depth of flow at these points. With this data, the loss of specific energy could be determined through the structure. It is known, however, that the loss of specific energy in a standing wave is about five percent. The loss of specific energy in a hydraulic jump was seen previously to be much greater. Assume that, at check drop no. 4, sufficient energy is available to form a hydraulic jump. Then, let $d_1 = 1.50$ feet, and, if the discharge is 1,264 second-feet, $V_1 = 26.21$ feet per second. The conjugate depth, therefore, will be $d_2 = 7.30$ feet with a velocity $V_2 = 5.38$ feet per second (figure 4B). Now, $E_1 = 12.30$, and $E_2 = 7.76$; the loss $E_1 - E_2 = 4.54$ (figure 4C). The loss in energy will be

$\frac{4.54}{12.30}$ x 100, or 36.8 percent as compared to about 5 percent in a standing wave.

9. The hydraulic jump compared to standing wave at check drop no. 4. In present-day designs of small drop structures, the hydraulic jump is usually adapted to dissipate the energy of the flow as it passes through the structure. It has just been shown that, when this is done, the energy of the flow is not only greatly dissipated, but the velocity of the flow is considerably reduced. For best efficiency, the hydraulic jump is confined in a rectangular stilling pool, which usually includes dentated sills or teeth to aid in the formation of a uniform velocity distribution below the jump. If the canal section is riprapped for a short distance downstream from the stilling pool, very little scour, if any, will occur in the canal. But severe scour will occur downstream from a drop structure, even though a hydraulic jump forms, when the jump is not properly confined or controlled. Many problems of this nature have been corrected by model studies.

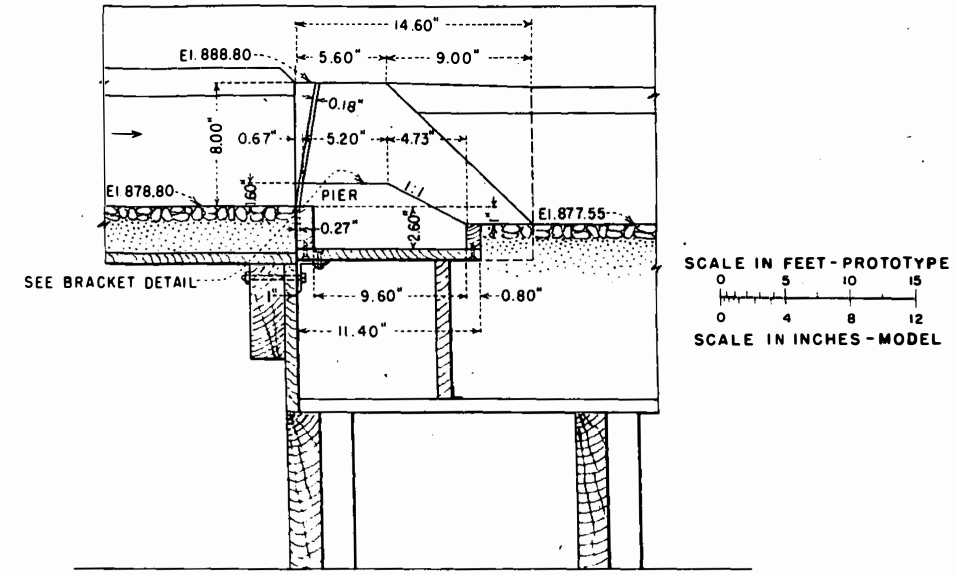
Unfortunately the design of check drop no. 4, and similar drop structures in the Sunnyside Main canal, provides but little energy dissipation to the flow. The classification above showed that standing waves form which dissipate only about five percent of the specific energy in the flow. Under this condition, the velocity reduction is very little; hence, the canal downstream is subjected to velocities which are excessive for the fine material, volcanic tuff, through which the Main canal flows. Observations of the prototype and model reveal that these velocities are concentrated near the center of the canal. Further observations made on the model, discussed more fully below, disclose: (1) That, as a result of the nonuniform velocity distribution, large eddies form along the sides of the canal, which have sufficient velocity to cut into the canal banks and produce the excessive scour that exists downstream from most of the check drops in the Main canal; (2) that, even though the standing waves at the drop were confined within a rectangular stilling pool provided with teeth, there was very little improvement in the flow; and (3) that riprap placed along the canal banks below the drop structure was washed into the canal.

10. The model. The model of check drop no. 4 was built to a scale of 1 to 15 (figures 5 and 6). The drop structure was constructed of redwood, and the steel brackets were made of light-gage sheet metal. The drop structure was installed in a large, metal-lined box, which provided sufficient length of approach upstream from the drop and sufficient width

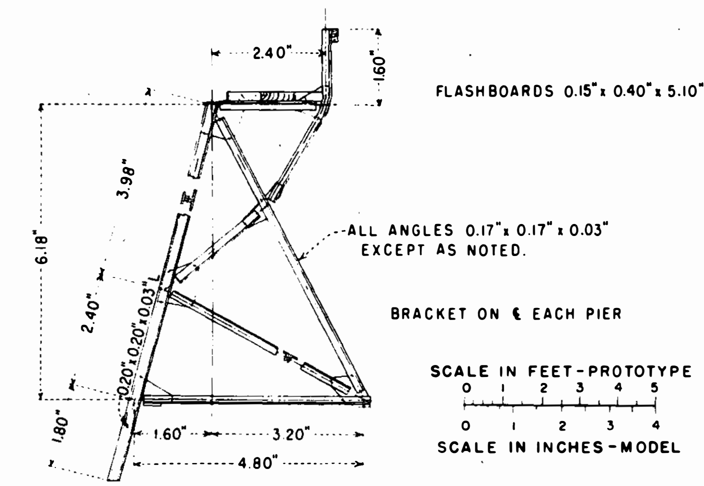


PLAN

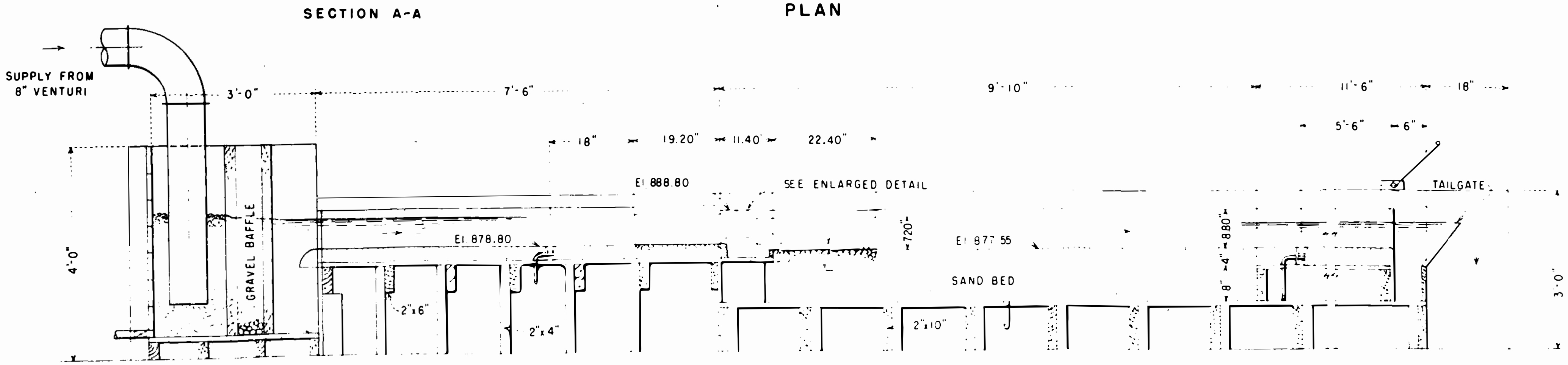
SECTION A-A



ENLARGED DETAIL OF DROP



BRACKET DETAIL



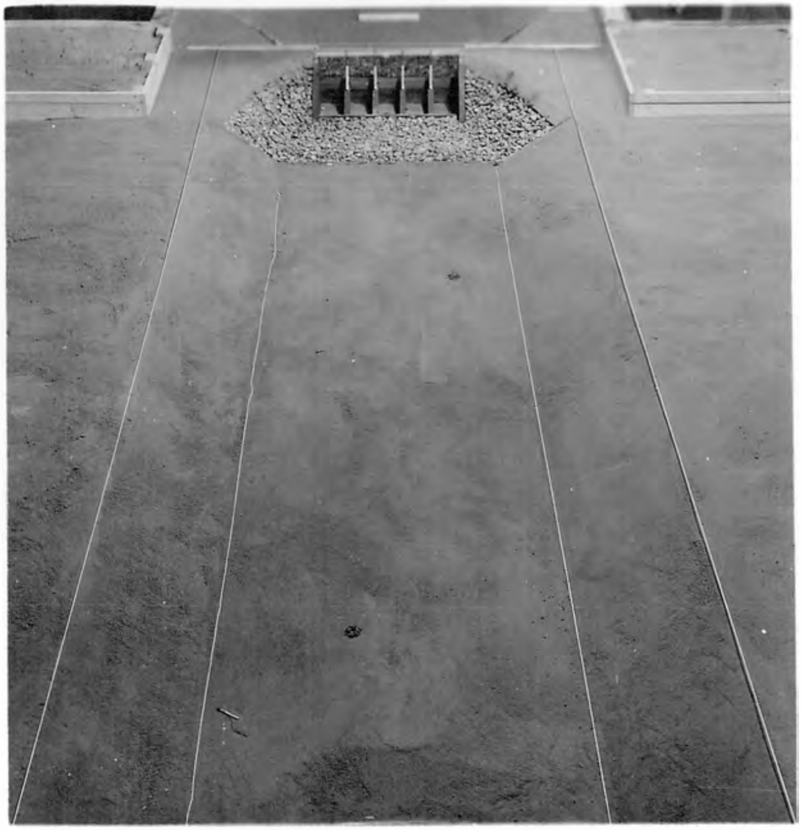
SECTION ON CENTER LINE

SCALE IN FEET - PROTOTYPE
0 15 30 45 60 75
0 12 24 36 48 60
SCALE IN INCHES - MODEL

FOR DETAILS OF EXISTING STRUCTURE SEE FIGURE 2

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YAKIMA PROJECT - WASHINGTON
REDESIGN OF CHECK DROPS - SUNNYSIDE MAIN CANAL
HYDRAULIC MODEL STUDIES - 1:15 SCALE
MODEL OF EXISTING STRUCTURE AT CHECK DROP 4

DRAWN H.G.D.JR.		SUBMITTED
TRACED F.A.J.-F.B.		RECOMMENDED
CHECKED H.G.D.JR.		APPROVED
	DENVER, COLORADO - DEC. 17, 1938	33-D-1774



