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UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL TESTS OF WASH OVERCHUTES AND WASTEWAYS - GILA GRAVITY MAIN CANAL GILA VALLEY PROJECT

Hydraulic Laboratory Report No. 185

ENGINEERING LABORATORIES BRANCH

DESIGN AND CONSTRUCTION DIVISION
DENVER, COLORADO

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UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Branch of Design and Construction
Engineering and Geological Control
and Research Division
Denver, Colorado
October 22, 1945

Laboratory Report No. 185
Hydraulic Laboratory

Compiled by: J. H. Douma
Reviewed by: J. W. Ball

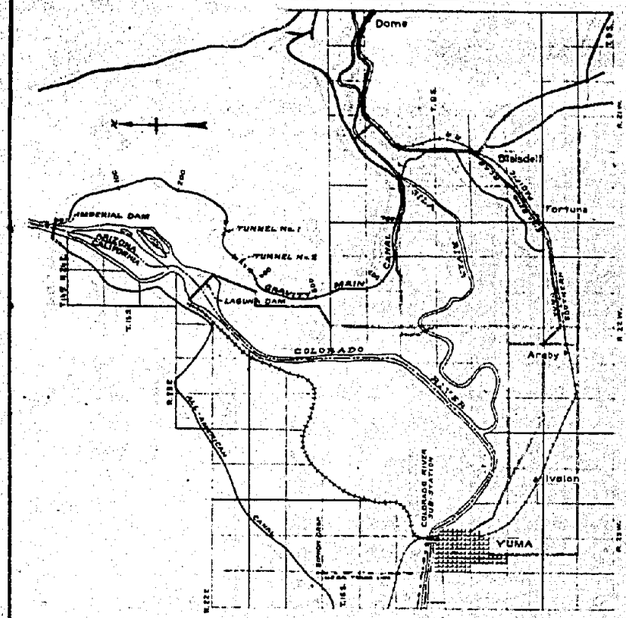
Subject: Hydraulic Model Tests of Wash Overchutes and Wasteways - Gravity Main Canal - Gila Valley Project.

INTRODUCTION

1. The prototype.

The Gravity Main Canal of the Gila Valley Project heads at Imperial Dam on the Colorado River fourteen miles northeast of Yuma, Arizona (figure 1). From the dam the course of the canal moves southeastward for approximately three and one-half miles, then turns southwest to a point near the old North Gila Canal one and one-half miles due south of Laguna Dam. From this point it moves in a southerly direction, turns due east, crosses the Gila River and circles again to the south, crossing the Fortuna Wash to move southwest through the Gila Valley.

The canal intersects numerous washes which remain dry except for run-off during severe rains when they flow heavily laden with detritus. Several types of structures were provided to handle the flow from these washes. Small concrete chutes or drainage inlets spill the flow from small drainage areas directly into the canal, while siphons carry the canal flow under the wash channels or large flumes bridge the canal to convey the floodwaters across it when the drainage areas are large. Automatic wasteways or spillways are placed at various points along the canal to handle the excess water from the drainage inlets during the



LOCATION MAP

KEY MAP

SCHEDULE No. 1
ESTIMATED QUANTITIES PLUGS, DIKES AND LINING

CANAL STATIONS	EXCAVATION PLUGS (C.Y.)	EXC. DRAINAGE DIKES (C.Y.)	COMPACTED LINING (C.Y.)	UNCOMPACTED LINING (C.Y.)
30+	1550	490		
33+51 - 34+62			660	120
37+14 - 38+48			960	150
67+50 - 69+50	4360	620		
108+57 - 110+25	3500	550		
166+04 - 169+00	6550	1700		
224+28 - 226+17	3930	590		
258+52 - 261+21	6010	610		
275+10 - 281+98			3410	730
285+30 - 295+10			8860	1140
325+40 - 328+00			3050	300
334+00 - 334+50			800	2000
TOTAL	25900	4560	17740	4440

SCHEDULE No. 2
ESTIMATED QUANTITIES PLUGS, DIKES AND LINING

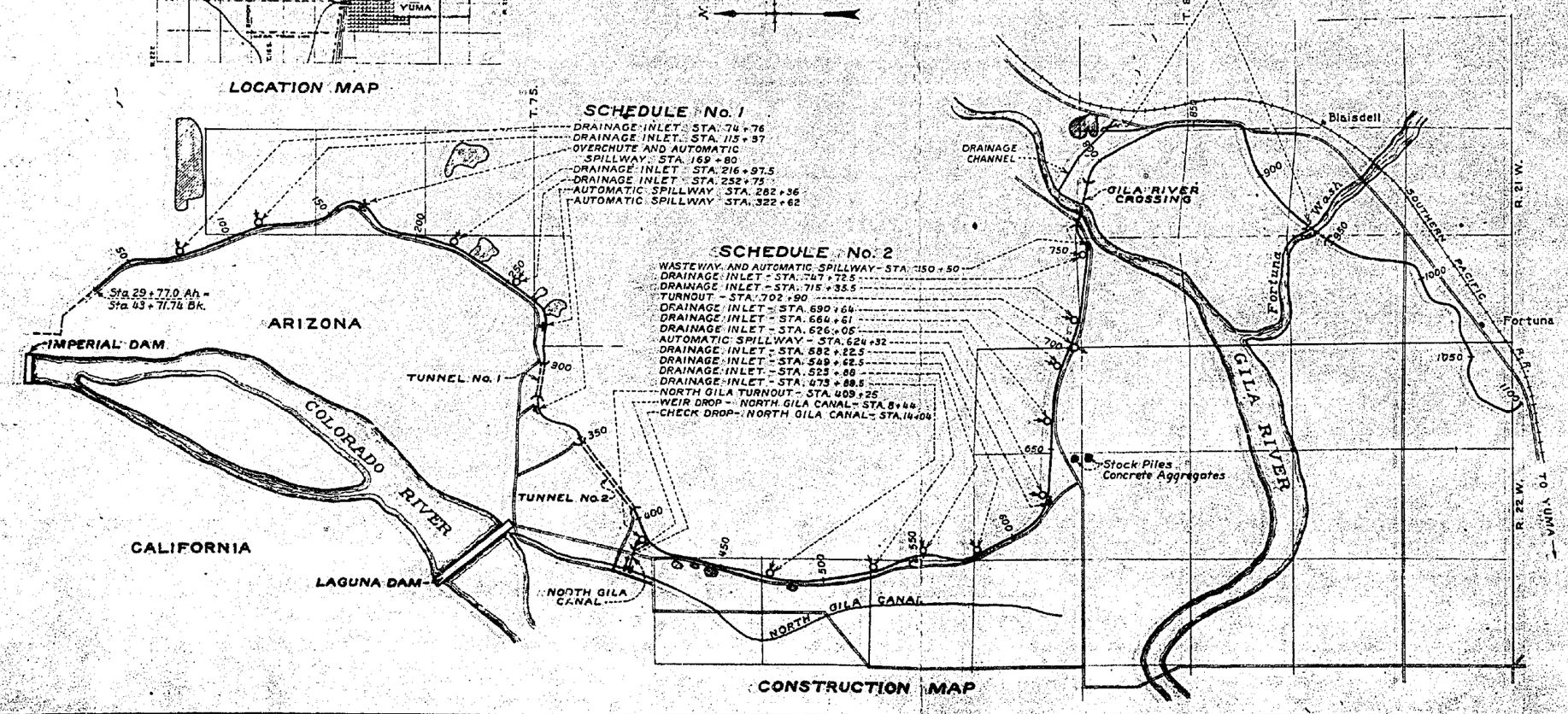
CANAL STATIONS	EXCAVATION PLUGS (C.Y.)	EXC. DRAINAGE DIKES (C.Y.)	COMPACTED LINING (C.Y.)	UNCOMPACTED LINING (C.Y.)
431+10 - 433+85	1770		2380	290
439+20 - 440+24			660	120
450+95 - 454+15	9300		1350	220
470+45 - 472+40	6130	410		
485+40 - 487+13			990	200
490+45 - 493+30	3360		1480	210
499+96 - 503+30	8680		1050	190
507+45 - 510+60	5190		980	170
521+50 - 522+24			470	80
522+96 - 524+03			590	120
525+20 - 527+68			1850	290
527+65 - 529+50	7160	700		
533+47 - 536+54			1850	350
550+95 - 552+92	3500	590		
556+63 - 560+38			2270	430
573+96 - 576+22	4610	1980		
623+63 - 625+00	4240			
677+15 - 679+00	13890	2040		
702+32 - 704+00	6580	1860		
712+09 - 713+00	3560	620		
730+60 - 732+10	2230	790		
744+02 - 745+10	1500	260		
759+15 - 760+00	2320			
North Gila Canal	2000			
TOTAL	80080	9250	15900	2670

(C) ---- Indicates borrow areas for compaction material

SCHEDULE No. 3
DRAINAGE CHANNEL CHUTE DROP - STA. 820+
FORTUNA WASH SIPHON - STA. 936+40

SCHEDULE No. 1
DRAINAGE INLET - STA. 74+76
DRAINAGE INLET - STA. 115+97
OVERCHUTE AND AUTOMATIC SPILLWAY - STA. 169+80
DRAINAGE INLET - STA. 216+97.5
DRAINAGE INLET - STA. 252+75
AUTOMATIC SPILLWAY - STA. 282+36
AUTOMATIC SPILLWAY - STA. 322+62

SCHEDULE No. 2
WASTEWAY AND AUTOMATIC SPILLWAY - STA. 150+50
DRAINAGE INLET - STA. 687+72.5
DRAINAGE INLET - STA. 715+35.5
TURNOUT - STA. 702+90
DRAINAGE INLET - STA. 690+64
DRAINAGE INLET - STA. 664+61
DRAINAGE INLET - STA. 626+05
AUTOMATIC SPILLWAY - STA. 624+32
DRAINAGE INLET - STA. 582+22.5
DRAINAGE INLET - STA. 549+62.5
DRAINAGE INLET - STA. 523+68
DRAINAGE INLET - STA. 473+89.5
NORTH GILA TURNOUT - STA. 409+25
WEIR DROP - NORTH GILA CANAL - STA. 8+44
CHECK DROP - NORTH GILA CANAL - STA. 14+04



CONSTRUCTION MAP

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
GILA PROJECT, ARIZONA
GRAVITY MAIN CANAL - STA. 30+00 TO STA. 946+70
LOCATION AND CONSTRUCTION MAP

DRAWN: T.F.R. SUBMITTED:
TRACED: R.R.L. RECOMMENDED:
CHECKED: APPROVED:
DENVER, COLORADO, JULY 10, 1958

SCD-915

period the washes are in flood stage and to by-pass the flow in case it becomes necessary to unwater the canal for repairs or other reasons. It was to check the adequacy of the proposed designs of the overchute flumes and the automatic wasteways that model studies were instigated in 1937.

2. Overchute Structure at Station 169+80.

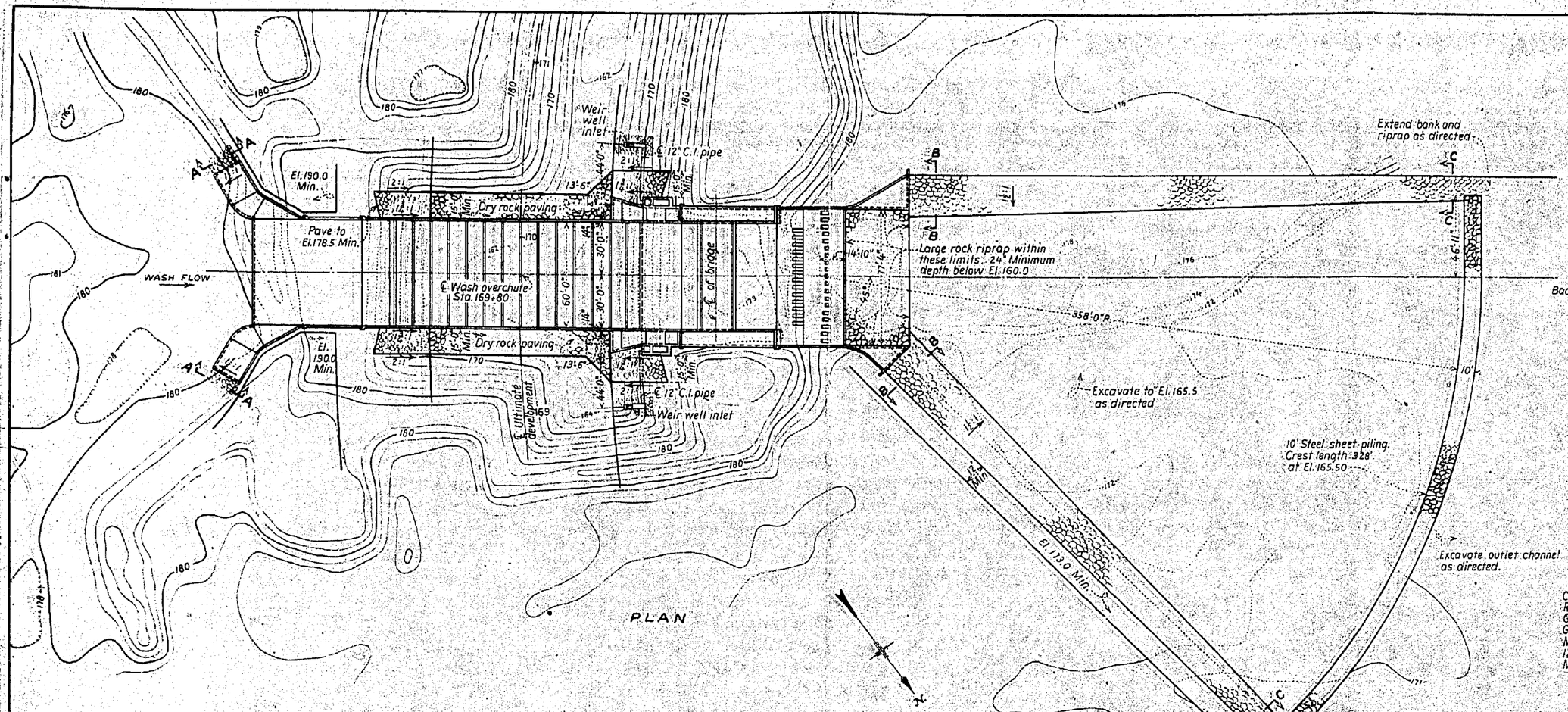
The studies concerned mainly the structure at Station 169+80 of the Gravity Main Canal (figure 2) but were expanded to include other similar structures. The originally proposed overchute structure was to handle an estimated maximum wash flood of 2,500 second-feet, passing it over the canal without interfering with normal operation. The structure was essentially a rectangular flume with an inlet transition on the upper side of the canal and terminating in a concrete stilling basin on the lower side.

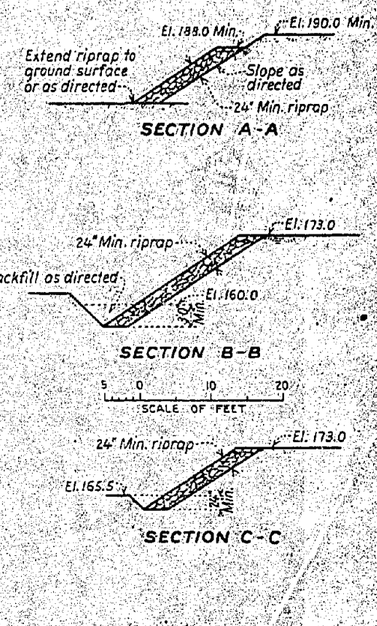
After completion of the construction of a model of this design the plans were changed to incorporate a 6-foot by 7-foot 6-inch rectangular wasteway on each side of the overchute to allow wasting of the main canal water when this becomes necessary. The wasteways terminate in the same stilling basin as the overchute and are designed to waste 750 second-feet each with the maximum canal water surface at elevation 177.47. The normal discharge of the main canal at a normal water surface, elevation 175.97, at Station 169+80 is 6,000 second-feet. Flow through the wasteways is controlled by automatically operating radial gates.

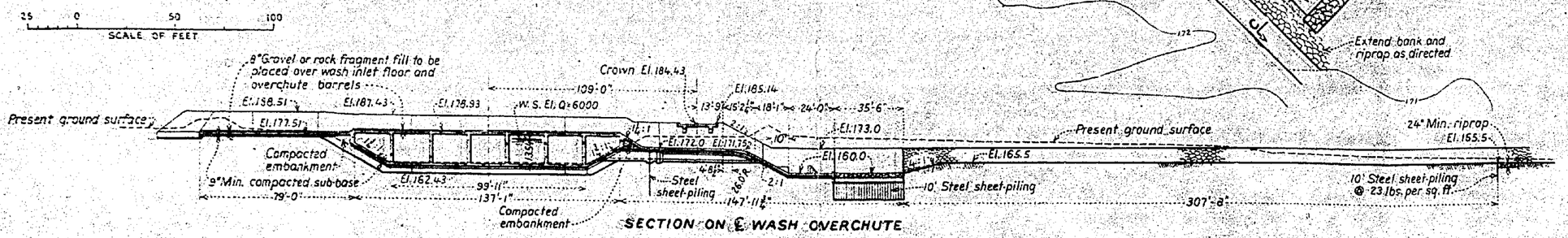
Due to insufficient pump capacity, inclusion of the wasteways in the 1:18 scale model (figure 3) was impossible. A second model to a scale of 1:24, including the wasteways, was designed, constructed, and tested (figure 4).

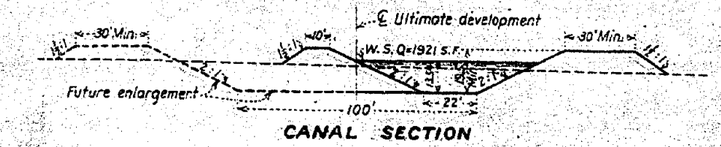
Since a number of structures similar to the original design without wasteways were being designed by the Bureau of Reclamation, general hydraulic studies were made of this model. A report of these studies

FIGURE 2









ESTIMATED QUANTITIES

Concrete	2,425 Cu.Yds.
Reinforcement steel	410,000 Lbs.
Gates	7,100 Lbs.
Gate hoists and controls	14,100 Lbs.
Miscellaneous metal work	2,500 Lbs.
12" C.I. Pipe	5,500 Lbs.
Metal pipe in reverse filters	4,000 Lbs.

REFERENCE DRAWINGS

PLAN AND SECTIONS	50-D-620
BARREL - INTERIOR UNITS	50-D-621
BARREL - INLET END UNIT - PLAN AND SECTIONS	50-D-622
BARREL - INLET END UNIT - REINFORCEMENT	50-D-623
BARREL - OUTLET END UNIT - PLAN AND SECTIONS	50-D-624
BARREL - OUTLET END UNIT - OVERCHUTE REINF.	50-D-625
BARREL - FLOAT WELLS AND GATE SECTION REINF.	50-D-626
WASH INLET	50-D-627
APRON, CHUTE AND STILLING POOL	50-D-628
BRIDGE ACROSS WASH CHANNEL	50-D-629
WASH OUTLET TRANSITIONS	50-D-630
COUNTERWEIGHTS AND JOINT DETAILS	50-D-631
CONSTRUCTION SECTIONS FOR BARRELS	50-D-632
WEIR (WELL INLETS	50-D-633
TIMBER AND PIPE RAILINGS	50-D-634
7'6"x6'0" TOP SEAL RADIAL GATE	50-D-611, 612

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
GILA PROJECT - ARIZONA

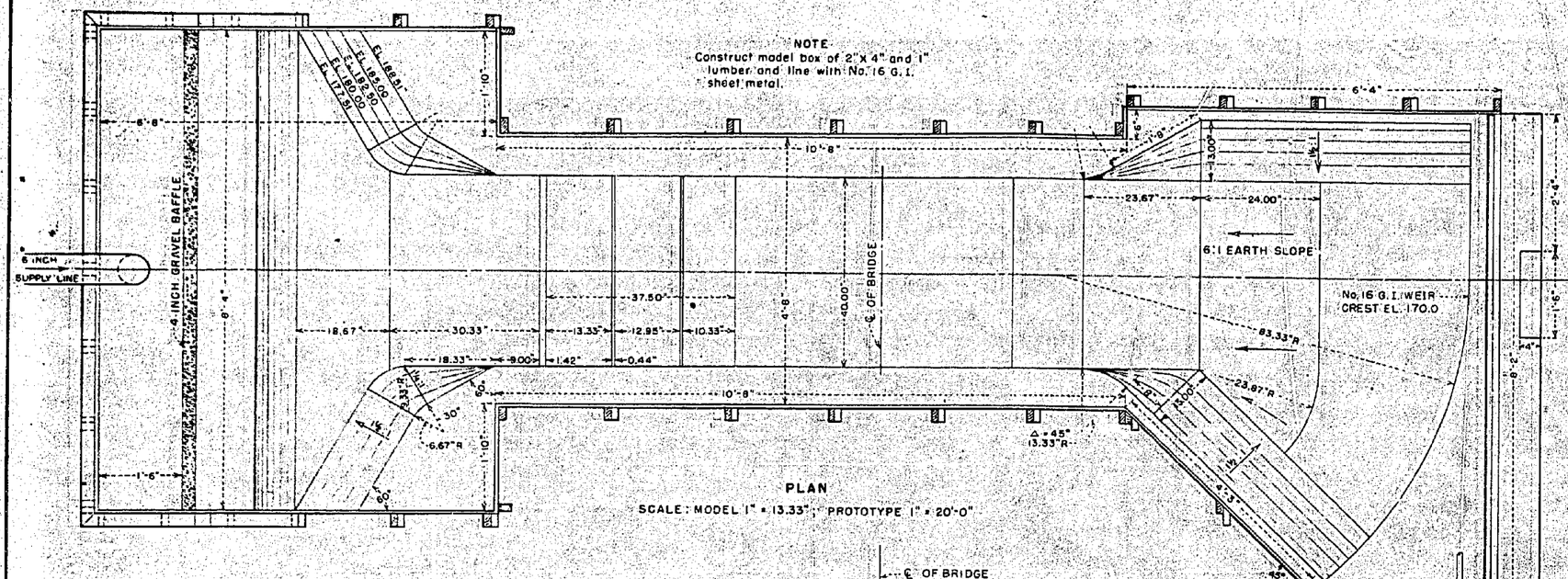
GRAVITY MAIN CANAL - STA. 169+80 OVERCHUTE AND AUTOMATIC SPILLWAY GENERAL PLAN AND TOPOGRAPHY

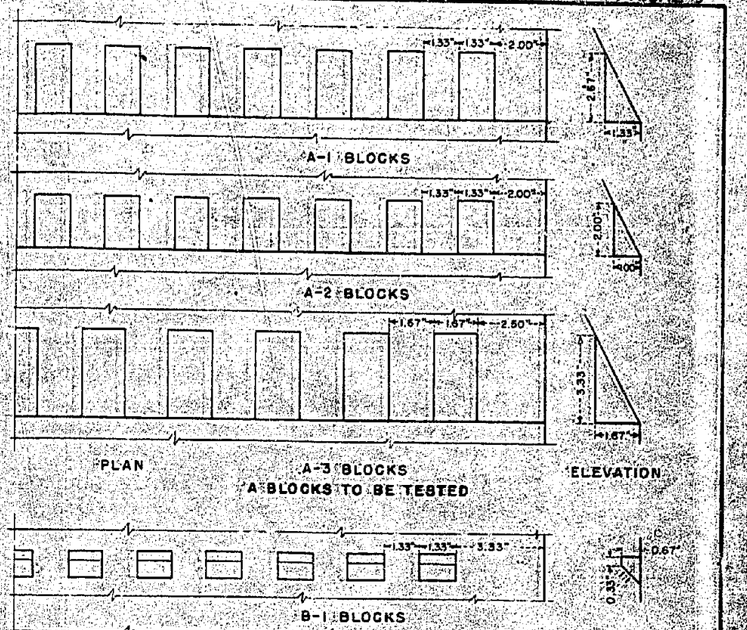
DRAWN: W.E.S. SUBMITTED:
TRACED: C.A.M. RECOMMENDED:
CHECKED: APPROVED:

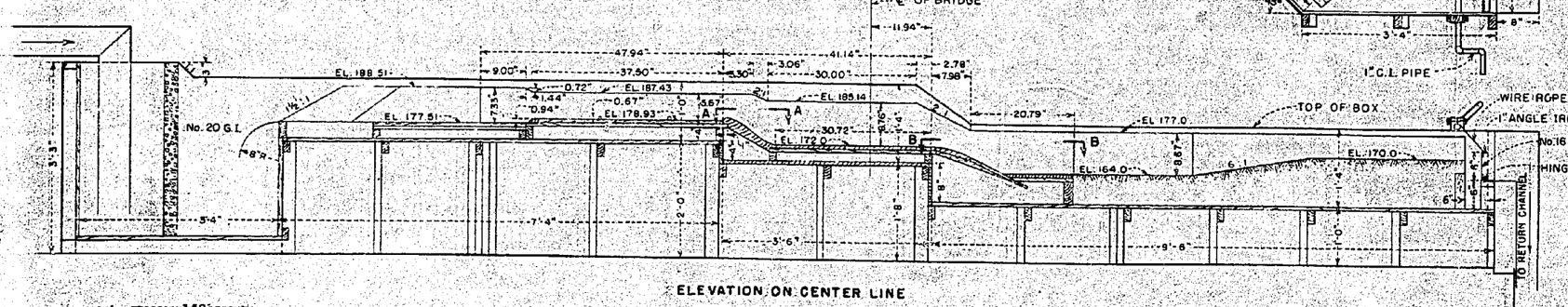
DENVER, COLORADO, DEC. 6, 1937 | 50-D-619

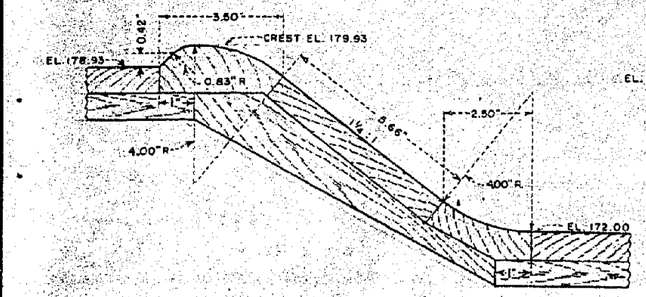
FIGURE 3

NOTE
Construct model box of 2"x 4" and 1" lumber and line with No. 16 G.I. sheet metal.

