Hyd - 54

See also Hyd 50 & 56.2

Denver, Colorado, April 17, 1939.

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

Subject: Thesis submitted to universities for degrees in engineering, by Bureau employees - Denver office.

1. In compliance with office memorandum No. 186, dated March 2, 1939, the final draft of a thesis entitled, "An Analytical and Experimental Study of Energy Dissipation and Scour Prevention at Small Check Drop Structures" is hereby submitted for release.

2. This thesis will be presented to the University of Colorado, Boulder, Colorado, as partial fulfillment of the requirements for the degree Master of Science in Civil Engineering.

3. The material for this thesis was obtained by the author in the hydraulic laboratory from a model study of check drop No. 4 of the Sunnyside Main Canal, Yakima Project, Washington. A report of this study was submitted to the chief designing engineer February 28, 1939, and it is tentatively planned to present this study in the form of a technical memorandum at a later date.

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OK

4/19/39

A well-prepared and very interesting thesis.

AN ANALYTICAL AND EXPERIMENTAL STUDY OF ENERGY DISSIPATION AND SCOUR PREVENTION AT SMALL CHECK DROP STRUCTURES

by

Haywood Guion Dewey, Jr.

C.E., Cornell University, 1935

A Thesis submitted to the Faculty of the Graduate School of the University of Colorado in partial fulfillment of the requirements for the Degree

Master of Science

Department of Civil Engineering

1939

This Thesis for the M.S. degree, by

Haywood Guion Dewey, Jr.

not proof-read, has been approved for the

Department of

Civil Engineering

by

Roderick L. Downing

C. L. Eckel

Date May 6 1939

ACKNOWLEDGEMENT

The author is indebted to the Bureau of Reclamation, United States Department of the Interior, Denver, Colorado, for permission to use the data contained in this paper, and for furnishing the illustrative material. The suggestions given by Mr. J. E. Warnock, director of the hydraulic laboratory in which the author conducted this study, are greatly appreciated. To Opal Gibbs Dewey, the author is grateful for her aid in preparing the manuscript.

Haywood Guion Dewey, Jr.

University of Colorado
April, 1939

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Introduction

The hydraulic engineer is frequently required to design open channel structures for controlling the flow of water from a higher to a lower level. In the design of flood control or power development structures, it is essential to include spillways for bypassing excess flow. In irrigation systems, many drop structures are required at changes of grade in a canal and at turnouts into the laterals. The total drop in water surface at these structures may be only a few feet or several hundred feet, but in any event, it is necessary to provide some means at the lower level for dissipating a large part of the energy of flow, and for reducing the velocity before the flow proceeds along its natural course.

It has been only recently recognized that the hydraulic jump is one of the best means of obtaining effective energy dissipation and velocity reduction. Accordingly, concrete stilling pools are placed at the lower end of spillways, overfall dams, or canal drop structures, enabling the dissipation of energy and velocity reduction to occur in a hydraulic jump confined within a stilling pool.

If this is not done, severe scour may occur where the accelerated flow enters the less rapidly moving flow of a river or canal. In the case of overfall dams, the toe of the structure may be gradually undermined by the scouring action of the rapid flow upon the channel just downstream from the dam, or the lower end of a spillway may be similarly affected.

In earth canals, scouring may be excessive below the lower end of a drop structure, even though the drop is small, since the material through which a canal flows is usually readily scoured.

Unfortunately, it is not an easy problem to design a stilling pool or hydraulic jump for any given drop and discharge. In any design, from one of a high drop to one of a low drop structure, there are certain hydraulic problems pertaining to a stilling pool which are characteristic of the amount of drop.

From the author's experience in studying these problems by models in a hydraulic laboratory, it has been recognized that drop structures may be divided into three distinct groups:

Group I: Those structures having a drop of several hundred feet to 20 feet.

Group II: Those structures having a drop of 20 feet to 4 feet.

Group III: Those structures having a drop of 4 feet or less.

This is purely an arbitrary grouping, but it permits a more definite study to be made of the characteristic hydraulic problems pertaining to the design of stilling pools at drop structures. To acquaint the reader with some of these problems, mention will be made of them in Chapter I before proceeding with the main discussion.

The main discussion of this paper is limited to a problem characteristic of one type of small drop structure included under Group III; namely, that of obtaining effective energy dissipation and velocity reduction in flow which is too near critical depth, due to submergence, to permit the use of a hydraulic jump. To illustrate this problem, a discussion will be made of the analytical and experimental study of the existing check drop structures in the Sunnyside Main Canal, Washington. This study was made by the author in the hydraulic laboratory of the Bureau of Reclamation, United States Department of the Interior, Denver, Colorado.

Numerous references may be found in hydraulic literature to small drop structures at which there is no submergence, enabling effective energy dissipation and velocity reduction to occur within a hydraulic jump and stilling pool. Unfortunately, little or nothing has been written on the problem of submergence at small drop structures. Accordingly, it is hoped as a result of the analytical and experimental study discussed in this paper, that the problem and the effect of submergence at small check drop structures will be recognized as one of more importance than heretofore, particularly in canals which flow through material easily scoured.

Chapter I

Hydraulic Problems in Design of Stilling Pools of Drop Structures

Before proceeding with the main discussion, mention is made in this chapter of some of the characteristic hydraulic problems pertaining to stilling pool designs of drop structures included under each of the groups mentioned in the Introduction. It is believed that this discussion is a more direct method of presenting these problems. Instead of assuming a broad application, future papers treating these problems and their solutions could limit their discussion to a particular group or groups. This should provide a more comprehensive record of hydraulic problems pertaining to stilling pool designs of drop structures.

Group I

This group includes the higher drop structures required in the design of flood control or power development structures. A spillway in this group, for example, is designed to carry larger discharges than drop structures in Group II or Group III. This requires considerably more energy and higher velocities to be reduced in a stilling pool.

The most recent problem concerning stilling pool design of drop structures in this group is the effect of air entrainment in water as it flows down a spillway or face of a dam. Observations[1] of such flow reveal a definite demarcation

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[1] Lane, E.W., Entrainment of Air in Swiftly Flowing Water. Civil Engineering, Vol. 9, No. 2., P. 89, February, 1939

between water containing very little entrained air and water containing considerable entrained air(figure 1).

This entrainment of air is believed to depend upon: (1) The turbulence created at the crest or slightly upstream; (2) the roughness of the face of a dam or spillway; (3) the depth of flow; and (4) the total drop. Its effect is to reduce the velocity of flow to some value less than that obtained by considering only solid water, and to increase the volume of the flow. The application of Manning's formula for computing depth and velocity of flow down the spillway will evidently give erroneous values. Since the design of a hydraulic jump is based on principles of momentum, it is of utmost importance to ascertain, with reasonable accuracy, the velocity and mass of water as it enters a stilling pool.

With increasing volume due to air entrainment, it would be necessary to increase the freeboard at the sidewalls of a spillway structure. This would be even more important in design of steep chutes which usually have less width and depth of flow, but more length than spillways or overfall dams. Under these conditions, the increase in volume of flow would be comparatively greater, since the effect of sidewall friction to increase air entrainment would be greater with less width and depth of flow. Evidently, increased length of chute would allow more time for air entrainment to develop.

It is also believed, but not fully verified, that air entrainment begins at about 10 feet per second, and that the water-air mixture will reach a terminal velocity of nearly

Figure 1. Air Entrainment in Alcova Spillway-Wyoming

80 feet per second.[2]

Considerable research must be made before a thorough understanding can be reached regarding air entrainment and its effect on stilling pool design. The problem is being studied in this country by the hydraulic laboratory of the Bureau of Reclamation, and at the University of Iowa and the University of Minnesota.

Group II

The drop structures included in this group are usually required in the design of irrigation systems, or whenever the drop is relatively small, from 20 feet to 4 feet. Although some air entrainment occurs in the flow in these structures, its effect on the design of a stilling pool is not as serious as in the case of higher drop structures.

The problem in this group is to obtain a stilling pool design which will give effective energy dissipation and velocity reduction, but at the same time be economical. The importance of economy is realized when consideration is given to the number of these structures required in any irrigation system.

Studies recently made in the hydraulic laboratory of the Bureau of Reclamation, reveal the designs of these intermediate type drop structures to be conservative. It has been found that stilling pool designs based on theoretical considerations have excessive length and depth; but the

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2 Ehrenberger, R., Osterreichischen Ingenieur - Und Architektenvereines. Nos. 15-16 and 17-18, 1926.

greatest error has been the use of stilling pools having a trapezoidal section. Although this type has been used because its cost of construction is cheaper than a rectangular stilling pool, the flow is concentrated nearer the center of the pool, resulting in a poor hydraulic jump and very little energy dissipation and velocity reduction. In some cases, the hydraulic jump is nearly out of the stilling pool (figure 2). Where trapezoidal stilling pools have been used, constant maintenance is usually required to prevent excessive erosion in the canal (figure 3). It has definitely been shown, however, that a hydraulic jump in a rectangular stilling pool is uniformly distributed across the pool, resulting in effective energy dissipation and velocity reduction, and less maintenance to the canal below the pool (figure 4).

Group III

The drop at structures in this group is very small, 4 feet or less. These drop structures are required mostly in irrigation canals, laterals, or ditches. Even though the drop is small, some energy dissipation should occur, or the material through which the canal or lateral flows may be heavily scoured. If the average allowable velocity for canals be taken as 2.50 to 3.00 feet per second, then a drop in water surface of only one foot produces a free fall velocity of about 8 feet per second at the drop structure. This velocity may be excessive in most canals. Velocities in excess of the allowable are permissible in only a few cases: Particularly when the canal below the

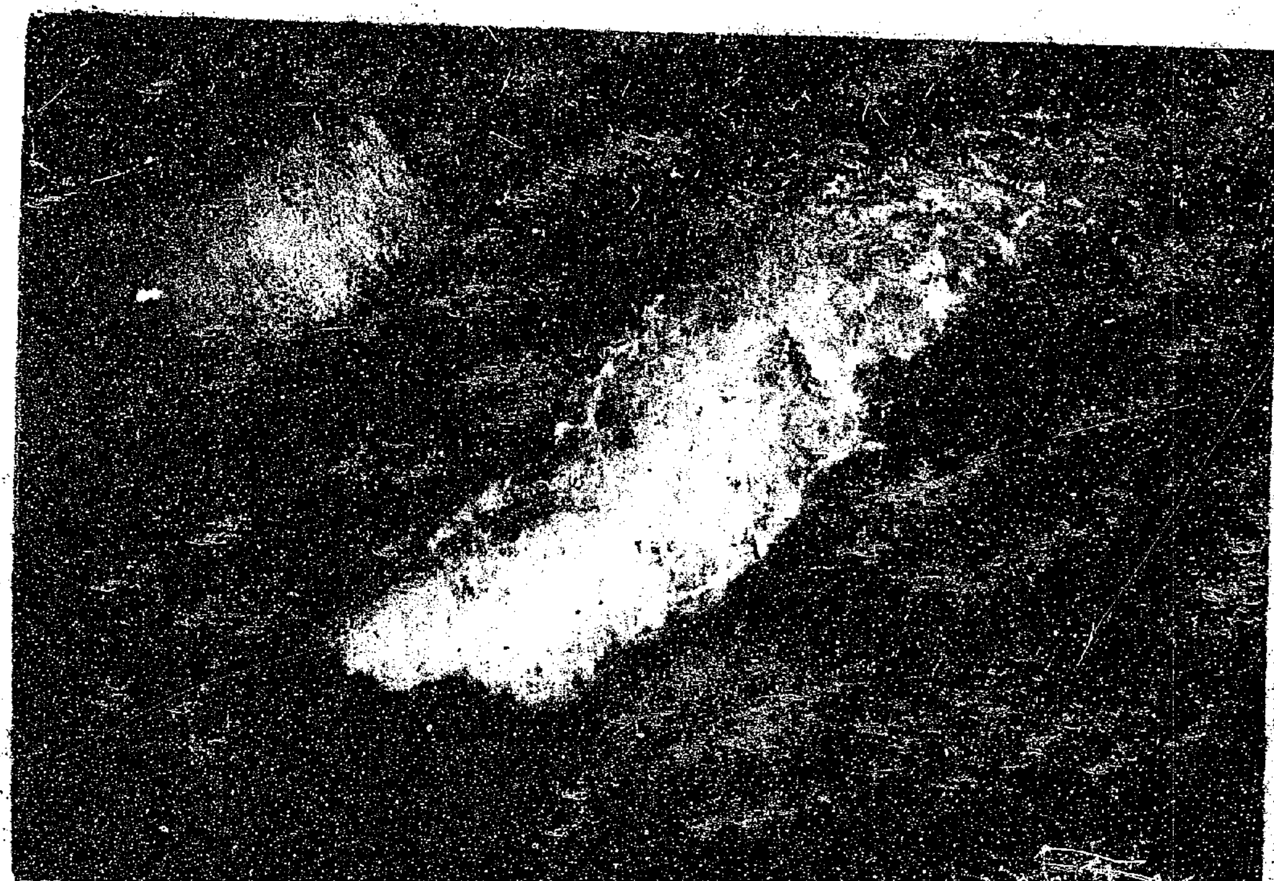


Figure 2. Trapezoidal Stilling Pool in Owyhee North Canal- Oregon

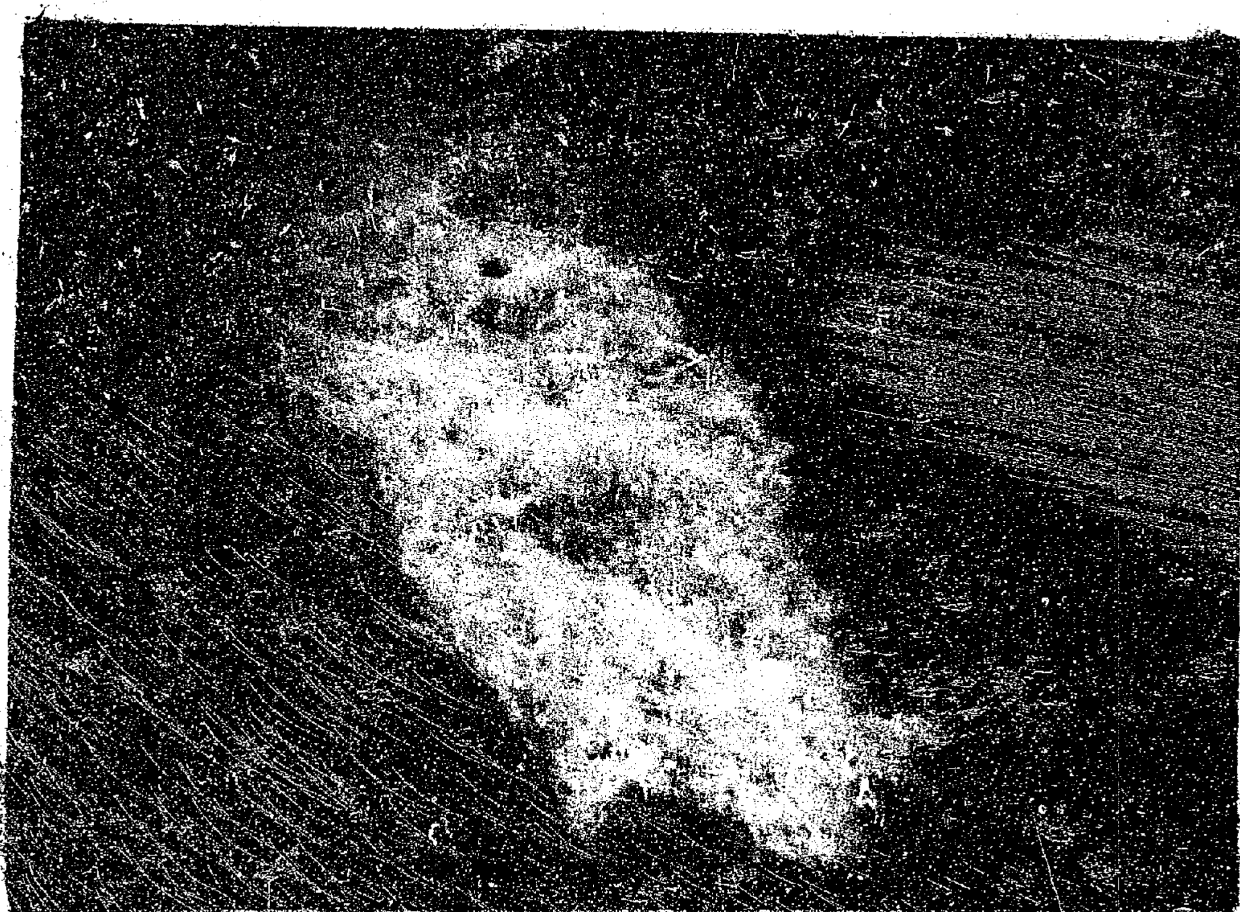


Figure 5. Trapezoidal Stilling Pool in Owyhee Kingman Lateral-[illegible]

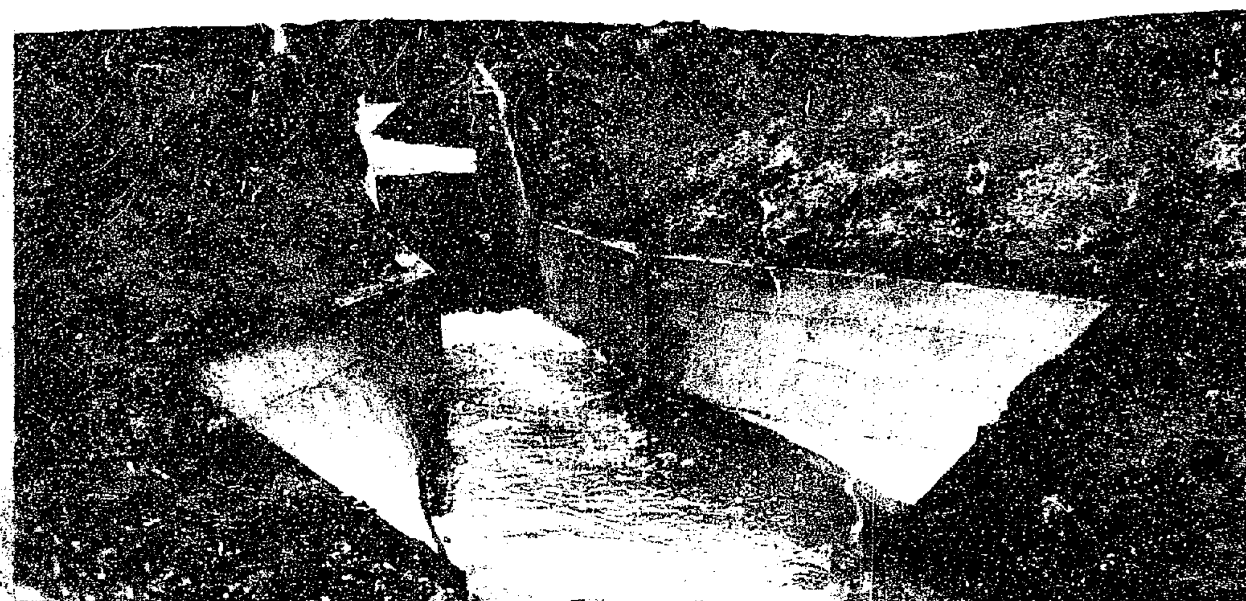


Figure 4. Rectangular Stilling Pool (and Transition) in Owyhee South Canal-Idaho

drop structure is concrete lined, or the material is not readily scoured.