A. LOOKING UPSTREAM

B. DROP STRUCTURE

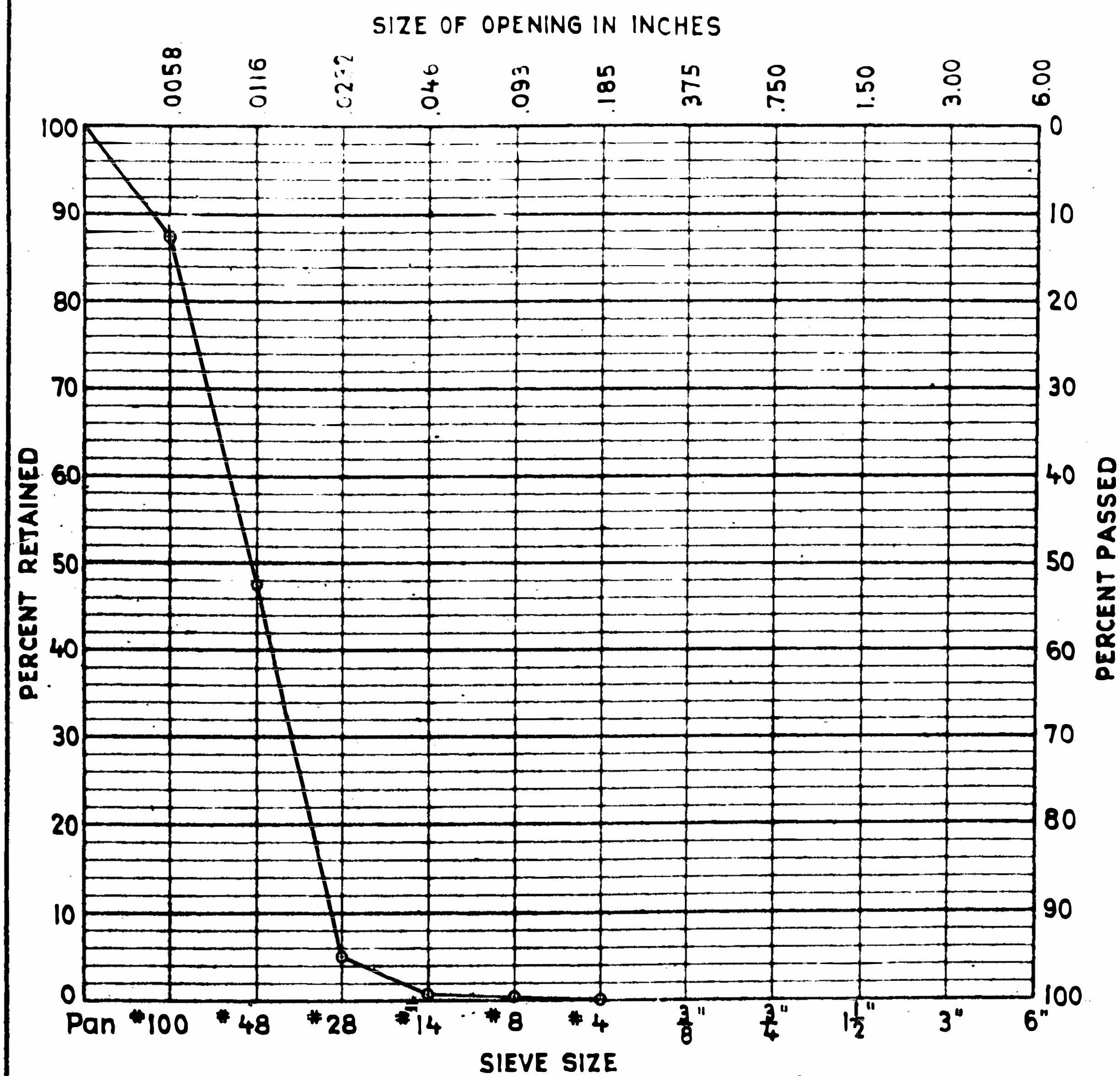
EXISTING STRUCTURE

and length downstream so that the scouring of the canal would not be restrained. Gages were installed to measure the depth of flow, which was controlled upstream by flashboards and downstream by a tail gate. Because the model velocities were considerably less than the prototype velocities, it was necessary to use, in the model, the finest sand available to reproduce the prototype scour which occurs in the volcanic tuff. A sieve analysis of the model sand is shown on figure 7.

11. The initial scour test. The purpose of the initial scour test was to make careful observations of the flow and scour in the model for comparison with the prototype. It was necessary to be certain that the model would reproduce qualitatively the prototype conditions before additional tests were made to determine a recommended design. For the initial test, the model was run for 26 hours at a discharge representing 1,300 second-feet. During this time, the number and symmetry of the flashboards and the tail-water depth were changed at intervals. At the time of the initial test, sufficient data were not available to determine the exact field relation between the depth of flow upstream and downstream for a certain discharge. Accordingly, during the initial scour test, the depths of flow were varied so that the model flow conditions at the drop would approximate those as witnessed in the field and as shown on available prototype photographs. It was determined, at a later date, upon receipt of observed field data, that the depths of flow maintained in the model were slightly less than observed depths. This had no effect on the final analysis. Figures 8 and 9 show the model in operation and the scour at the completion of the test.

12. Analysis of initial model test and comparison to prototype. As a result of the studies made from the initial test, it became evident that the following existing conditions had to be removed in order to have a satisfactory design: (1) Unsymmetrical flow distribution through the drop structure; (2) standing waves and excessive velocities below the drop; (3) eddies along the side of the canal immediately below the drop; and (4) scour to the canal.

The model showed that the poor approach condition to the drop structures was due to the wing walls being at right angles to the direction of flow. This caused a drawdown or dished effect as the flow entered the panels on each side of the structure (figures 9A and 18A). After passing over the flashboards, the flow along the side walls surged upward and became partially submerged by the return flow of the side eddy mentioned below. This disturbance caused an unbalanced condition in the region of the standing wave and further aggravated



GRADING

Sieve Size	% Ret.	Cum. %	Comb. % Ret.	Comb. % Cum.
6"				
3"				
1 1/2"				
3/4"				
3/8"				
#4	0.0	0.0		
F.M.				
#8	0.1	0.1		
#14	0.4	0.5		
#28	4.7	5.2		
#48	42.6	47.8		
#100	39.7	87.5		
Pan	12.5	100.0		
F.M.	1.41			
Comb. F.M.				
G/S				

Remarks: Sieve analysis of sand used in 1:15 scale model of check drop 4.

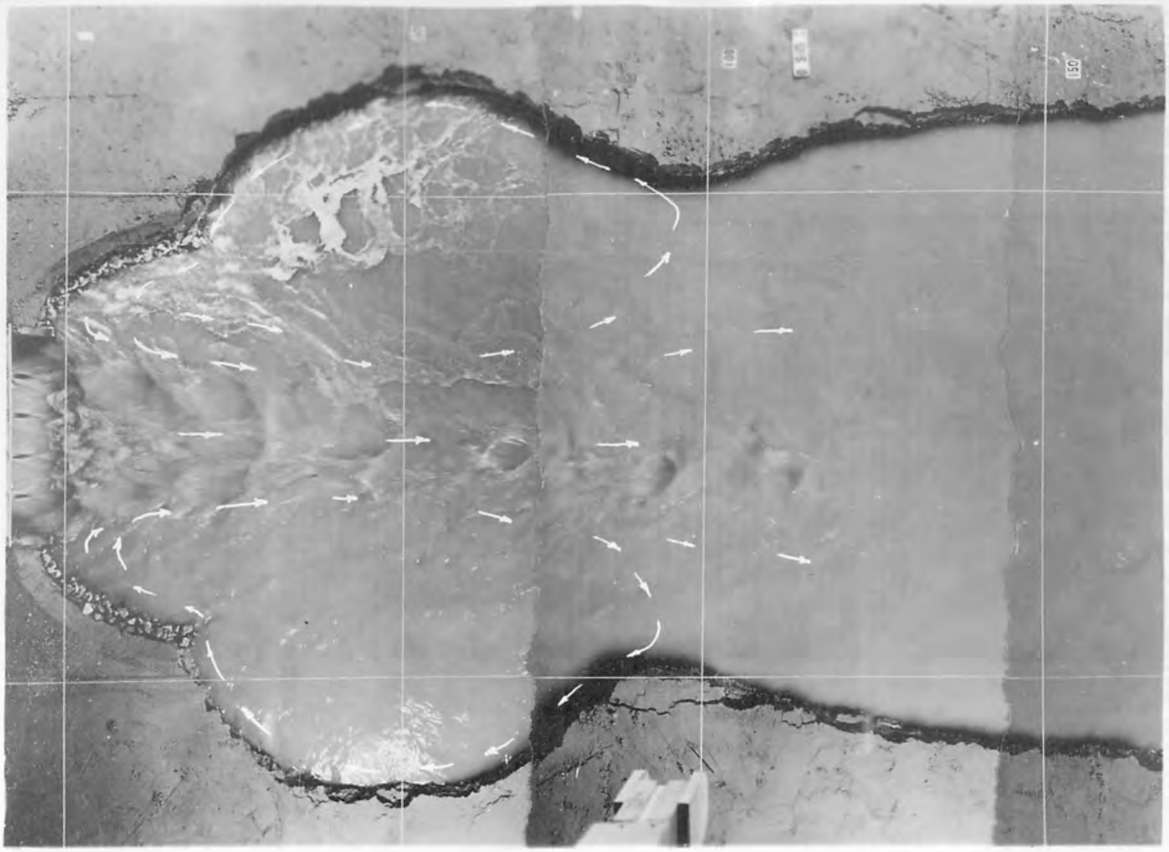
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

PROJECT: YAKIMA - WASHINGTON
FEATURE: REDESIGN OF CHECK DROPS
AGGREGATE: ....
SAMPLE: ....
DRAWN: H.G.D. CHECKED: ....

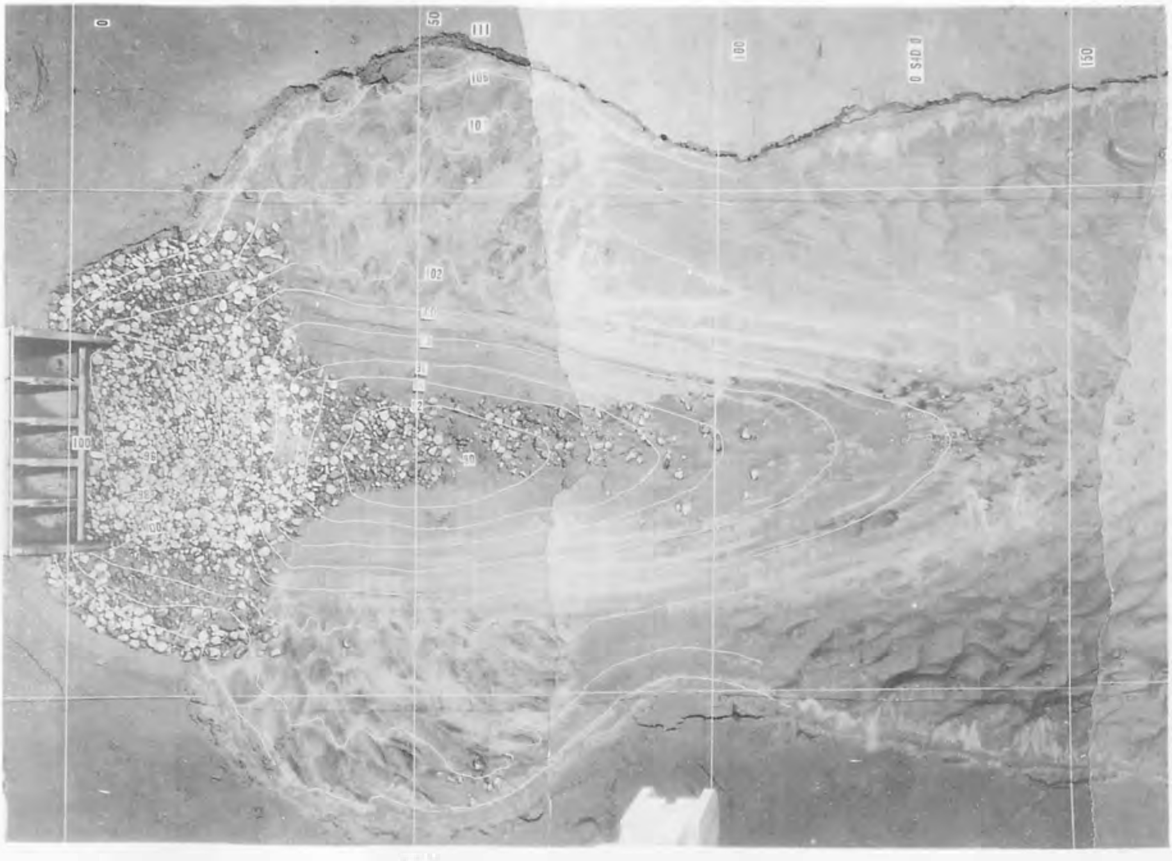
Date: 1-9-39 | At: DENVER, COLORADO | 33-D-1775