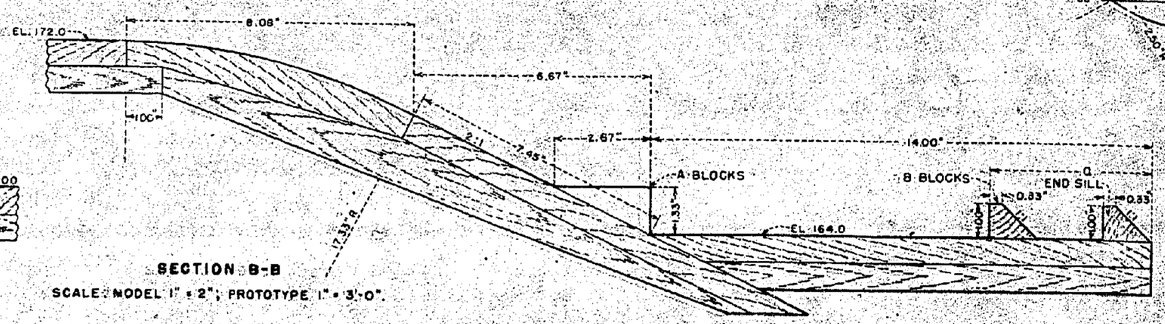




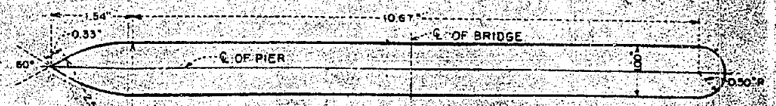




SECTION A-A
SCALE: MODEL 1" = 2"; PROTOTYPE 1" = 3'-0".



SECTION B-B
SCALE: MODEL 1" = 2"; PROTOTYPE 1" = 3'-0".



PLAN OF BRIDGE PIER
SCALE: MODEL 1" = 5.33", PROTOTYPE 1" = 2'-0".

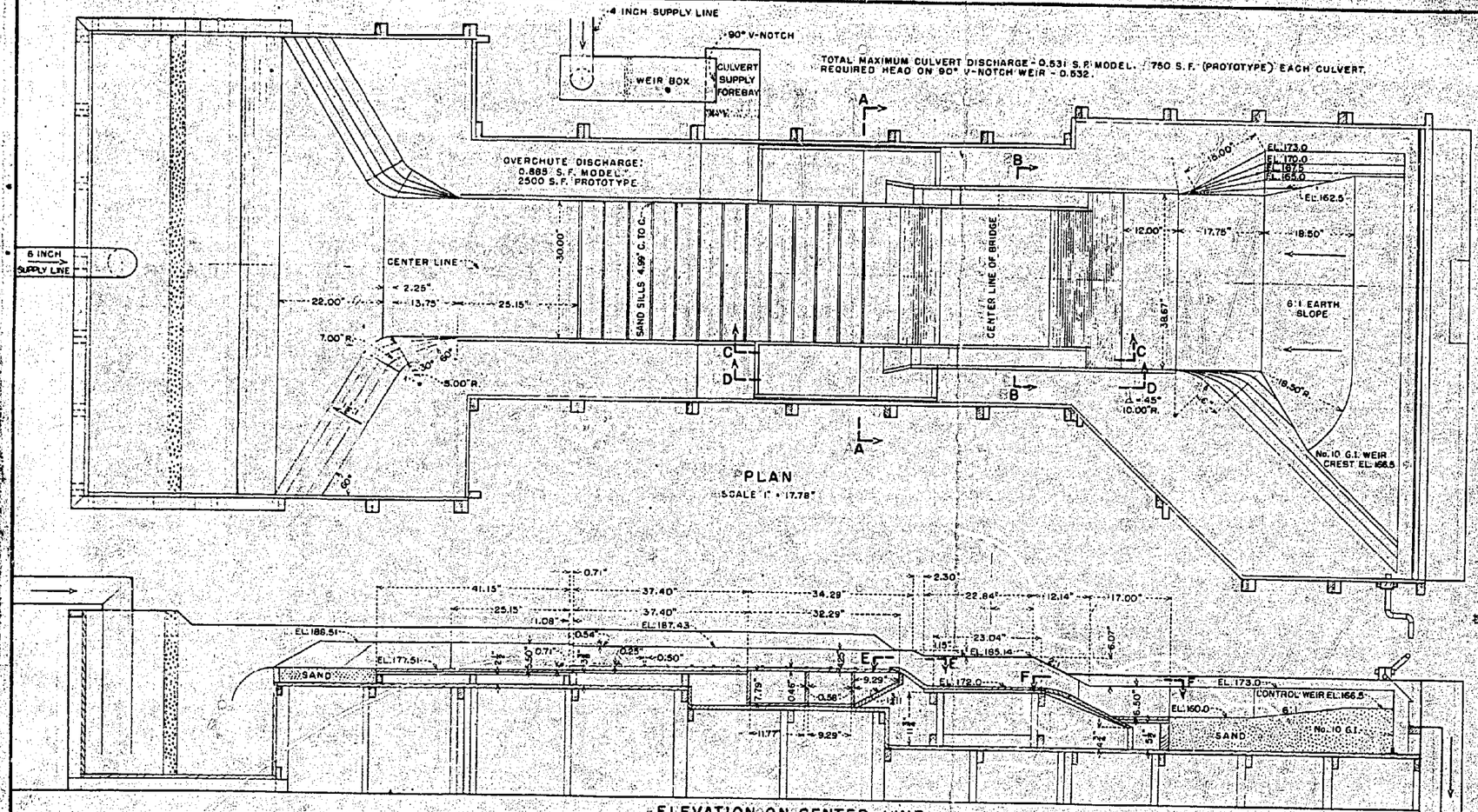
NOTE
Make pier 8" high.

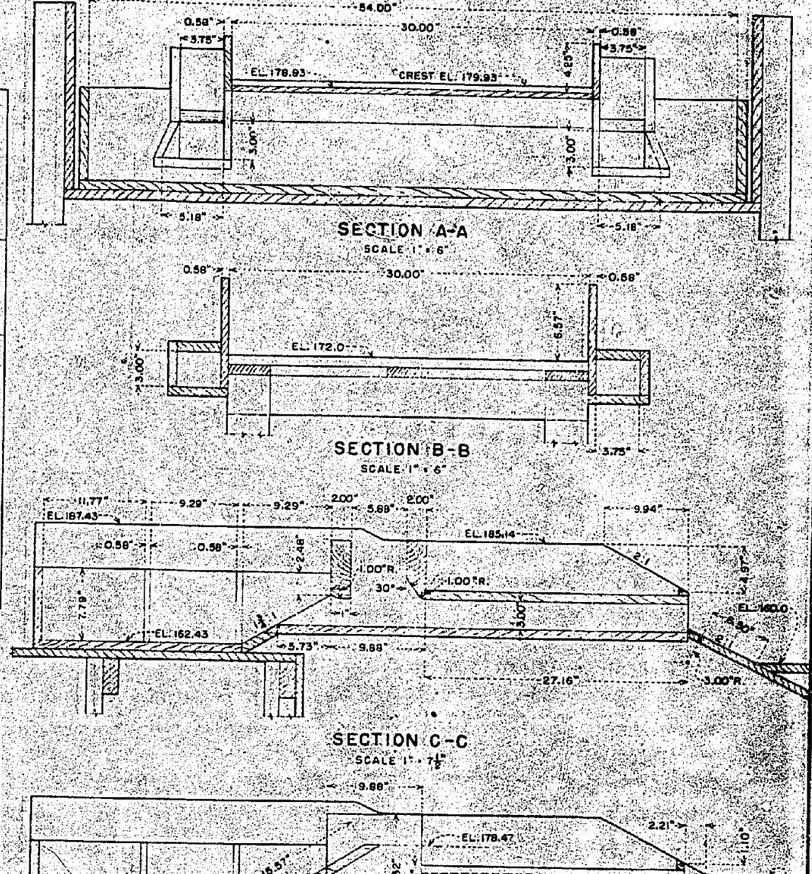
NOTES
1. Test end sills of the same cross-section as the B blocks.
2. Test stilling pool with and without B blocks.
3. Determine best "a" distance from tests.

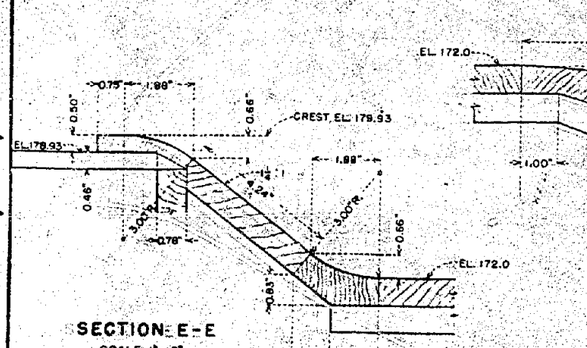
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
GILA VALLEY PROJECT - ARIZONA
GRAVITY MAIN CANAL - STATION 169+80
WASH OVERCHUTE
HYDRAULIC MODEL STUDIES
GENERAL MODEL LAYOUT
MODEL SCALE 1:18 Q_m = 1.82 S.F. Q_p = 2500 S.F.

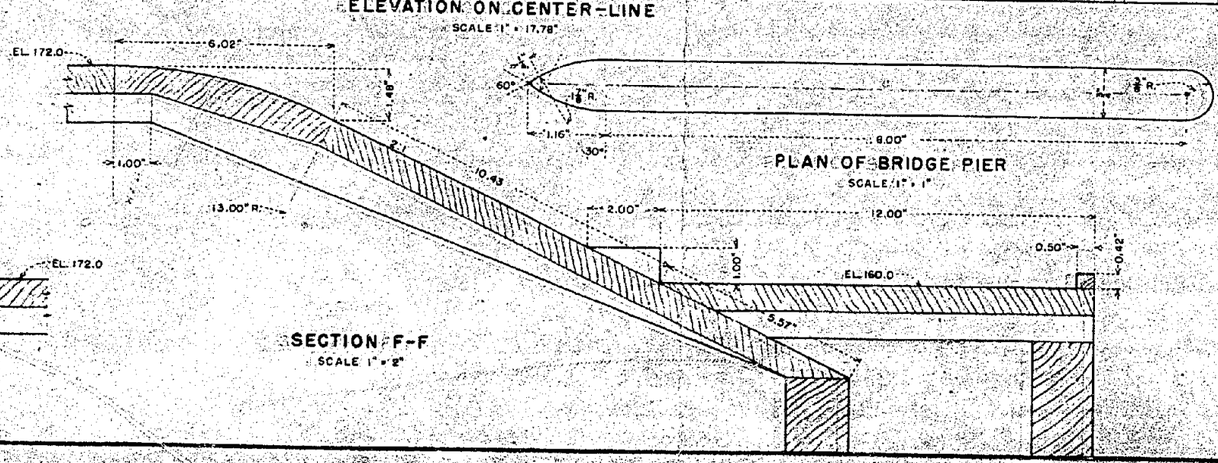
DRAWN J.M.D. SUBMITTED
TRACED E.M.W. & F.B. RECOMMENDED
CHECKED J.M.D. APPROVED
DENVER, COLO. NOV. 13, 1937

FIGURE 4









NOTES

1. Use same model box as constructed for first model.
2. Make bridge pier 6.57" high.
3. Test stilling basin for original design as shown in section F-F.
4. Determine best size, shape and spacing of chute blocks and stilling basin blocks and end sill from hydraulic jump performance and scour tests.

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
GILA VALLEY PROJECT - ARIZONA
GRAVITY MAIN CANAL - STATION 169+60

WASH OVERCHUTE AND WASTEWAY
HYDRAULIC MODEL STUDIES
GENERAL MODEL LAYOUT
MODEL SCALE 1:24

DRAWN	J.H.D.	SUBMITTED	
TRACED	E.F.F.-T.B.	RECOMMENDED	
CHECKED	J.H.D.	APPROVED	
		PROJECT ENGR.	FEB. 12, 1938

is included in this report.

The studies were made in the hydraulic laboratory of the Bureau of Reclamation, Denver, Colorado, between November 1937 and March 1938. A draft of the report was completed in July 1938, but was not reviewed at that time because of more urgent work.

3. Scope of Model Tests.

In the first model, flow conditions in the inlet transition, overchute section over the main canal, and the stilling basin were studied. In the second model, conditions of flow were studied in the overchute inlet, section over the main canal, wasteway inlets, and the stilling basin for separate and concurrent operation of the wasteway and overchute. Studies were also made to determine the most efficient bridge pier to be used on the elevation 172.0 level floor of the overchute.

4. Summary of Results.

From the model study it was accertained that the original design inlet to the wash overchute was satisfactory.

A satisfactory height and spacing for the mud sills, which were for the purpose of compelling sand and silt to deposit on the floor of the chute channel under receding floods to protect the concrete floor slabs against heat from the sun, was established by the investigation. The height was changed from 12 to 8 inches and the spacing from 19'-9" to 9'-10½".

It was considered desirable to place a protective coat of sand or gravel on the chute floor and not to rely upon nature for the initial deposit. Available literature indicated that the material should be 1½- to 3-inch gravel.

From a study of various combinations of apron steps, blocks and sills the most effective combination of these features was established. By using this combination it was found that the downstream depth to form a satisfactory hydraulic jump on the apron could be reduced to 85 percent

of that indicated by the momentum theory. Curves from which it is possible to determine the pool characteristics for similar structures were compiled from these data and are contained in this report.

A satisfactory pool design, including apron steps and sills, adequate for handling the individual or combined flows of the combined overchute and wasteway structure, was evolved from the model studies.

The flow through the original design wasteway gate section was found to be satisfactory.

Flow conditions adjacent to the bridge pier in the overchute channel below the canal were improved by streamlining the pier nose.

THE INVESTIGATION

5. Flow conditions in the overchute inlet.

Very smooth flow conditions existed in the inlet to the overchute for both the 1:18 and 1:24 scale models (plate 1). The original inlet design was recommended.

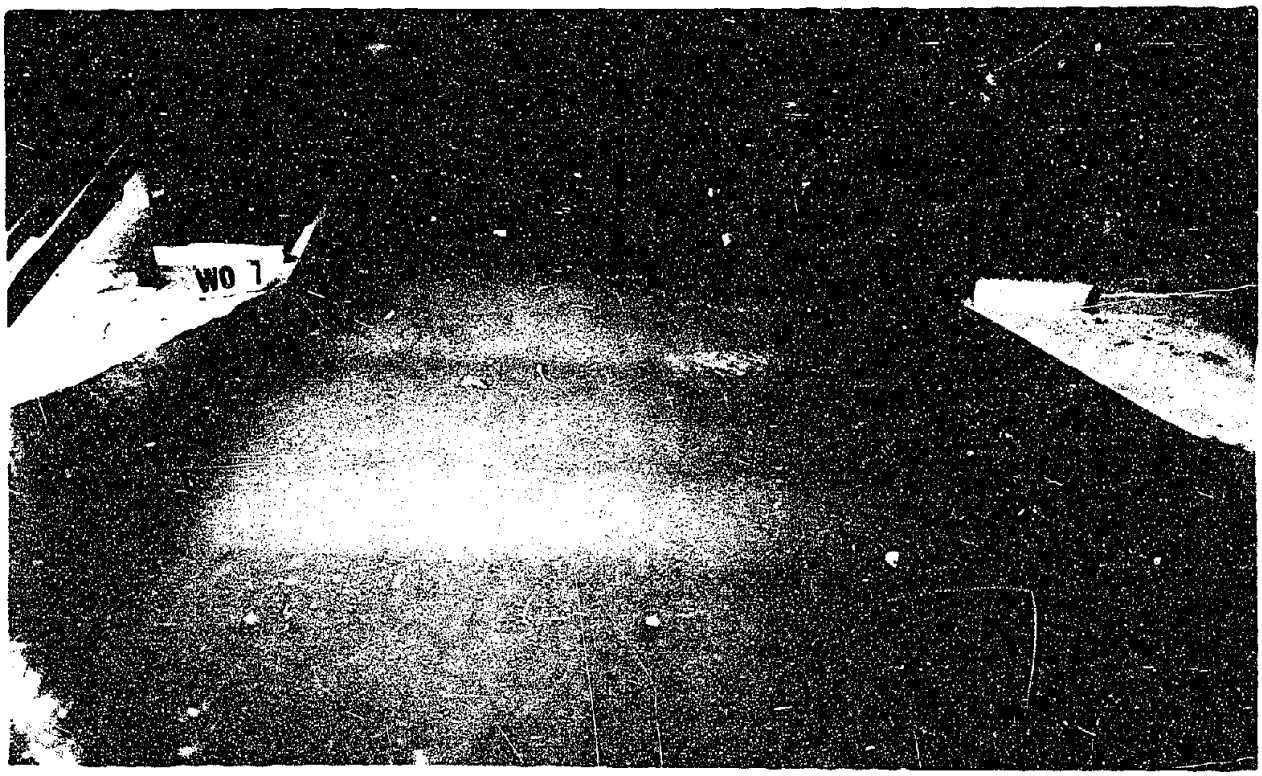
6. Flow conditions in the rectangular section over the main canal.

The original design provided for six mud sills eight inches wide and twelve inches high, parallel to the canal, spaced nineteen feet nine inches center to center in the channel section over the main canal. The purpose of the sills was to aid in the deposition of a layer of sand and mud for protection of the overchute floor from the heat of the summer sun. Because of space requirements in the laboratory, a section containing only three of these sills was duplicated in the first model.