At the drop structures included in Group I and Group II, there was sufficient energy to cause the depth of flow upstream from a stilling pool to be less than critical depth. It will be recalled that this condition is essential for the formation of a hydraulic jump. At small drop structures included in this group, however, it is not always possible to obtain this condition.

One of the stilling pool problems in this group occurs, therefore, when the depth of flow at a drop structure is prevented from being less than critical depth due to submergence at the point of drop. This condition exists, for example, when the depth of flow in a canal is maintained uniformly between drop structures. This depth, which is maintained in order to divert water into laterals, is usually sufficient to cause submergence (figure 5.). As a result, it is very difficult to obtain effective energy dissipation and velocity reduction, since a hydraulic jump is not readily adapted. One solution to this problem will be discussed in Chapter V.

Many canals or laterals are operated, however, without causing submergence at a drop structure. A simple H-drop (figure 6.), for example, will operate satisfactorily under this condition. It is not a difficult problem to obtain sufficient energy dissipation and velocity reduction at these structures, and many standard designs have been made which are readily adapted to a particular problem.

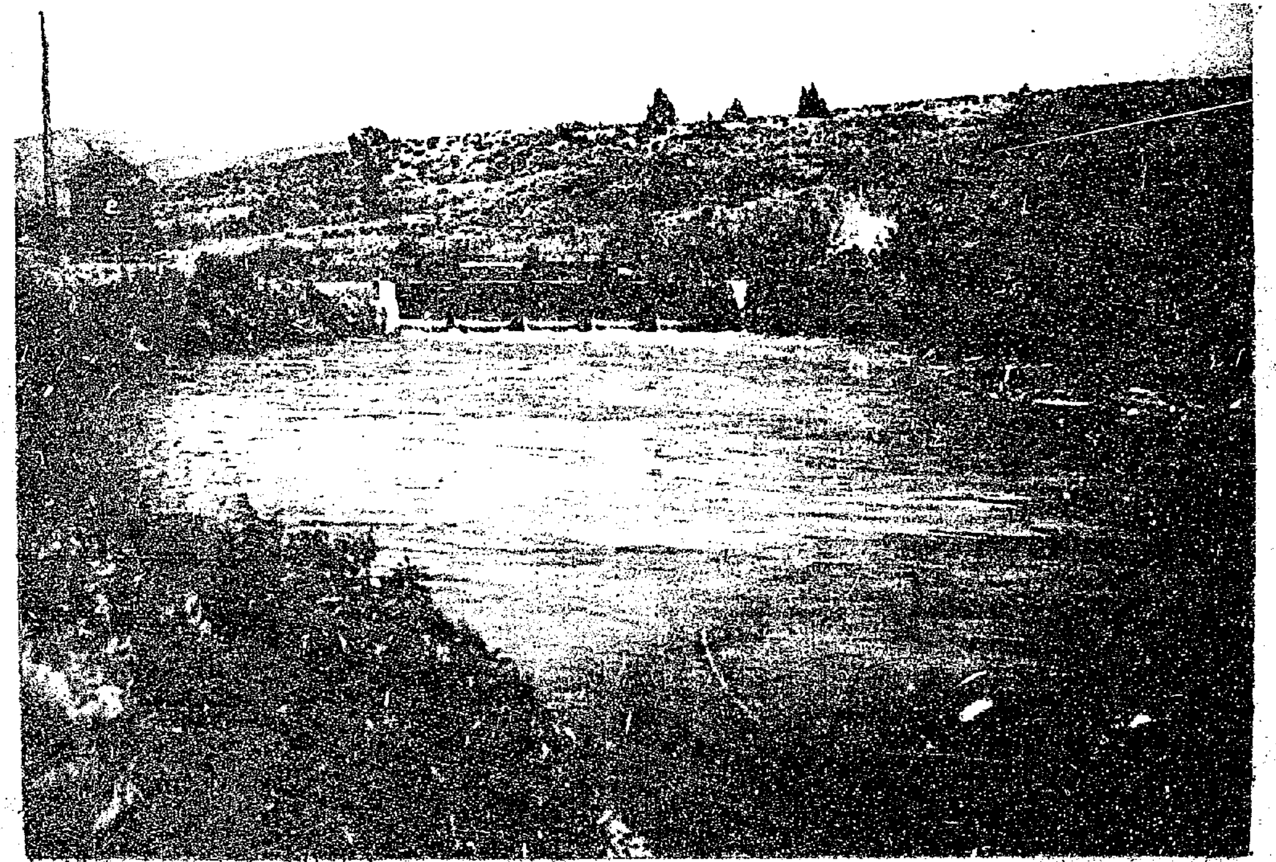


Figure 6. Check Drop No.2 in Sunnyside Main Canal-Washington

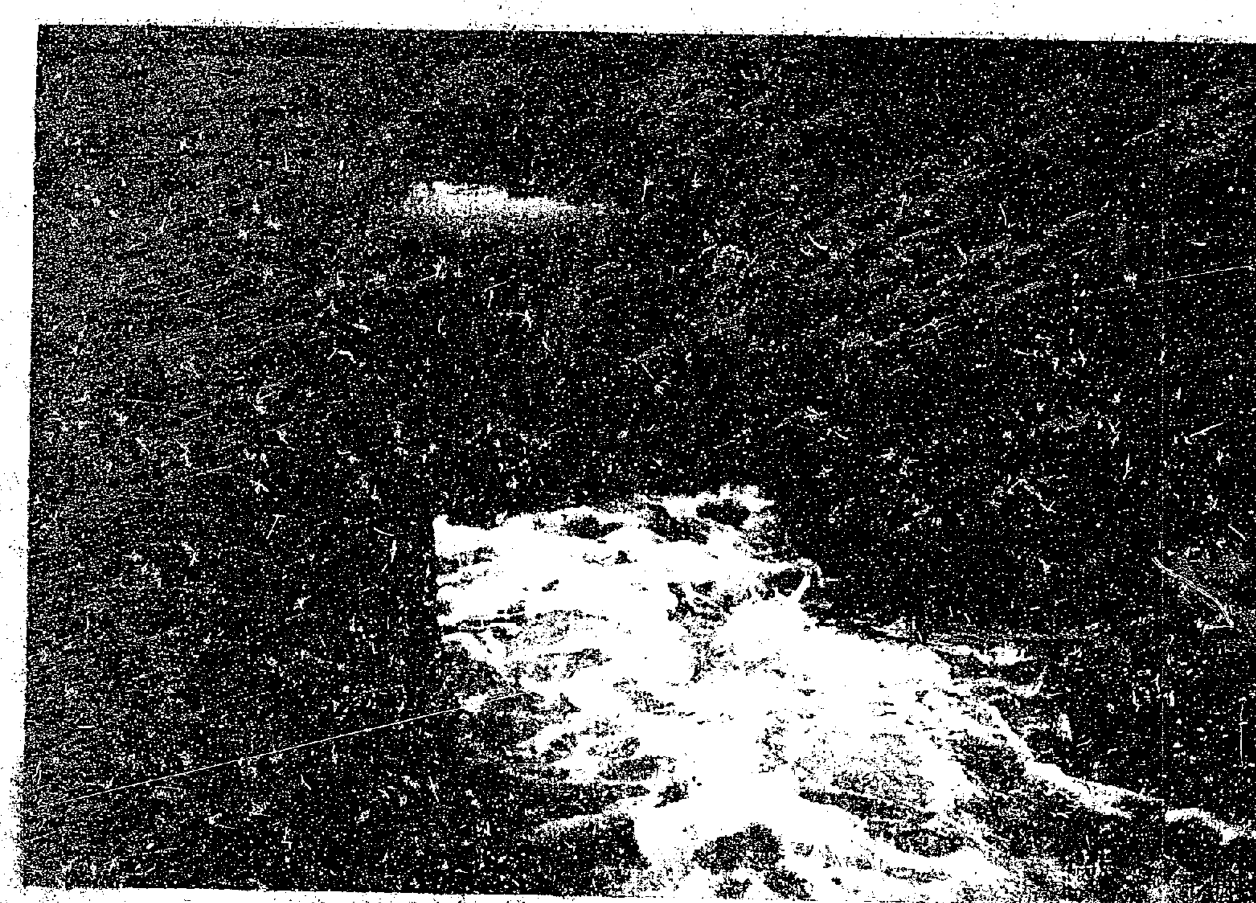


Figure 6. H-Drop in Gem District D-Line, Idaho

Solution of Individual Problems

Although any particular hydraulic problem pertaining to a stilling pool design of drop structures may be recognized, its solution is not always apparent. It is now generally recognized that hydraulic model studies are necessary in most cases to obtain a satisfactory stilling pool design. This method has proved successful in closed conduit and open channel studies. Such well-known structures as Boulder Dam and Grand Coulee Dam, not to mention many others, have been investigated by model studies and in every case, the stilling pools in particular have been evolved by this method. In many cases, model studies have been made of existing structures to eliminate unsatisfactory flow conditions.

The problems of Group I are readily solved by model studies, except the problem of air entrainment. This problem must be studied on prototype drop structures, since the velocity of flow in models is not sufficient to entrain much air. A stilling pool, however, may be readily designed by a model study neglecting effect of air entrainment. This probably gives a conservative design, but one that will nevertheless be effective. In any event, the stilling pool so evolved will operate as indicated by the model, which is more assuring than depending on purely analytical considerations.

The problems of Group II have been solved in many instances by only analytical considerations, but there are many existing drop structures included in this group which have been, or should be, corrected by model studies. Studies at the Bureau of Reclamation (page 7) have developed a nomographic solution for the design of drop structures of this group. This type of solution permits a design of drop structures which will not only operate effectively, but will also be economical.

The problems of Group III yield more readily to analytical considerations than those of Group I or Group II, if submergence does not occur. This is probably due to less discharge and drop. The problem of designing a stilling pool for the drop structure shown on figure 6 is not difficult, but when submerged flow occurs at the point of drop, model studies are usually necessary before a satisfactory stilling pool can be evolved. It would be nearly impossible to determine analytically the scour effect of flow at a submerged drop structure. It is for this reason that a model study was made of the existing check drop structures (figures 5 and 9) of the Sunnyside Main Canal, Washington.

It is important to note that, although it may be predetermined that a hydraulic jump will form in a stilling pool of any drop structure, model studies can greatly improve the efficiency of the jump. A hydraulic jump based on theoretical considerations alone, has been found somewhat unstable with high velocities "corkscrewing" out of the stilling pool. This unfavorable condition is practically eliminated if a Rehbock sill, or a row of dentated steps and baffle piers, is placed along the floor of a stilling pool. In addition, this may also permit the depth and length of a stilling pool to be less than that determined by analytical considerations.

Chapter II

A Characteristic Problem of a Drop Structure in Group III

The Sunnyside Main Canal

The Sunnyside Division of the Yakima irrigation project consists of about 100,000 acres. Nearly 90,000 acres are 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 7). At the time of its purchase by the Reclamation Service, the canal had a capacity of 650 second-feet at the headworks. Plans were made after its purchase in 1906 to enlarge the canal and increase its capacity to 1,076 second-feet at the headworks. This work was completed by 1912. Additional improvements were made from 1917 to 1922 which allowed the capacity at the headworks to be increased to 1,300 second-feet, its present capacity.

Check Drop Structures

Due to increasing the capacity of the canal, the hydraulic gradient was changed, causing increased velocities and a lower water surface. To meet this condition, 23 check drops (figure 8) were built in the canal about 2 miles apart. Most of the check drops were built in the winter of 1907-08, and the remaining ones were built during the period 1909-16. These structures raised the water surface enabling the desired amount of water to be diverted through adjoining laterals and, at the same time, they reduced the velocity of