FIGURE 7

X-D-1694

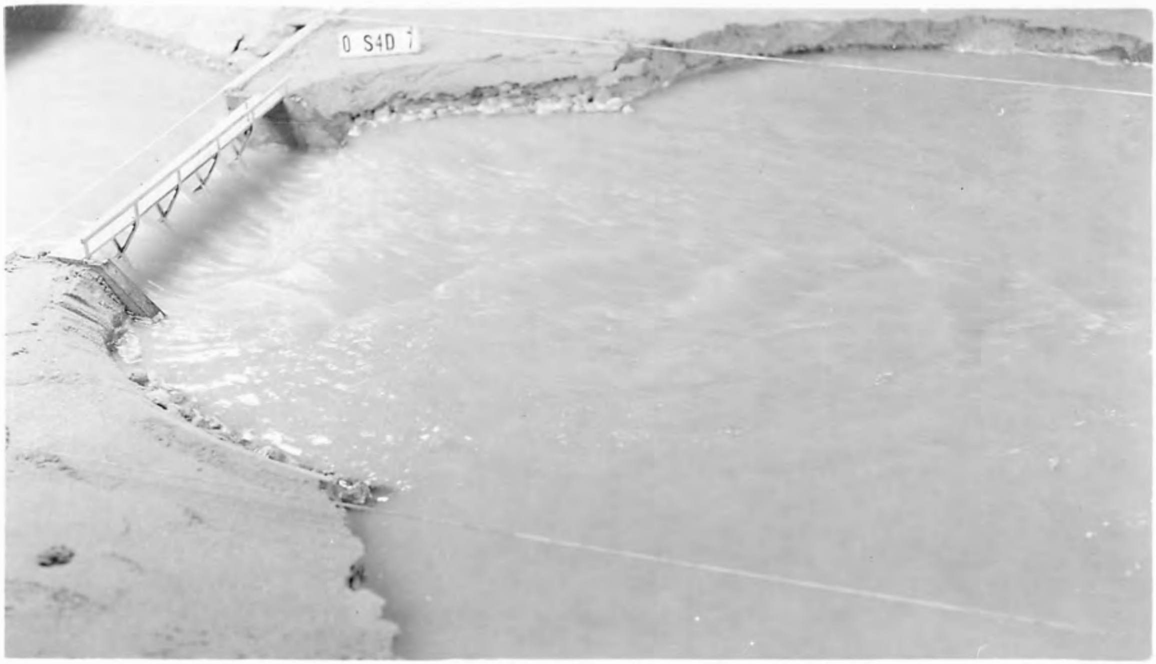


A. SCOUR AFTER 21 HOURS - DISCHARGE 1300 SECOND-FEET
UPSTREAM WATER SURFACE ELEVATION 886.25
DOWNSTREAM WATER SURFACE ELEVATION 884.55
FOUR FLASHBOARDS PER PANEL



B. SCOUR AFTER 26 HOURS
MODEL ELEVATION 100.0 EQUALS PROTOTYPE ELEVATION 877.55

EXISTING STRUCTURE



A. SCOUR AFTER 21 HOURS - DISCHARGE 1300 SECOND-FEET
UPSTREAM WATER SURFACE ELEVATION 886.25
DOWNSTREAM WATER SURFACE ELEVATION 884.55
FOUR FLASHBOARDS PER PANEL

B. SCOUR AFTER 26 HOURS
MODEL ELEVATION 100.0 EQUALS PROTOTYPE ELEVATION 877.55

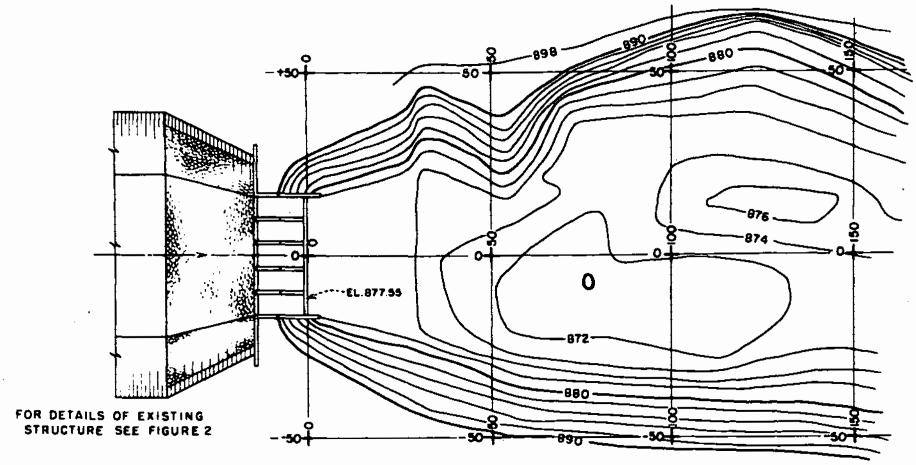
EXISTING STRUCTURE

the poor flow distribution extending downstream (figure 8A). The flow through the other panels was symmetrical, but it had a tendency to pile up on the upstream face of the steel brackets and remain split as it plunged into the check basin. This disturbance did not prevail after the flow had reached the region of the standing wave (figure 8A).

The standing wave, as discussed on page 8, formed over the check basin (figures 8A and 9A), and additional waves, accompanied by excessive velocities, extended downstream about 150 feet. The higher velocity flow was concentrated near the center of the canal, but, on each side of the canal, starting at a point about 75 feet below the drop, the direction of flow was upstream. This peculiar combination of flow downstream at the center of the canal and upstream along the $1\frac{1}{2}$ to 1 side slopes formed a large eddy along each side of the canal (figure 8A). The upstream component of these eddies had sufficient velocity to cut into the canal banks and produce excessive scour.

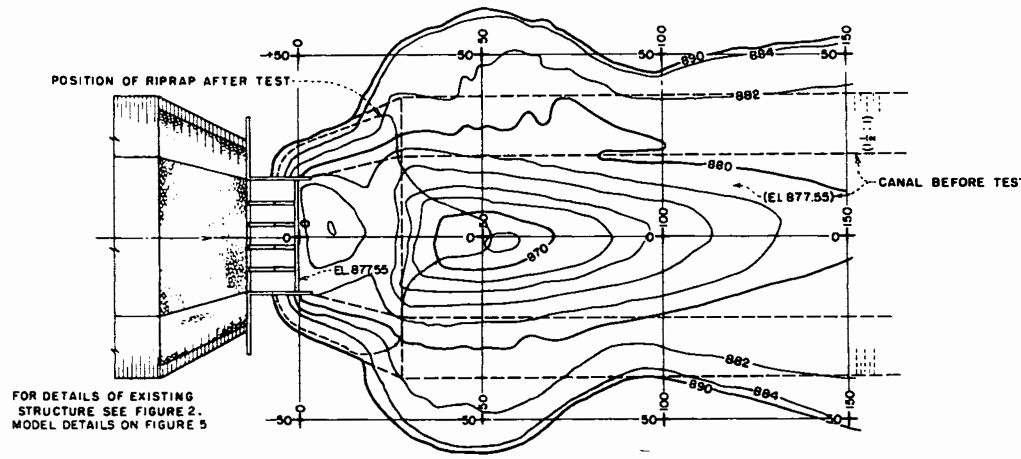
The scour in the model appears to be more severe than in the prototype, as shown on figures 1, 10, and 11. This was expected since the sand in the model was more saturated and less compacted than the material in the field. The scour to the riprapped canal bottom for a distance of 20 feet downstream from the drop was not excessive, but, beyond this point, the canal bottom was heavily scoured. Observations on the model revealed that, as the flow passed over the flashboards, it was partially deflected upwards by the weir wall at the lower end of the check basin. This had a tendency to reduce the amount of scour just below the drop. But, as the flow passed downstream to the end of the riprapped transition, high velocities prevailed along the bottom of the canal, which scoured a large hole (figure 9B) into which the riprap near this region was deposited. This may explain the reason why the riprap at the prototype structure is continually washed into the pool below the drop.

It will be noted that the scour pattern in the model (figure 8) is symmetrical about the center line of the canal whereas the scour at the prototype (figure 1) is on the left side of the canal. This difference in scour pattern is due principally to the fact that there was no curvature in the canal alinement in the model above or below the drop structure whereas the canal curves to the left upstream from the prototype drop structure and to the right downstream. Another factor contributing to the prototype scour pattern has been the method of placing the flashboards. These have been placed unsymmetrically so as to divert the flow from the scoured section of the canal. The model showed definitely, however, that scour will be excessive below