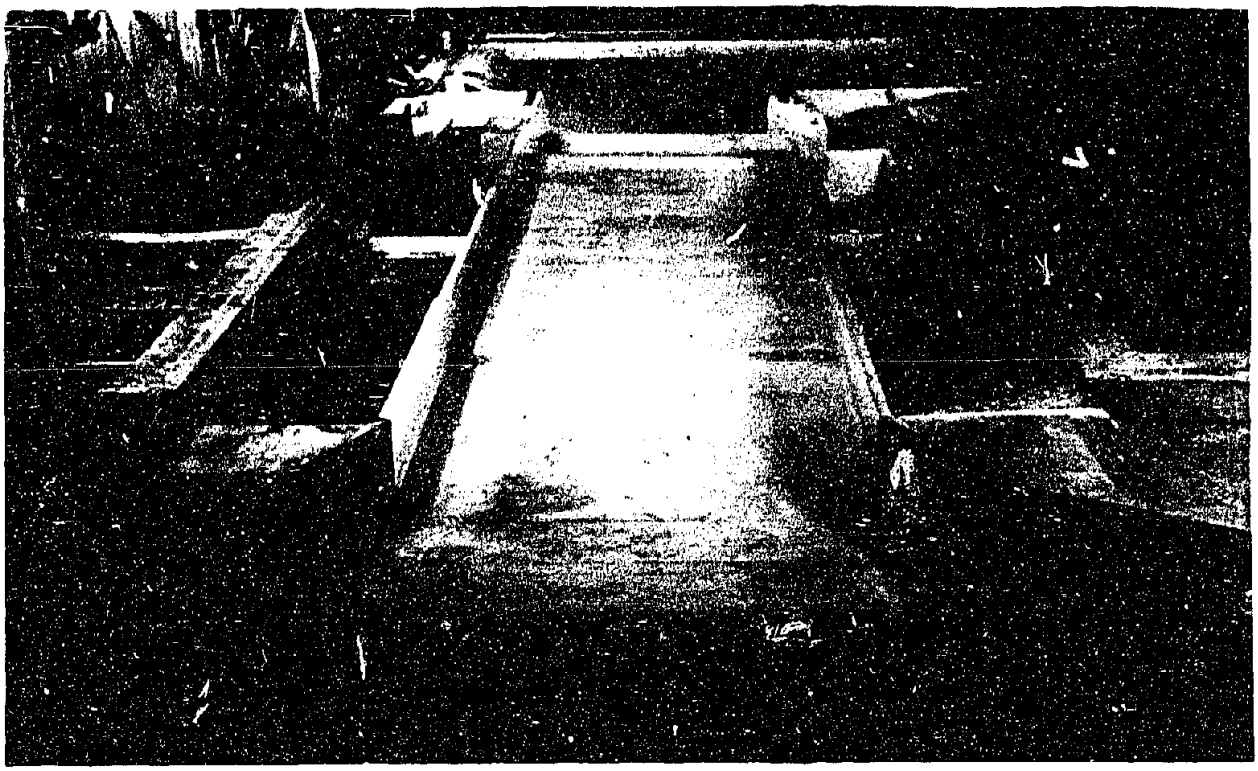
The character of flow over these sills was very rough with the formation of a standing wave or a very small hydraulic jump near each of the sills. These waves or jumps were located approximately midway between succeeding sills except for the last, which was carried over the crest (plates 2 and 3)

The phenomenon satisfies theoretical treatment given to the hydraulic jump. The critical depth for the 60-foot wide section and

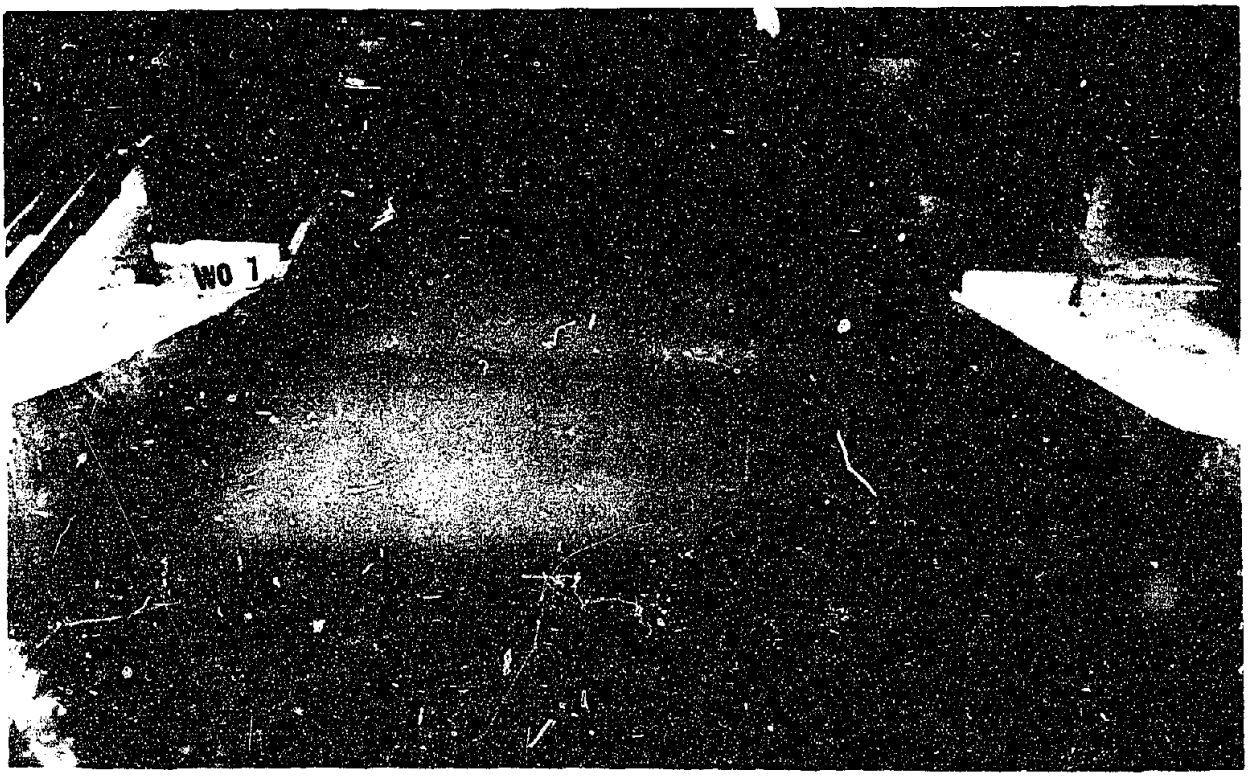
A. OVERCHUTE INLET.



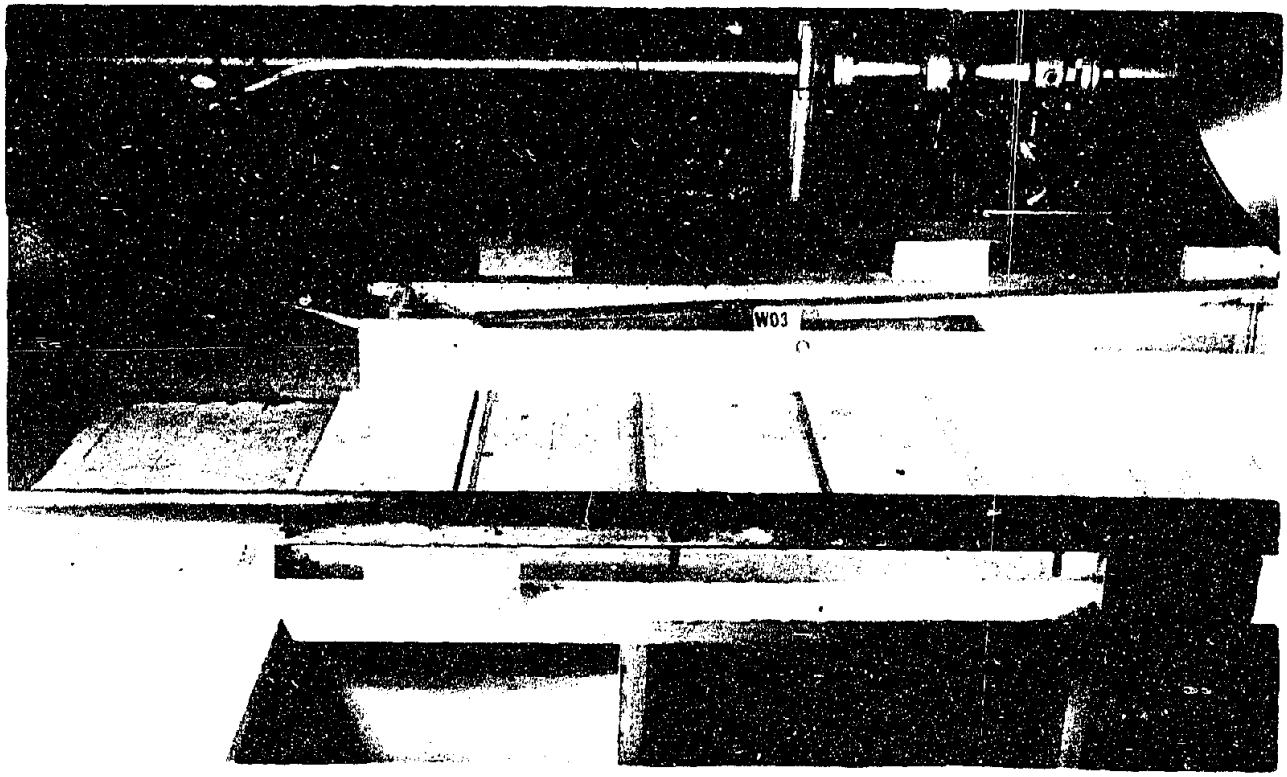
B. OVERCHUTE INLET. DISCHARGE 2500 SECOND-FEET.



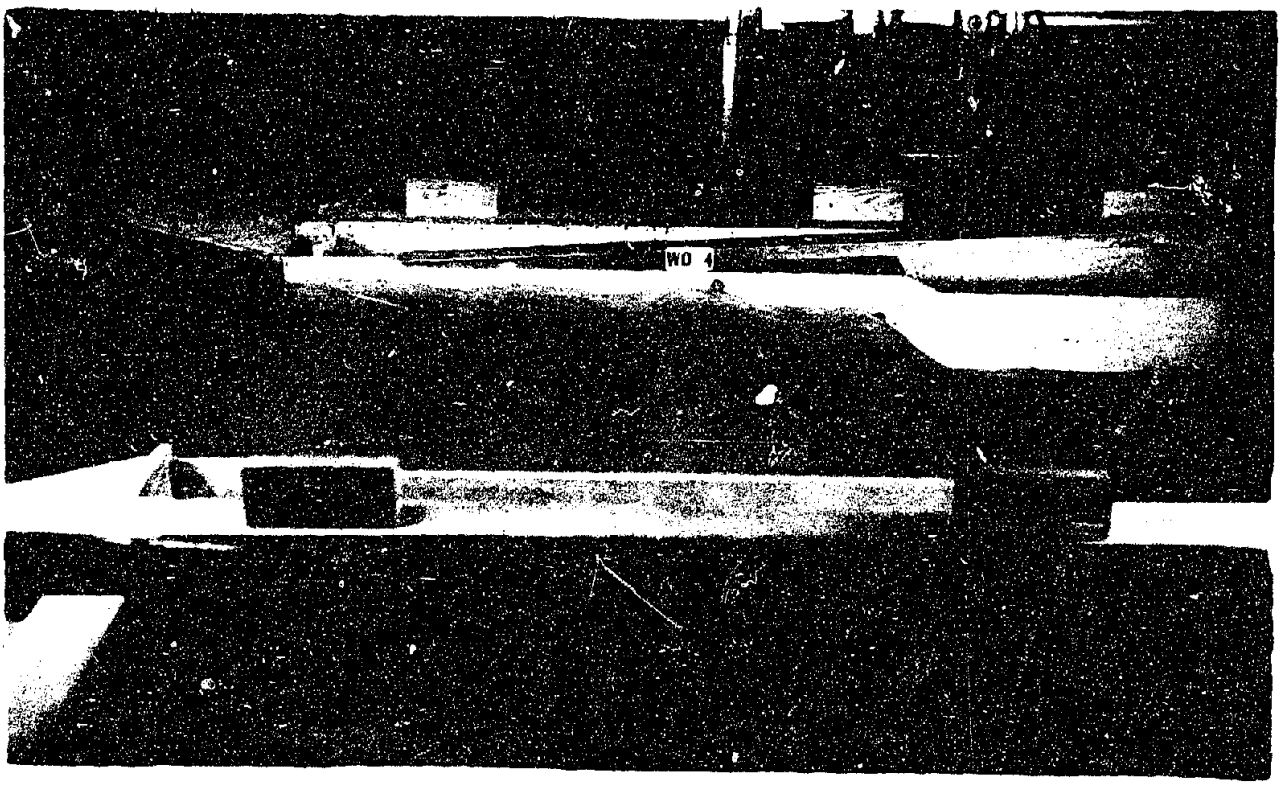
A. OVERCHUTE INLET.



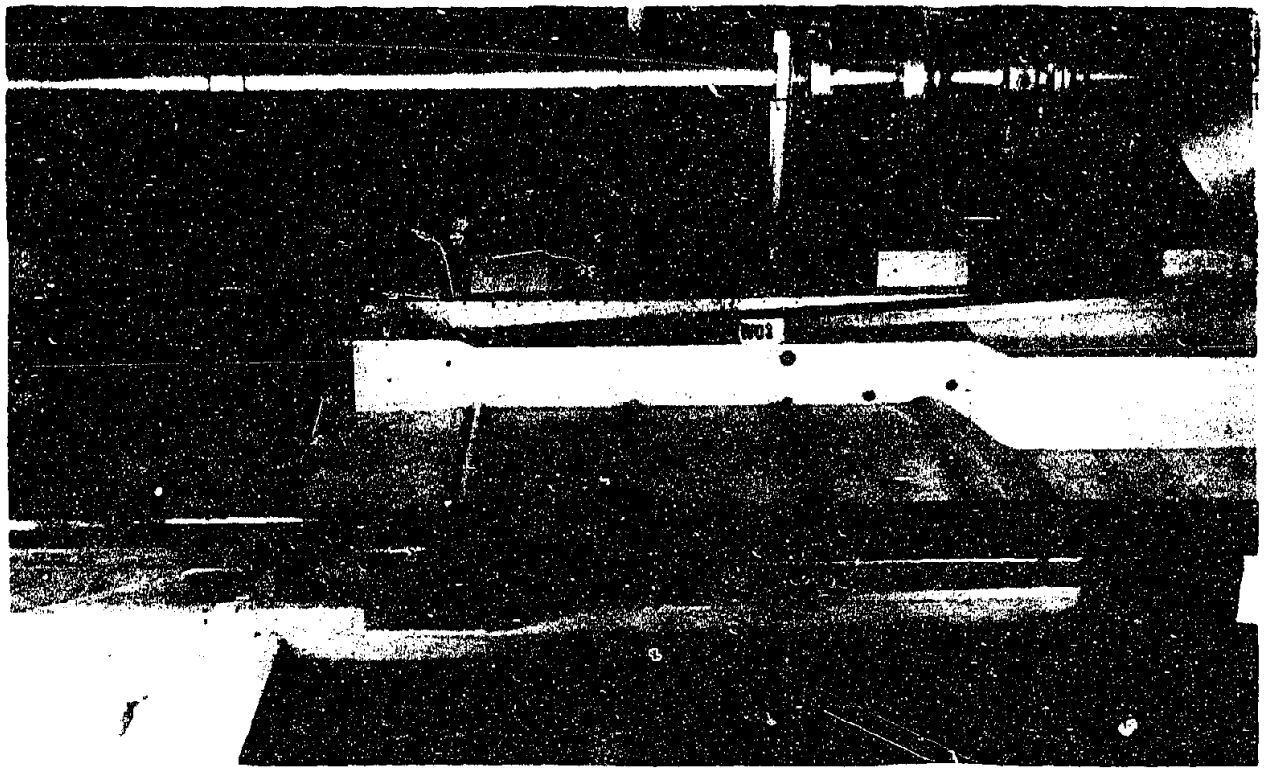
B. OVERCHUTE INLET. DISCHARGE 2500 SECOND-FEET.



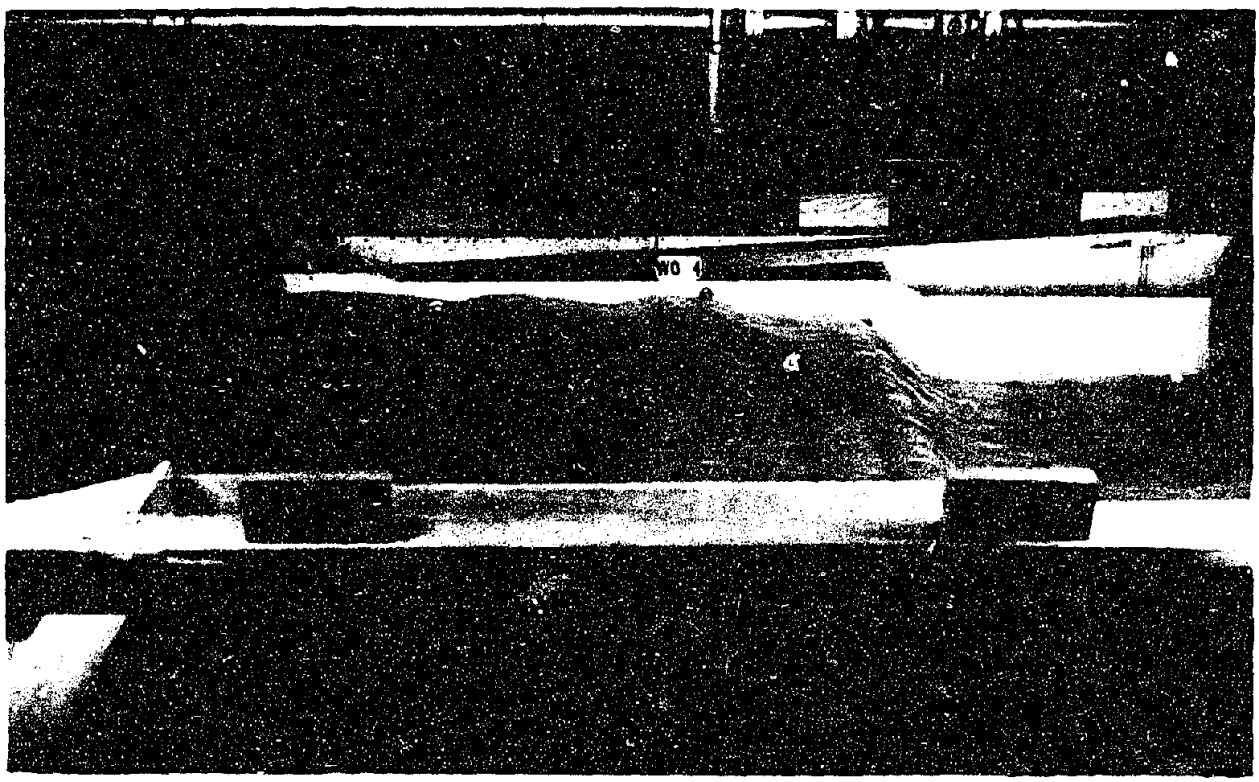
A. 1:18 SCALE MODEL. 12-INCH HIGH SILL SPACED 19 FEET 9INCHES.



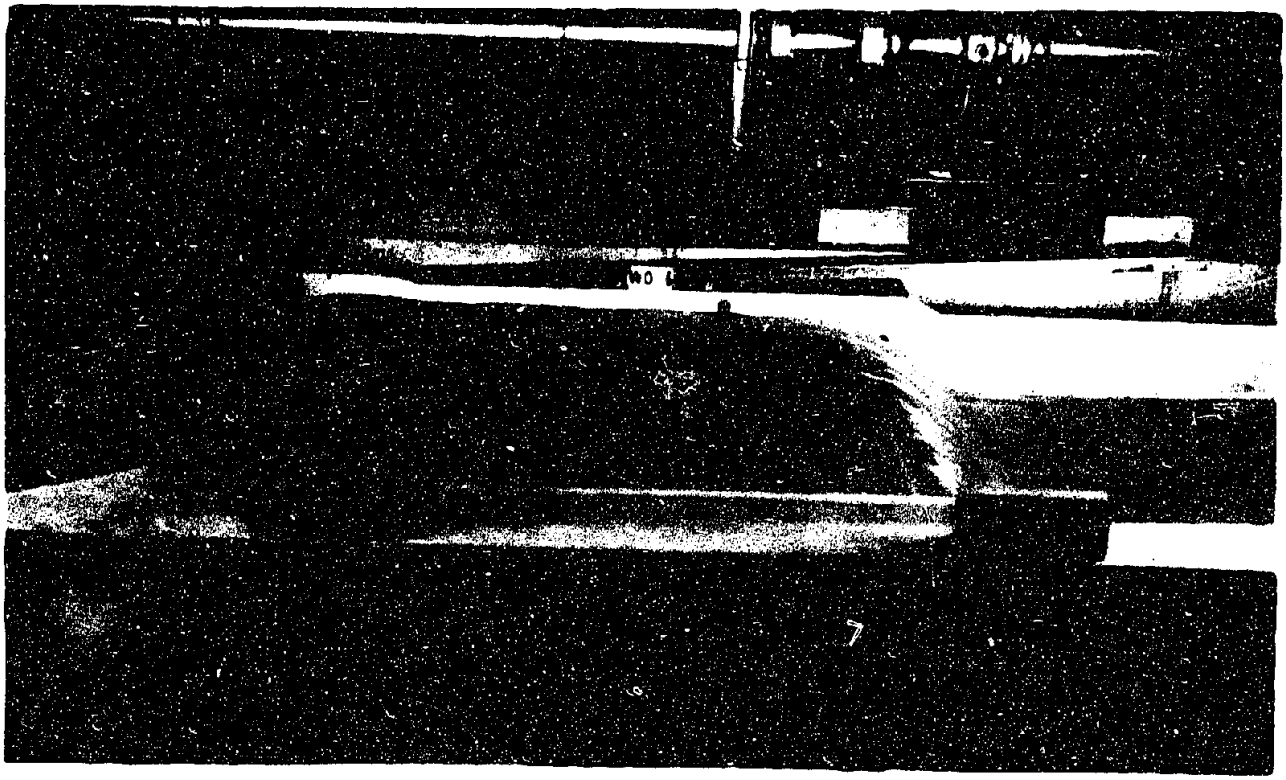
B. 1:18 SCALE MODEL. 12-INCH HIGH SILL SPACED 19 FEET 9 INCHES. DISCHARGE 2500 SECOND-FEET.



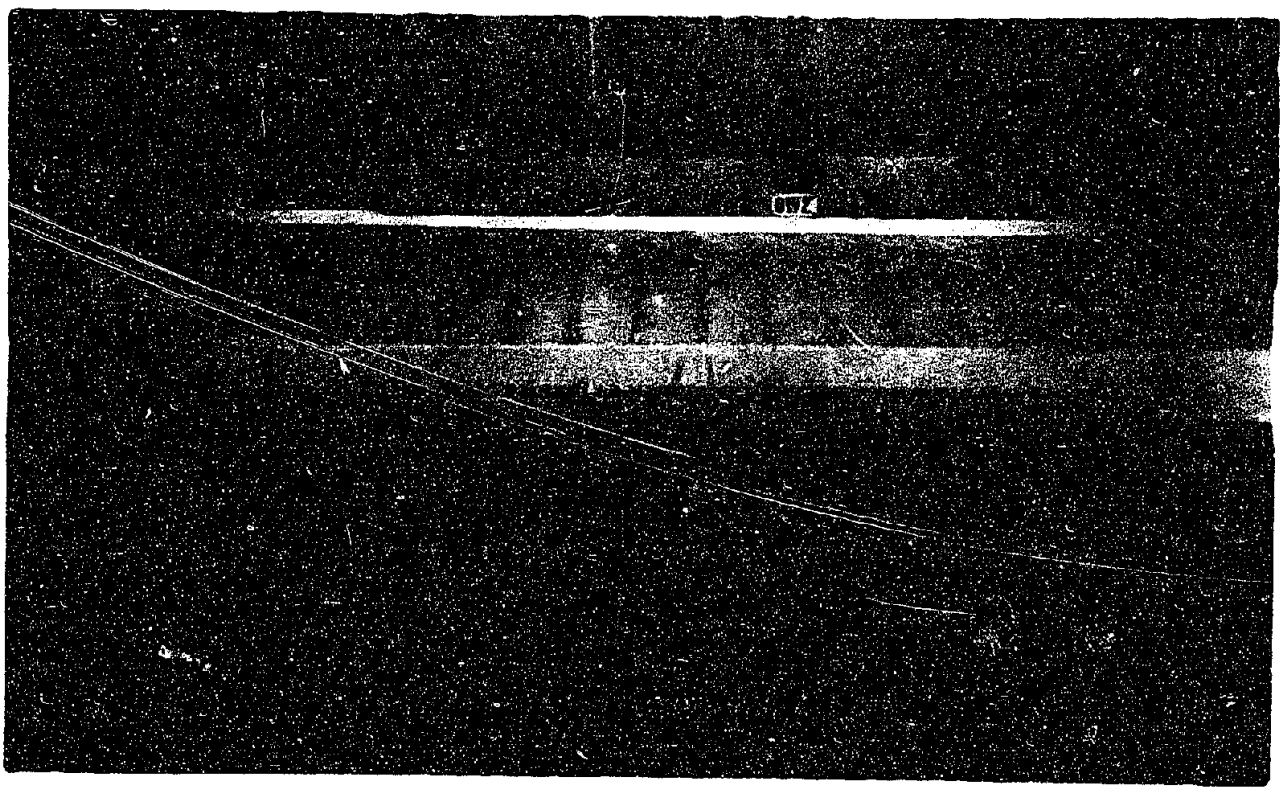
A. 1:18 SCALE MODEL. 12-INCH HIGH SILL SPACED 19 FEET 9INCHES.