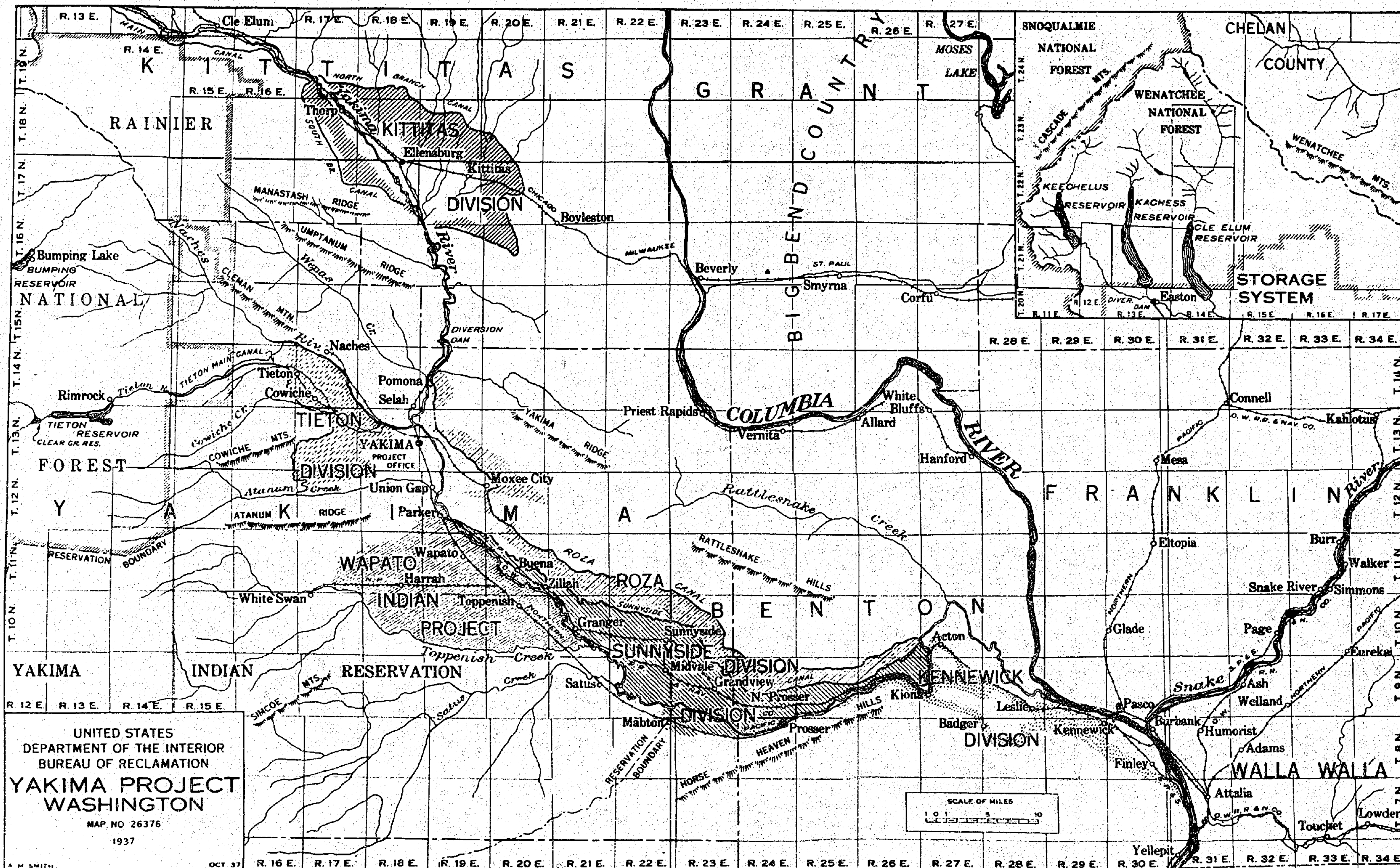
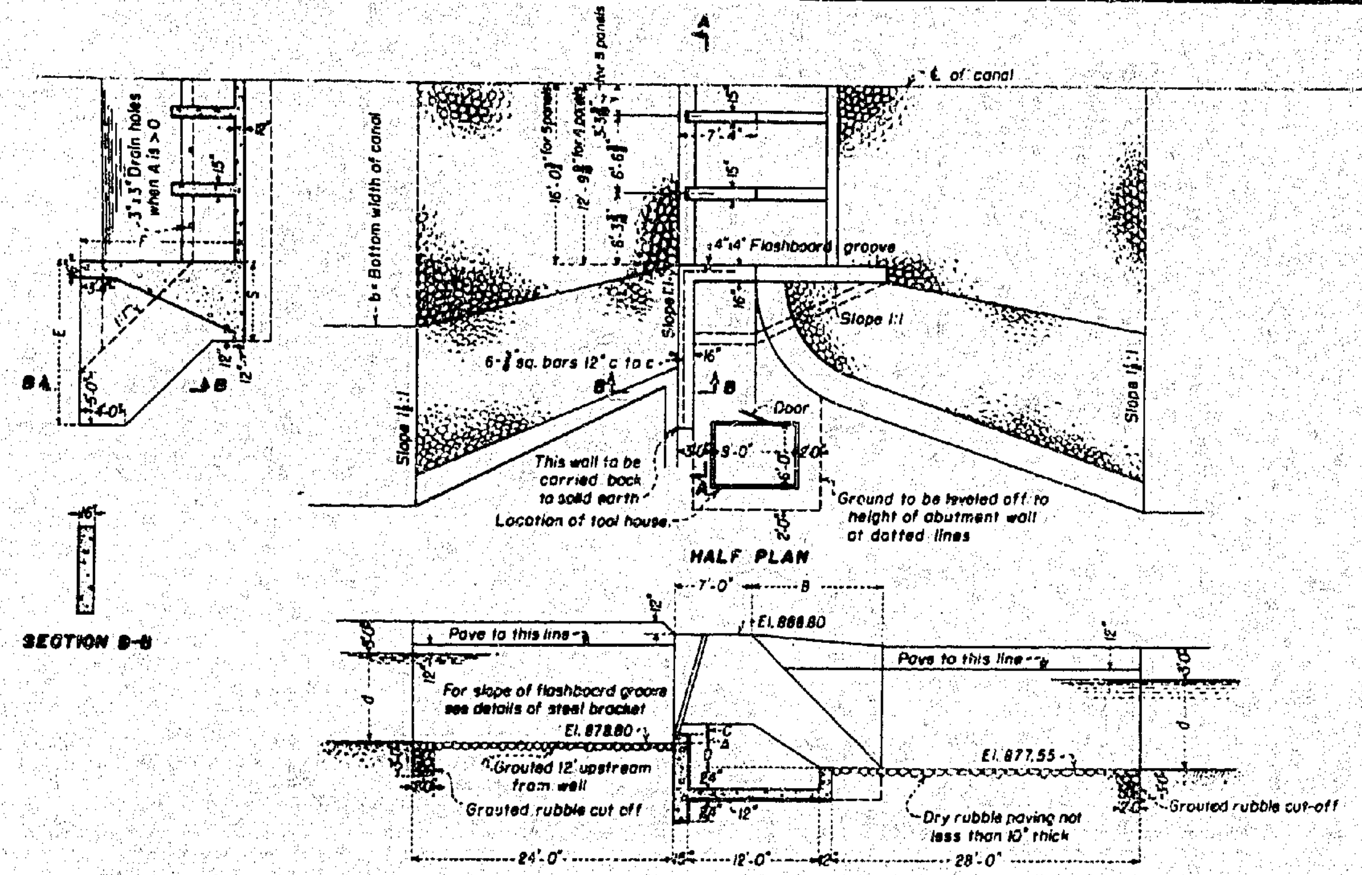

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YAKIMA PROJECT
WASHINGTON
MAP NO. 26376
1937
KITTITAS DIVISION
TIETON DIVISION
WAPATO INDIAN PROJECT
ROZA
SUNNYSIDE DIVISION
KENNEWICK DIVISION
STORAGE SYSTEM
KITTITAS
GRANT
BIG BEND COUNTRY
YAKIMA
BENTON
FRANKLIN
WALLA WALLA
CHELAN COUNTY
RAINIER NATIONAL FOREST
SNOQUALMIE NATIONAL FOREST
WENATCHEE NATIONAL FOREST
YAKIMA INDIAN RESERVATION
COLUMBIA RIVER
Yakima River
Snake River
KEECHELUS RESERVOIR
KACHESS RESERVOIR
CLE ELUM RESERVOIR
BUMPING RESERVOIR
TIETON RESERVOIR
SCALE OF MILES

Figure 7

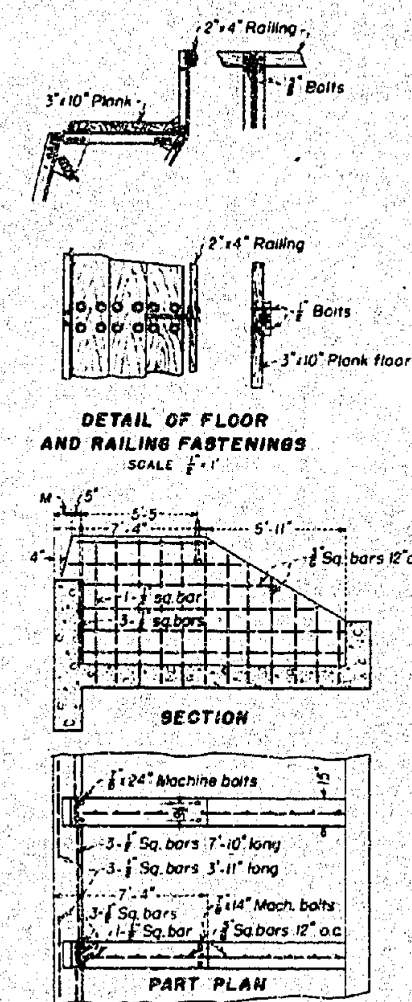


HALF PLAN

SECTION B-B

SECTIONAL ELEVATION
SCALE ⅛"=1'

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

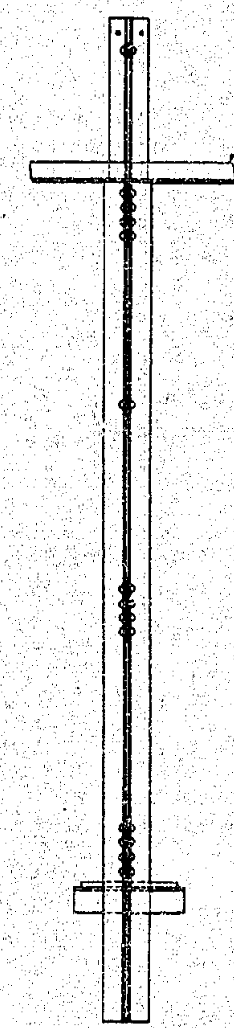


DETAIL OF FLOOR AND RAILING FASTENINGS

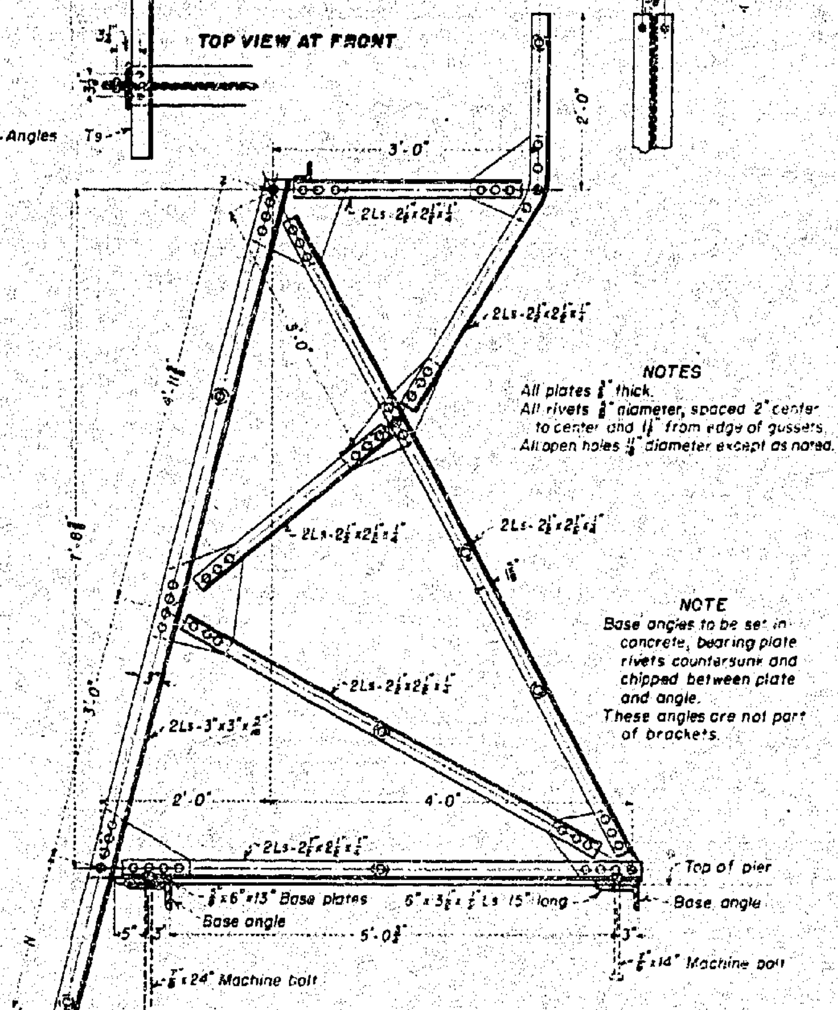
SECTION

PART PLAN

DETAIL OF REINFORCEMENT AND BOLT CENTERS



FRONT VIEW

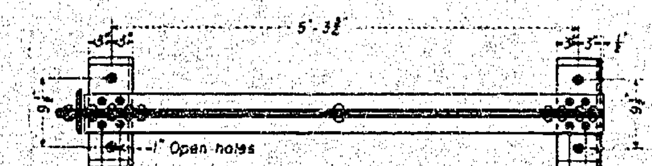


TOP VIEW AT FRONT

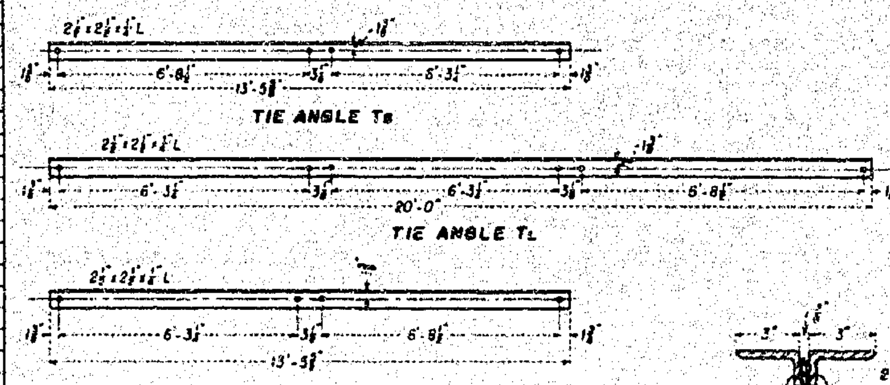
DETAILS OF STEEL BRACKET
SCALE 1"=1'

NOTES
All plates ⅜" thick.
All rivets ⅝" diameter, spaced 2" center to center and 1¼" from edge of gussets.
All open holes 1" 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.



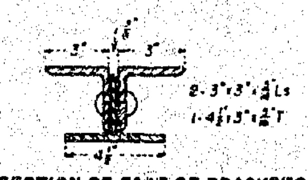
TOP VIEW OF BASE



TIE ANGLE T_B

TIE ANGLE T_L

TIE ANGLE T_R



CROSS SECTION OF FACE OF BRACKETS AS RECONSTRUCTED IN WINTER OF 1908
SCALE 3"=1' SEE S.P. 600 FOR DETAILS

TABLE OF DIMENSIONS AND QUANTITIES

Revised	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
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'-8⅝"	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'-10⅝"	0'-8½"	1'-9"	97.7	441	410	4	2280	80	322	B-3	239
3	45	8.0	1053	2'-0"	5	0'-6"	1'-6"	12'-0"	15'-0"	15'-0"	5'-10⅝"	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'-7⅜"	0'-10"	2'-3"	89.6	441	410	4	2280	85	322	B-5	386
5	44	8.0	1039	1'-7½"	5	0'-0"	2'-0"	11'-7½"	15'-0"	14'-7½"	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'-11⅜"	0'-7"	1'-2⅜"	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'-6⅜"	0'-10"	2'-3"	88.2	441	404	4	2280	85	322	B-8	635
8	42	8.0	992	1'-9"	5	0'-6"	1'-6"	11'-9"	15'-0"	14'-9"	5'-9⅜"	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'-6⅜"	0'-10"	2'-3"	88.2	441	401	4	2280	85	322	B-10	812
10	41	8.0	982	2'-0"		Ziliah Wasteway																693
11	40	8.0	966	1'-0"	5	0'-0"	2'-0"	11'-0"	15'-0"	14'-0"	5'-6⅜"	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'-6⅜"	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'-6⅜"	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'-0⅜"	0'-4"	0'-2⅜"	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'-10⅜"	0'-7"	1'-2⅜"	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'-10⅜"	0'-7"	1'-2⅜"	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'-6⅜"	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'-1½"	9'-7½"	14'-0"	12'-7½"	5'-1"	0'-7½"	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'-0⅜"	0'-7"	1'-2⅜"	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'-5½"	0'-7½"		See S.P. 677				56		B-24	1965
22	22	6.7	490	0'-9"	4	0'-1½"	0'-7½"	9'-5½"	13'-8½"	12'-5½"	5'-0½"	0'-5½"	0'-9½"	58.6	349	346	3	1710	56	240	B-23	2141
23	22	6.0	430	0'-7½"	4	0'-0"	0'-0"	8'-7½"	13'-0"	11'-7½"	4'-9"	0'-4"	0'-2½"	54.3	349	332	3	1710	48	240	B-26	2420

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

NOTES
This drawing traced from Dwg. SP628, dated January 1909.
Elevations shown are for Check Drop 4 only.
For detail of model see Figure 3.

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YAKIMA PROJECT - WASHINGTON
MAIN CANAL STANDARD DROPS

DRAWN E.A.M.	SUBMITTED
TRACED C.A.G.	RECOMMENDED
CHECKED C.R.E.	APPROVED

33-D-1772

Figure 8

flow in the canal.

Examination of figure 8 shows the check drops to be of a standard design. A check basin is placed between two vertical walls. Included in the check basin are piers on which steel brackets are placed. These permit the placing of flashboards in the flow to regulate the depth of water between successive check drops. Riprap is placed in the approach and downstream from the check basin to reduce scouring. Although the check drop structures were designed nearly thirty years ago, this type of design is occasionally used today.

The Problem

After the check drop structures had been in operation only a few years, scouring began to occur in the canal section immediately downstream. Maintenance of the canal banks was required continually. This consisted of placing riprap in the scoured section of the canal during the winter; however, the riprap was washed into the canal during an irrigation season. Gradually the scouring continued, regardless of maintenance, until recently it became necessary to eliminate it, since the scour was encroaching upon valuable orchard and crop lands adjacent to the drop structures. In some instances, the canal below a drop was nearly twice its original width (figures 5 and 9). Accordingly, the problem was referred to the Bureau of Reclamation, Denver, Colorado.