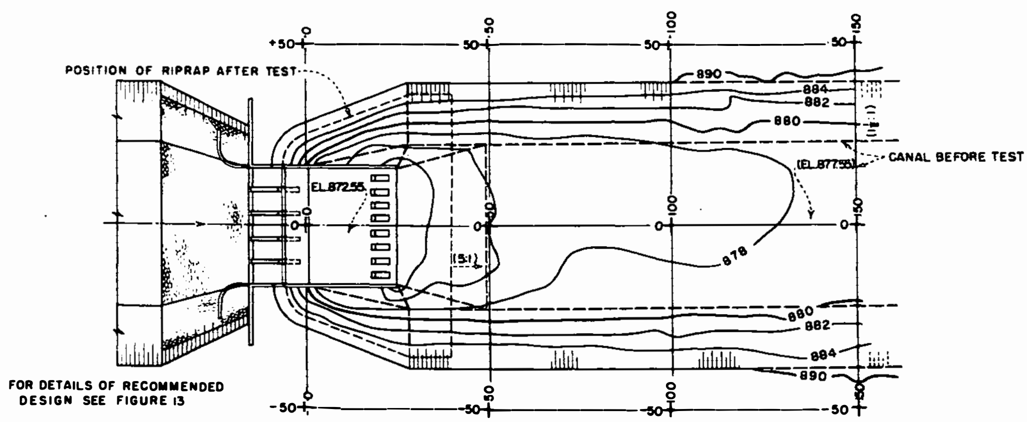
A

TOPOGRAPHY IN POOL BELOW DROP FROM FIELD SURVEY AT EXISTING STRUCTURE CHECK DROP 4



B

TOPOGRAPHY IN POOL BELOW DROP FROM TEST OF 1:15 SCALE MODEL OF EXISTING STRUCTURE CHECK DROP 4

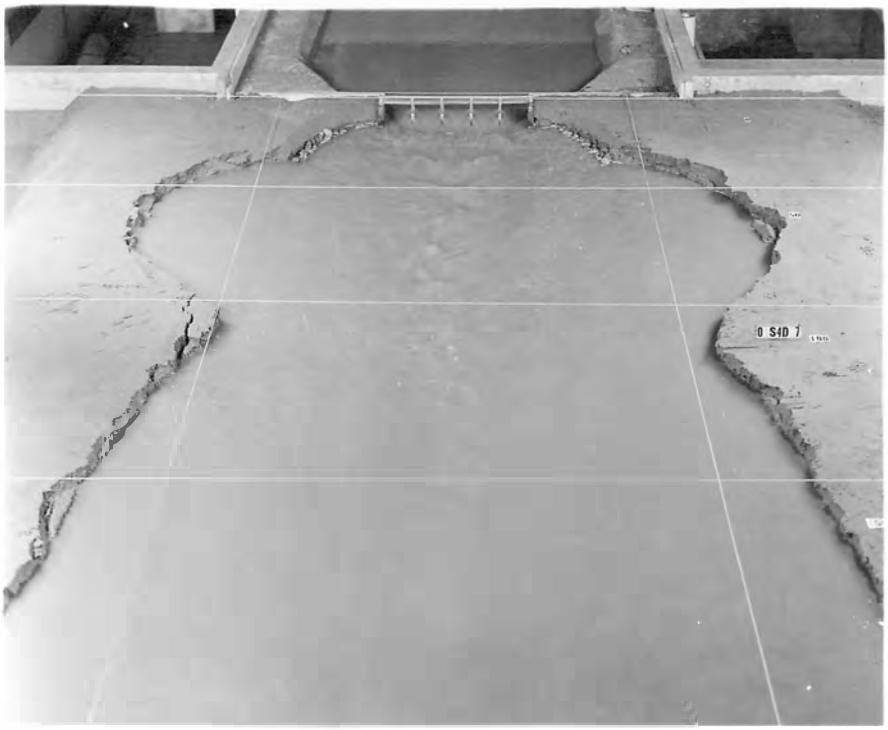


C

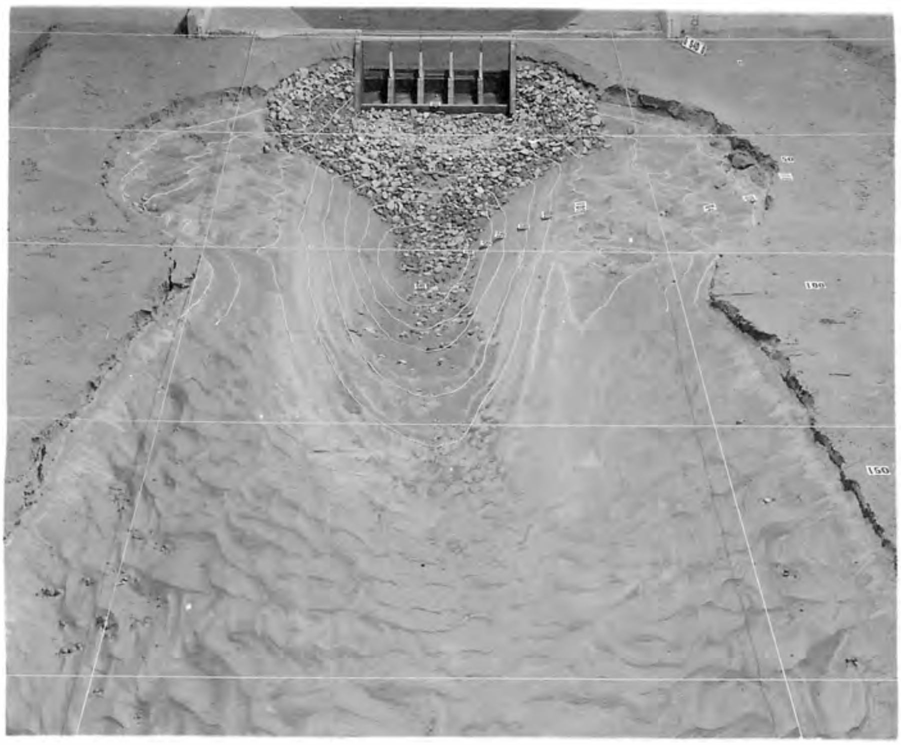
TOPOGRAPHY IN POOL BELOW DROP FROM TEST OF 1:15 SCALE MODEL OF RECOMMENDED REDESIGN OF EXISTING STRUCTURE CHECK DROP 4

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YAKIMA PROJECT – WASHINGTON
REDESIGN OF CHECK DROPS – SUNNYSIDE MAIN CANAL
HYDRAULIC MODEL STUDIES – 1:15 SCALE
COMPARISON OF SCOUR AT EXISTING STRUCTURE WITH SCOUR IN MODEL

DRAWN H. G. D. JR. SUBMITTED
TRACED B.B. – F.S. RECOMMENDED
CHECKED H. G. D. JR. APPROVED
DENVER, COLORADO – DEC. 6, 1938
33-D-1776



A. SCOUR AFTER 21 HOURS - DISCHARGE 1300 SECOND-FEET
UPSTREAM WATER SURFACE ELEVATION 886.25
DOWNSTREAM WATER SURFACE ELEVATION 884.55
FOUR FLASHBOARDS PER PANEL



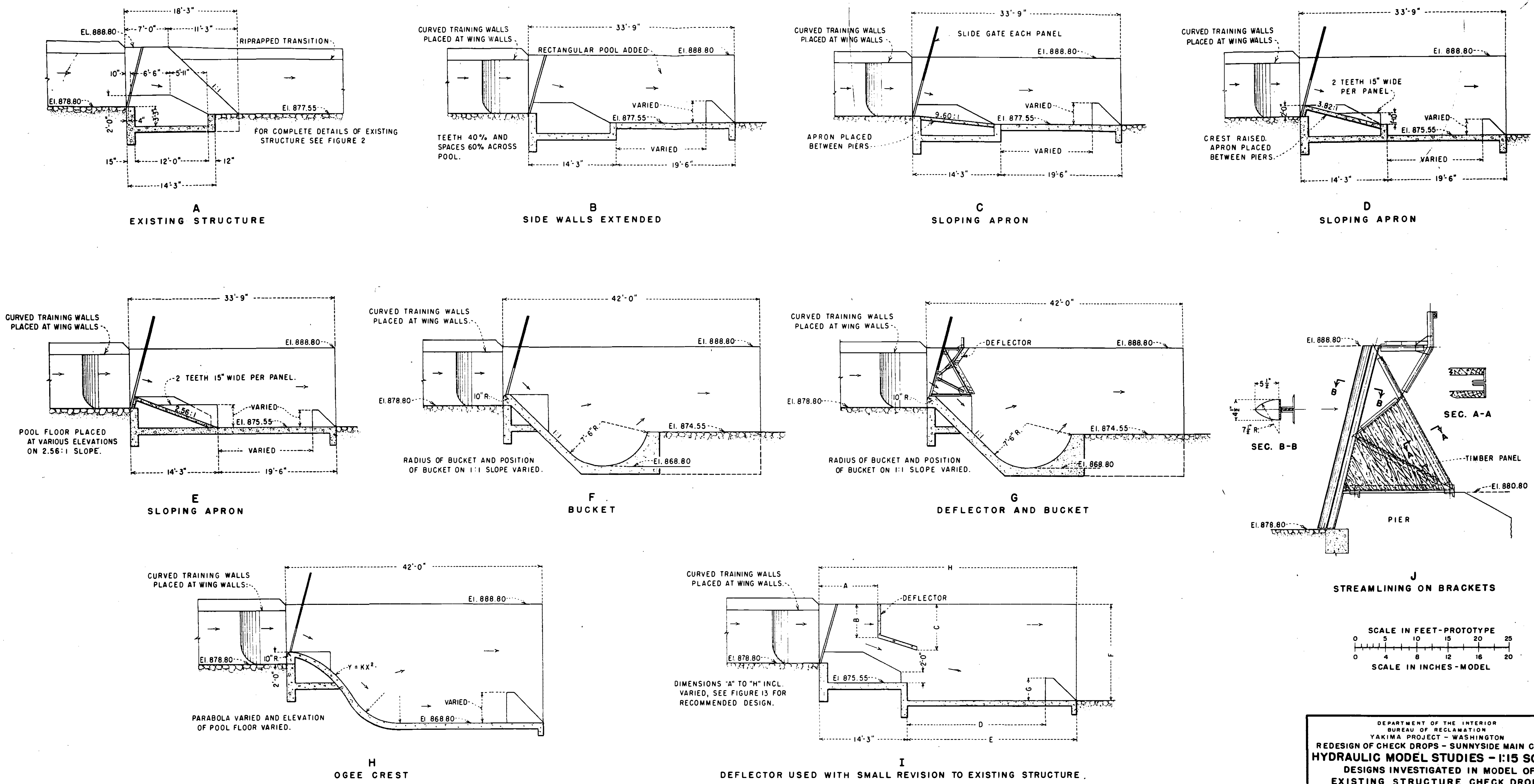
B. SCOUR AFTER 26 HOURS
MODEL ELEVATION 100.0 EQUALS PROTOTYPE ELEVATION 877.55

EXISTING STRUCTURE

the drop regardless of the alinement in the canal, the compactness of the material, or the symmetry of the flashboards.

13. Model tests leading to the recommended design. Before a satisfactory solution to this problem could be obtained, it was necessary to investigate many designs in the model and use the best features of each. Each design investigated was tested with various discharges, tail-water depths, and flashboard arrangements. Careful study was made on the performance of each design especially at maximum discharge since, at that discharge, the most unfavorable conditions existed. Before studies could be made on the elimination of the standing waves and excessive velocities, it was necessary to make the flow symmetrical at the approach and below the check basin. To improve the flow at the approach, the steel brackets were streamlined (figure 12J), and curved training walls were added to the wing walls (figure 12B). The flow was improved to such an extent that curved training walls were adapted to succeeding designs since they eliminated the draw-down or dished effect in the flow at each side panel. Although the streamlining to the brackets improved the flow, it was found later that the streamlining would not be required. To improve the flow below the check basin, a rectangular stilling pool was formed by extending the vertical side walls downstream (figure 12B). This prevented the return flow of the side eddies from submerging the main stream as it left the check basin.

With the symmetry of the flow greatly improved through the drop structure, it was possible to make a careful study of the flow to determine the procedure necessary for the elimination of standing waves and excessive velocities. Observations revealed that, as the jet plunged over the flashboards into the check basin, it did not closely follow along the stilling-pool floor but was deflected upwards by the weir wall at the downstream end of the check basin. This condition caused most of the higher velocity flow to be concentrated near the surface, which made it not only difficult to eliminate the standing waves but made teeth in the stilling pool (figure 12B) ineffective in dispersing the high velocity. If the jet could be made to follow the stilling-pool floor, it would be possible to greatly reduce the tendency for standing waves to form, and teeth would have some effect on the velocities. To accomplish this, the design shown on figure 12C was tried. In this design, a sloping apron was placed between each pier from the top of the upstream weir wall to the top of the downstream weir wall of the check basin. As a result, the entering jet had a tendency to follow the stilling-pool floor. This tendency was slightly increased when slide gates were used at the steel brackets instead of flashboards (figure 12C). With slide gates in place, the direction and the velocity of the jet



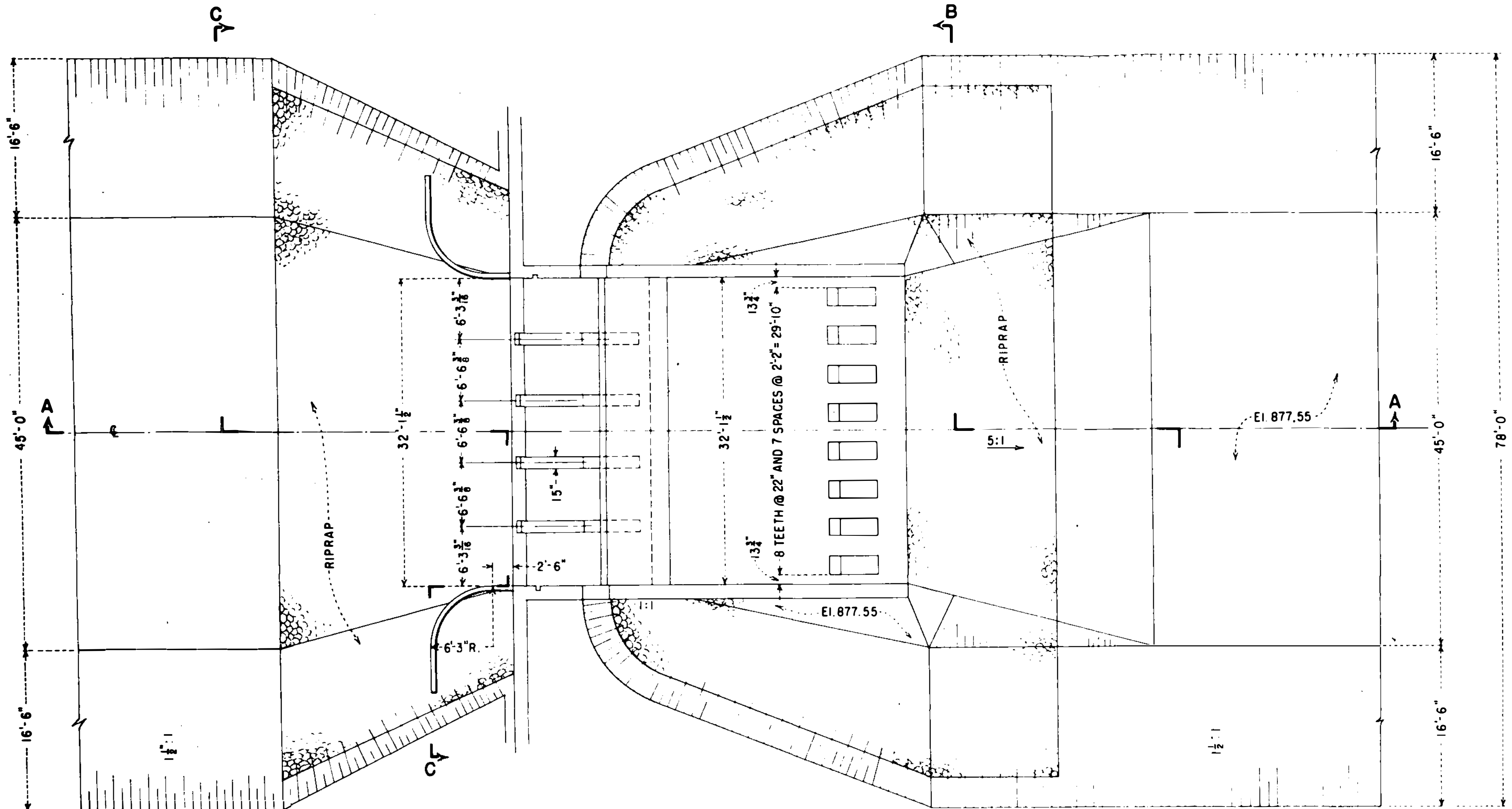
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YAKIMA PROJECT - WASHINGTON
REDESIGN OF CHECK DROPS - SUNNYSIDE MAIN CANAL
HYDRAULIC MODEL STUDIES - 1:15 SCALE
DESIGNS INVESTIGATED IN MODEL OF
EXISTING STRUCTURE CHECK DROP 4