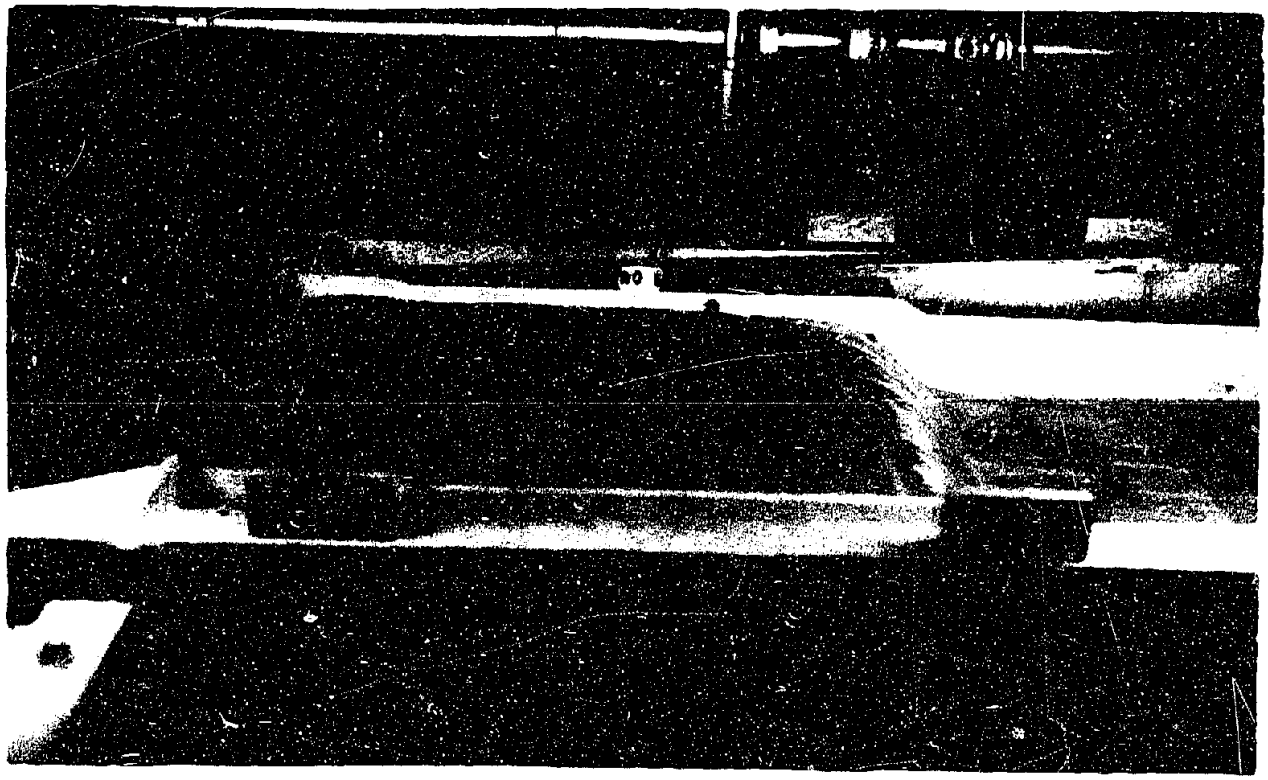
B. 1:18 SCALE MODEL. 12-INCH HIGH SILL SPACED 19 FEET 9 INCHES. DISCHARGE 2500



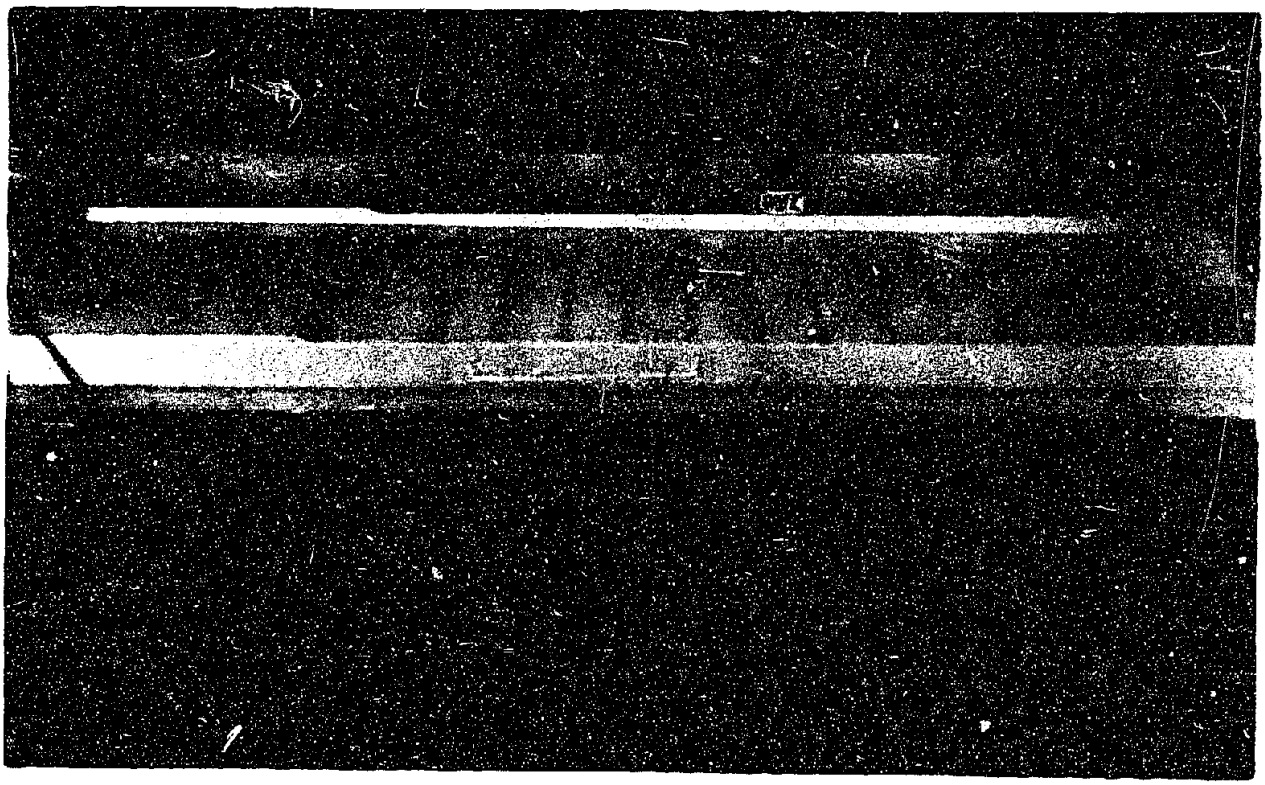
A. 1:18 SCALE MODEL. 6-INCH HIGH SILL SPACED 9 FEET 10⅛ INCHES. DISCHARGE 2500 SECOND-FEET.



B. 1:24 SCALE MODEL. 6-INCH HIGH SILL SPACED 9 FEET 10⅛ INCHES. DISCHARGE 2500 SECOND FEET.



A. 1:16 SCALE MODEL. 6-INCH HIGH SILL SPACED 9 FEET 10½ INCHES. DISCHARGE 2500 SECOND-FEET.



B. 1:24 SCALE MODEL. 8-INCH HIGH SILL SPACED 9 FEET 10½ INCHES. DISCHARGE 2500 SECOND FEET.

maximum flow of 2500 second-feet is computed to be 3.79 feet. As the flow passes over each 1-foot high sill, the velocity must increase, which must be accompanied with a drop in head or decrease in depth to produce the increase. It is plain that the depth and velocity over the sill cannot be the same as over the floor, nor can the velocity decrease with an increase in depth, since both of these cases require an increase in total energy content which is an impossibility. The influence of an upstream sill on decreasing the depth of flow over the sill is sufficient to reduce the depth below critical, and the lower floor between adjacent sills and the backwater effect of the next downstream sill produces an increase in depth to above critical, resulting in the formation of the standing wave, or small hydraulic jump. The phenomenon will occur at each sill until a point is reached upstream where the depth reduction due to velocity increase over the sill is insufficient to establish flow below critical depth. From this point on upstream the flow is relatively smooth.

In the first model, the small jump existed below each of the three sills, but for the second model, in which all six mud sills were installed, the size of the jump decreased progressively upstream until the fifth sill was reached, where there was only a ripple. The water surface was smooth over the sixth sill and upstream (plate 3-B).

In both models, tests with several sill sizes and spacings were made to determine the arrangement which would result in the smoothest flow conditions. The character of flow accompanying each arrangement tested was identical in the two models. Several sizes of sills were tested at the original nineteen-foot nine-inch spacing. The small jumps formed in every case and their size increased with the sill height. Tests were made for several spacings and it was found that a spacing of ten feet was the maximum giving smooth flow in the chute. Apparently, for this spacing, the character of flow lines becomes similar to the character of

flow lines over a smooth floor at the same elevation as the tops of the sills. In this case the decrease in depth over the sills is insufficient to establish flow below critical depth, resulting in smooth flow over the entire length of section. With the 10-foot spacing, several heights of sills showed no change in the smoothness of flow. Thus the sill height must be dependent upon other factors.

Figure 5 shows the water-surface profile along the overchute center line in the first model for the original design and for 6-inch high sills with 9-foot 10⅓-inch spacing. Figure 6 shows the water-surface profile in the second model for 12-inch high sills spaced 19 feet 9 inches and 8-inch high sills spaced 9 feet 10½ inches. The latter arrangement was adopted in the final design.

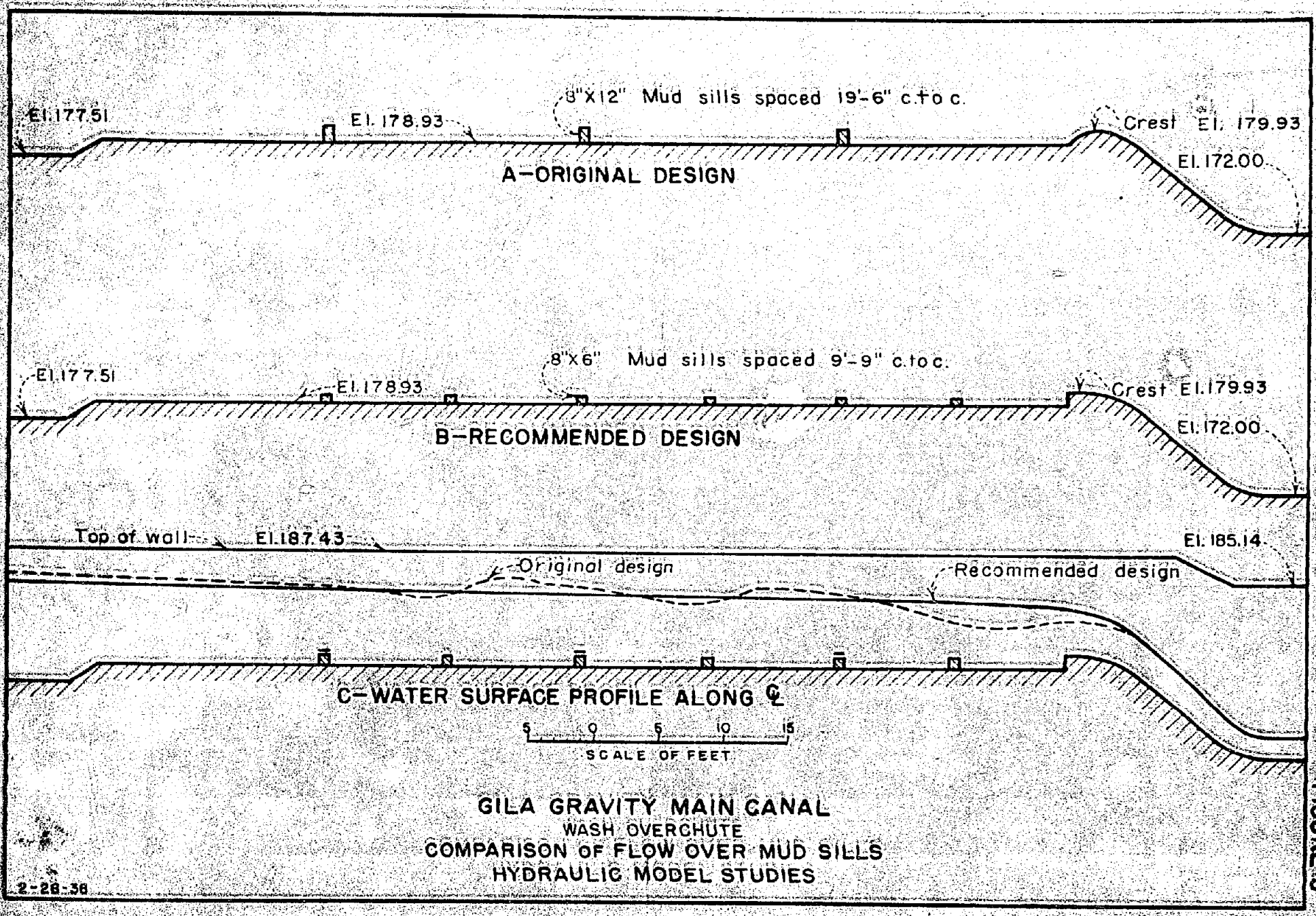
7. Material covering for protection of the concrete floor slabs.

Because of the extreme summer temperatures in the vicinity of the project, it is desirable to protect the concrete, especially the thin floor slab over the main canal of the overchute structure. The most economical and perhaps effective means of protection is a covering of sand and mud deposited from the transported and suspended loads of receding floods.

The question arises as to whether sufficient material will be deposited on the floor to form an effective covering. Factors influencing the quantity of deposited material are quantities of transported and suspended loads and rate of flood subsidence, the quantity being greater for greater loads and smaller rates.

A theoretical approach may be made to determine the probability that a protective covering will remain in the overchute. The case of no sills will be treated first and an attempt will then be made to modify the conclusions to include the influence of the sills.

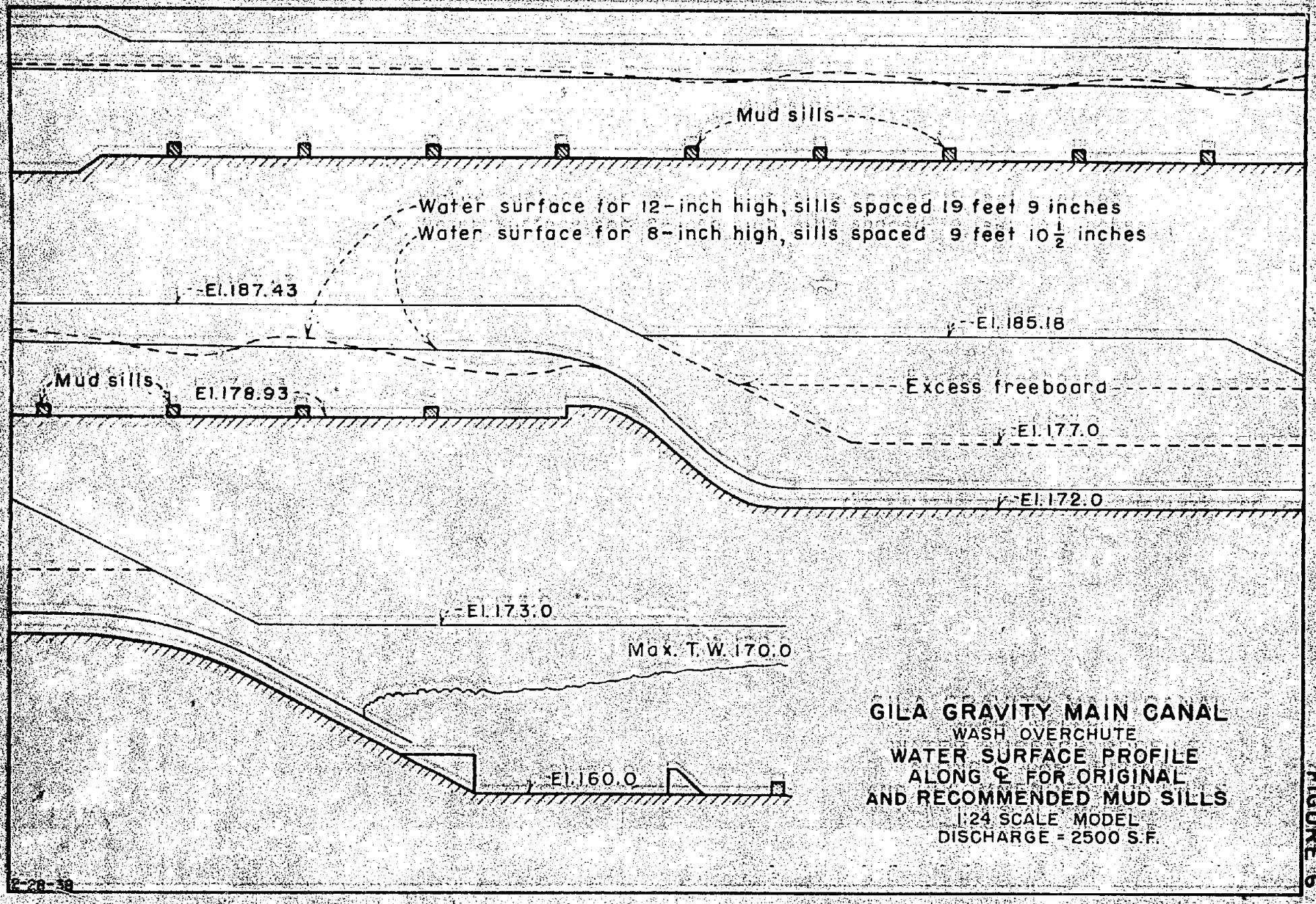
Du Boy's law, $T_c = wDS$, may be used in determining the conditions for which a sand covering will remain on the floor. T_c is the critical



GILA GRAVITY MAIN CANAL
WASH OVERCHUTE
COMPARISON OF FLOW OVER MUD SILLS
HYDRAULIC MODEL STUDIES

2-28-38

FIGURE 5



GILA GRAVITY MAIN CANAL
WASH OVERCHUTE
WATER SURFACE PROFILE ALONG ℄ FOR ORIGINAL AND RECOMMENDED MUD SILLS
1:24 SCALE MODEL
DISCHARGE = 2500 S.F.

FIGURE 6

E-28-38

tractive force for impending motion of the sand, w is the unit weight of water, D the depth of flow and S the hydraulic gradient. If all other values are known, the required depth of water to leave a sand covering may be computed. Use of the Manning formula allows the elimination of the hydraulic gradient. The solution for depth follows:

$$T_c = wDS \quad \ldots\ldots (1)$$

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \quad \ldots\ldots (2)$$

Since the channel is broad and shallow D may replace R. Replacing V by $\frac{Q}{BD}$ and solving (1) and (2) gives

$$D^{7/3} = \frac{w n^2 Q^2}{2.22 B^2 T_c} \quad \ldots\ldots (3)$$

From a study of tests by several experimenters, the critical tractive force of an average sand of natural mix, mean grain size of 0.02 inch, uniformity modulus of 0.50, and a specific gravity of 2.65 varies little from 0.025 pound per square foot. w is 62.5 pounds per cubic foot, B is 60 feet, Q is 2500 second-feet, and the value of n for a sand bottom and concrete side walls has been found to be 0.016. Solving equation (3) for D gives a value equal to 12.6 feet.

The mean velocity corresponding to a depth of 12.6 feet and discharge of 2500 second-feet is 3.3 feet per second, which is in agreement with the recommended safe mean velocity to prevent erosion as recommended by several authorities. For the computed critical depth of 3.8 feet, the upstream water surface will be approximately 3/2 x 3.8 or 5.7 feet above the crest elevation. This means that the crest must be 7.9 feet above the floor to insure a 12-inch cover of sand on the floor. A reduction of the crest height to five feet would probably result in all the sand except that immediately in front of the crest

being washed off the floor.

For the original designed crest height of 12 inches above the floor, and on the basis of a critical mean velocity of 3.3 feet per second, any sand entering the overchute with flows less than 480 second-feet will be deposited on the floor. For larger flows the sand will be carried through the structure.

Since the water cross-sectional area is larger in the inlet transition than in the rectangular section of the overchute, any sand carried to the structure will be deposited before the rectangular section is reached. Sand will deposit in the inlet during the low stages of the flood and will be carried out during the high stages providing conditions for deposit during the next low stage. Only when there is a small flow over sufficient duration or a number of small floods of shorter duration, to fill the inlet to its capacity for holding sand, will sand be carried to and deposited in the rectangular section. It is clear then that for the 12-inch high crest and no sills there will probably not be a satisfactory covering on the floor most of the time. To raise the crest 7.9 feet above the floor would make the cost excessive.

Now the case of the use of sills will be considered. When the depth of water above the sills is greater than D_c, computed by Du Boy's formula, then the sills will be covered and they will be of no use. When the depth is less than D_c the sills will not be covered, and with too great spacing of the sills the flow lines near the bottom will be disturbed, resulting in more eddies and increasing local tractive force values. The conditions for depositing material are less favorable than for a smooth floor. Only when sills are spaced sufficiently close so that the vertical velocity gradients are sensibly the same as for a smooth floor will the sills be effective in holding sand on the floor. The spaces between the sills act as traps into which the sand settles

from the layers of flow immediately above the tops of the sills. For any material, having fixed the height of sill, the spacing will depend on the excess mean velocity over the critical mean scouring velocity, being smaller the greater the excess. Due to the complexity of the behavior of the flow lines in the neighborhood of an obstruction, the correct spacing of sills must be determined by field operation. Sills will maintain a covering for any flow provided they are properly spaced. The most economical sill height and spacing and crest height will depend on the character of floods and amount of material transported.

A suggested alternative to the use of sills was to place large size stones on the floor as a covering. A study was made of the required depth of water for the maximum flood of 2500 second-feet to prevent removal of any specified material from the floor. An extensive search was made for literature from which to determine the allowable critical mean velocities to prevent erosion of materials of the sizes available in the field which may be used as a covering. In general, the data were very meager, but the results of measurements for larger size materials found in an unpublished translation of a Russian paper seem to be very good and applicable to this work. The following is a tabulation of the necessary hydraulic properties for several sizes of material:

Material Size	Critical Mean Velocity to Prevent Erosion	Required Depth to Pass 2500 sec.-feet	Required Height of Crest Above Floor
1/2 to 1"	5.4 f.p.s.	7.7 ft.	3.0 ft.
1 to 1½"	6.2 f.p.s.	6.7 ft.	2.0 ft.
1½ to 3"	7.7 f.p.s.	5.4 ft.	0.7 ft.

A sample computation will show how the values of the above table were obtained. For material from 1/2 to 1 inch in size, the measured critical mean velocity to prevent erosion is 5.4 feet per second. The required depth to pass 2500 second-feet at this mean velocity for the

60-foot wide channel is 7.7 feet. With a 12-inch covering on the floor, the water surface must be 8.7 feet above the floor. The required crest height is then 8.7-5.7 or 3.0 feet above the floor for this size material.

The last column of the table gives the required height of sill to prevent removal of the several size materials listed in the first column. With the original 1-foot crest height, 1-1/2- to 3-inch size material should be used. It is to be expected that some finer material will settle out of suspension during the receding flood to partly fill the voids, increasing the effectiveness of the covering. This fine material, however, will be carried out again during the next high flood and more will be deposited during the low stages of the same flood.

Of the two methods of providing for a protective covering herein discussed, placement of 1-1/2- to 3-inch size material on the floor is recommended, since it is probably the most economical and appears to give more assurance that a satisfactory covering will be provided.