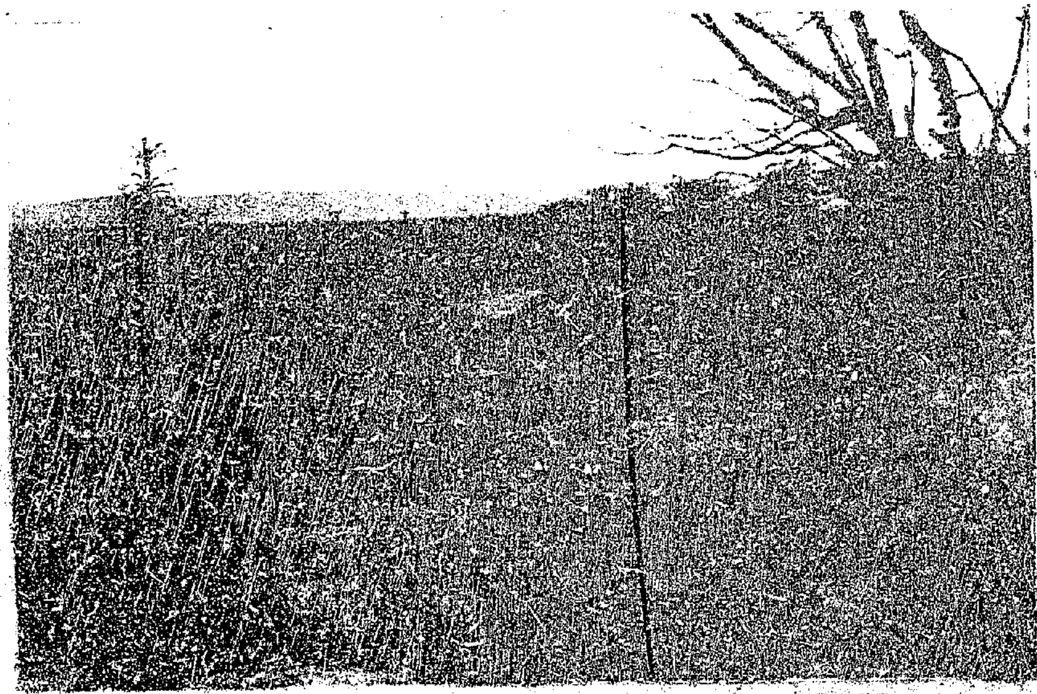
It will be seen presently, from an analytical study of a typical check drop structure, that the scouring in the canal was due principally to submergence at the drop which prevented effective energy dissipation and velocity reduction. This is recognized as a problem of some of the drop structures included in Group III.

Solution by Model Study

Since there is no analytical method which can be readily applied to the scour effect of flow at existing check drop structures, it was necessary to make a hydraulic model study of the check drops in the Sunnyside Main Canal. When the problem was submitted to the author, it was decided to study Check Drop No. 4, as that structure was typical of the unfavorable conditions existing (figure 9).

A model study was to be made, therefore, to determine, in the form of a recommended design, the necessary revisions which could readily be applied to Check Drop No. 4 to eliminate excessive scouring in the canal. Since the check drop structures are of a standard design, the recommended design determined for Check Drop No. 4 could be applied to the other drop structures.

A. Looking Upstream-Discharge 1275 Second-Feet
July 1938



B. Looking Upstream-Scour October 1938

Figure 9. Check Drop NO.4 in the Sunnyside
Main Canal-Washington

Chapter III

Hydraulic Analysis

Type of Flow

Although it is not possible to determine analytically the scour effect of the flow at a check drop structure, it is entirely possible to make an analytical study of the type of flow, for example, hydraulic jump or standing wave. If it is determined a hydraulic jump exists, then it is known that considerable energy dissipation is occurring. If a standing wave is found to exist, however, then it is known that very little energy dissipation occurs. Having once established the type of flow that exists at a small drop structuré, it is then often possible to explain their unsatisfactory operation, in addition to any explanations given as a result of observations made on the actual structure and on the model.

In hydraulic literature, some authors refer to the hydraulic jump as a standing wave, while others (including the author) consider these to be two distinct phenomena. In this discussion, a hydraulic jump shall be considered that phenomenon which occurs when flow less than critical depth, rapid flow, passes in an abrupt manner to flow greater than critical depth, tranquil flow. A standing wave shall be considered as surface undulations characterized by flow at or very near critical depth.

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[3]. Figure 10C shows the specific energy diagram for Check Drop No. 4. This diagram has been plotted for a discharge of 1,264 second-feet through the rectangular section having a width of 32.13 feet (figure 8).

By referring to the specific energy diagram, it is possible to trace the change of specific energy in the flow with the change of depth in a section for the given discharge. It must be understood, however, that specific energy is referred to the bottom of a channel as a datum which may change from section to section. Unless otherwise stated, the terms "specific energy" and "energy" will be synonymous.

It can be seen on figure 10C that 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 content of specific energy, is called "critical depth."

The Hydraulic Jump

Referring to figures 10B and 10C, 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 lost in the jump is represented by

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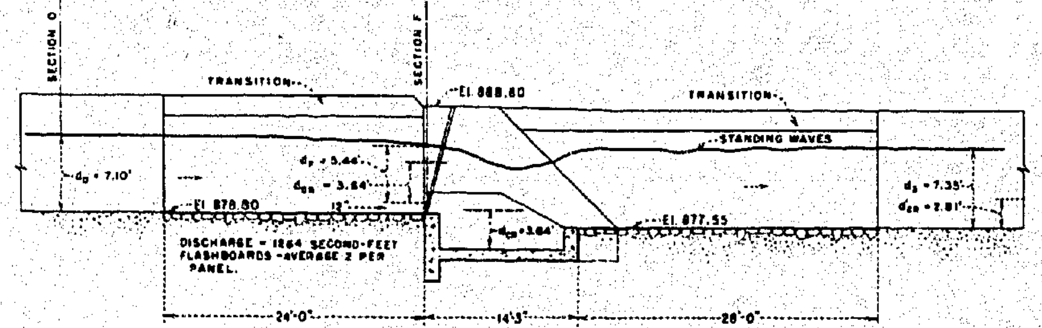
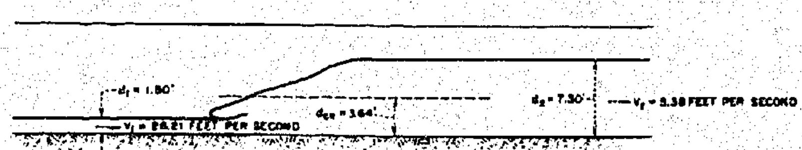
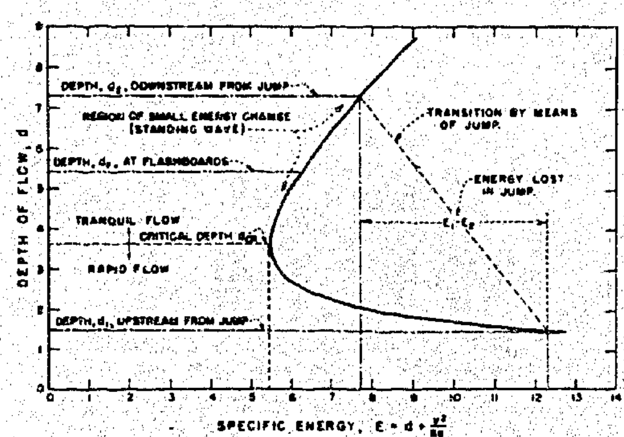
3 Bakhmeteff, Boris A., Hydraulics of Open Channels; McGraw-Hill, 1932; p. 32.



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 ... SUBMITTED
TRACED ... RECOMMENDED
CHECKED ... APPROVED
DENVER, COLORADO - JAN. 3, 1939 | 33-D-1773

Figure 10

$E_1 - E_2$. 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 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, however, that this relation applies only to a hydraulic jump.

Assume a rectangular channel of constant cross-section in which it is possible to vary d_2, the depth of flow downstream; and assume d_1, the depth of flow upstream to be constant, and the discharge to be constant. Now, by referring to the specific energy diagram (figure 10C), it can be seen that if d_2 be less than d_1 (let $d_1 = 1.50$ feet), there will be a gain in specific energy. Under these conditions, the flow could be represented as passing over a vertical curve of a spillway. But now, let d_2 be greater than d_1, and it can be seen that there will be loss in specific energy. In the particular case where the increase in d_2 above d_1 passes through critical depth, then, as has been seen, a hydraulic jump forms, causing a large loss in specific energy. It will also be observed in the case of a hydraulic jump, that the ratio, $\frac{d_1}{d_2}$, is much less than unity. This is an important fact, since large energy losses are accompanied by great changes in depth, and great changes in depth result in considerable velocity reduction.

The Standing Wave

If sufficient energy is not present in the flow to cause d_1 to be considerably 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.

Assuming again the conditions of the rectangular channel and referring to the specific energy diagram in the region just above critical depth, it can be seen that: If the depth of flow, d_2, be less than d_1 (let $d_1 = 5$ feet), there will be a loss of 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 occurring in the hydraulic jump. Furthermore, standing waves may form even if d_2 is slightly less than or greater than d_2, and they may also form if d_1 is slightly less than critical depth.

There is no abrupt change of depth when standing waves occur, so 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. This means, furthermore, that the change of depth is slight, resulting in very little velocity reduction.

Classification of Check Drop No. 4

It is difficult to completely determine, analytically, the type of flow that exists at Check Drop No. 4, since 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 upstream from 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 at Check Drop No. 4:

$Q = 1{,}264$ second-feet
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 0 to section F (figure 10A):

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

$+ S\ell$

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),

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

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

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 be readily seen, however, that only one value applies here. If the tailwater 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 below critical depth at the flashboards 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 standing waves will form when d_1 and d_2 are nearly equal and are either slightly above or below critical depth.

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 discharges not only results in d_2 being above critical depth, but it prevents the depth of flow at the flashboards, d_F, from being less than critical depth.

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 these depths, 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; whereas the loss of specific energy in a hydraulic jump was seen previously to be much greater.

Since it is not possible to accurately determine the changes in depth of flow through the drop structure, assume standing wave flow in which $d_1 = 5.50$ feet and $d_2 = 5.00$ feet. Then from figure 10C the corresponding specific energy values will be $E_1 = 6.30$ and $E_2 = 5.90$, respectively. The loss, $E_1 - E_2 = 0.40$, or 6.4 percent. Assuming a constant rectangular section, it is evident that since the decrease in depth from d_1 to d_2 is small, the reduction in velocity will be practically nothing.

Now assume that sufficient energy is available to form a hydraulic jump at Check Drop No. 4. Then let $d_1 = 1.50$ feet, and with a discharge of 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 10B). Now, $E_1 = 12.30$ and $E_2 = 7.76$; the loss, $E_1 - E_2 = 4.54$ (figure 10C). The loss in energy will be $\frac{4.54}{12.30} \times 100$, or 36.8 percent as compared to about 6 percent in the standing wave flow assumed. Although there was practically no velocity reduction in that flow, the velocity reduction through the assumed hydraulic jump is about 80 percent.

<u>The Hydraulic Jump Compared to Standing Wave at Check Drop No. 4</u>

As previously mentioned, the hydraulic jump is used in stilling pools of drop structures when sufficient energy is available for a jump to form. When this occurs, it has been shown that the energy of the flow is not only greatly dissipated, but the velocity of the flow is considerably reduced. For best efficiency of the hydraulic jump, the stilling pool should include dentated sills or teeth. This will also aid 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 it is important to note that severe scour will occur downstream from a drop structure, even though a hydraulic jump forms, if the jump is not properly confined or controlled. Many problems of this nature have also 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 in the flow. The classification reveals that there is not sufficient energy to form a hydraulic jump; instead, standing waves form. Under this condition, there is little velocity reduction; hence, the canal downstream must be subjected to velocities which are excessive for the fine material, volcanic ash or tuff, through which the canal flows.

To digress for the moment, it may seem paradoxical to state that it is necessary to dissipate energy of flow to prevent scour, and then to state that sufficient energy must be available to cause effective energy dissipation. This is evidently a matter of the result of losing specific energy. The loss of energy in a hydraulic jump is considerable and it is accompanied by a great change in depth, and considerable velocity reduction. The standing wave, however, causes very little energy loss, and therefore only a small change in depth and very little velocity reduction. Hence, to reduce the amount of energy and to reduce the velocity in order to prevent scouring, it is necessary to cause the flow to pass through a hydraulic jump. But to do this, there must be sufficient energy to make the depth upstream, d_1, be considerably less than critical depth. Consequently, if standing waves occur in the flow, more energy must be added before a hydraulic jump will form.

Now, as has just been mentioned, the use of a hydraulic jump will give satisfactory flow conditions in and below a stilling pool; but observations of the prototype and model reveal the standing wave flow at Check Drop No. 4 to be responsible for the unsatisfactory conditions. Instead of a uniform flow distribution, high velocity flow is concentrated near the center of the canal. This is evidently due to insufficient energy dissipation and velocity reduction, and to the sudden transition below the check basin (figure 8). Further observations made on the model, discussed presently, discloses: (1) As a result of the nonuniform velocity distribution, large eddies form along the sides of the canal. These eddies have sufficient velocity to cut into the canal banks and produce the excessive scour which exists downstream from most of the check drops in the Sunnyside Main Canal: (2) there was very little improvement in the flow, even though the standing waves at the drop were confined within a rectangular stilling pool; and (3) riprap below the drop structure was washed into the canal.

Chapter IV

Hydraulic Model Study of Check Drop No. 4

The Model

The model of Check Drop No. 4 was built to a scale of 1 to 15 (figures 11 and 12). 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 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 at the drop structure, 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 volcanic tuff. A sieve analysis of the model sand is shown on figure 13.

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 could be made to determine a recommended design for eliminating scour.

For the initial test, the model was run for 26 hours at

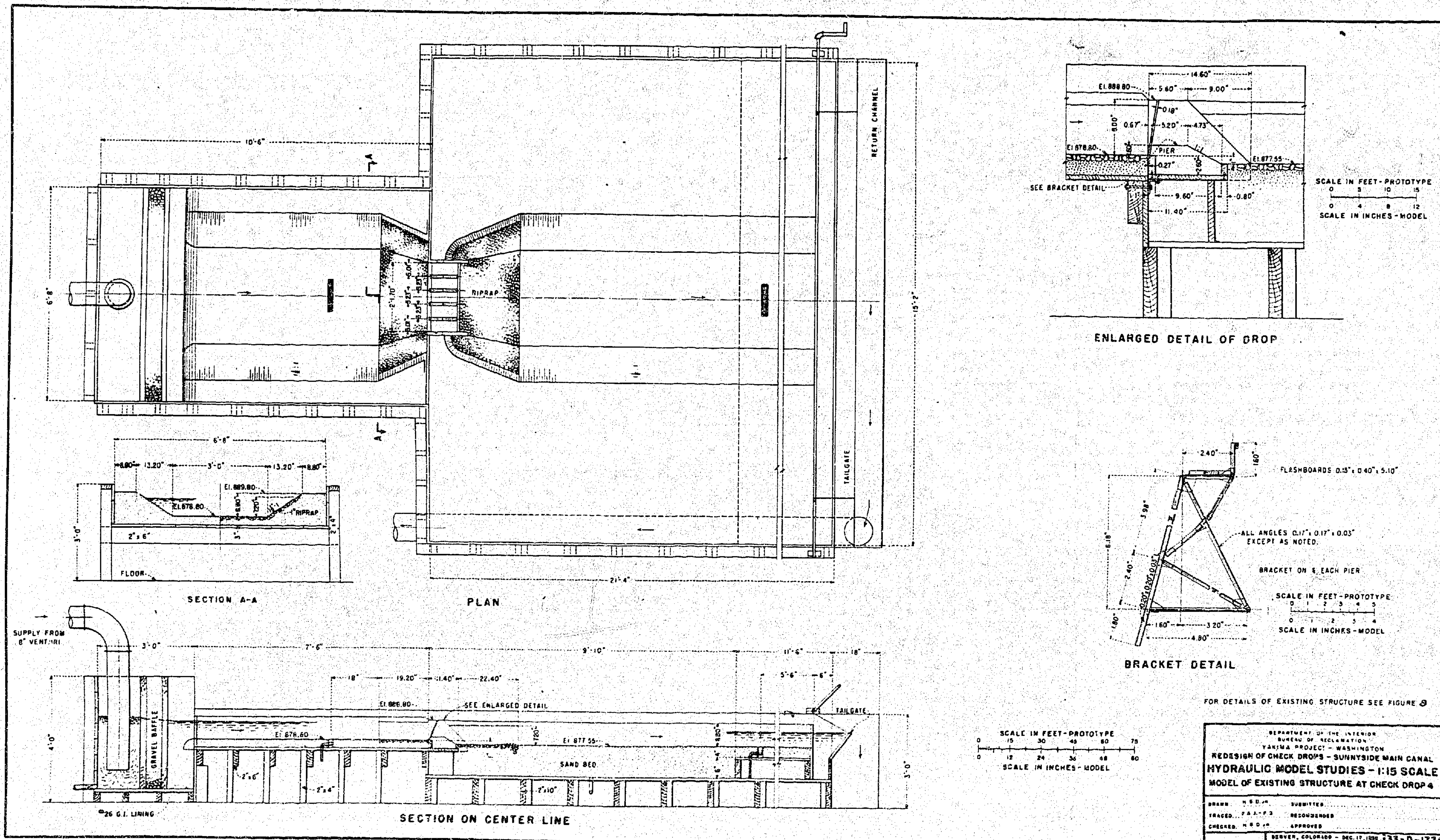
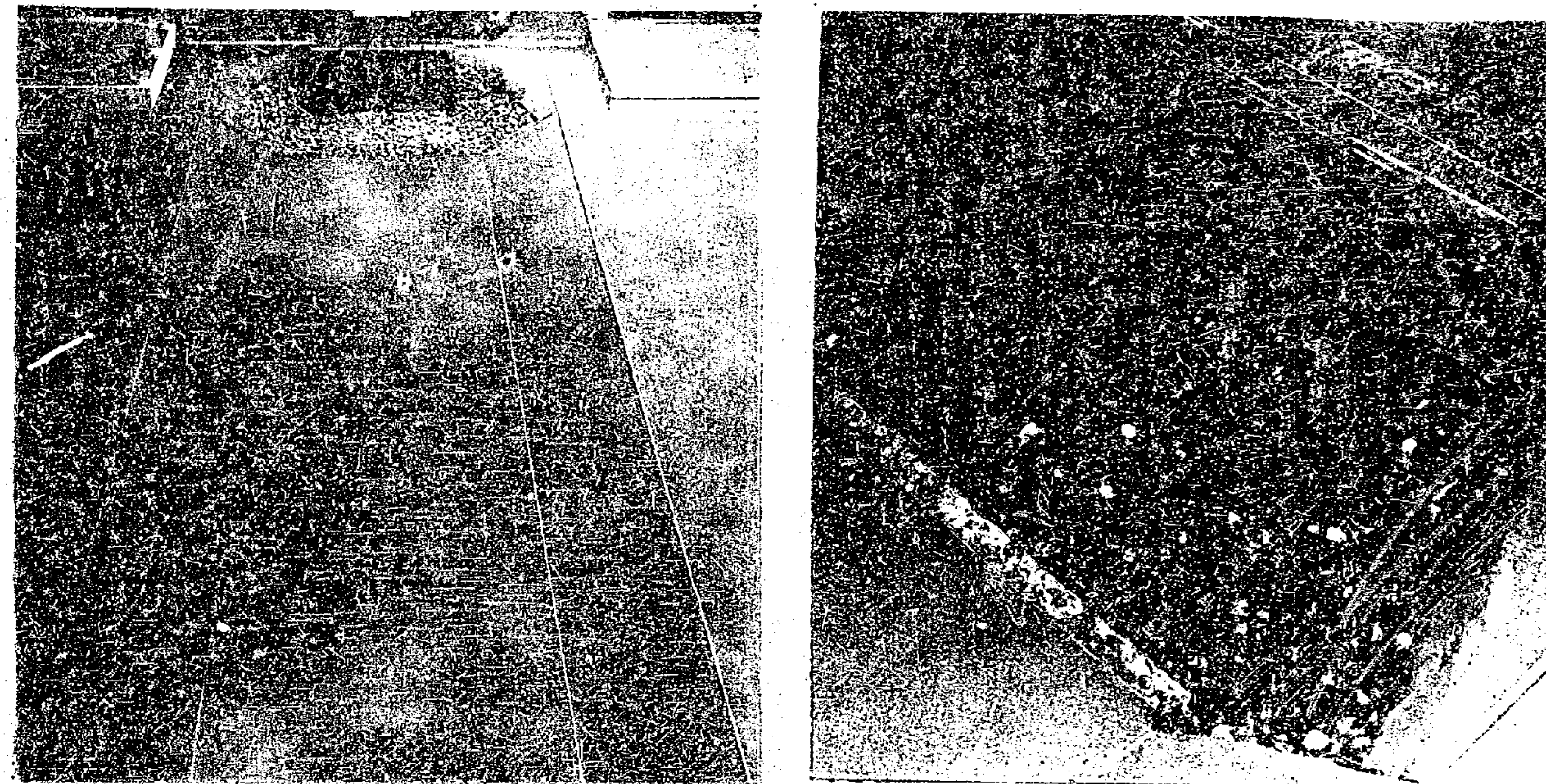