DRAWN H. G. D. JR. SUBMITTED
TRACED F.A.J. - F.B. RECOMMENDED
CHECKED H. G. D. JR. APPROVED
DENVER, COLORADO - DEC. 15, 1938 33-D-1777

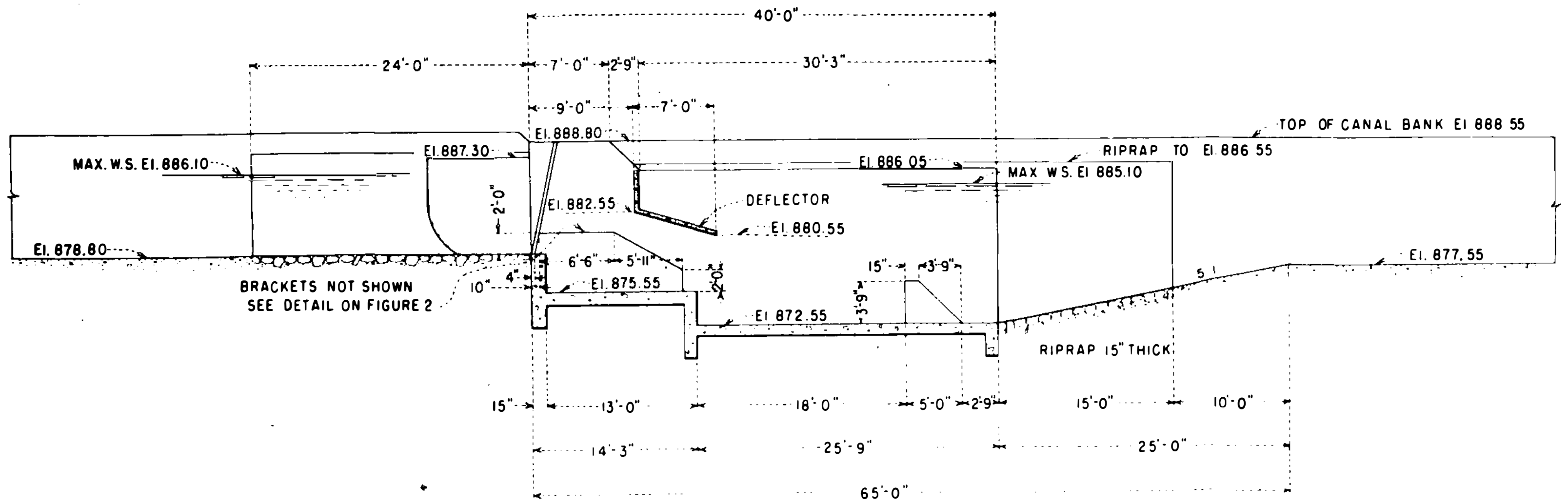
were more nearly parallel to the apron between the piers. The teeth in the stilling pool were now more effective, and it was noted that the velocities below the drop were slightly less. For small discharges, the standing waves were partially eliminated, but, for maximum discharge, it was necessary to raise the slide gates to such an extent that the jet no longer followed the apron into the stilling pool. As a result, the standing waves still formed. This design, however, was improved by providing more depth of stilling pool and increasing the slope of the apron (figures 12D and 12E). With these improvements, the tendency of the jet to follow the apron into the stilling pool was further increased. This rendered the teeth still more effective, and the velocities were further decreased, especially with the added depth of pool. The standing waves still persisted, however, as in the other designs.

When it was seen that the velocities below the drop decreased with an increase in pool depth and the tendency for the jet to follow a steeper apron was more favorable, the design shown on figure 12F was investigated. This provided still greater pool depth but added a bucket. The bucket has, in certain instances, been successfully used to dissipate energy by creating a large roller in the flow. If proper tail-water depth is furnished, the bucket may produce good results; otherwise, the roller will cause excessive boil or be drowned. It was hoped that the jet would follow the 1 to 1 slope into the bucket and result in the formation of a roller instead of the standing wave. This was not realized, however, because, when normal tail-water depth was maintained, which was necessary to prevent excessive boil, the jet failed to follow the slope into the bucket sufficiently to eliminate the standing waves. The velocities below the pool were still excessive. This design was improved in one respect by placing a deflector at the steel brackets (figure 12G). The deflector forced the jet to follow the slope into the bucket so that the standing wave was completely eliminated; however, the velocity of the entering jet was increased as it passed under the deflector, and this caused excessive boil in the pool in addition to excessive scour and velocities downstream from the pool. Although the use of a bucket did not warrant further study, additional study was made in an attempt to make the jet follow a slope into a stilling pool without using a deflector. The design shown on figure 12H provided an ogee crest, which more nearly fit the trajectory of the entering jet. The jet followed the ogee crest surprisingly well at maximum discharge and for all ranges of tail-water depth. But, if flashboards were added, the trajectory of the jet was changed, and the flow then oscillated between following the ogee crest and forming a standing wave. The use of slide gates also changed the trajectory of the

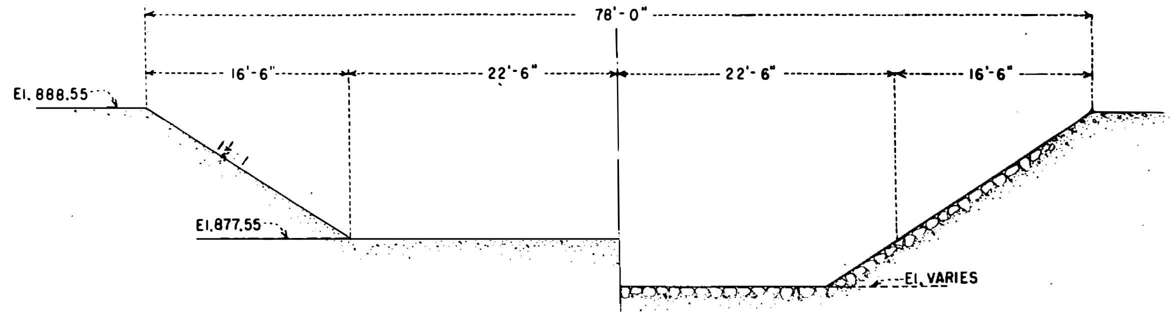
FIGURE 13



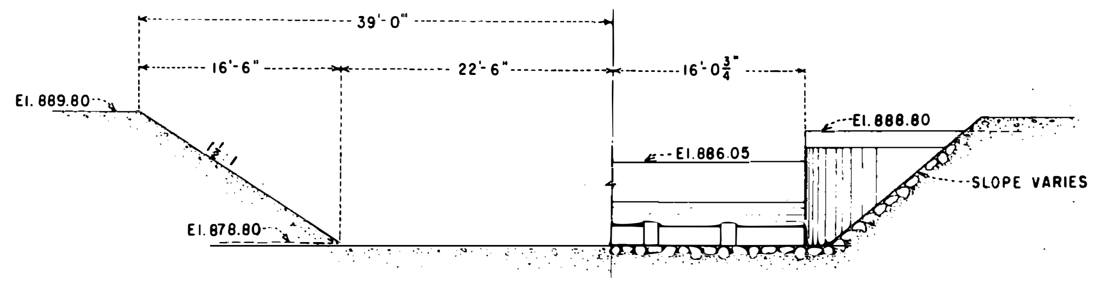
PLAN



SECTION A-A



SECTION B-B



SECTION C-C

SCALE IN FEET-PROTOTYPE

0 5 10 15 20 25

0 4 8 12 16 20

SCALE IN INCHES-MODEL

NOTES

1. All riprap shown is included in existing structure except the length of 15'-0" added below pool in recommended design.
2. No fill is required in scoured banks of canal below drop.
3. Riprap added to bottom of canal is to be placed on 5:1 slope or at El. 872.55.
4. See Figure 2 , for details of existing structure.

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YAKIMA PROJECT – WASHINGTON
REDESIGN OF CHECK DROPS – SUNNYSIDE MAIN CANAL
HYDRAULIC MODEL STUDIES – 1:15 SCALE
RECOMMENDED DESIGN OF CHECK DROP 4

DRAWN H.G.D.JR. SUBMITTED
TRACED F.A.J. – F.B. RECOMMENDED
CHECKED H.G.D.JR. APPROVED

DENVER, COLORADO – NOV. 26, 1938 33-D-1778

FIGURE 14

A. STILLING POOL

B. TRAINING WALLS

C. DISCHARGE 1300 SECOND-FEET

RECOMMENDED DESIGN

jet at maximum discharge and at lower discharges, and, even though slide gates or flashboards were added, the standing wave still formed. Velocity reduction was again impossible.

The best features of the designs just discussed were the deflector, which eliminated the standing waves, and the deeper stilling pool, which caused a reduction in the velocities below the drop. Additional investigation using these features was not warranted on the latter designs since the cost of changing the existing structure would be excessive. Further study was made, therefore, using the deflector and deeper stilling pool with a minimum change to the existing structure. Figure 12I shows this revision, which was developed to give a satisfactory recommended design.