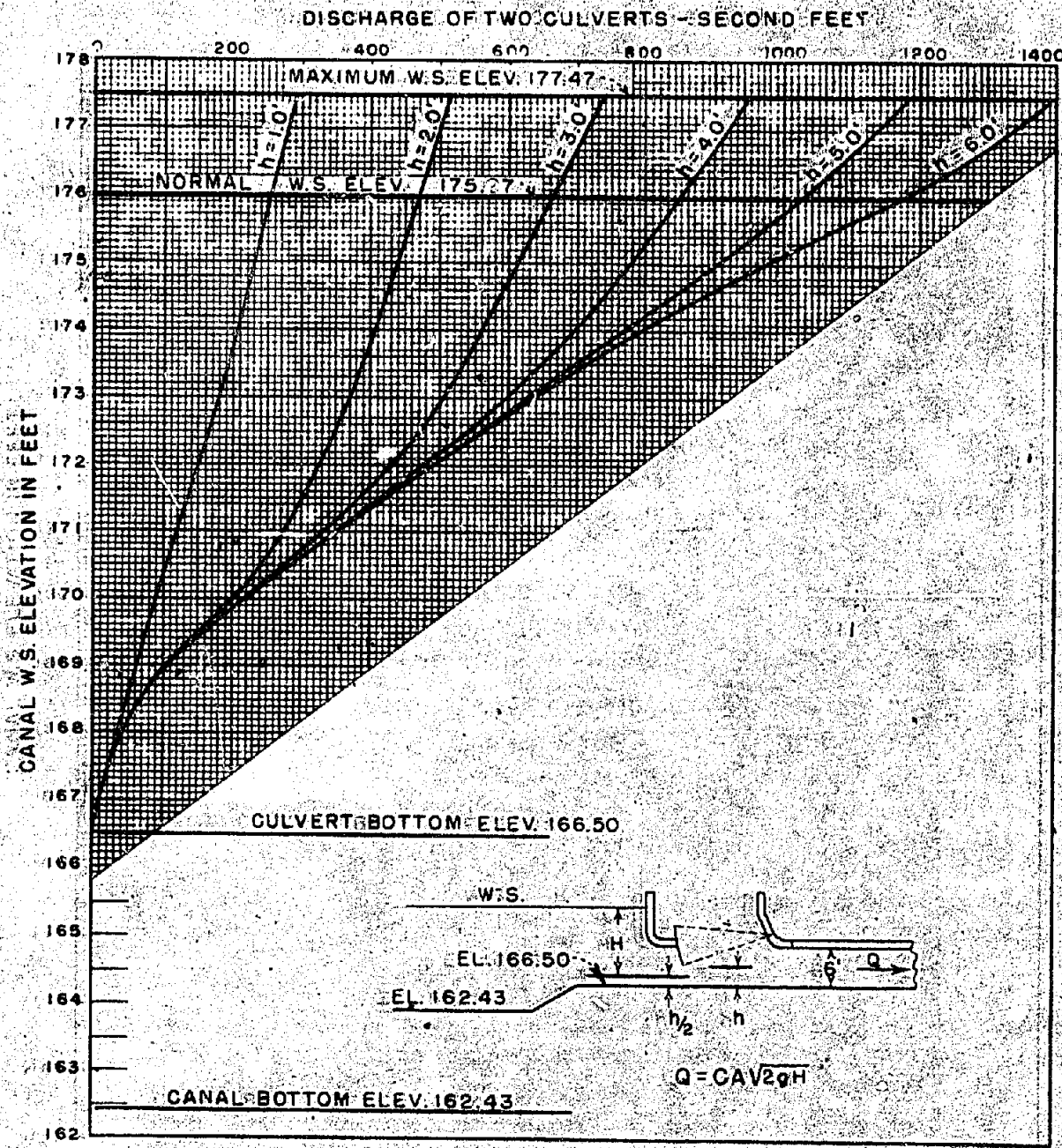
8. Flow through the wasteway gates.

The flow into the wasteway gates was smooth (plate 4). The gates were calibrated and their combined discharge against canal water-surface elevation plotted for several gate openings (figure 7). From this diagram with any known gate opening and canal water-surface elevation the discharge may be determined.

Figure 8 shows the discharge plotted against gate opening for the normal and maximum canal water-surface elevations 175.97 and 177.47.

Figure 9 shows how the discharge coefficient of the formula $Q = CA\sqrt{2gH}$ varies with the canal water surface elevation and the head on the culvert. The head was defined as the distance from the water-surface elevation to the center of the gate opening.

The value of C for the maximum canal water-surface elevation is 0.682 and the corresponding discharge is 1390 second-feet. The design discharge for the wasteways was 1500 second-feet.

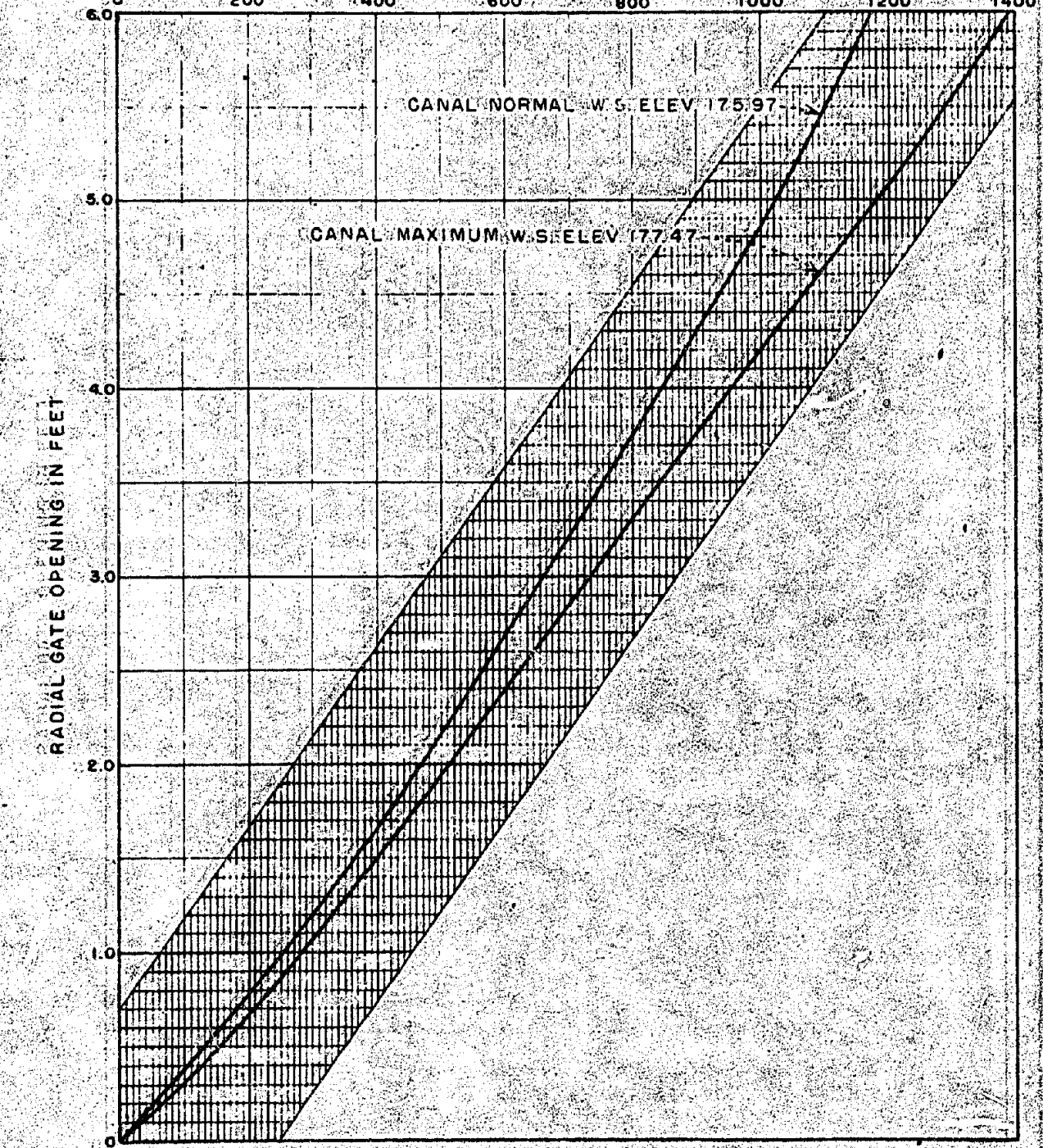


GILA VALLEY PROJECT - ARIZONA

GRAVITY MAIN CANAL - STATION 169 + 80

WASH OVERCHUTE AND WASTEWAY

CANAL W.S. ELEV. - DISCHARGE CURVES FOR VARIOUS RADIAL GATE OPENINGS

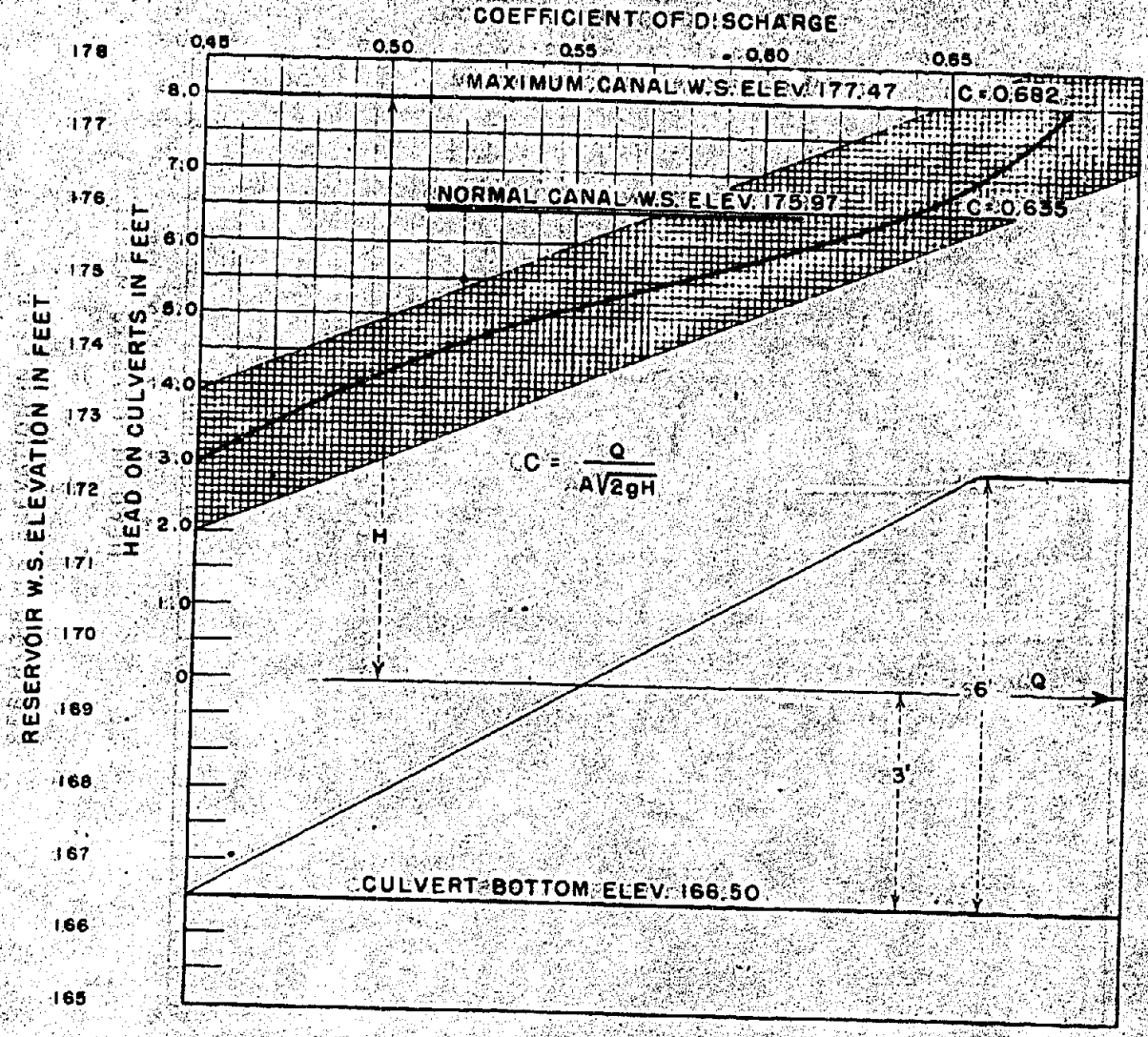


GILA VALLEY PROJECT - ARIZONA

GRAVITY MAIN CANAL - STATION 169 + 80

WASH OVERCHUTE AND WASTEWAY

WASTEWAY DISCHARGE - RADIAL GATE OPENING CURVES FOR NORMAL AND MAXIMUM CANAL W.S. ELEVATION

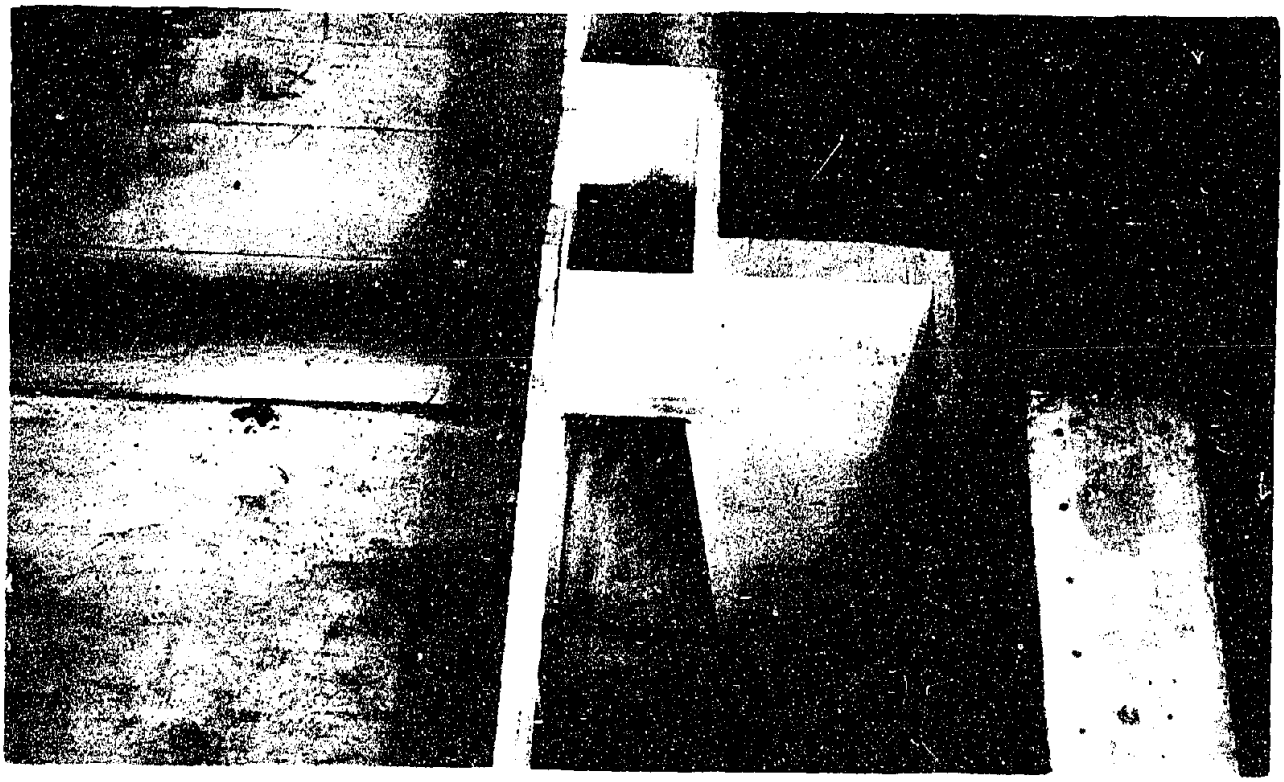


GILA VALLEY PROJECT - ARIZONA

GRAVITY MAIN CANAL - STATION 169+80

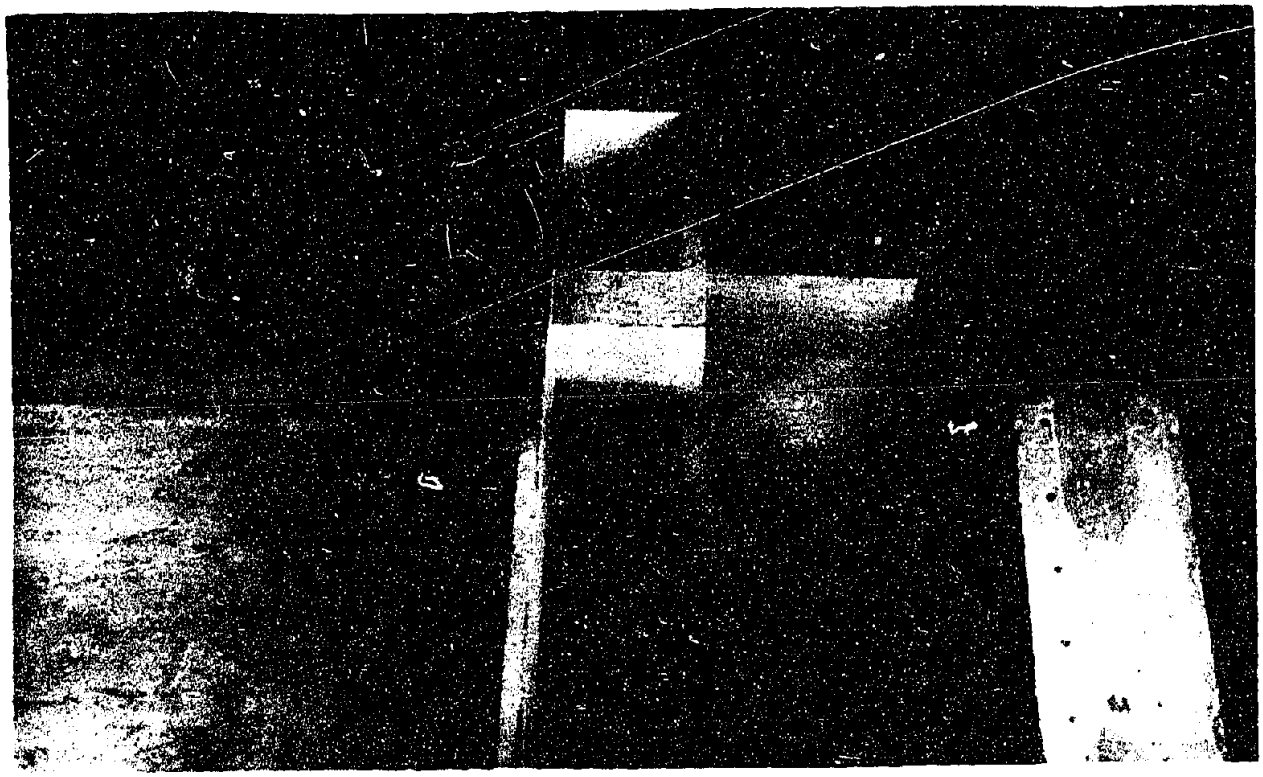
WASH OVERCHUTE AND WASTEWAY

CULVERTS COEFFICIENT OF DISCHARGE HEAD CURVE FOR RADIAL GATES OPEN

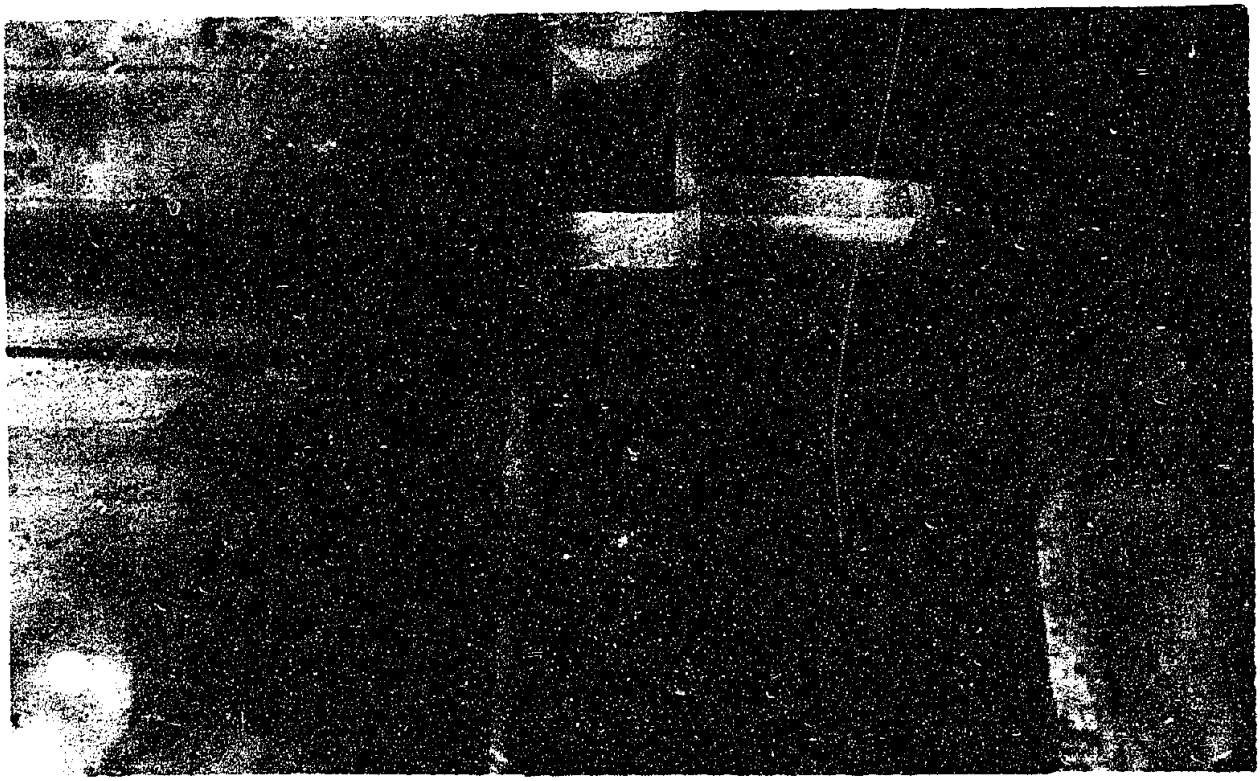


A. MAIN CANAL WASTEWAY INLET.

B. MAIN CANAL WASTEWAY INLET. WASTEWAY DISCHARGE 695 CUBIC FEET PER SECOND. WATER-SURFACE ELEVATION 177.47.



A. MAIN CANAL WASTEWAY INLET.



B. MAIN CANAL WASTEWAY INLET. WASTEWAY DISCHARGE 695 SECOND-FEET. MAIN CANAL WATER-SURFACE ELEVATION 177.47.

9. Stilling basin design for wash overchutes without adjoining wasteways.

Wash overchutes without adjoining wasteways are frequently designed by the Bureau, so a series of general tests on the first model were undertaken to determine the best form of stilling basin. The effectiveness of the various forms of stilling basins was determined by the scouring effects in a bin in which sand was placed to represent the wash bed downstream from the pool and by the smoothness of the water-surface through the jump, determined by mean watersurface measurements taken along the center line. Preliminary tests showed that duplicate runs would produce the same scour results, that the amount of compacting of the sand was of little importance, and that a run of 30 minutes was sufficient to secure practically the same result as a flood of great duration.

10. The hydraulic jump.

In the most perfect form of the hydraulic jump, the energy of the high velocity water is dissipated so thoroughly in internal impact of water against that little energy remains to be used up in eddies and boils downstream. In the case of a structure with a well-formed jump, the velocity of the water is so quickly and uniformly reduced that little scour of beds and banks results. It is therefore, desirable, whenever possible, to design the stilling basin so that such a jump will occur, as the protection required for the bed and banks is thus reduced to a minimum.