ENLARGED DETAIL OF DROP
SEE BRACKET DETAIL
PIER
SCALE IN FEET - PROTOTYPE
SCALE IN INCHES - MODEL
RETURN CHANNEL
RIPRAP
TAILGATE
SECTION A-A
FLOOR
PLAN
BRACKET DETAIL
FLASHBOARDS 0.13" x 0.40" x 5.10"
ALL ANGLES 0.17" x 0.17" x 0.03" EXCEPT AS NOTED.
BRACKET ON ℄ EACH PIER
SUPPLY FROM 8" VENTURI
GRAVEL BAFFLE
SEE ENLARGED DETAIL
SAND BED
26 G.I. LINING
SECTION ON CENTER LINE
FOR DETAILS OF EXISTING STRUCTURE SEE FIGURE 9
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
DENVER, COLORADO
33-D-1774

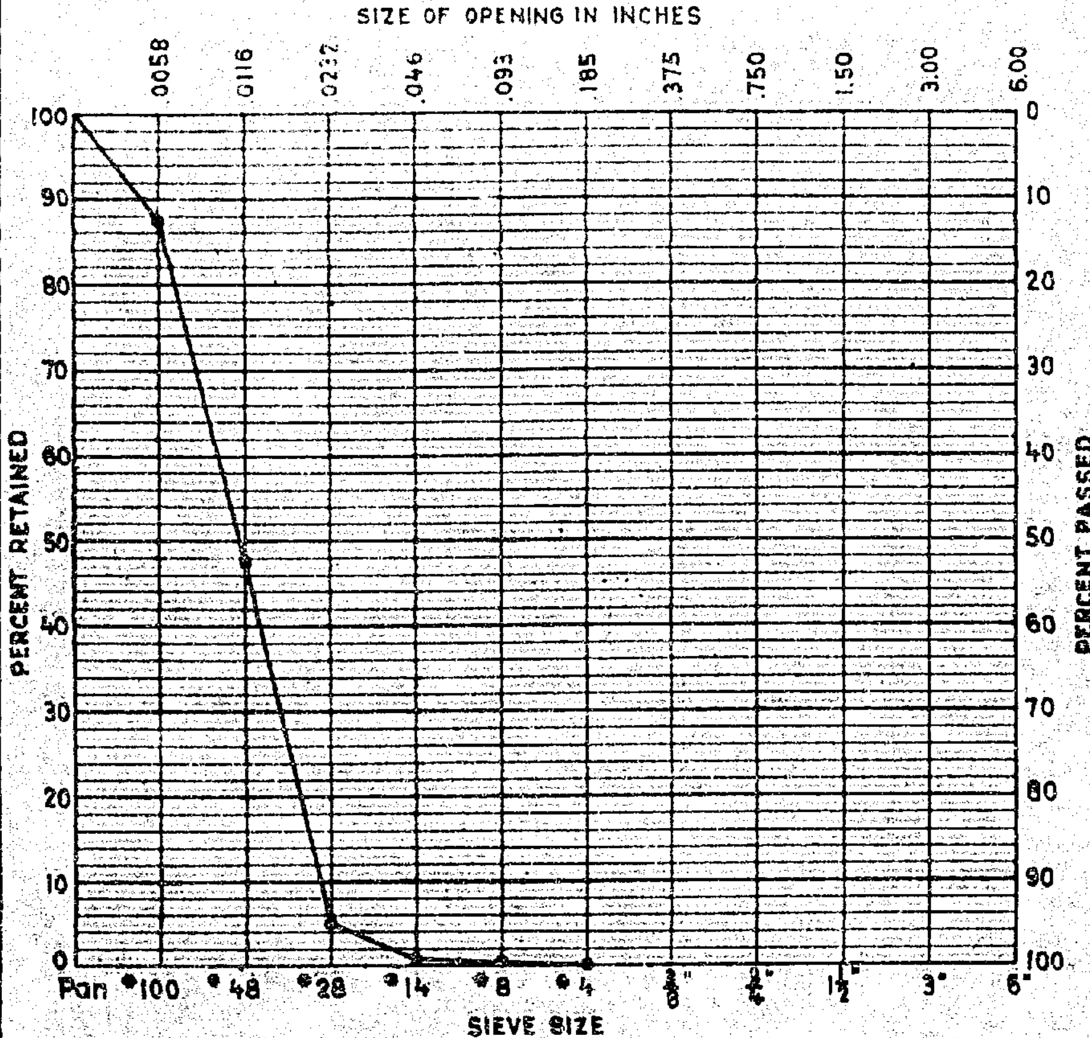
Figure 11



A. Looking Upstream

B. Drop Structure

Figure 12. Existing Structure



GRADING

Sieve Size	% Ret.	Cum. %	Comb. % Ret.	Comb. % Cum.
6"				
3"				
1½"				
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.B.D. CHECKED
Date 1-9-39 At DENVER, COLORADO 33-D-1775

Figure 13

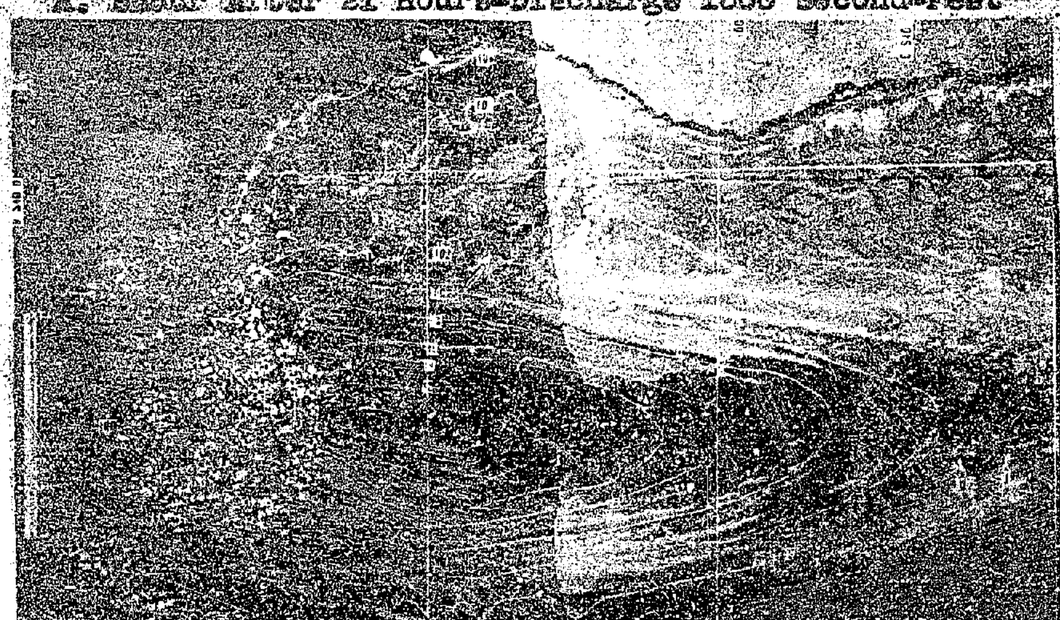
a discharge representing 1,300 second-feet. During this time, the number and symmetry of the flashboards and the tailwater depth was changed at intervals. At the time of the initial test, sufficient data were not available to determine the exact field relation between the depth of flow upstream and downstream 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 14 and 15 show the model in operation, and the scour at the completion of the test.

Analysis of Initial Model Test and Comparison to Prototype

As a result of the studies made during the initial test, it became evident that the following existing conditions had to be removed in order to reach 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.

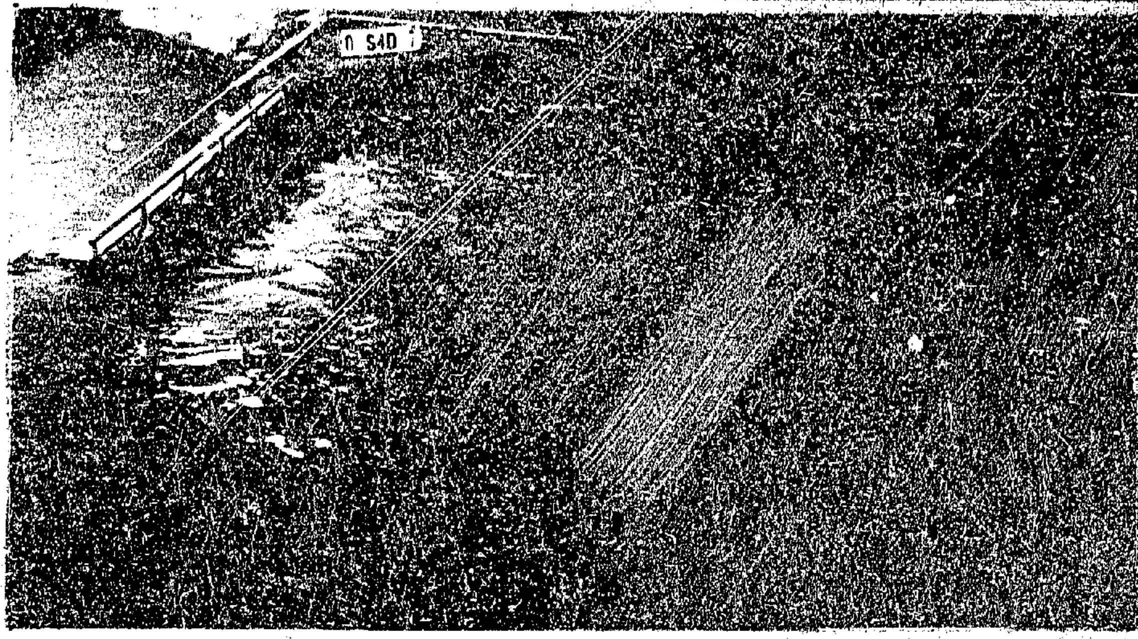
The model showed that the poor approach condition to the drop structures was due to the wing walls being at right angles to the direction of flow. This caused a drawdown or dished effect as the flow entered the panels on each side of

A. Scour After 21 Hours-Discharge 1300 Second-Feet

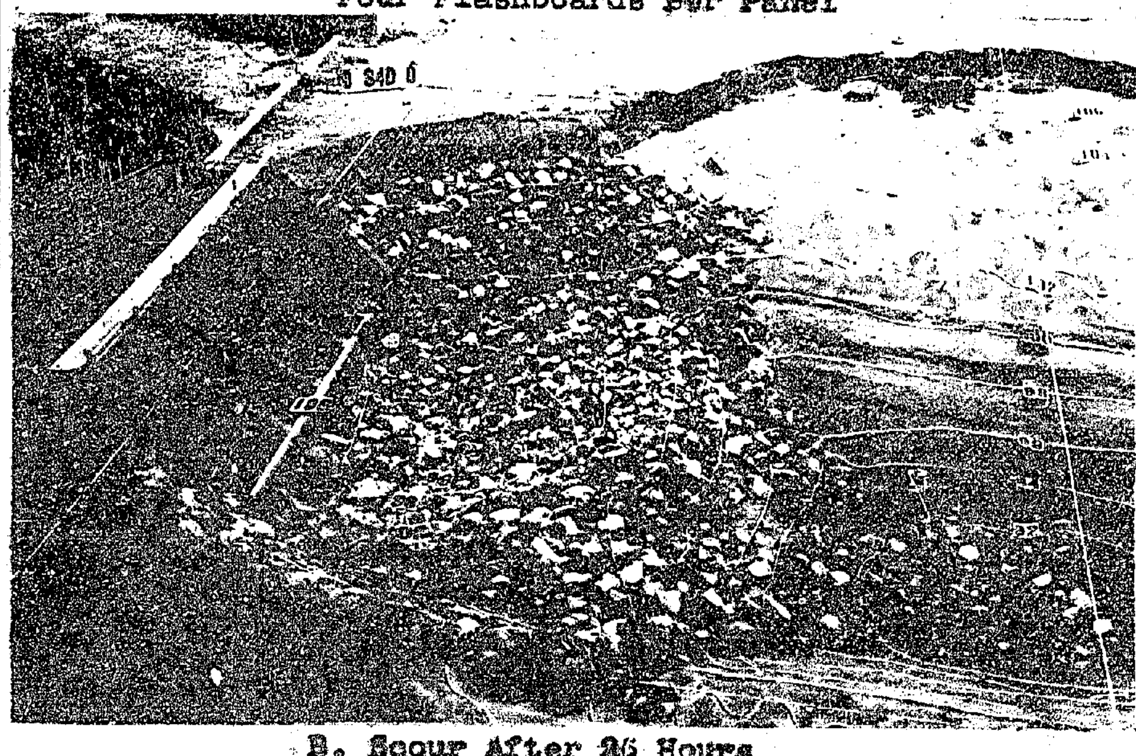


B. Scour After 2½ Hours

Figure 14. Existing Structure



A. Scour After 21 Hours-Discharge 1500 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 100.00 Equals
Prototype Elevation 877.55

Figure 15. Existing Structure

the structure (figures 15A and 26A). 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 the poor flow distribution extending downstream (figure 14A). 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, however, after the flow had reached the region of the standing wave.