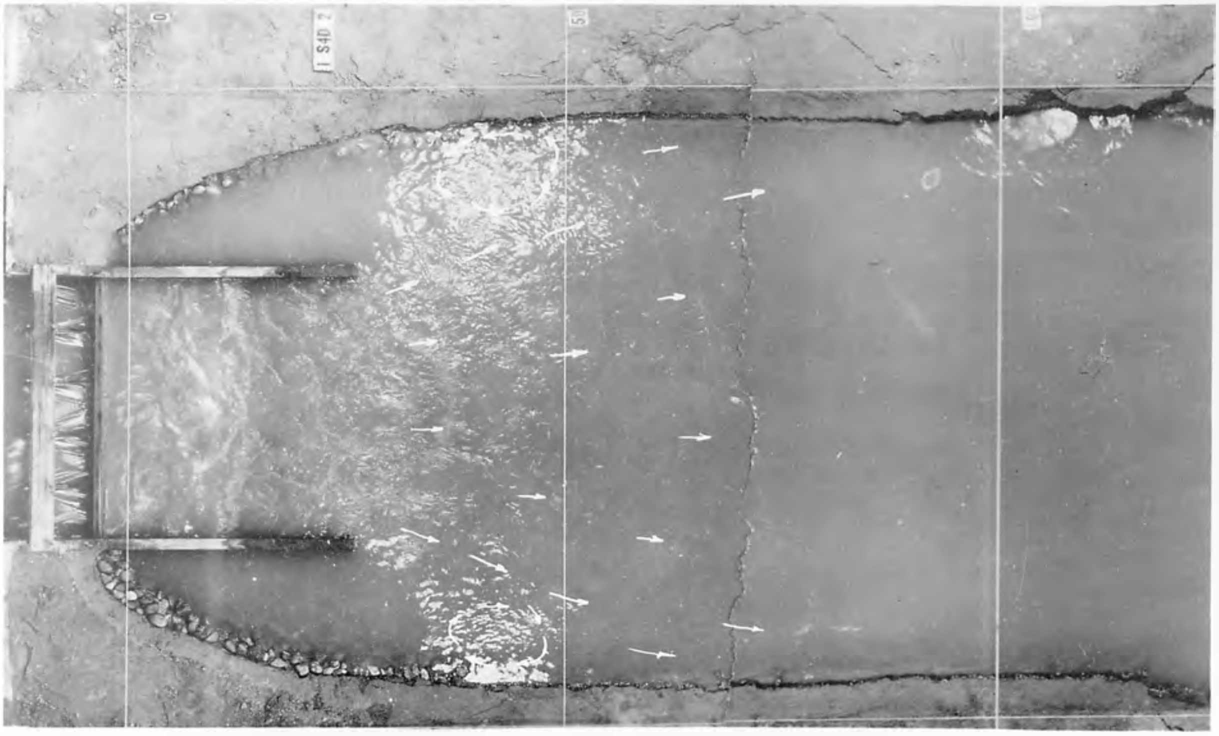
14. The recommended design structure. The recommended design (figures 13 and 14) requires the removal from the existing structure of the downstream weir wall of the check basin and part of the vertical side walls (figure 19). Curved training walls are added in the approach, and the vertical side walls are lengthened to form a stilling pool with a concrete floor on which eight baffle piers are placed. A deflector is added above the check basin, consisting of timbers supported by a structural-steel framework attached to the existing steel brackets (figures 19 and 20). Where it is possible in the field, the canal section at the stilling pool may be built in as shown on figure 13. In any event, it is necessary to place riprap 15 feet below the pool on the canal banks and along the bottom, either on a 5 to 1 slope or at elevation 872.55. If backfill is not placed along the stilling-pool walls, additional riprap beyond that existing will not be required in the transition. No backfill or riprap is required in the heavily scoured banks of the canal beyond the stilling pool.

15. Flow through the recommended design. By placing curved training walls in the approach and adapting a deflector and stilling pool at the check basin, the recommended design has eliminated the following unfavorable conditions that exist at the prototype: (1) Unsymmetrical flow distribution through the drop structure; (2) standing waves and excessive velocities below the drop; (3) eddies along the side of the canal immediately below the drop; and (4) scour to the canal.

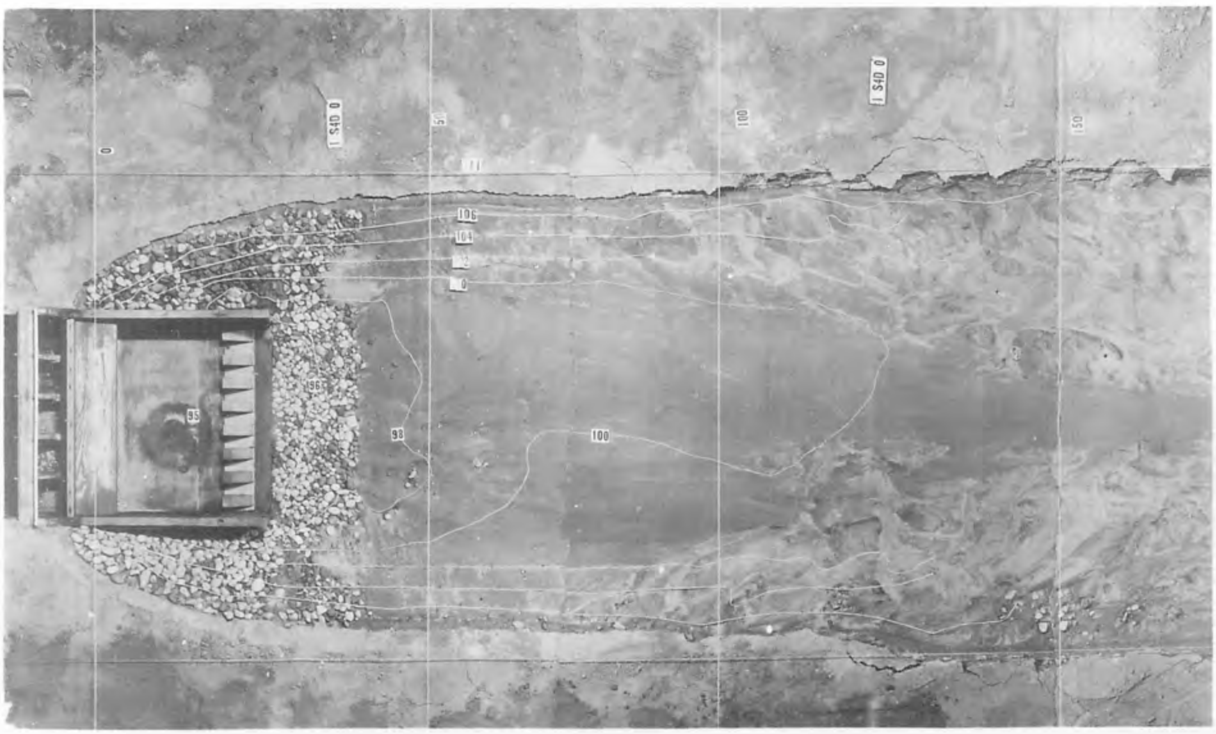
The addition of curved training walls in the approach results in a uniform depth of flow at the entrance to the drop and below the check basin (figures 14B, 14C, and 18). Present design practice usually adopts a warped transition where the section of a canal changes from trapezoidal to rectangular. This automatically provides uniform depth of flow at the rectangular section. The curved training walls were adopted, however, to keep the cost of revision to a minimum. Streamlining of the brackets is not necessary since the forward velocity at this section is reduced by the deflector. As the flow passes over the flashboards, instead of a standing wave forming, the jet shoots under the deflector and follows the stilling-pool floor. The deflector is so placed that the maximum discharge can be passed at normal tail-water depths without any splashing occurring over the deflector or excessive depths forming upstream. Additional discharge may be passed in the event of a cloudburst or similar cause. If 1,264 second-feet are flowing and the tail water is maintained at elevation 884.90, the discharge may be increased to 1,525 second-feet before the flow spills over the deflector. The water surface upstream would then be at elevation 886.56. If the discharge is 1,264 second-feet and the tail-water depth is allowed to increase with the discharge, the structure will pass 1,490 second-feet before the flow tops the deflector. The water surface upstream would then be at elevation 886.60, and the tail water would be at elevation 885.13. The flashboards in place for these two conditions are 3-2-2-1-2, right to left looking downstream.

As the flow passes under the deflector and along the stilling-pool floor, it is diffused in the tail water, and its velocities are greatly reduced by the baffle piers. It is difficult to classify the type of flow existing in the stilling pool, but, from the analysis previously made, it is evident that a hydraulic jump cannot form. A roller, however, is created in the pool by the action of the baffle piers so that the flow could be called a submerged roller. The reduction in energy head and velocity depends on the depth of pool and the baffle piers. The effectiveness of this combination to give smooth flow conditions under the existing circumstances only is seen on figures 15A, 16A, and 17A. In the scour test discussed below, it was observed that, although the tendency to scour was not increased when the flashboards were placed unsymmetrically, the stilling pool operated more efficiently and better velocity distribution could be obtained when the flashboards were symmetrical. The prototype structure has been operated with such flashboard arrangements as 6-6-4-2-2 to divert the flow away from the scoured section of the canal. The function of the stilling pool in the recommended design is to produce uniform flow distribution below the drop, but it is necessary to avoid placing the flashboards unsymmetrically if the efficiency of the pool is to be maintained.

A scour test was made on this design similar to the one made on the model of the existing structure. At the end of 26 hours, the scour in the model was slight (figures 10C, 15B,



A. SCOUR AFTER 21 HOURS - DISCHARGE 1300 SECOND-FEET
UPSTREAM WATER SURFACE ELEVATION 886.35
DOWNSTREAM WATER SURFACE ELEVATION 884.55
FOUR FLASHBOARDS PER PANEL



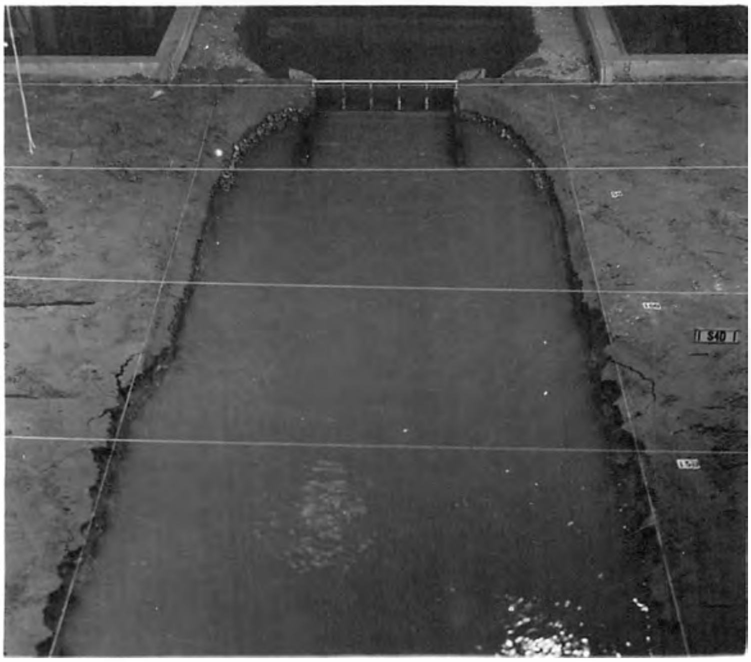
B. SCOUR AFTER 26 HOURS
MODEL ELEVATION 100.0 EQUALS PROTOTYPE ELEVATION 877.55

RECOMMENDED DESIGN

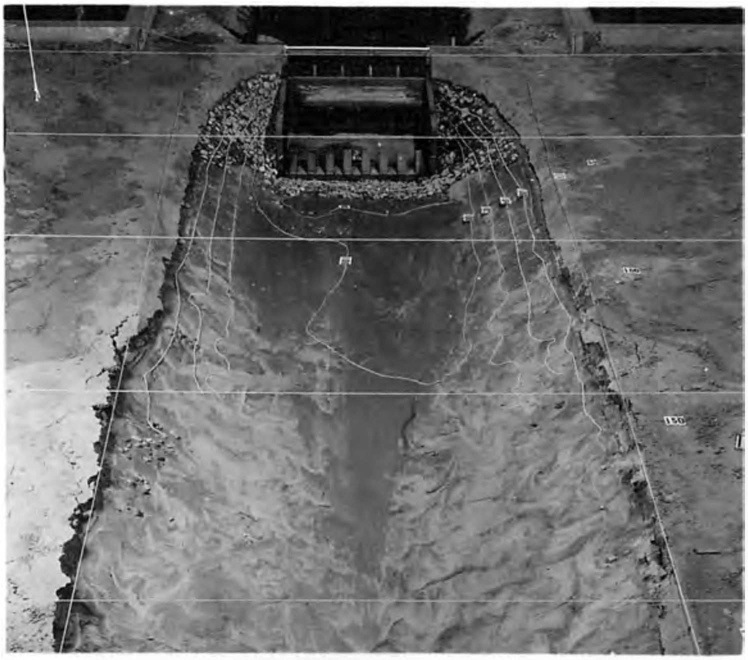
A. DISCHARGE 1264 SECOND-FEET
UPSTREAM WATER SURFACE ELEVATION 885.90
DOWNSTREAM WATER SURFACE ELEVATION 884.90
FLASHBOARDS 3-2-2-1-2 RIGHT TO LEFT LOOKING DOWNSTREAM