The ideal condition would be to have a tailwater at such a height above the wash bed for each discharge that it would form a perfect jump for the depth and velocity which would exist at the entrance to the stilling pool for that discharge. The height of the tailwater at the structure in question, however, is controlled by a steel sheet-piling weir to be placed at elevation 170.0 across the stream channel 230 feet downstream from the end of the pool. The relative elevations of stilling basin floor and downstream weir crest were designed such that the required

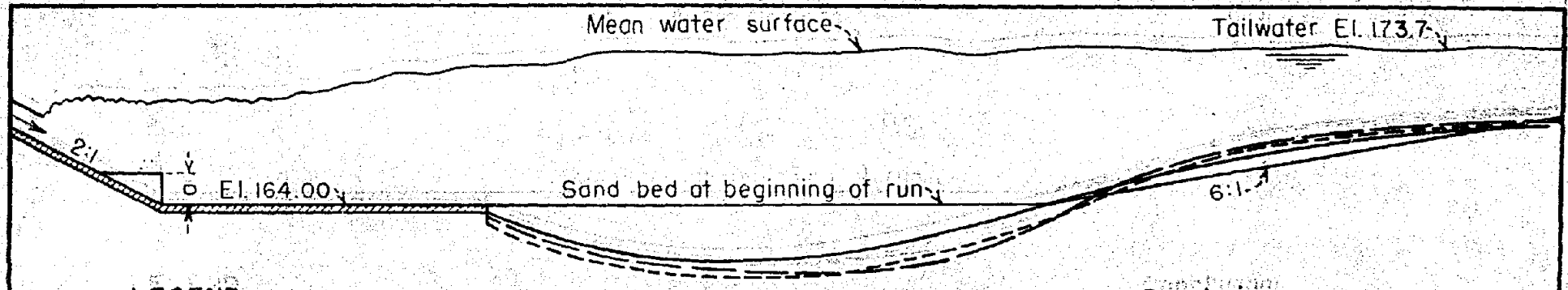
downstream depth, d_2, exists for the maximum discharge. With 6.0 feet difference in elevation between the floor and the weir crest, this depth will be excessive for flows less than the maximum, but it is more important to have the correct tailwater depth for the maximum flow than for the smaller quantities.

11. Chute steps, apron blocks, and end sill.

A toothed step placed in the chute at the entrance to the stilling basin is effective in forming a more efficient jump. In such cases, the incoming jet is broken up into a number of small jets each of which sets up its own system of eddies, therefore, resulting in a greater dissipation of energy. With the numerous jets entering the pool over a greater range in elevation, there is less tendency for the incoming flow to dive under the water in the pool without causing a well-formed jump.

Figure 10-A shows the effect of the size of this step on scour when no end sill or apron blocks are used. There was very little difference in the watersurface with or without the chute step, but the scour becomes more severe as the size of the step becomes excessively large. This is explained by the fact that the jets over the step reached the relatively short floor at about the end of the basin, resulting in more severe scour. Later tests showed, however, that correct size of chute step was effective in reducing the scour when apron blocks and an end sill were used.

Any arrangement of baffle blocks and end sill placed across the apron will, of course, check the velocity of the entering sheet of flowing water. Blocks and sills with vertical upstream faces are more effective than those with sloping upstream faces so that smaller sizes may be used to produce the required results. However, the vertical upstream face blocks and sills will receive greater impact as the high velocity carries to the floor with little retardation. This may call for facing

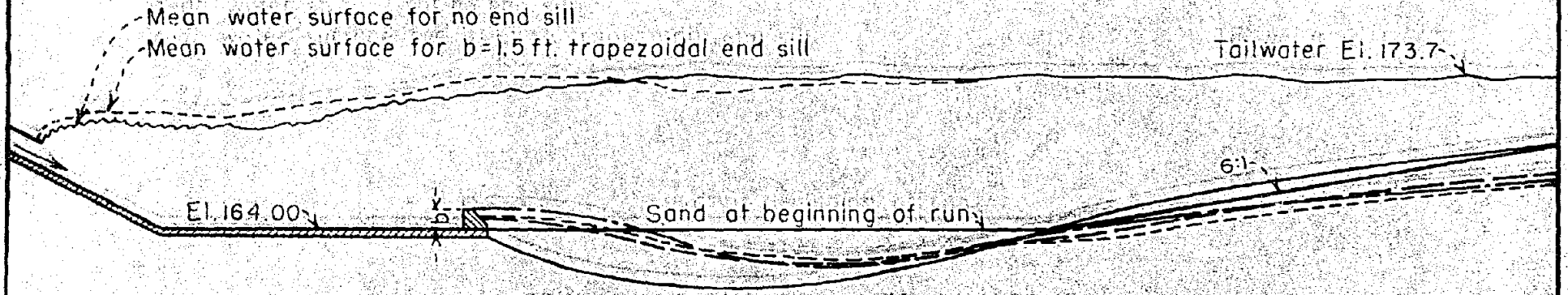


LEGEND

- no chute step
- a = 1.5 feet
- a = 2.0 feet

A – EFFECT OF CHUTE STEP SIZE ON SCOUR
NO END SILL OR APRON BLOCKS

Conclusion:
The least scour is given for no chute steps when no end sill is used.



LEGEND

- no end sill
- 10" x 12" rect. sill
- b = 1 ft. (trap. sill)
- b = 1.5 ft. (trap. sill)

B – EFFECT OF END SILL SIZE ON SCOUR
NO CHUTE STEP OR APRON BLOCKS

Conclusion:
The 1 ft. high trapezoidal end sill shows the least scour.

5 0 5 10 15
SCALE OF FEET

GILA GRAVITY MAIN CANAL

WASH OVERCHUTE
STILLING POOL STUDIES
EFFECT OF CHUTE STEP AND END SILL ON SCOUR DOWNSTREAM

3-3-38

FIGURE 10

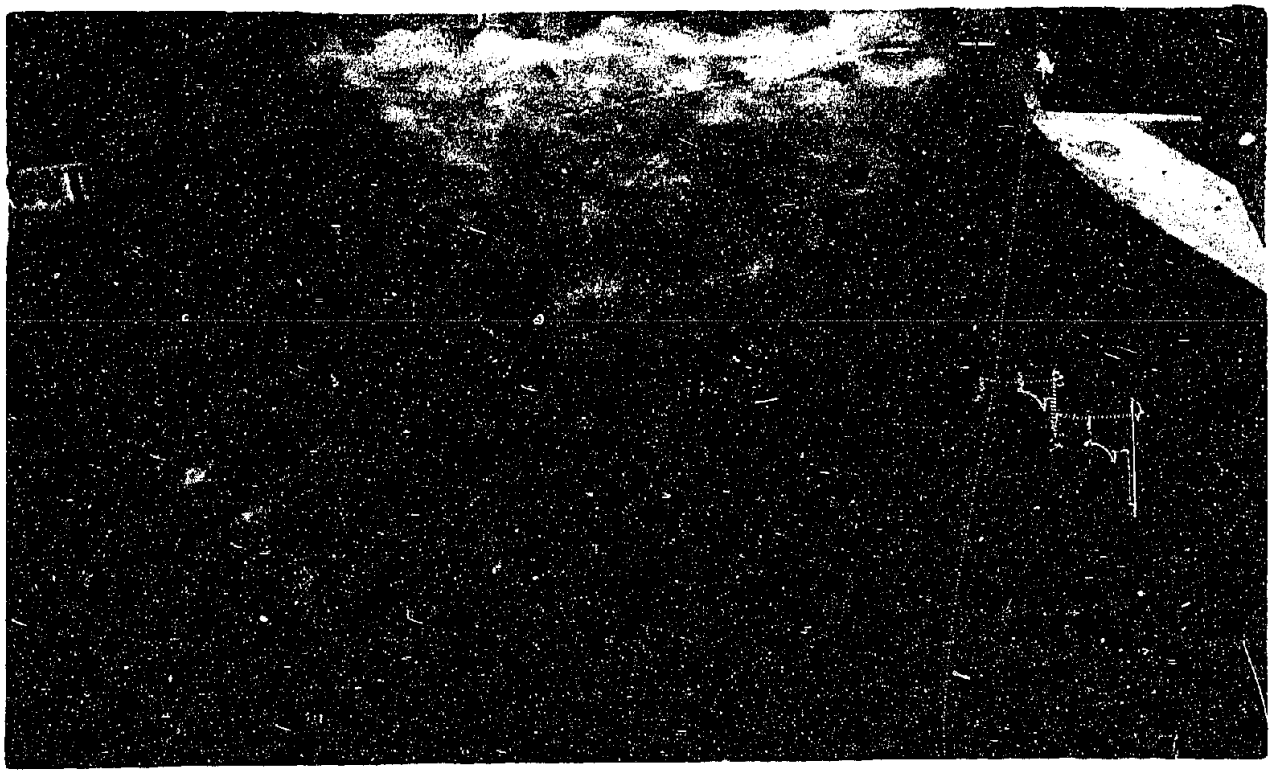
on the blocks, reinforcement, and anchorage.

The apron blocks and end sill should be of proper height. If too high, they will receive excessive impact, develop an excessively large ground roller which results in more severe scour, lift the high velocity flow from the bottom to the surface where retardation is much slower and expose the river banks to higher erosive forces. The top of the sill must be sufficiently below the channel bottom downstream so that a drop of the water surface will not result at any discharge from the obstruction caused by the sill. If the sill is made only high enough to deflect the flow just off the river bed, the velocity is quickly retarded by the static pressure of the tailwater, and the current of the ground roller formed immediately downstream from the end of the floor in contact with the river bed moves upstream and drags bed material up towards the end of the floor, rather than scours it away.

Figure 10-B shows that a trapezoidal sill one foot high placed at the end of the pool results in the least scour, although the 10-inch rectangular sill of the original design is practically as effective. There is no apparent difference between rectangular and trapezoidal sills of the same height, nor in the top width of the sills, so the most economical shape and width may be used. As the sill height becomes greater than one foot there is an excessive drop in water surface over the sill, resulting in rougher flow conditions and greater bed and bank scour. Scour is excessive when no end sill is used (plates 5, 6 and 7).

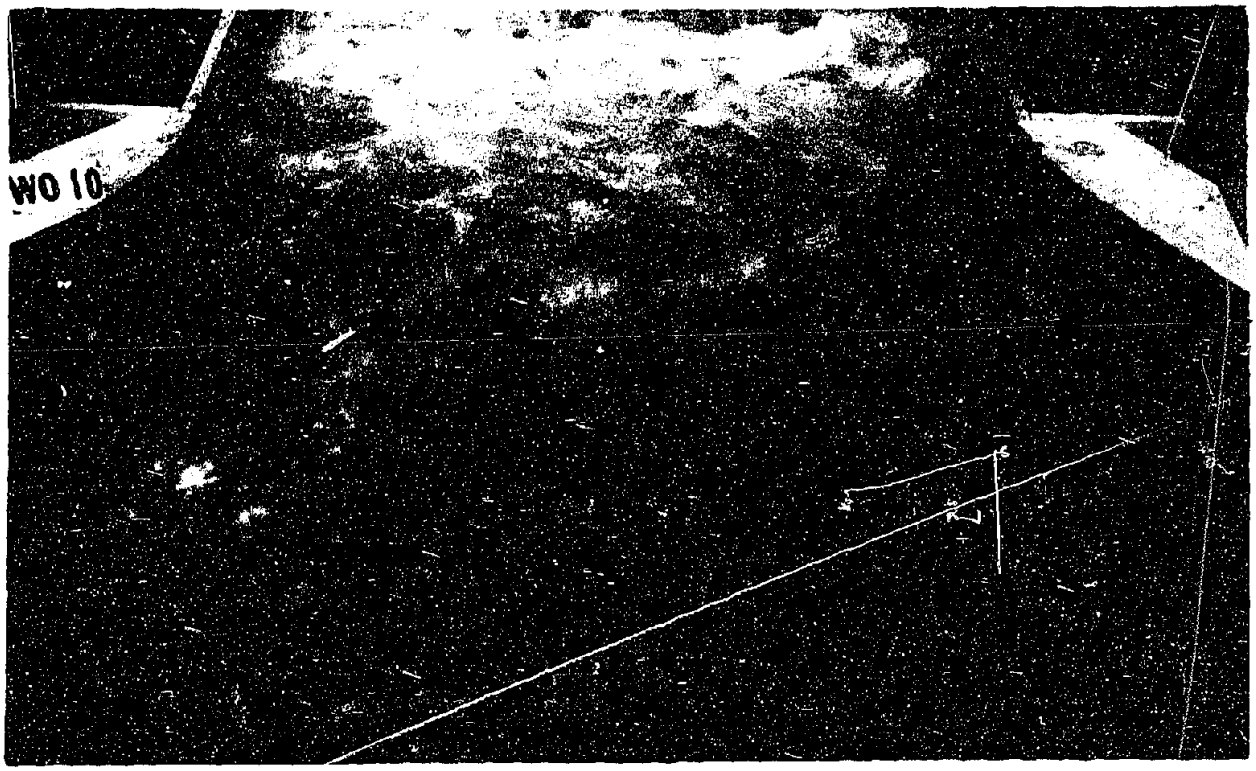
Figure 11-A shows that the chute step is effective in reducing scour when an end sill is used. The step two feet high gave the least scour (plate 8). However, the size does not greatly affect the scour. With the depth of water entering the pool equal to 1.2 feet, the height of the step is 5/3 of this or $\frac{5}{3}d_1$.

Figure 11-B shows that a row of baffle blocks placed on the basin floor in front of the end sill is effective in reducing both scour and



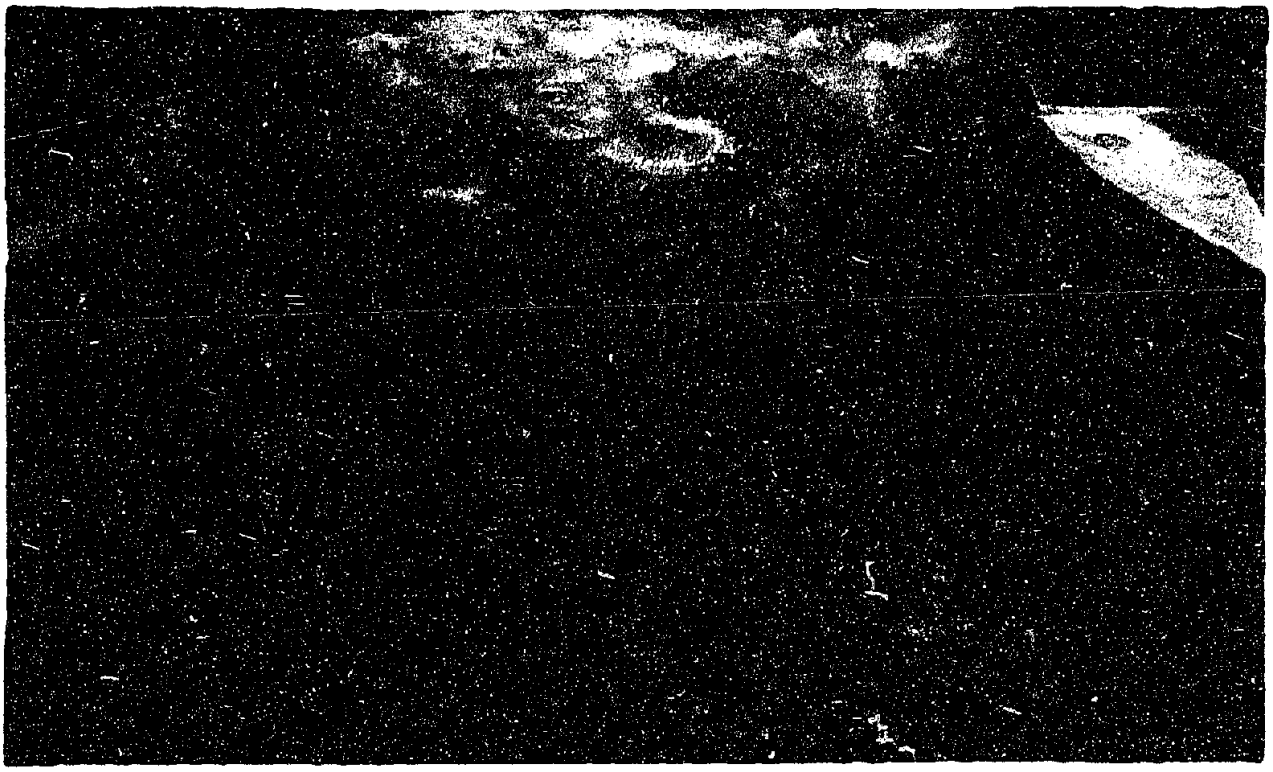
A. 1:18 SCALE MODEL OF OVERCHUTE STILLING BASIN. NO BLOCKS OR SILL. DISCHARGE 2500 SECOND-FEET.

B. REVERSE OF PLATE 5-A.

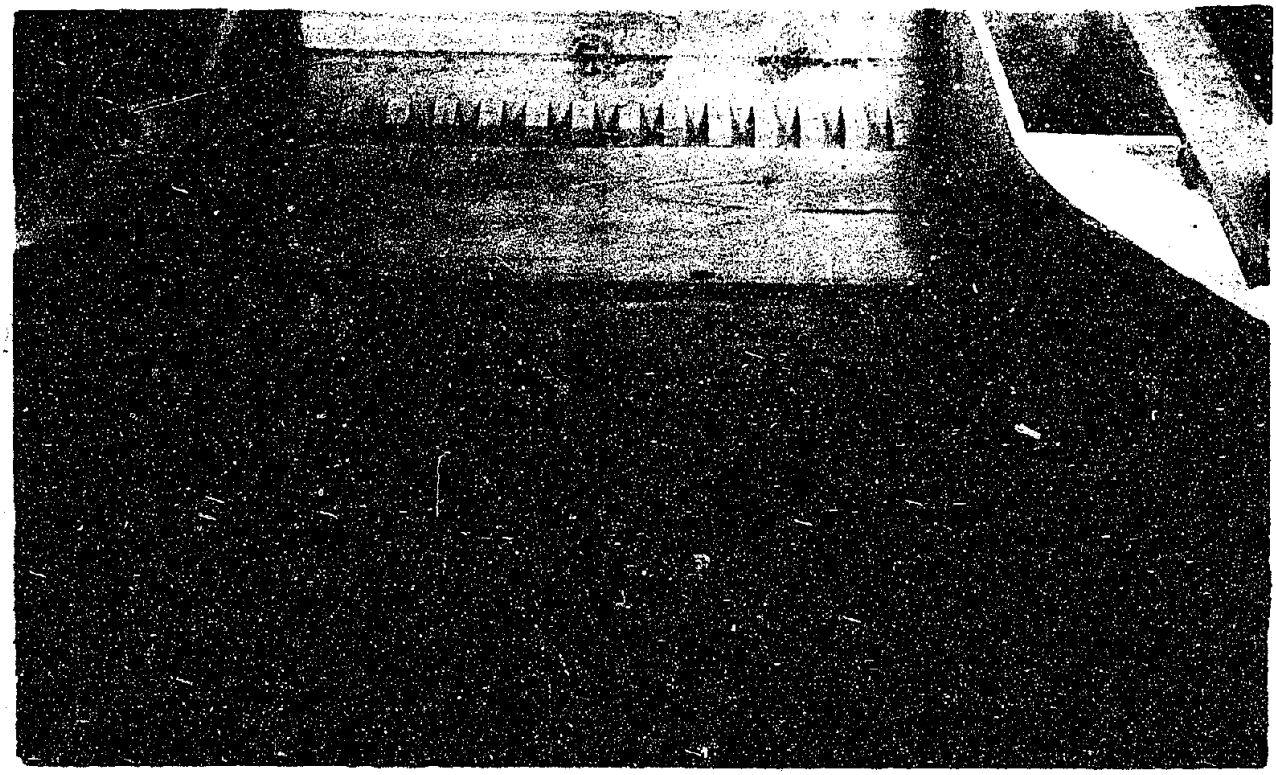


A. 1:16 SCALE MODEL OF OVERCHUTE STILLING BASIN. NO BLOCKS OR SILL. DISCHARGE 2500 SECOND-FEET.

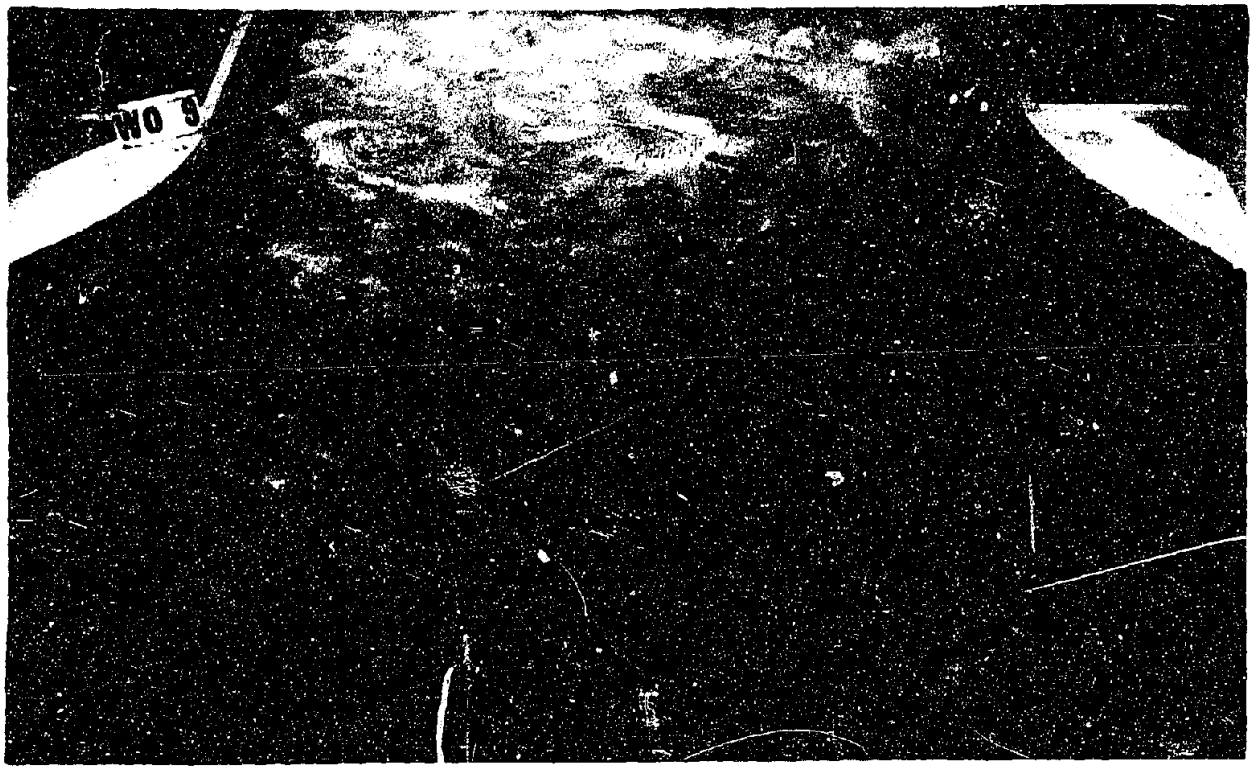
B. SCOUR OF PLATE 5-A.