A standing wave, as discussed on page 28, formed over the check basin (figures 14A and 15A), and additional waves, accompanied by high velocity flow, 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 1-1/2 to 1 side slopes formed a large eddy along each side of the canal (figure 14A). 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 9, 16, and 17. This was anticipated 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 was not excessive for a distance

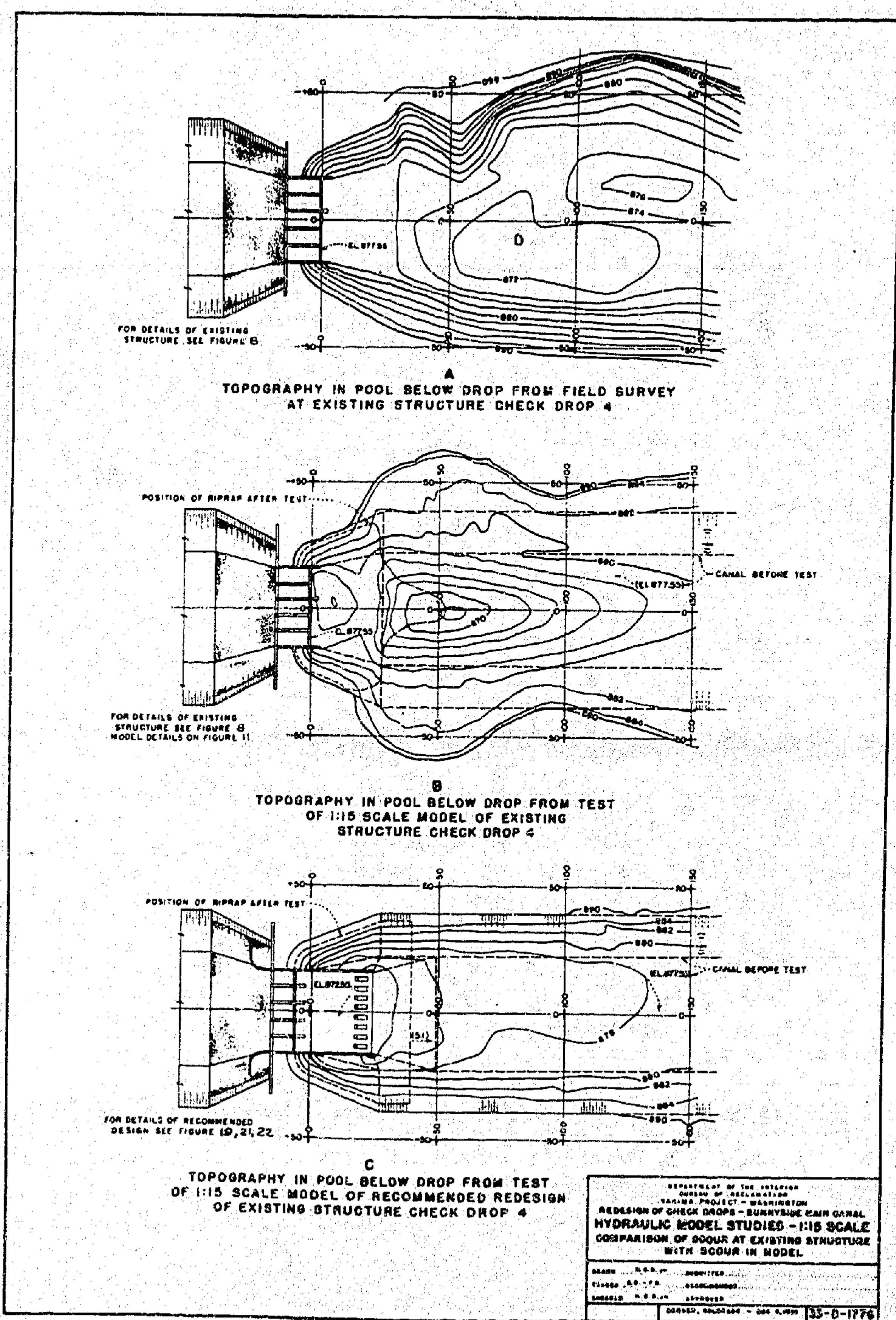
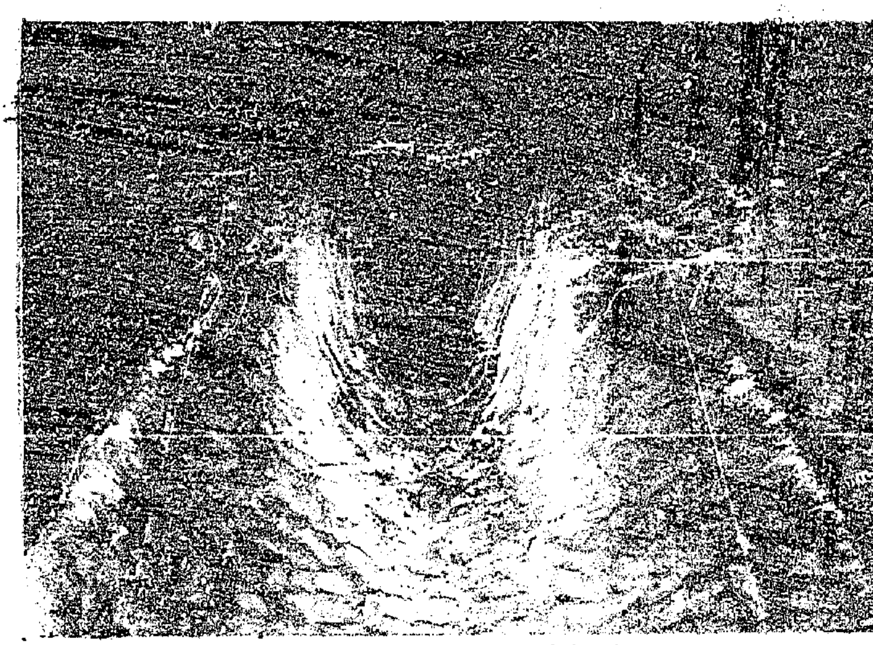

FOR DETAILS OF EXISTING STRUCTURE, SEE FIGURE 8
A
TOPOGRAPHY IN POOL BELOW DROP FROM FIELD SURVEY AT EXISTING STRUCTURE CHECK DROP 4
POSITION OF RIPRAP AFTER TEST
CANAL BEFORE TEST
FOR DETAILS OF EXISTING STRUCTURE SEE FIGURE 8
MODEL DETAILS ON FIGURE 11
B
TOPOGRAPHY IN POOL BELOW DROP FROM TEST OF 1:15 SCALE MODEL OF EXISTING STRUCTURE CHECK DROP 4
POSITION OF RIPRAP AFTER TEST
CANAL BEFORE TEST
FOR DETAILS OF RECOMMENDED DESIGN SEE FIGURE 19, 21, 22
C
TOPOGRAPHY IN POOL BELOW DROP FROM TEST OF 1:15 SCALE MODEL OF RECOMMENDED REDESIGN OF EXISTING STRUCTURE CHECK DROP 4
REDESIGN OF CHECK DROPS - SUNNYSIDE MAIN CANAL
HYDRAULIC MODEL STUDIES - 1:15 SCALE
COMPARISON OF SCOUR AT EXISTING STRUCTURE WITH SCOUR IN MODEL
33-D-1776

Figure 16

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 100.00 Equals
Prototype Elevation 877.55

Figure 17. Existing Structure

of 20 feet downstream from the drop, but beyond this point the canal bottom was heavily scoured. Observations on the model revealed that the flow as it passed over the flash-boards, 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 downstream from the drop. But, as the flow passed downstream to the end of the rip-rapped transition, high velocities prevailed along the bottom of the canal. This caused a large hole to be scoured, into which riprap near this region was deposited (figure 15B). This may explain why 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 14) is nearly symmetrical about the center line of the canal; but the scour at the prototype (figure 9) 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 at the steel brackets to divert the flow from the scoured section of the canal. The model showed definitely, however, that scour will be excessive below the drop regardless of the alinement in the canal, the compactness of the material, or the symmetry of the flashboards.

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. Every design investigated was tested with various discharges, tailwater depths, and flashboard arrangements. Careful study was made of their performance, especially at maximum discharge (1300 second-feet), since at that discharge the most unfavorable conditions existed.

Before studies could be made to eliminate the standing waves and excessive velocities, it was necessary to make the flow symmetrical through the drop structure and below the check basin.

To improve the flow through the drop structure, the steel brackets were streamlined (figure 18J), and curved training walls were added to the wing walls in the approach (figure 18B). Although the streamlining of the brackets improved the flow conditions, it was found later it would not be required. The use of training walls, however, improved the flow conditions to such an extent that they were adopted to succeeding designs investigated. The training walls eliminated the drawdown or dished effect which occurred in the flow at each side panel (figure 26A).

To improve the flow below the check basin, a rectangular stilling pool was formed by extending the vertical side walls downstream (figure 18B). This prevented the return flow of the side eddies from submerging the main stream as it left the check basin.

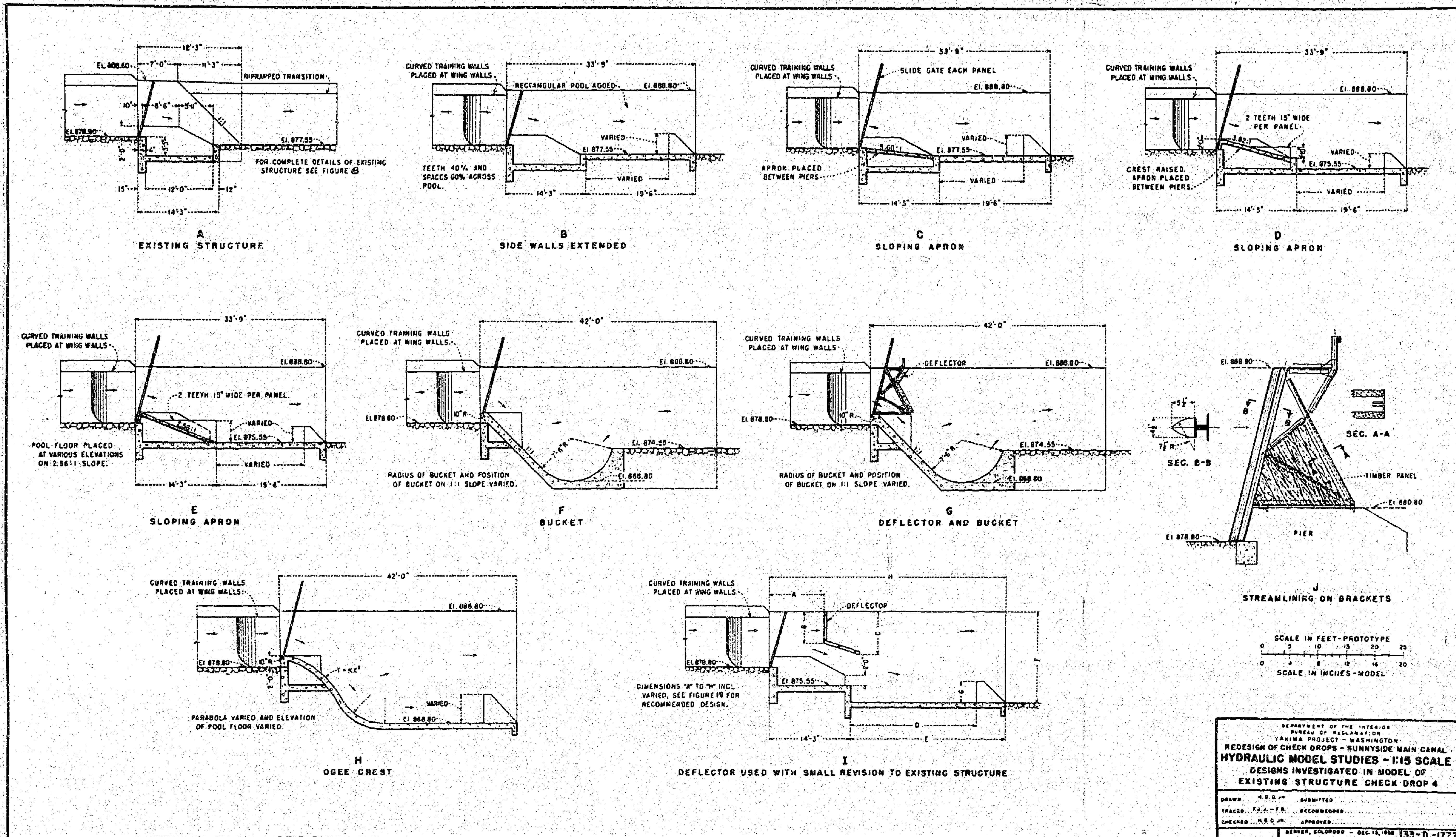

A
EXISTING STRUCTURE
RIPRAPPED TRANSITION
FOR COMPLETE DETAILS OF EXISTING STRUCTURE SEE FIGURE 8
B
SIDE WALLS EXTENDED
CURVED TRAINING WALLS PLACED AT WING WALLS
RECTANGULAR POOL ADDED
TEETH 40% AND SPACES 60% ACROSS POOL.
VARIED
C
SLOPING APRON
SLIDE GATE EACH PANEL
APRON PLACED BETWEEN PIERS
D
SLOPING APRON
2 TEETH 15" WIDE PER PANEL
CREST RAISED. APRON PLACED BETWEEN PIERS.
E
SLOPING APRON
POOL FLOOR PLACED AT VARIOUS ELEVATIONS ON 2.56:1 SLOPE.
F
BUCKET
RADIUS OF BUCKET AND POSITION OF BUCKET ON 1:1 SLOPE VARIED.
G
DEFLECTOR AND BUCKET
DEFLECTOR
J
STREAMLINING ON BRACKETS
SEC. A-A
SEC. B-B
TIMBER PANEL
PIER
H
OGEE CREST
PARABOLA VARIED AND ELEVATION OF POOL FLOOR VARIED.
I
DEFLECTOR USED WITH SMALL REVISION TO EXISTING STRUCTURE
DIMENSIONS "A" TO "H" INCL. VARIED, SEE FIGURE 19 FOR RECOMMENDED DESIGN.
SCALE IN FEET-PROTOTYPE
SCALE IN INCHES-MODEL
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
DENVER, COLORADO
33-D-1777

Figure 18

With the symmetry of the flow greatly improved through the drop structure, it was then 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 the jet, as it plunged over the flash-boards into the check basin, did not follow closely along the stilling-pool floor, but was deflected upwards by the weir wall at the downstream end of the check basin. This 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 18B) ineffective in dispersing this velocity. If the jet could be made to follow the stilling-pool floor, however, it would be possible to 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 18C 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 18C). With slide gates in place, the direction and the velocity of the jet were more nearly parallel to the apron between the piers, since the flow had to pass under the slide gates. The teeth in the stilling pool were then more effective, and it was noted that the velocities downstream from the drop were slightly less. For small discharges, slide gates in place, 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 existed.

This design, however, was improved by providing more depth of stilling pool and increasing the slope of the apron (figures 18D and 18E). 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 as a result of the added depth of pool. The standing waves still persisted, however, as in the other designs.

When it was observed 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 18F was investigated. This design provided still greater pool depth, but added a bucket. The bucket has been used successfully in many instances to dissipate energy by creating a large roller in the flow. If proper tailwater 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 cause a roller to form instead of standing waves. This was not realized, however, because at normal tailwater depth, which was necessary to prevent excessive boil, the jet failed to follow the slope into the bucket sufficiently to eliminate the standing waves; furthermore, the velocities below the pool were still excessive.

This design was improved in one respect by placing a deflector at the steel brackets (figure 18G). The deflector forced the jet to follow the slope into the bucket so that the standing waves were completely eliminated; however, the velocity of the entering jet was necessarily increased as it passed under the deflector. This caused excessive boiling above the bucket, 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 18H 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 tailwater depth. But if flashboards or slide gates were added, the trajectory of the jet was changed, causing the flow to oscillate between following the ogee crest and forming standing waves. Effective velocity reduction was still impossible with this design.

The best features of the designs just discussed were the deflector, which eliminated the standing waves, and the deeper stilling pool, which caused some 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 18I shows this revision, which was developed to give a satisfactory recommended design.

Chapter V

The Recommended Design Structure

Revisions to Check Drop No. 4

The recommended design (figures 19 and 20) requires the removal from the existing structure of the downstream weir wall of the check basin and part of the vertical side walls (figure 21). Curved training walls are added in the approach, and the vertical side walls are lengthened to form a stilling pool in 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 21 and 22). Where it is possible in the field, the canal section at the stilling pool may be built in as shown on figure 19. 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.