B. SCOUR AFTER 26 HOURS
MODEL ELEVATION 100.0 EQUALS PROTOTYPE ELEVATION 877.55

RECOMMENDED DESIGN

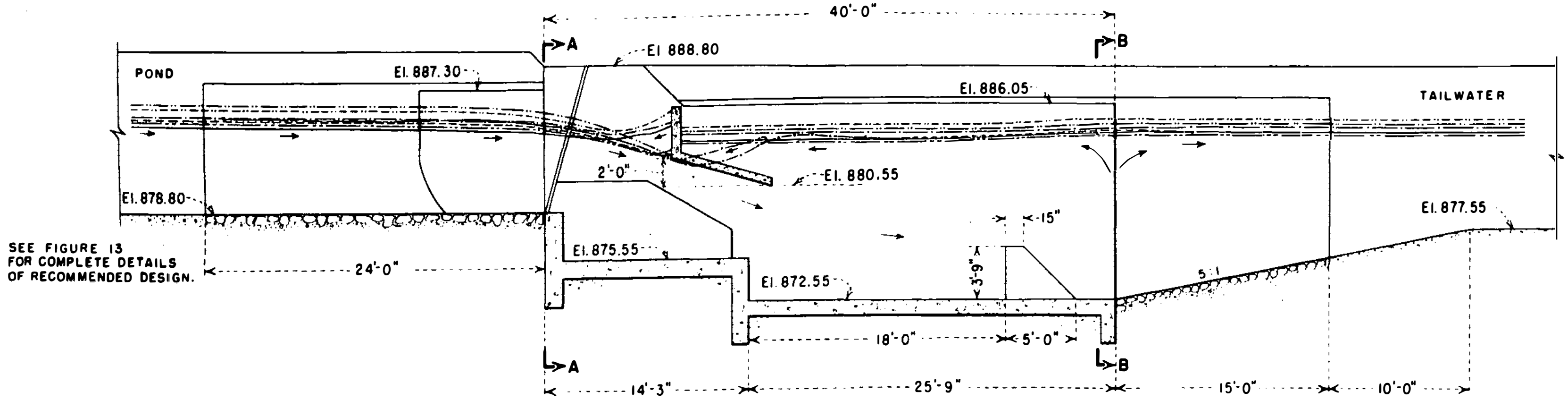


A. SCOUR AFTER 21 HOURS - DISCHARGE 1300 SECOND-FEET
UPSTREAM WATER SURFACE ELEVATION 885.22
DOWNSTREAM WATER SURFACE ELEVATION 883.95
FLASHBOARDS 1-2-1-2-1



B. SCOUR AFTER 26 HOURS
MODEL ELEVATION 100.0 EQUALS PROTOTYPE ELEVATION 877.55

RECOMMENDED DESIGN



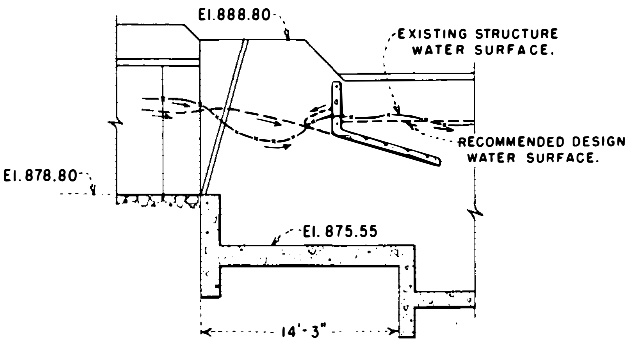
B

WATER SURFACE PROFILES ON CENTER LINE
CHECK DROP 4

EXPLANATION

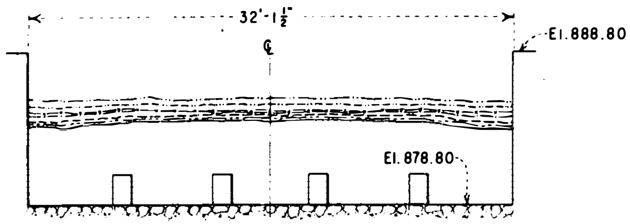
CONVENTION	Q C.F.S.	TAILWATER ELEV.	POND ELEV.	FLASHBOARDS
— - - — *	1264	884.90	885.99	3-2-2-1-2
—— · ——	1300	884.55	886.29	4-4-4-4-4
— · — · —	1300	884.30	885.42	1-2-1-2-1
— — — —	1300	883.95	885.19	1-2-1-2-1
————	1300	883.95	884.84	NONE
- · - · -	1300	883.80	885.06	1-1-1-1-1
x—x—x	1300^	883.95	885.36	1-2-1-2-1

^ PROFILE THROUGH MODEL OF EXISTING STRUCTURE. ALL OTHERS THROUGH MODEL OF RECOMMENDED DESIGN.
* THIS PROFILE ESTIMATED

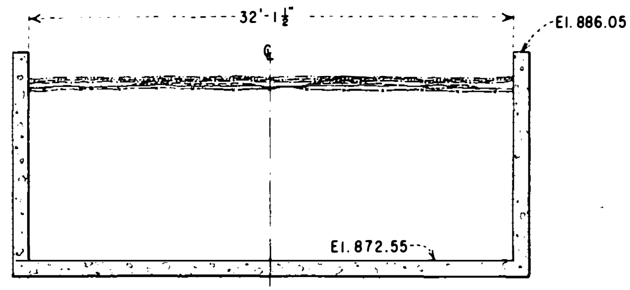


A

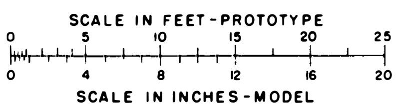
WATER SURFACE PROFILES AT SIDE WALL



SECTION A-A



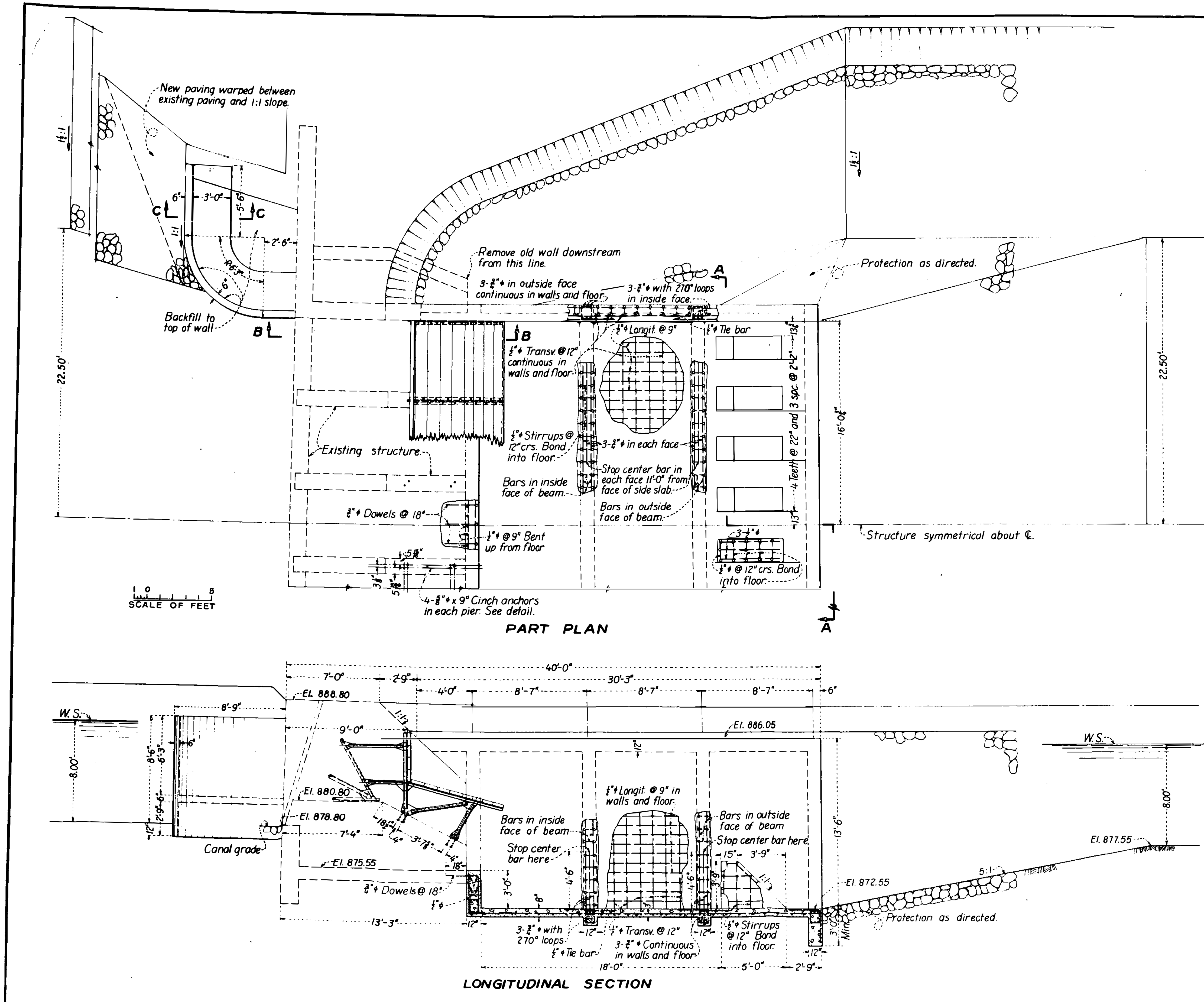
SECTION B-B

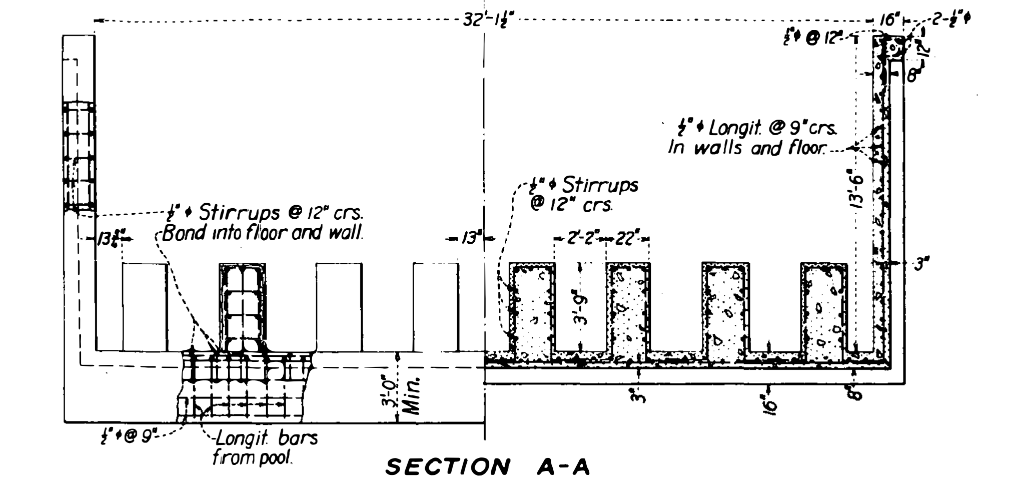


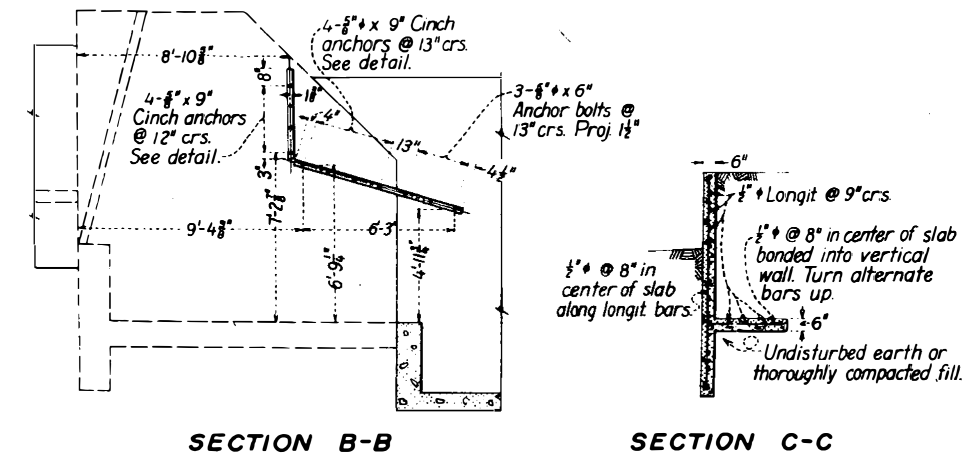
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YAKIMA PROJECT – WASHINGTON
REDESIGN OF CHECK DROPS – SUNNYSIDE MAIN CANAL
HYDRAULIC MODEL STUDIES – 1:15 SCALE
WATER SURFACES THROUGH MODEL OF RECOMMENDED DESIGN