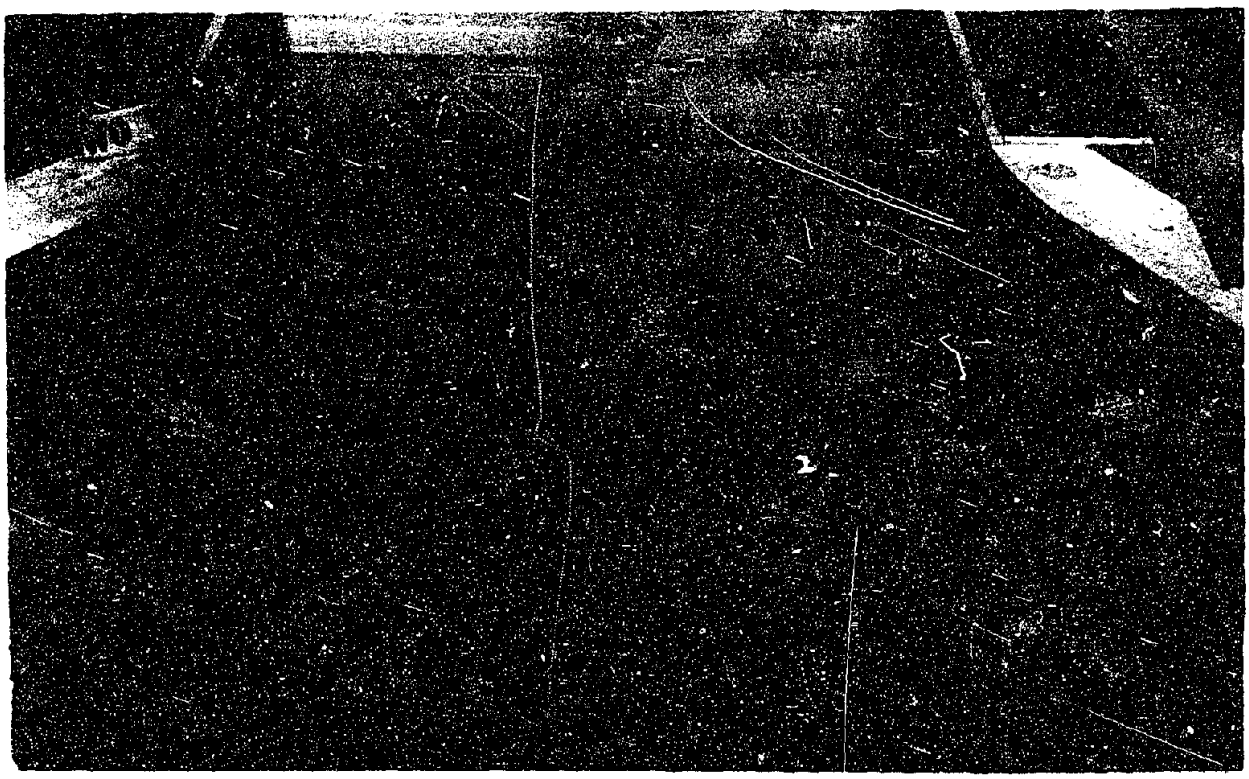
A. 1:18 SCALE MODEL OF OVERCHUTE STILLING BASIN. BEST SIZE CHUTE BLOCKS 2 FEET HIGH. NO FLOOR BLOCKS OR END SILL. DISCHARGE 2500 SECOND-FEET.



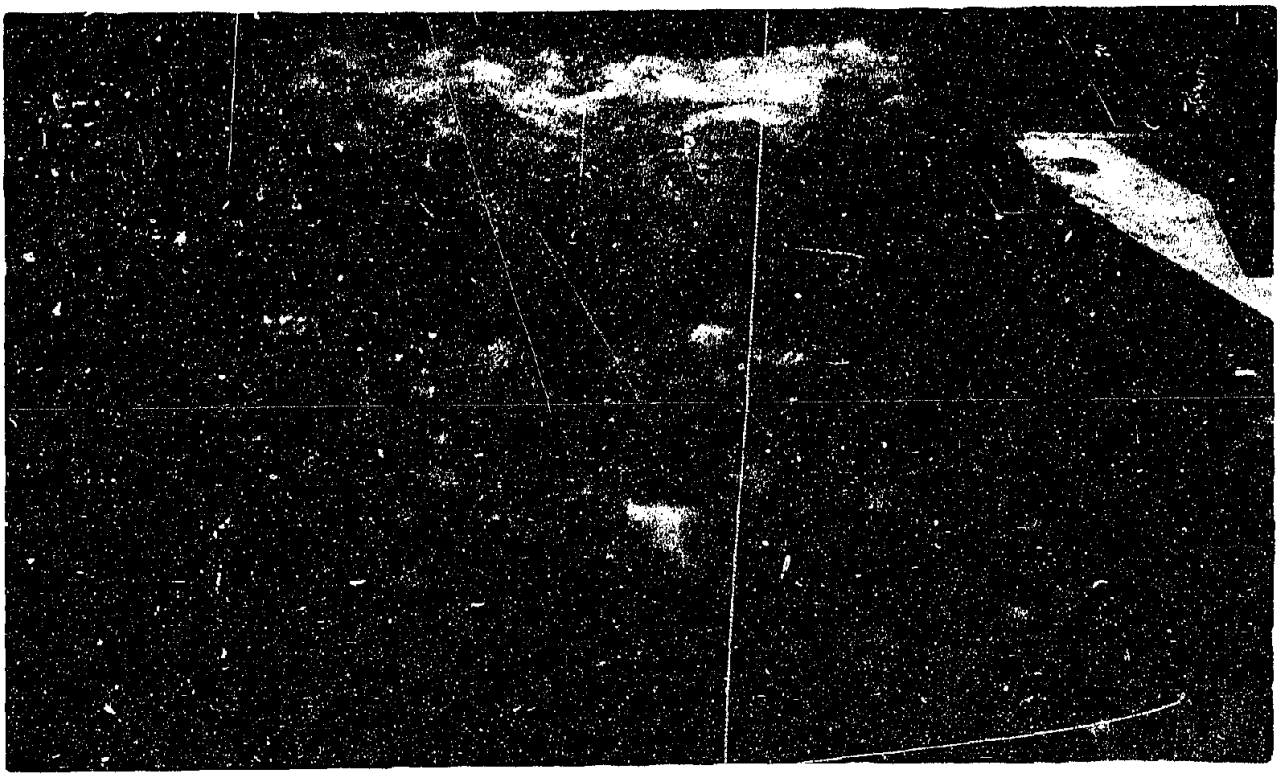
B. SCOUR OF PLATE 6-A.



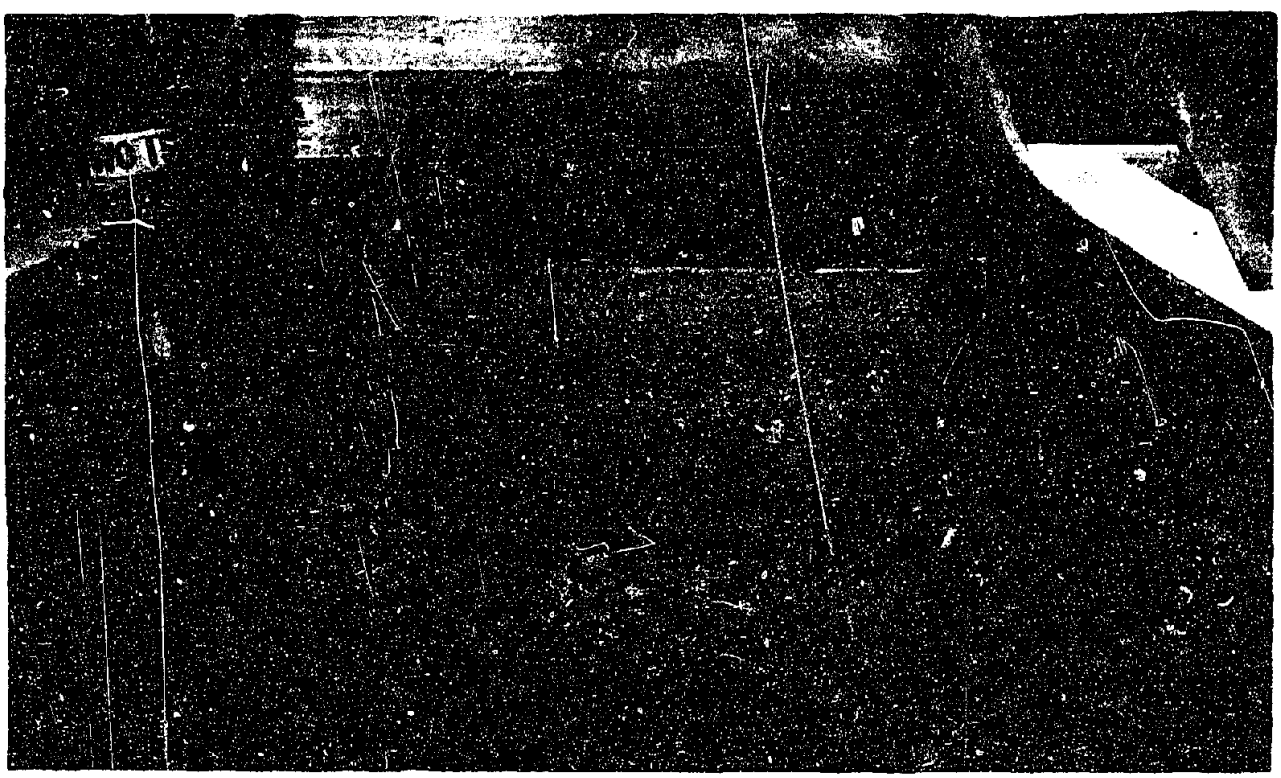
A. 1:18 SCALE MODEL OF OVERCHUTE STILLING BASIN. BEST SIZE CHUTE BLOCKS 2 FEET HIGH. NO FLOOR BLOCKS OR END SILL. DISCHARGE 2600 SECOND-FEET.



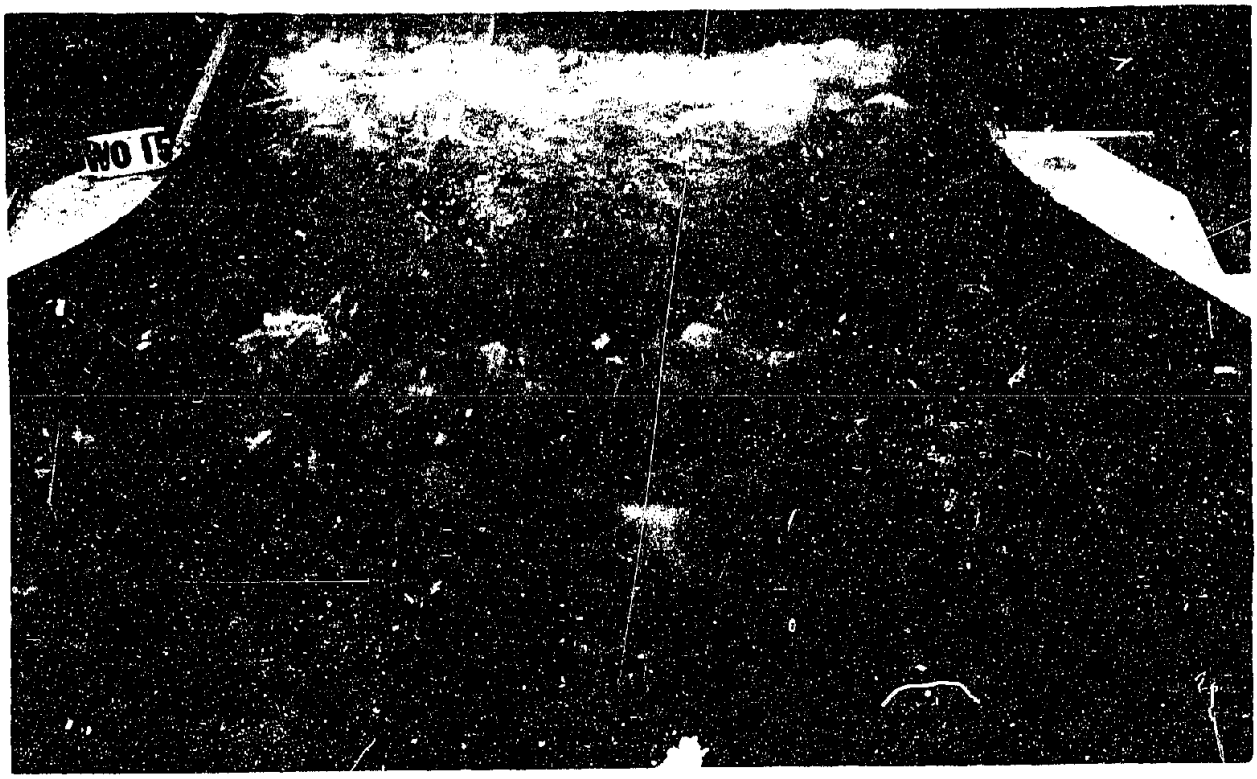
B. SCOUR OF PLATE 6-A.



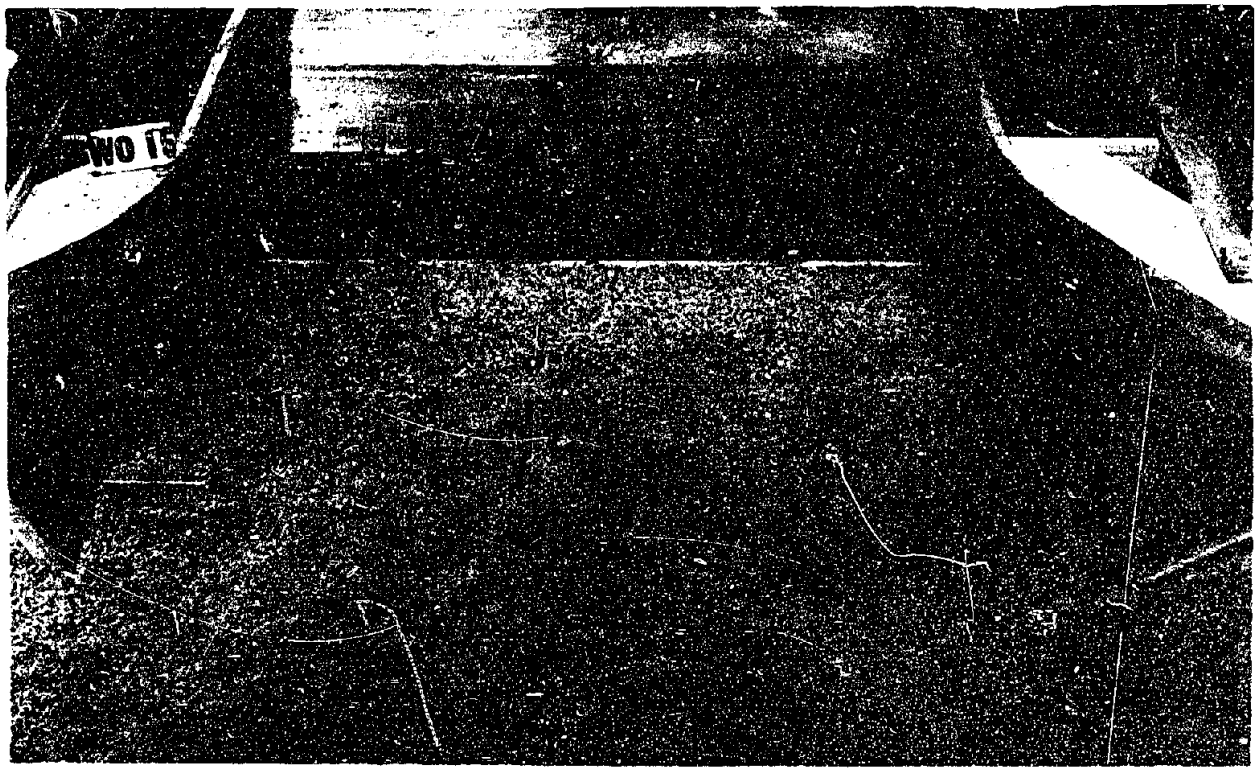
A. 1:18 SCALE MODEL OF OVERCHUTE STILLING BASIN. BEST END SILL 1 FOOT HIGH. NO CHUTE OR FLOOR BLOCKS. DISCHARGE 2500 SECOND-FEET.



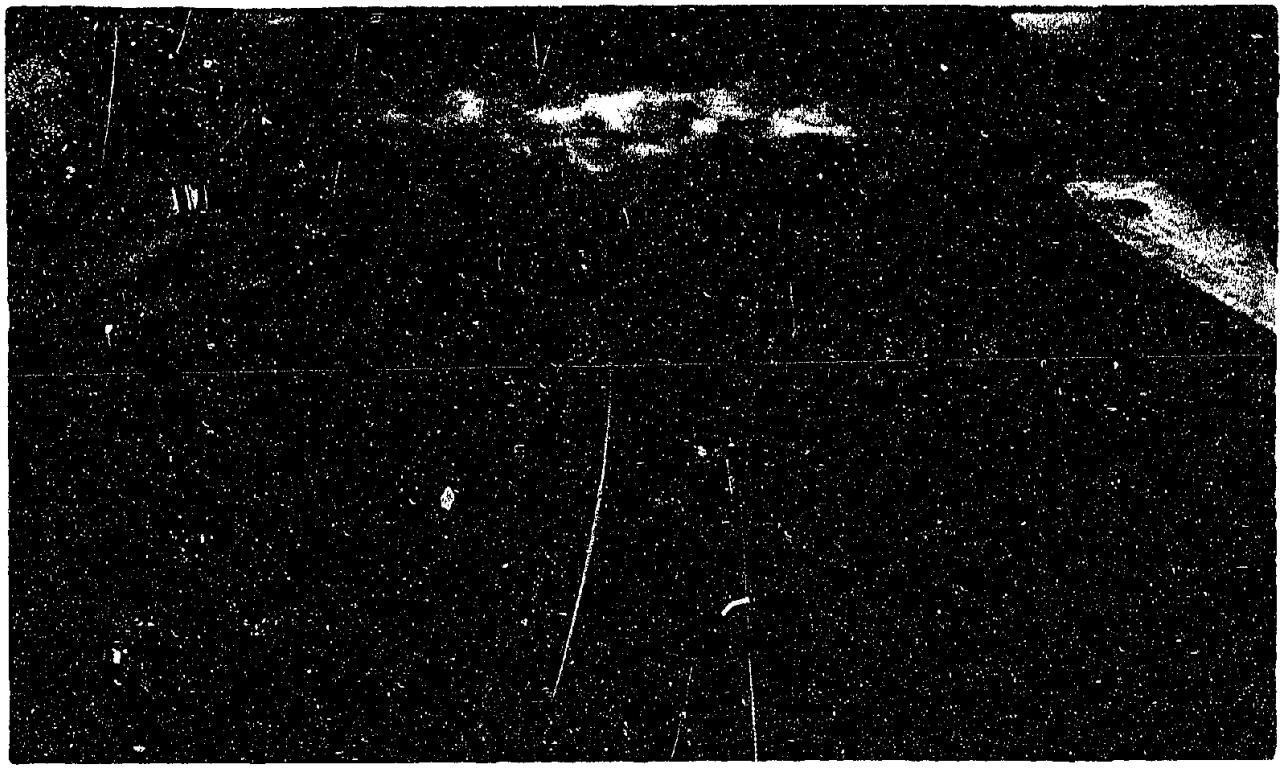
B. SCOUR OF PLATE 7-A.



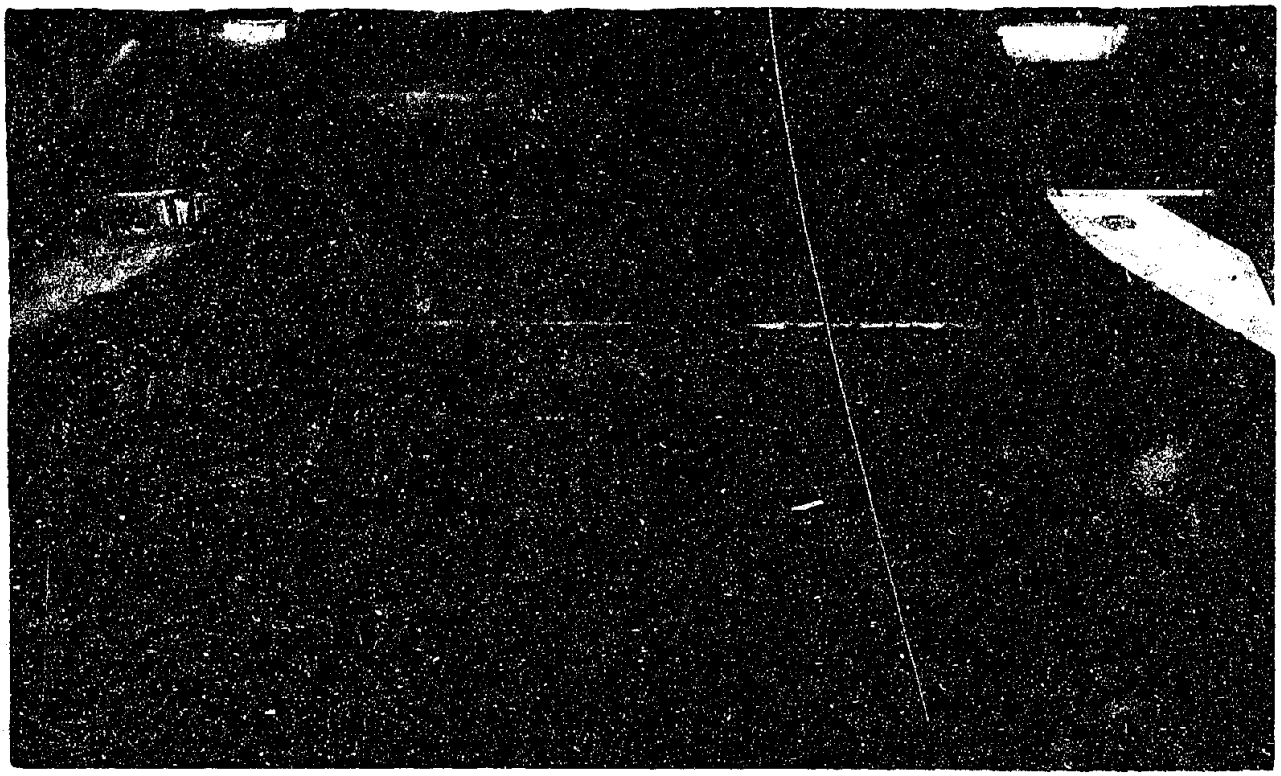
A. 1:18 SCALE MODEL OF OVERCHUTE STILLING BASIN. BEST END SILL 1 FOOT HIGH. NO CHUTE OR FLOOR BLOCKS. DISCHARGE 2500 SECOND-FEET.