It should be mentioned that if hydraulic model studies could have been made to test the design of the existing check drop structures before they were built in the canal, a design would have been evolved which no doubt would have operated satisfactorily to this day. This design in all probability would have been somewhat different than the

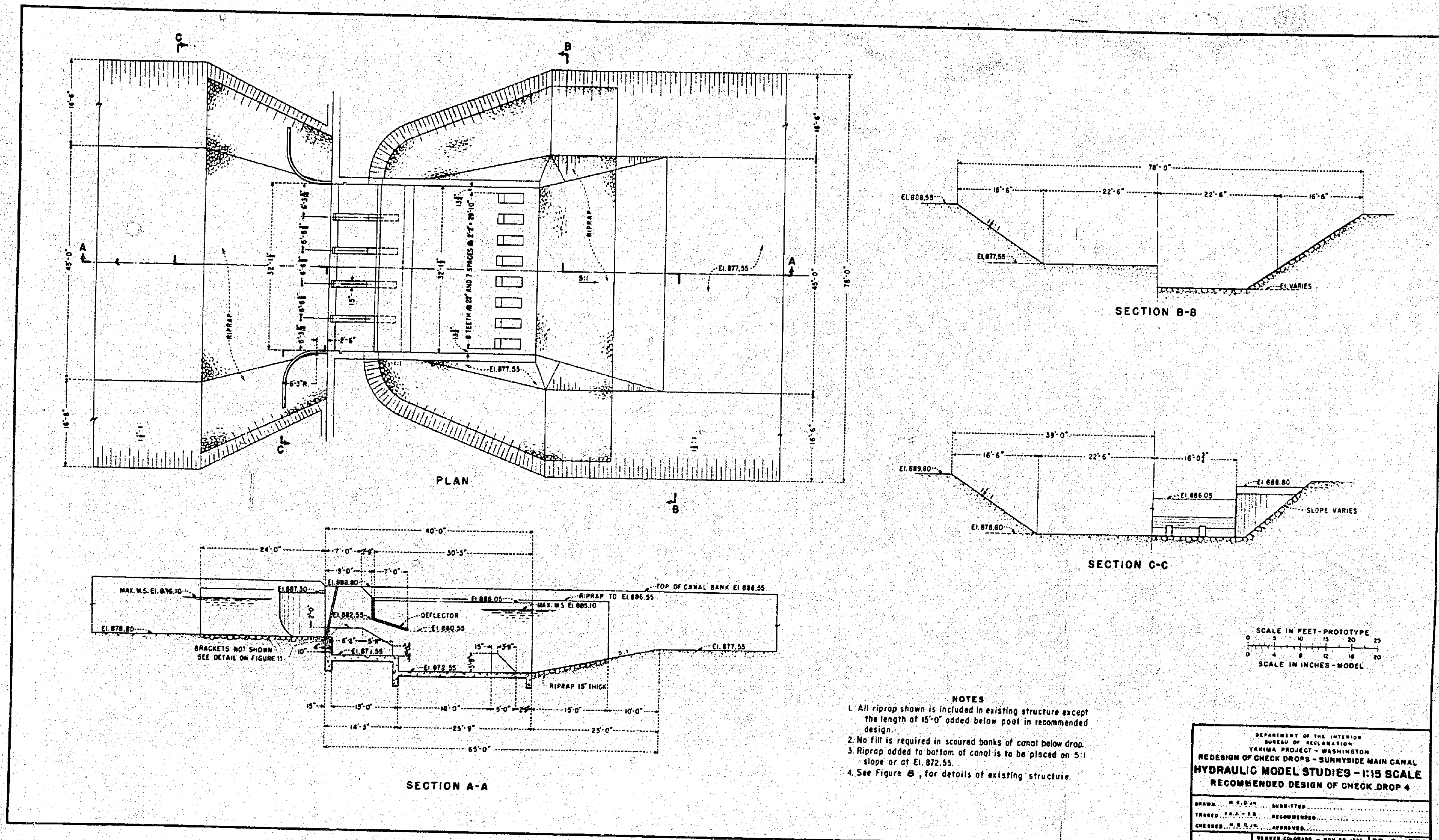
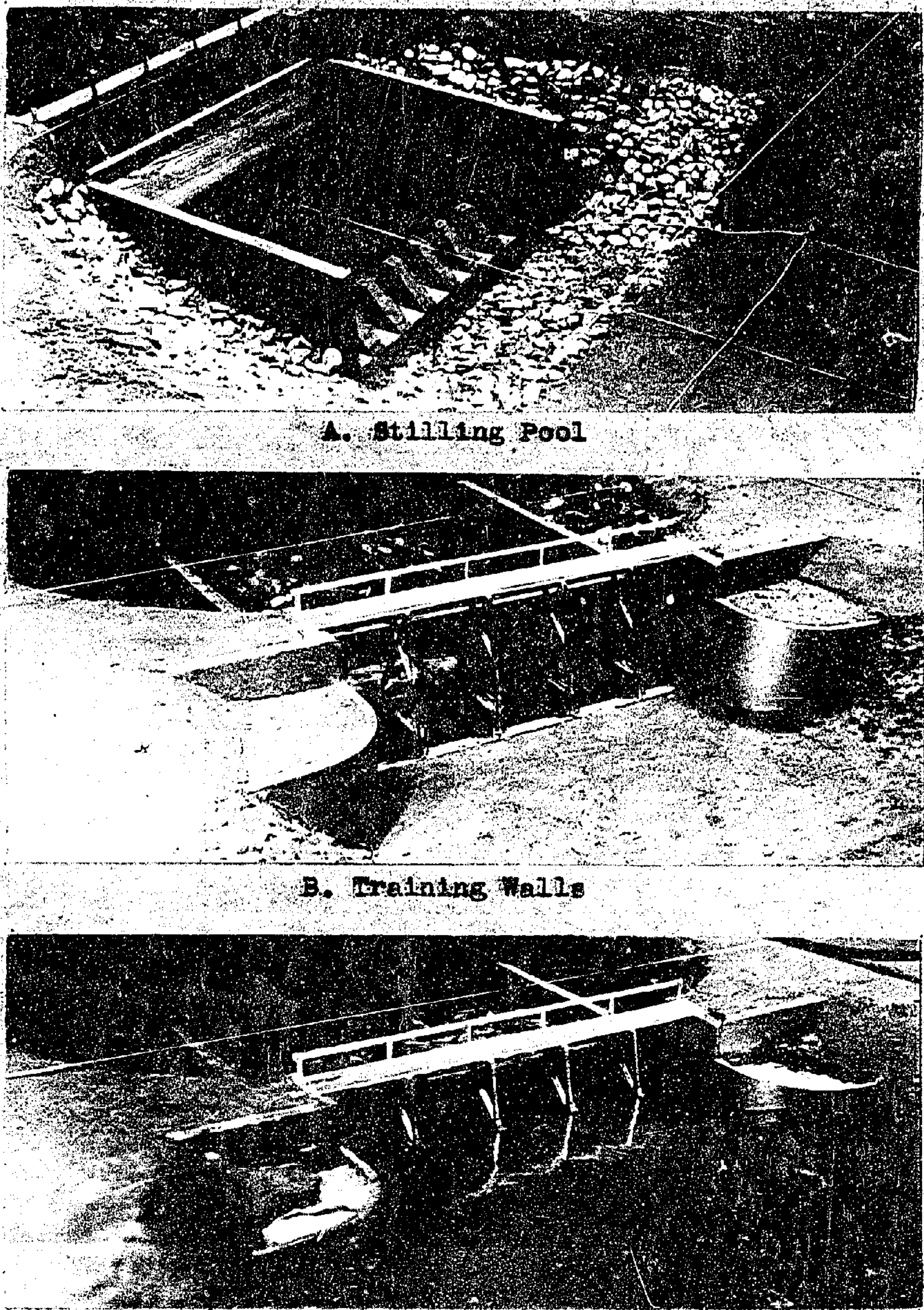

PLAN
RIPRAP
8 TEETH @ 22" AND 7 SPACES @ 2'-2 13/16"
EL. 877.55
SECTION B-B
78'-0"
16'-6"
22'-6"
EL. 888.55
EL. 877.55
EL. VARIES
SECTION C-C
39'-0"
EL. 889.80
EL. 878.80
EL. 886.05
SLOPE VARIES
SECTION A-A
40'-0"
24'-0"
30'-3"
MAX. W.S. EL. 896.10
EL. 887.30
EL. 888.80
EL. 886.05
RIPRAP TO EL. 886.55
TOP OF CANAL BANK EL. 888.55
MAX. W.S. EL. 885.10
EL. 882.55
DEFLECTOR
EL. 880.55
EL. 878.80
BRACKETS NOT SHOWN
SEE DETAIL ON FIGURE 11
EL. 871.55
EL. 872.55
EL. 877.55
RIPRAP 15" THICK
65'-0"
SCALE IN FEET-PROTOTYPE
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 8, 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... SUBMITTED...
TRACED... RECOMMENDED...
CHECKED... APPROVED...
DENVER, COLORADO
33-D-1778

Figure 19

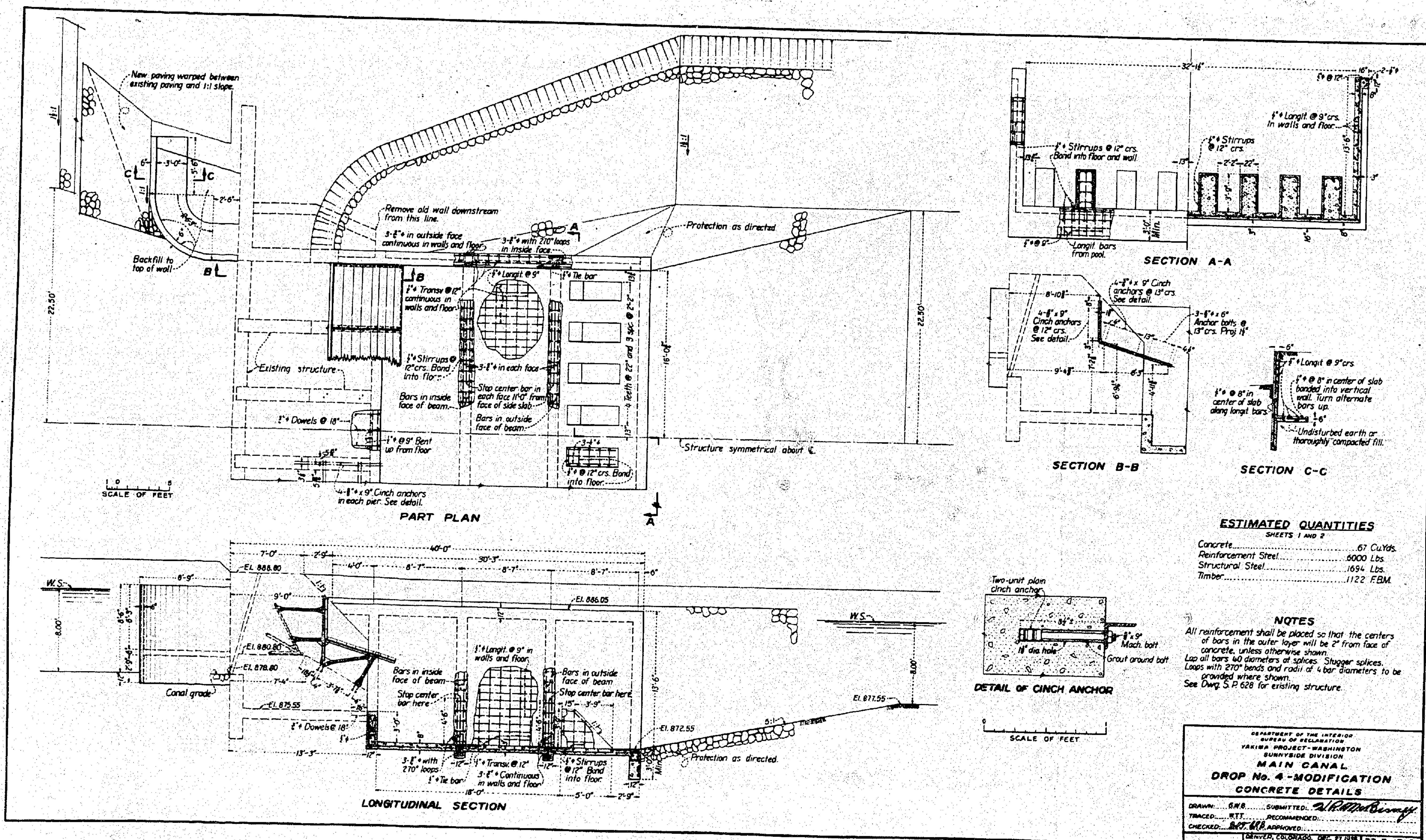


A. Stilling Pool

B. Training Walls

C, Discharge 1300 Second-Feet

Figure 20. Recommended Design



PART PLAN
SECTION A-A
SECTION B-B
SECTION C-C
LONGITUDINAL SECTION
DETAIL OF CINCH ANCHOR
SCALE OF FEET
Structure symmetrical about ℄
Protection as directed
Existing structure
Remove old wall downstream from this line.
New paving warped between existing paving and 1:1 slope.
Backfill to top of wall
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:
TRACED: R.T.T. RECOMMENDED:
CHECKED: APPROVED:
DENVER, COLORADO, DEC. 1938
SHEET 1 OF 2
33-D-1783

Figure 21

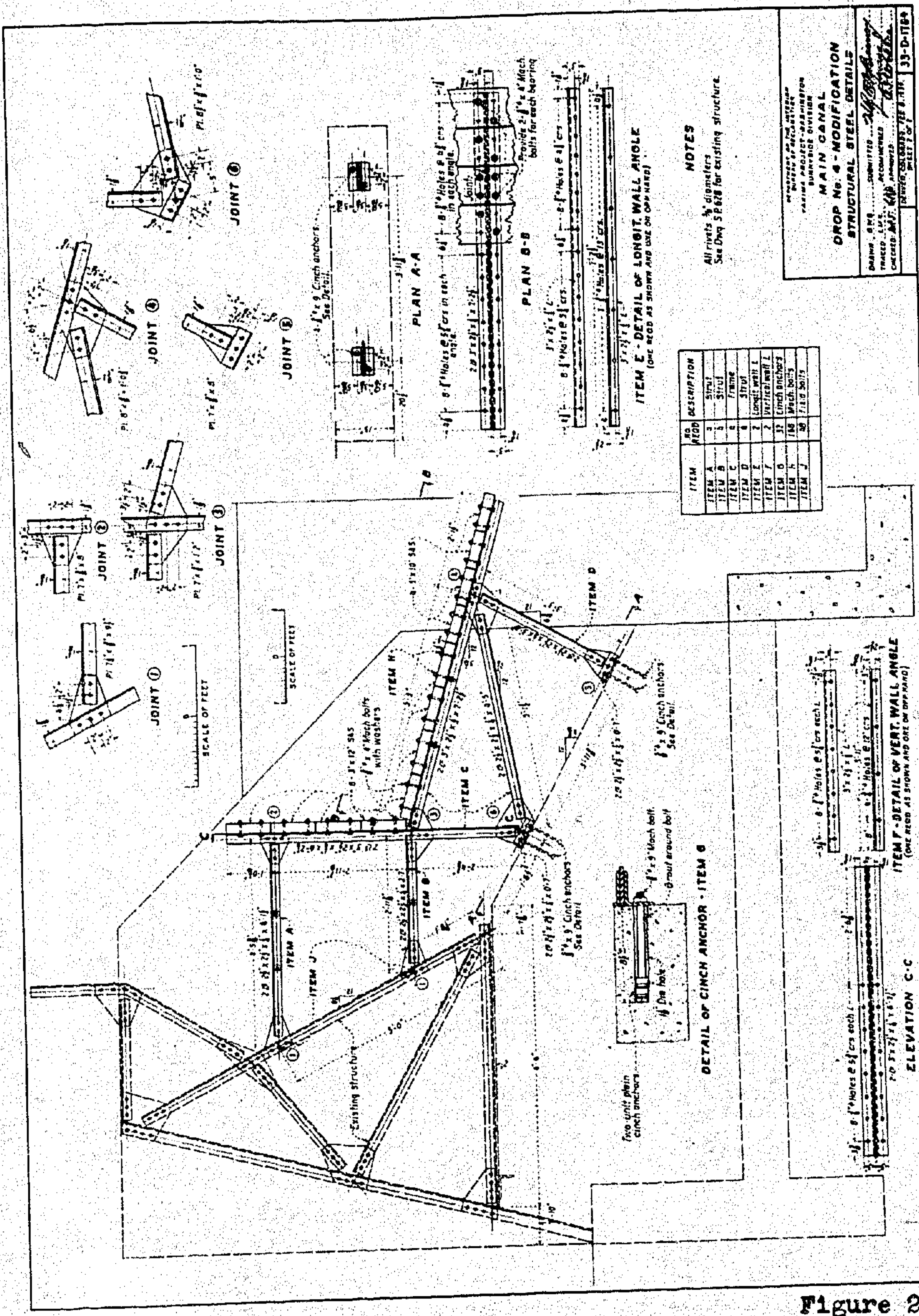

MAIN CANAL
DROP No. 4 - MODIFICATION
STRUCTURAL STEEL DETAILS
NOTES
PLAN A-A
PLAN B-B
JOINT 1
JOINT 2
JOINT 3
JOINT 4
JOINT 5
JOINT 6
ITEM E - DETAIL OF LONGIT. WALL ANGLE
DETAIL OF CINCH ANCHOR - ITEM G
ITEM F - DETAIL OF VERT. WALL ANGLE
ELEVATION C-C

Figure 22

recommended design just discussed. It was not possible, however, to investigate an entirely new design during this study, since the cost of removing the drop structures from the canal would have been excessive. Therefore, the design offered here is one which is not only economical, but one that renders a satisfactory solution to the problem for the conditions involved.

Flow Through 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 20B, 20C and 26). The curved training walls were adopted to keep the cost of revision to a minimum. Present design practice, however, usually adopts a warped transition where the section of a canal changes from trapezoidal to rectangular. This automatically provides a more uniform depth of flow at the rectangular section. Streamlining of the brackets, as previously mentioned, is not necessary, since the forward velocity at the entrance to the drop s+ructure is reduced by the deflector.