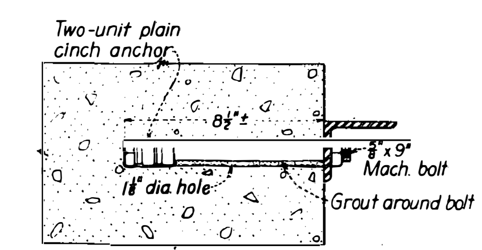
DRAWN	H.G.D. JR.	SUBMITTED	
TRACED	R.W.B.–F.B.	RECOMMENDED	
CHECKED	H.G.D. JR.	APPROVED	
	DENVER, COLORADO – DEC. 7, 1938		33-D-1779

FIGURE 18









DETAIL OF CINCH ANCHOR

SCALE OF FEET

ESTIMATED QUANTITIES

SHEETS 1 AND 2

Concrete	67 Cu.Yds.
Reinforcement Steel	6000 Lbs.
Structural Steel	1694 Lbs.
Timber	1122 F.B.M.

NOTES

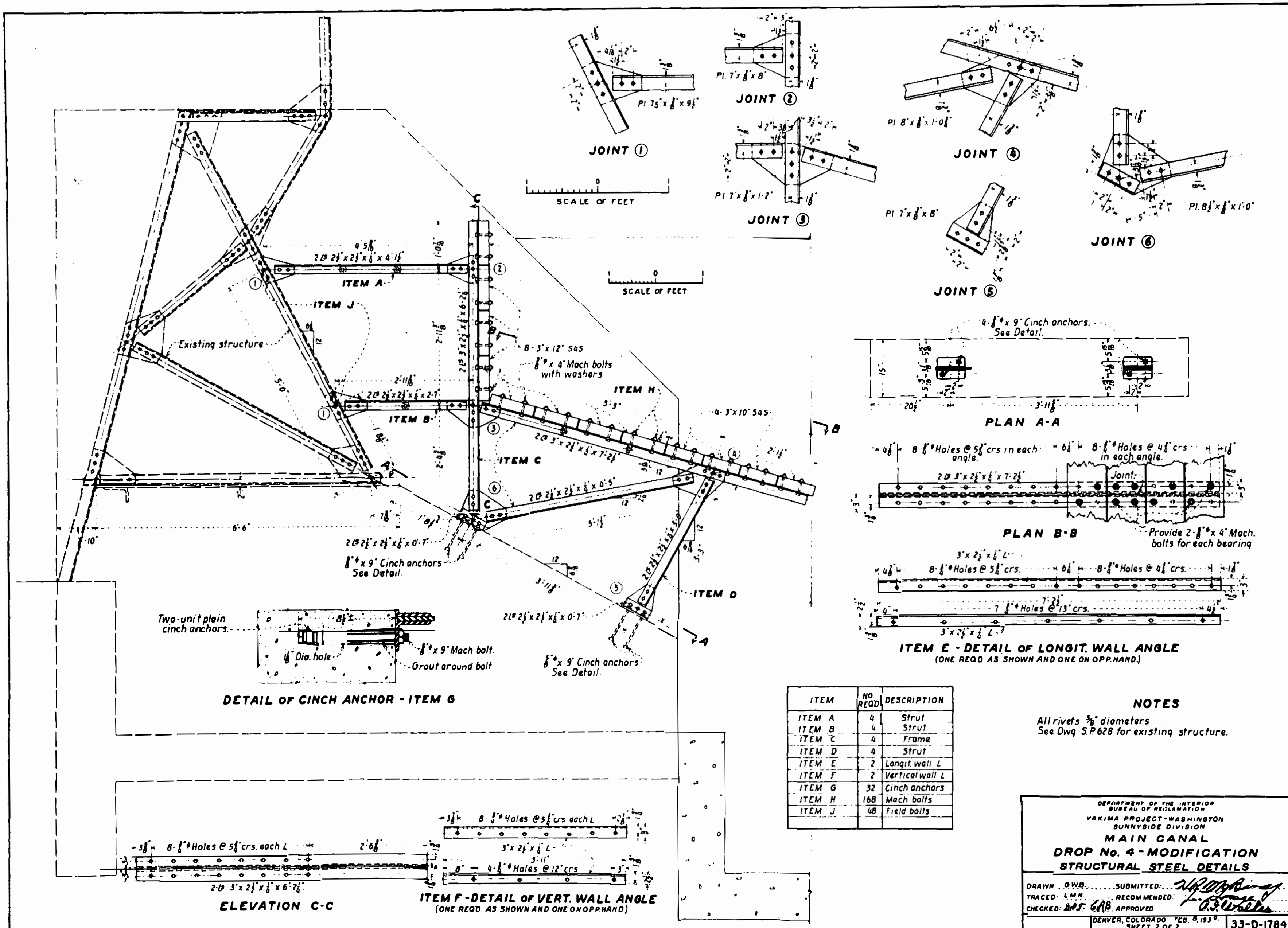
All reinforcement shall be placed so that the centers of bars in the outer layer will be 2" from face of concrete, unless otherwise shown.
Lap all bars 40 diameters at splices. Stagger splices.
Loops with 270° bends and radii of 4 bar diameters to be provided where shown.
See Dwg. S. P. 628 for existing structure.

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YAKIMA PROJECT-WASHINGTON
SUNNYSIDE DIVISION

MAIN CANAL
DROP No. 4 - MODIFICATION
CONCRETE DETAILS

DRAWN: G.W.B. SUBMITTED: [signature]
TRACED: W.T.T. RECOMMENDED:
CHECKED: R.A.T. C.R.B. APPROVED:

DENVER, COLORADO, DEC. 27, 1938 SHEET 1 OF 2 | 33-D-1783



ITEM	NO. REQ'D	DESCRIPTION
ITEM A	4	Strut
ITEM B	4	Strut
ITEM C	4	Frame
ITEM D	4	Strut
ITEM E	2	Longit. wall L
ITEM F	2	Vertical wall L
ITEM G	32	Cinch anchors
ITEM H	168	Mach bolts
ITEM J	48	Field bolts

NOTES

All rivets ⅝" diameters
See Dwg S.P. 628 for existing structure.

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YAKIMA PROJECT-WASHINGTON
SUNNYSIDE DIVISION

MAIN CANAL
DROP No. 4 - MODIFICATION
STRUCTURAL STEEL DETAILS

DRAWN: G.W.B. SUBMITTED:
TRACED: L.M.N. RECOMMENDED:
CHECKED: APPROVED:
DENVER, COLORADO FEB. 8, 1939 SHEET 2 OF 2
33-D-1784

FIGURE 20

16B, and 17B). The scour in the riprapped section just downstream from the stilling pool was slight (figure 16B). This demonstrates the ability of the teeth in the stilling pool to reduce the velocity along the bottom of the canal. The scour at the side slopes, starting about 125 feet below the drop, was due to a sloughing of the material at the start of the test and not due to excessive velocities. Before the canal was rebuilt for this test, observations were made to see what effect the flow from the recommended design structure would have upon the scoured canal section actually existing in the field. It was observed that, although eddies formed in the wide section of the scoured canal, their velocity was reduced to such an extent that cutting into the banks was stopped. A tendency to scour could not be observed at any point. At the existing structure, the velocities are greater near the center of the canal than at the sides. This condition, it was seen, caused side eddies to form. Below the recommended design pool, however, the velocity distribution is uniform. Under this condition, extensive side eddies do not form, and the tendency to scour is removed. Small eddies do form, however, just at the end of the riprapped section, but they extend only about 15 feet downstream, and their velocity is not sufficient to scour the canal banks (figure 15A). This condition could have been eliminated if it had been possible to use a warped transition from the rectangular stilling pool to the trapezoidal canal section. The cost of using a warp would be excessive, however, since it would first be necessary to rebuild the scoured canal section.

16. Conclusions. The design developed from this model study is recommended for application to the standard drop structure of the Sunnyside Main canal. The recommended design was developed from certain conditions of flow existing in the canal. Care should be exercised in applying this design to other structures where the flow conditions and other factors are not the same.

This model study reveals the need for better designs of small drop structures. In future problems of this type, the following recommendations may lead to a better design:

(a) When a section of a canal or channel changes from trapezoidal to rectangular or vice versa, it is desirable to use a warped transition and one in which the change in cross section is not too abrupt. If this is not possible where a section changes from trapezoidal to rectangular, add curved training walls where the flow enters the rectangular section.

(b) Hydraulic structures should be designed and operated so that the flow passing through them will be symmetrical;

otherwise an unbalanced condition forms, particularly in the flow below the structure. This results in the formation of eddies and creates nonuniform velocity distribution, the combination of which may cause abnormal scour downstream from the structure.

(c) Avoid the use of forming a hydraulic jump or similar energy dissipator in expanding or trapezoidal sections. A rectangular stilling pool should be used.

(d) When a series of small drops is planned, an attempt should be made to eliminate certain ones or increase the reach between. If this can be done, the head on any particular structure can be increased so that a hydraulic jump can be adapted at the drop.

- - -

ABSTRACT OF CORRESPONDENCE

7-16-38 From Superintendent, Yakima project, to Chief Engineer, regarding proposed reconstruction of check drops on Sunnyside Main canal.

7-25-38 Memorandum from H. R. McBirney to Chief Designing Engineer, recommending a limited program of model testing.

8-8-38 From Chief Engineer to Superintendent, Yakima project, stating that the redesign of check drops has been assigned to the hydraulic laboratory. Status of model design and outline of model tests given.

9-28-38 From Chief Engineer to Superintendent, Yakima project, stating that model tests have started and requesting prototype flow and scour data at check drop no. 4.

10-4-38 From Superintendent, Yakima project, to Chief Engineer, stating that prototype flow data at check drop no. 4 will be taken during the remainder of the irrigation season. Scour data will be taken at end of irrigation season.

10-12-38 From Acting Chief Engineer to Superintendent, Yakima project. Progress report of model studies and request for available prototype flow data. Plan to remove alternate drops submitted.

10-19-38 From Acting Superintendent, Yakima project, to Chief Engineer, stating objections to plan for removal of alternate drops. Available prototype flow data submitted.

11-22-38 From Acting Superintendent, Yakima project, to Chief Engineer. Prototype flow data and cross sections of canal at check drop no. 4 submitted. Error in data in letter of 10-19-38 noted.

12-21-38 From Chief Engineer to Superintendent, Yakima project, stating that the model studies have been completed and the recommended design is being detailed. Request made for prints of photographs at check drops no. 4 and no. 8.

1-4-39 From Acting Superintendent, Yakima project, to Chief Engineer. Photographs of check drops no. 4 and no. 8 submitted.