As the flow passes over the flashboards, instead of standing waves forming, the jet shoots under the deflector and follows the stilling pool floor. The deflector is so placed, moreover, that the maximum discharge can be passed at normal tailwater depths without any splashing occurring over the deflector or excessive depths forming upstream. It was determined in the model that additional discharge may be passed in the event of a cloudburst or similar cause. If 1,264 second-feet are flowing and the tailwater 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 tailwater depth is allowed to increase with an increase in 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 tailwater would be at elevation 885.13. The flashboards placed between the steel brackets 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 tailwater, and its velocities are greatly reduced by the baffle piers. It is difficult to classify the type of flow existing in the stilling pool, however, 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 flow impinging on the baffle piers; hence, 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 for this particular design is seen on figures 23A, 24A, and 25A.

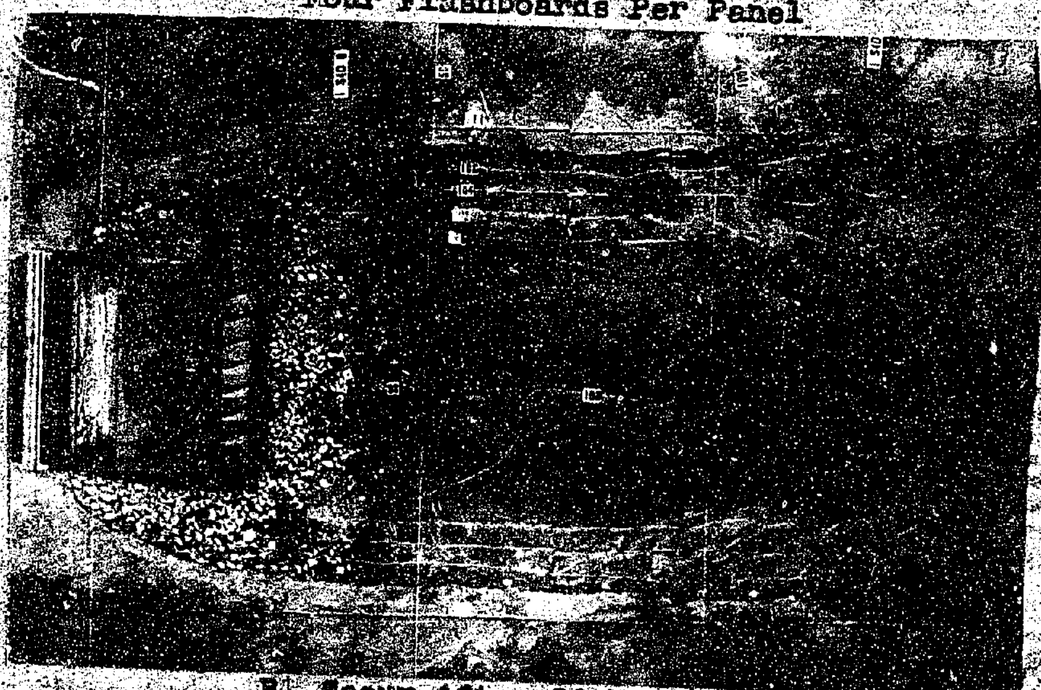
At the existing structure, the velocities are greater near the center of the canal than at the sides. This condition, it was shown, caused side eddies to develop which scoured the canal banks. Below the recommended design pool, however, the velocity distribution is uniform, and under this condition extensive side eddies do not form.

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 23A). Even these eddies 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.

<u>Scour Test</u>

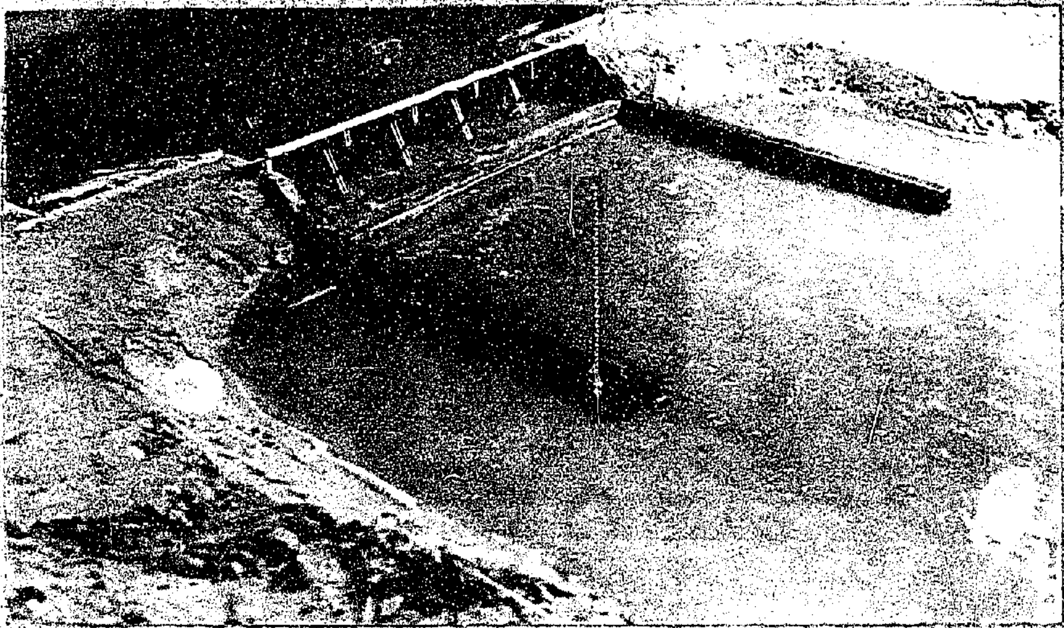
A scour test was made on this design similar to the one made on the model of the existing structure. This provided ready comparison between the performance of the two designs. However, before the canal was rebuilt in the model to its original unscoured section for this test (figure 12),

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

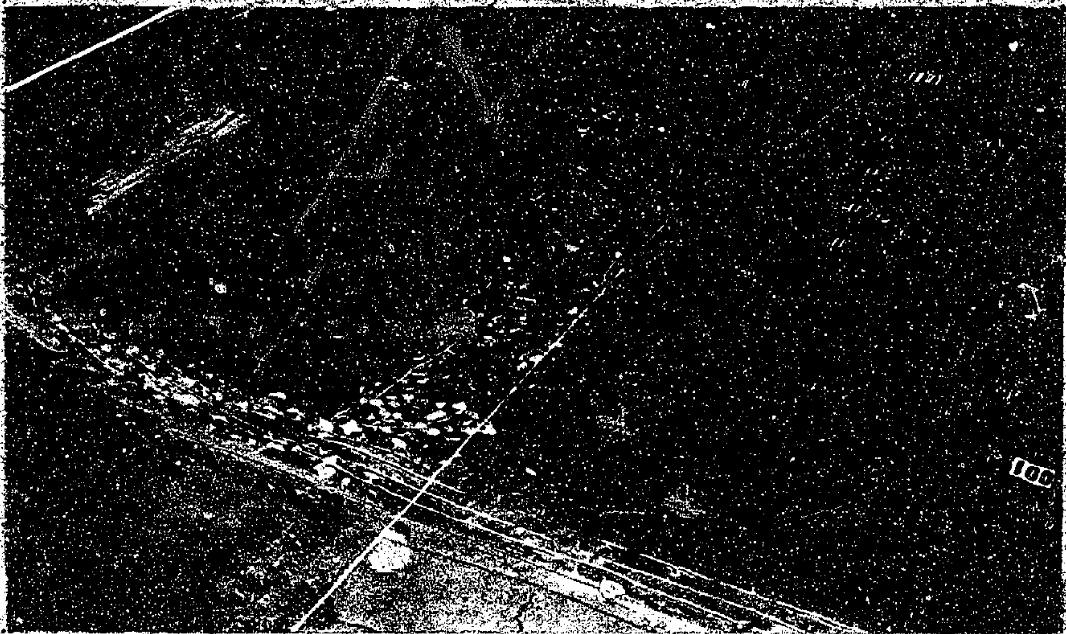


B. Scour After 26 Hours
Model Elevation 100.00 Equals
Prototype Elevation 877.55

Figure 23. 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 100.00 Equals
Prototype Elevation 877.55

Figure 28. Recommended Design

A. Scour After 21 Hours--Discharge [illegible] 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 100.00 Equals
Prototype Elevation 877.55

Figure 25. Recommended Design

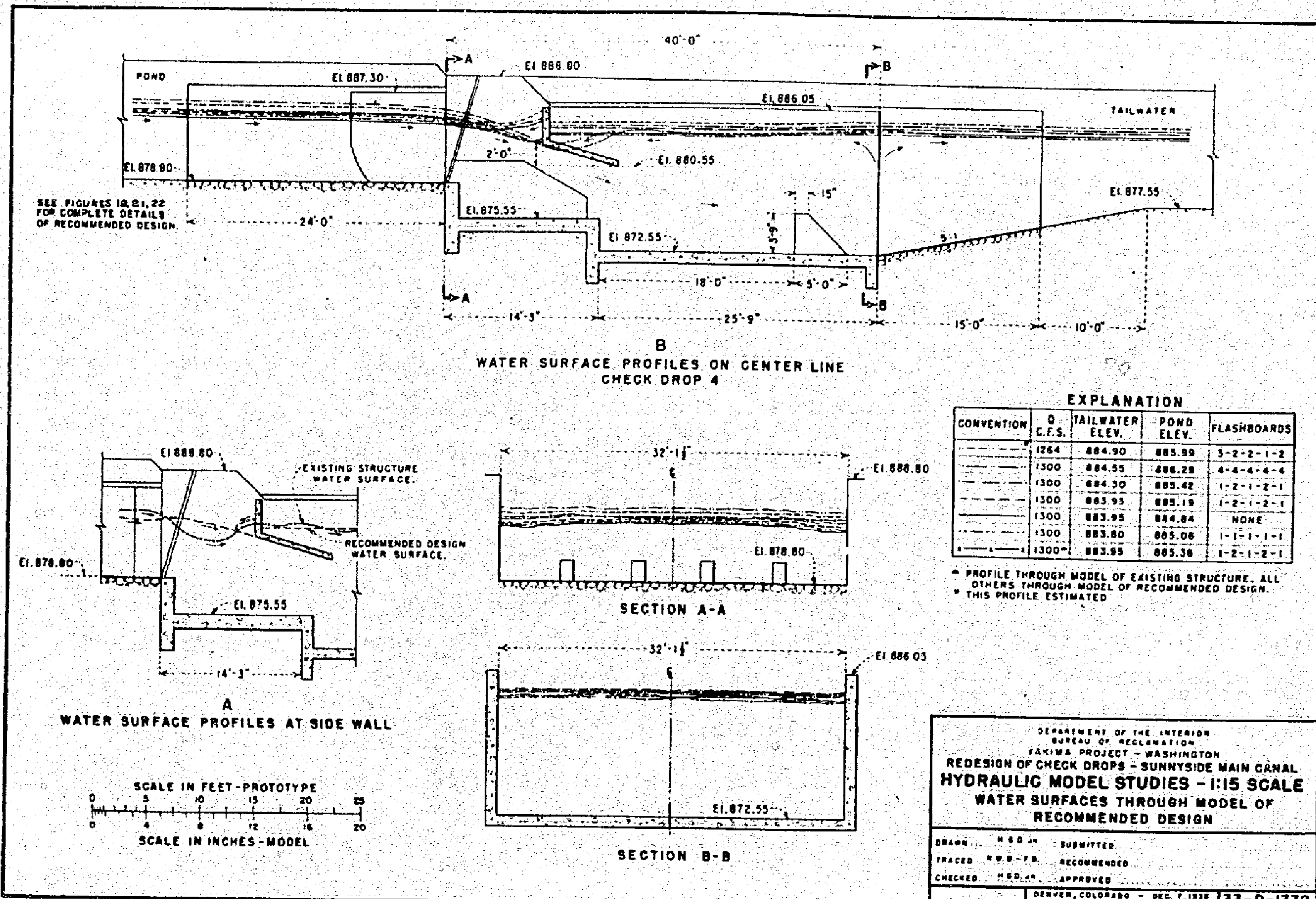

40'-0"
POND
El. 887.30
El. 888.00
El. 886.05
TAILWATER
El. 878.80
2'-0"
El. 880.55
SEE FIGURES 19, 21, 22 FOR COMPLETE DETAILS OF RECOMMENDED DESIGN.
24'-0"
El. 875.55
15"
El. 877.55
3'-9"
El. 872.55
5:1
18'-0"
5'-0"
14'-3"
25'-9"
15'-0"
10'-0"
B
WATER SURFACE PROFILES ON CENTER LINE
CHECK DROP 4
EXPLANATION
CONVENTION | Q C.F.S. | TAILWATER ELEV. | POND ELEV. | FLASHBOARDS
1264 | 884.90 | 885.99 | 3-2-2-1-2
1300 | 884.55 | 886.28 | 4-4-4-4-4
1300 | 884.30 | 885.42 | 1-2-1-2-1
1300 | 883.93 | 885.19 | 1-2-1-2-1
1300 | 883.95 | 884.84 | NONE
1300 | 883.80 | 885.06 | 1-1-1-1-1
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
El. 888.80
EXISTING STRUCTURE WATER SURFACE.
RECOMMENDED DESIGN WATER SURFACE.
El. 878.80
El. 875.55
14'-3"
A
WATER SURFACE PROFILES AT SIDE WALL
32'-1½"
El. 888.80
El. 878.80
SECTION A-A
32'-1½"
El. 886.05
El. 872.55
SECTION B-B
SCALE IN FEET-PROTOTYPE
0 5 10 15 20 25
0 4 8 12 16 20
SCALE IN INCHES-MODEL
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.B.B.-F.B. RECOMMENDED
CHECKED H.G.D. JR. APPROVED
DENVER, COLORADO - DEC. 7, 1938
33-D-1779

Figure 26

observations were made to see what effect the flow from the recommended design would have upon a scoured canal section similar to that existing in the field (figure 9). It was observed that eddies formed in the wide section of the scoured canal, but their velocity was reduced to such an extent that cutting into the banks was stopped. A tendency to scour at any other point was not observed.

After the canal had been rebuilt in the model, and the recommended design structure tested for 26 hours with a discharge representing 1300 second-feet, the resulting scour was very slight (figures 16C, 23B, 24B, and 25B). The scour in the riprapped section just downstream from the stilling pool was negligible (figure 24B; compare with figure 15B). 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 sand in the model at the start of the test and not due to excessive velocities.

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 if the flashboards were symmetrical. The prototype structure has been operated with such flashboard arrangements 6-6-4-2-2 to divert the flow away from the scoured section of the canal. The function of the stilling pool of the recommended design structure 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.

At the present time there are no data or observations on the revised Check Drop No. 4, since this structure will not be completely revised until the winter of 1940. However, since the model of the existing structure so nearly duplicated the flow and scour conditions of the prototype in the initial test, it is quite reasonable to predict that the revised structure will perform similar to the model of the recommended design. It is not unreasonable to expect this, since model studies are now universally adopted in hydraulic design to predict the performance of the prototype structure.

Conclusions

The revisions to Check Drop No. 4, in the form of a recommended design, are directly applicable to other check drop structures in the Sunnyside Main Canal. The use of a deflector, rectangular stilling pool, and baffles or dentates within a stilling pool, may be applicable to other small drop structures in Group III, if the flow conditions are similar to those in the Sunnyside Main Canal.

In general, the author believes that if submergence occurs at small drop structures, and as a result scouring is excessive in a canal, the unfavorable conditions may be eliminated by adapting to a drop structure the three features just mentioned.

As a result of this study, and from observations made in the field, it is apparent that the design of small drop structures could be greatly improved. Accordingly, the following recommendations are made in hope that they will help to improve not only the designs of drop structures included in Group III, but also those drop structures included in Group I and Group II:

1. When the section of a canal or channel changes from trapezoidal to rectangular or vice-versa, it is desireable to use a warped transition. The transition, however, must be long enough to prevent the change in cross-section from being too abrupt. If this is not possible when the section changes from trapezoidal to rectangular in the direction of flow, place curved training walls at the entrance to the rectangular

section, or extend wing walls upstream which will diverge from the rectangular section.

2. All drop structures should be so designed and so operated that the flow passing through them will be symmetrical or uniformly distributed. Avoid the use of unsymmetrical gate openings or flashboard arrangements, otherwise unbalanced flow conditions will exist in the stilling pool and canal section downstream. As a result, eddies will form which may cause abnormal scour in the canal.

3. Avoid the use of a hydraulic jump or similar energy dissipator in stilling pools of expanding section or trapezoidal section. A rectangular stilling pool should be used.

4. In planning an irrigation system, allow sufficient length of reach between successive drop structures in a canal, so that enough head or energy will be available to permit the formation of a hydraulic jump at the drop structure.

5. During the design of any drop structure, a study should be made of the energy loss and velocity reduction in the stilling pool by referring to a specific diagram. The procedure followed in Chapter III may be taken as an example.

6. Model studies have proven to be an invaluable aid in hydraulic design. It is recommended that whenever possible, model studies should be made to solve hydraulic problems which are not readily solved by analytical consideration. It is obviously better to rely on model behavior, than to depend on untried facts.

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