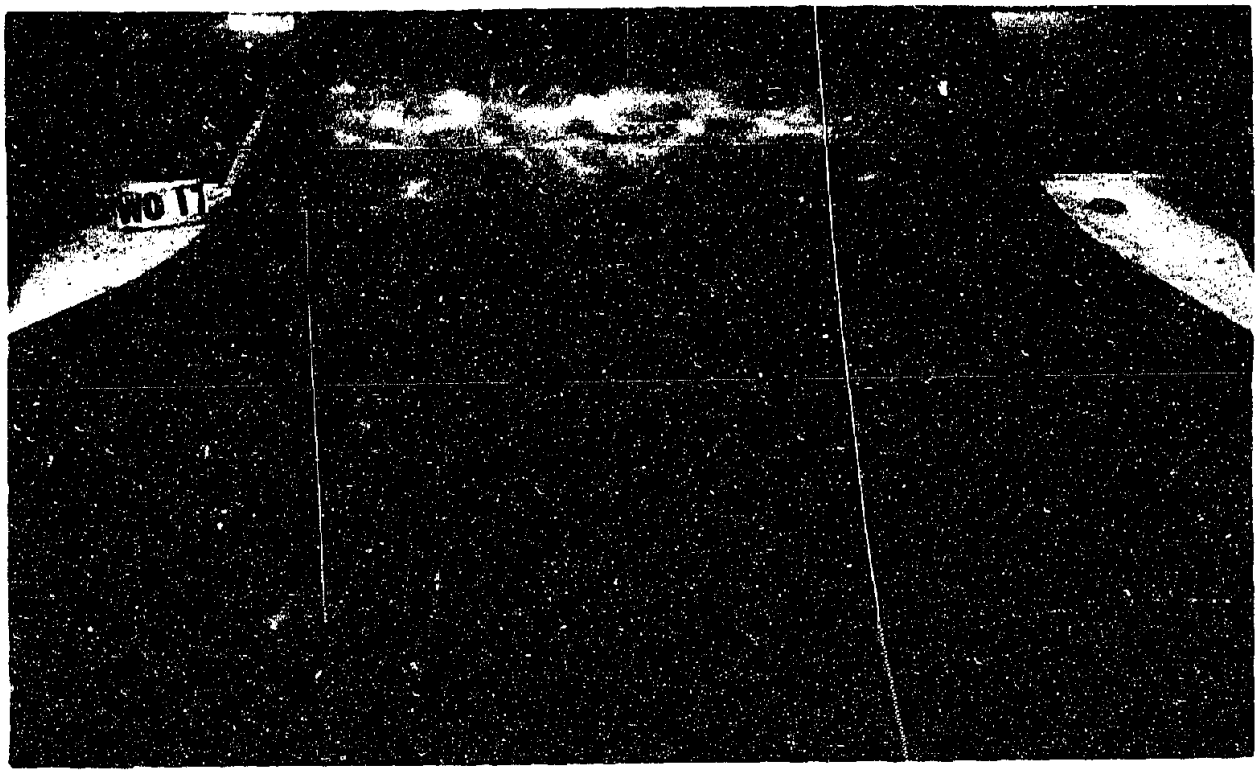
B. SCOUR OF PLATE 7-A.



A. 1:16 SCALE MODEL OF OVERCHUTE STILLING BASIN. 2 FEET HIGH CHUTE BLOCKS AND 1 FOOT HIGH END SILL. NO FLOOR BLOCKS. DISCHARGE 2500 SECOND-FEET.

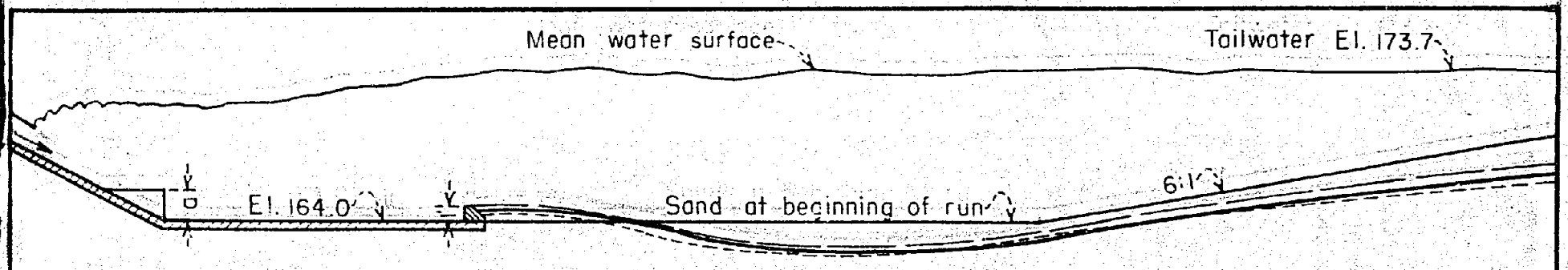


B. SCOUR OF PLATE 8-A.



A. 1:18 SCALE MODEL OF OVERCHUTE STILLING BASIN. 2 FEET HIGH CHUTE BLOCKS AND 1 FOOT HIGH END SILL. NO FLOOR BLOCKS. DISCHARGE 2500 SECOND-FEET.

B. SCOUR OF PLATE 8-A.

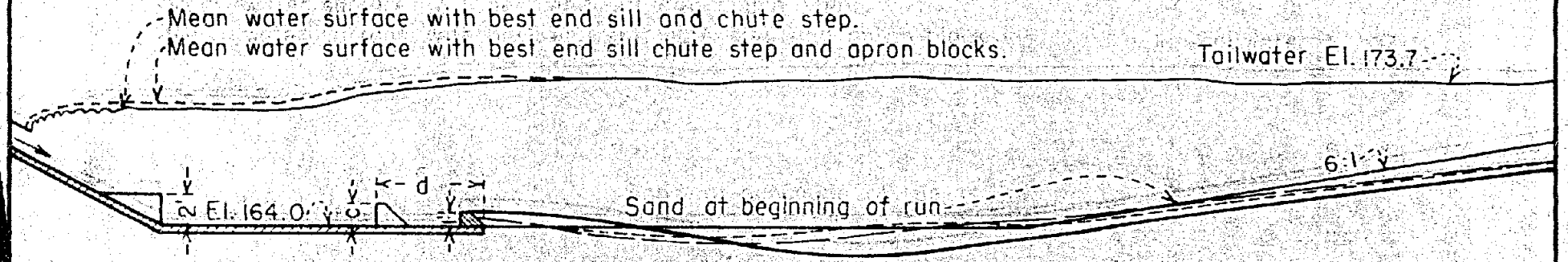


A- EFFECT OF CHUTE STEP SIZE ON SCOUR WITH BEST END SILL AND NO APRON BLOCKS

LEGEND

- no chute step
- c = 1.5 ft.
- c = 2.0 ft.
- c = 2.5 ft.

Conclusion:
The 2.0 ft. chute step is best.



B- EFFECT OF APRON BLOCK SIZE ON SCOUR WITH BEST END SILL AND CHUTE STEP

LEGEND

- no apron blocks
- c = 1.0 ft. d = 6 ft.
- c = 1.0 ft. d = 7.5 ft.
- c = 1.5 ft. d = 7.0 ft.

5 0 5 10 15
SCALE OF FEET

Conclusion:
The 1.5 ft. high trapezoidal apron blocks 7.0 ft. from the end of the apron are best.

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EFFECT OF CHUTE STEP AND APRON BLOCK SIZE ON SCOUR DOWNSTREAM

FIGURE 11

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length of jump. A row of blocks 1.5 feet high placed with their upstream faces 7.0 feet from the end of the floor results in very little scour and a smooth water surface.

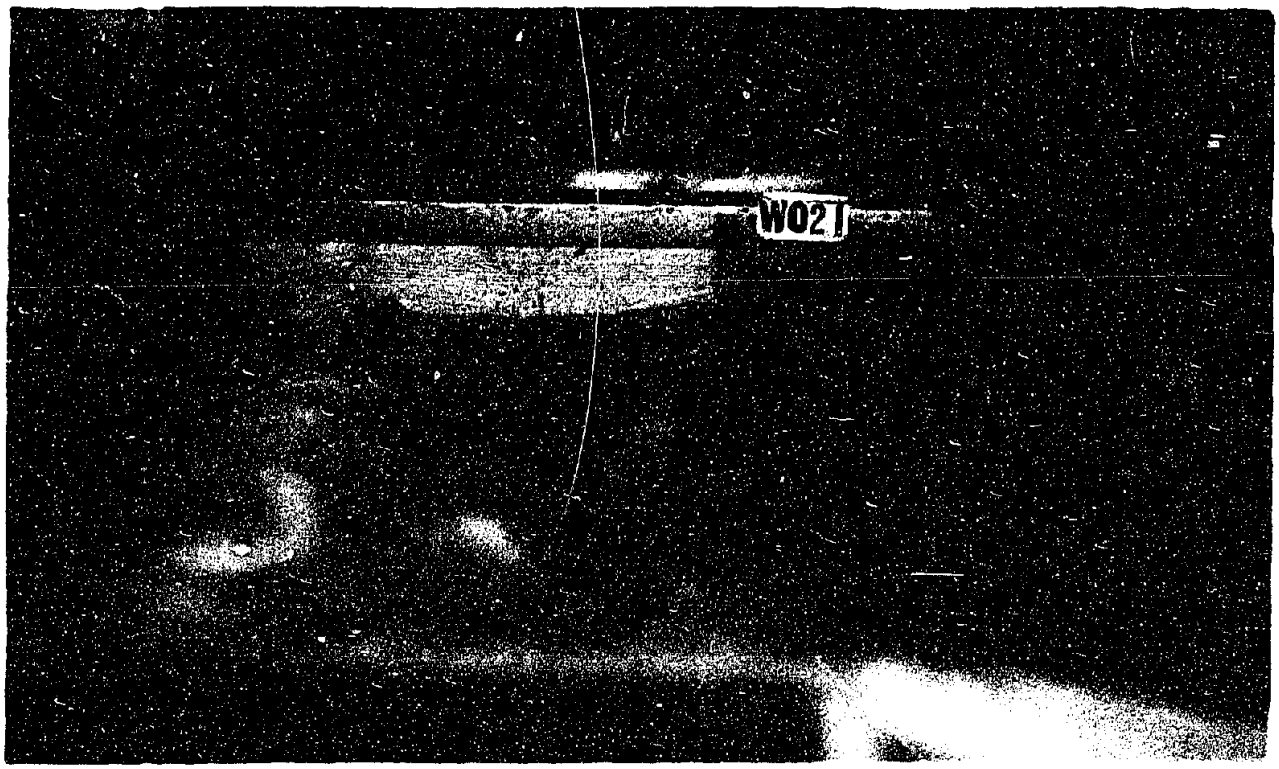
The arguments for the correct sill height are applicable to determining the correct block height. The block height has more effect on scour than does the location; however, if placed too far upstream they receive excessive impact and if placed too near the end sill they become less effective. These and other experiments indicate that the correct location is approximately one-third the floor length from the end of the pool.

Figure 12-A shows the improvement of the best stilling pool arrangement over the original design of 15-inch high chute blocks and a 10-inch by 12-inch rectangular end sill for the elevation 164.0 floor. The maximum difference in scour is about 3 feet and the greater depth of water in the pool gives a greater factor of safety against the jump sweeping out of the pool (plates 9 and 10).

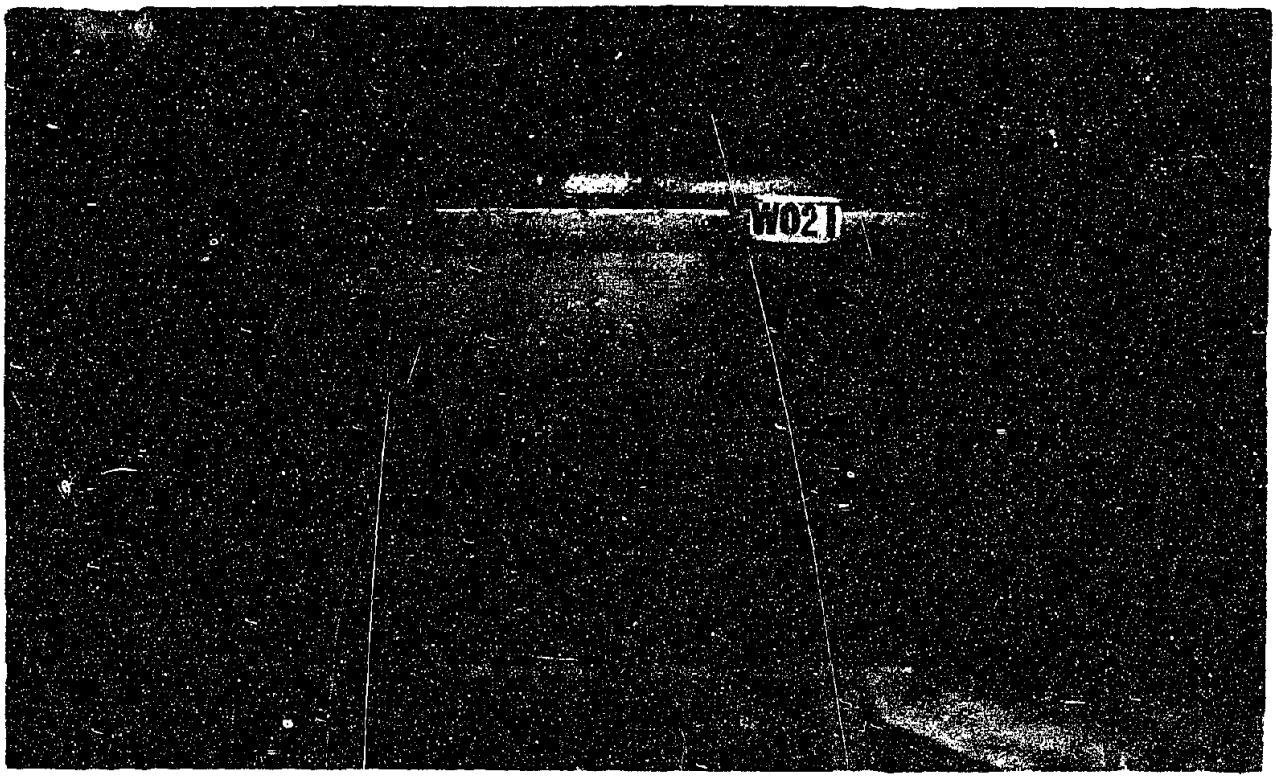
12. Stilling basin floor elevation.

The stilling basin floor elevation is determined from the sheet-piling weir elevation and the necessary tailwater depth to maintain the hydraulic jump. It was found possible to have considerably less tailwater depth than the jump theory indicated without causing the pool to sweep out. Part of this difference is due to the fact that the blocks and sill obstruct the flow, producing higher water surface elevation in the pool.

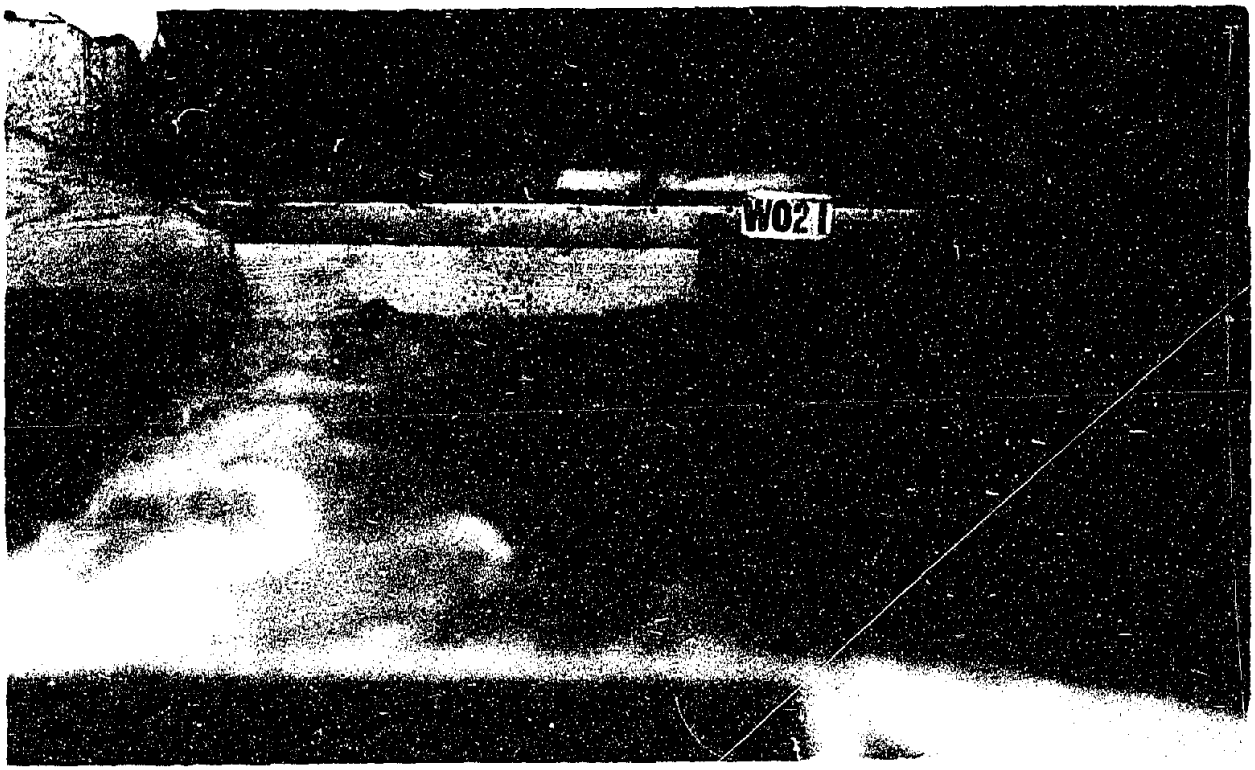
The minimum tailwater elevation for the maximum discharge may be determined when the weir crest elevation is fixed. With the weir crest at elevation 170.0, the critical depth over the weir is 1.22 feet, for 2500 second-feet, and the minimum elevation of the energy gradient at the pool is 170.0 + 3/2 x 1.22 or 171.83. The necessary downstream depth determined by the momentum formula is 8.9 feet. With blocks and



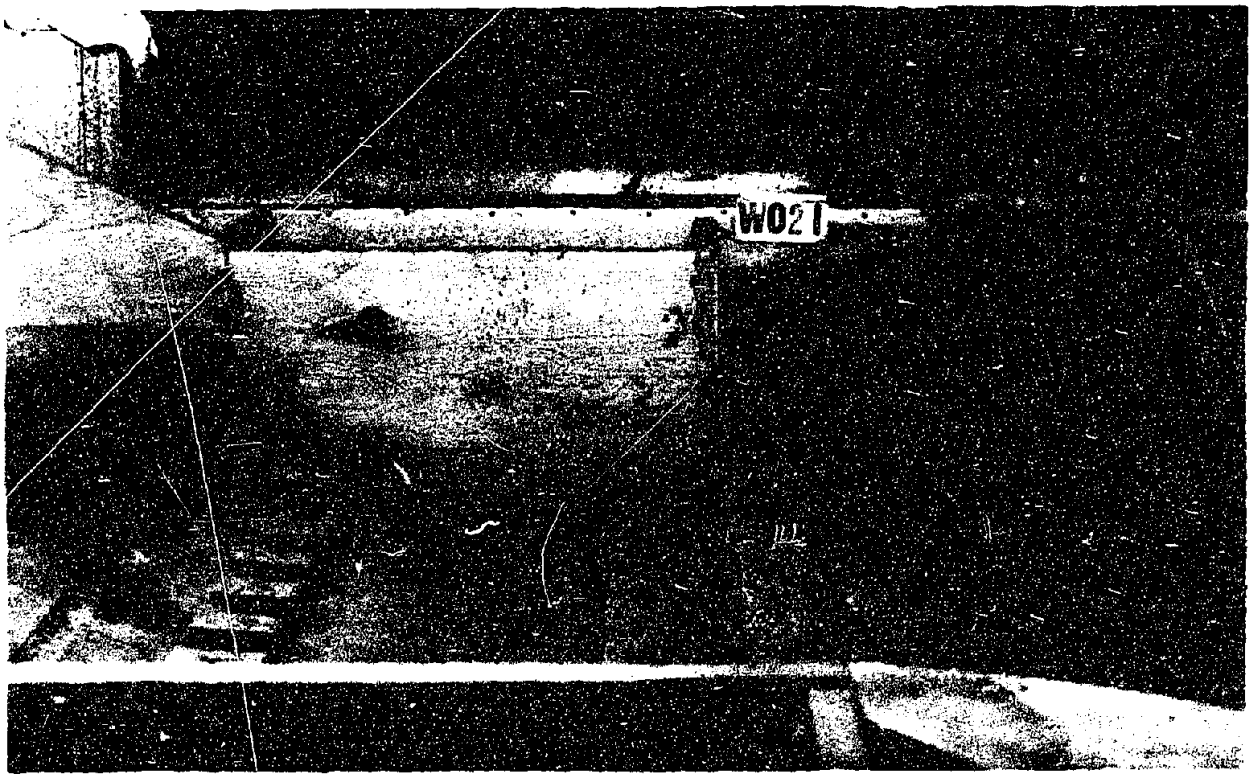
A. 1:18 SCALE MODEL OF OVERCHUTE STILLING BASIN. RECOMMENDED DESIGN. FLOOR ELEVATION 164.0. 2 FEET HIGH CHUTE BLOCKS, 2 FEET HIGH FLOOR BLOCKS 9 FEET FROM THE END OF THE BASIN AND 1 FOOT HIGH END SILL. DISCHARGE 2500 SECOND-FEET.



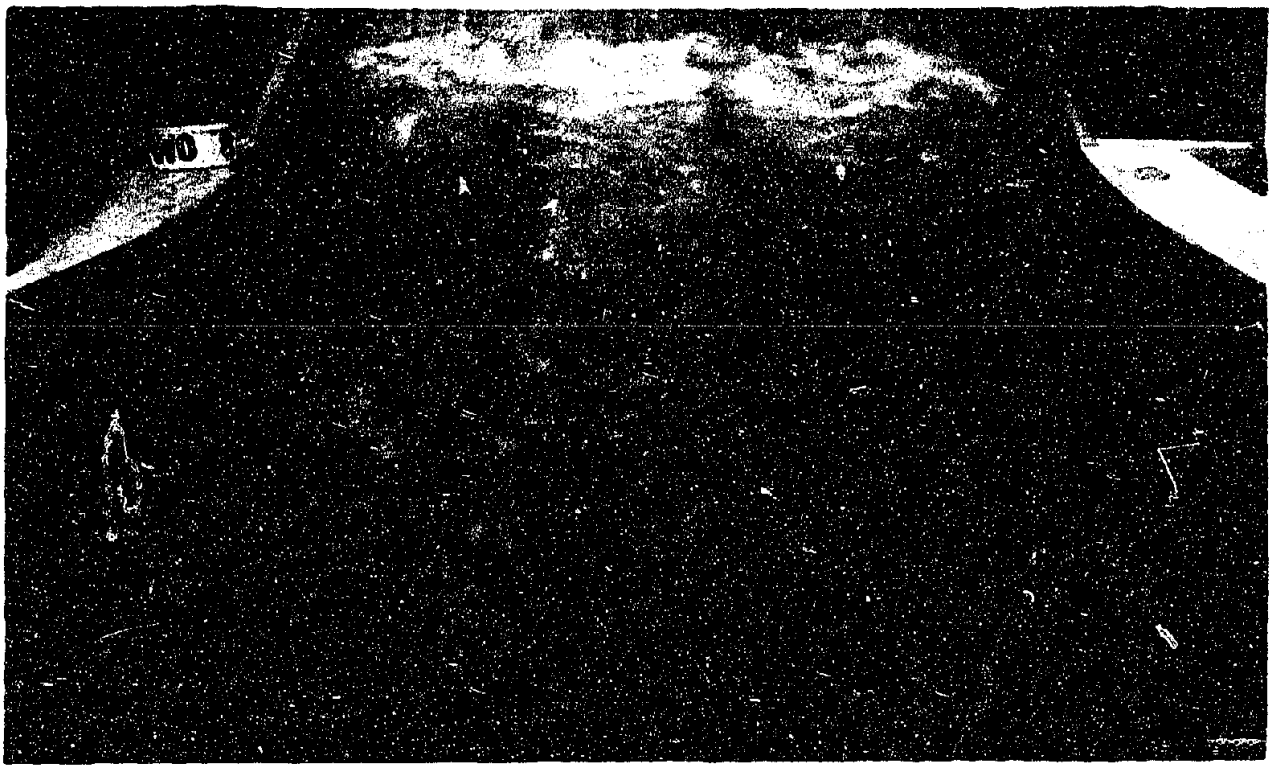
B. SCOUR OF PLATE 9-A.



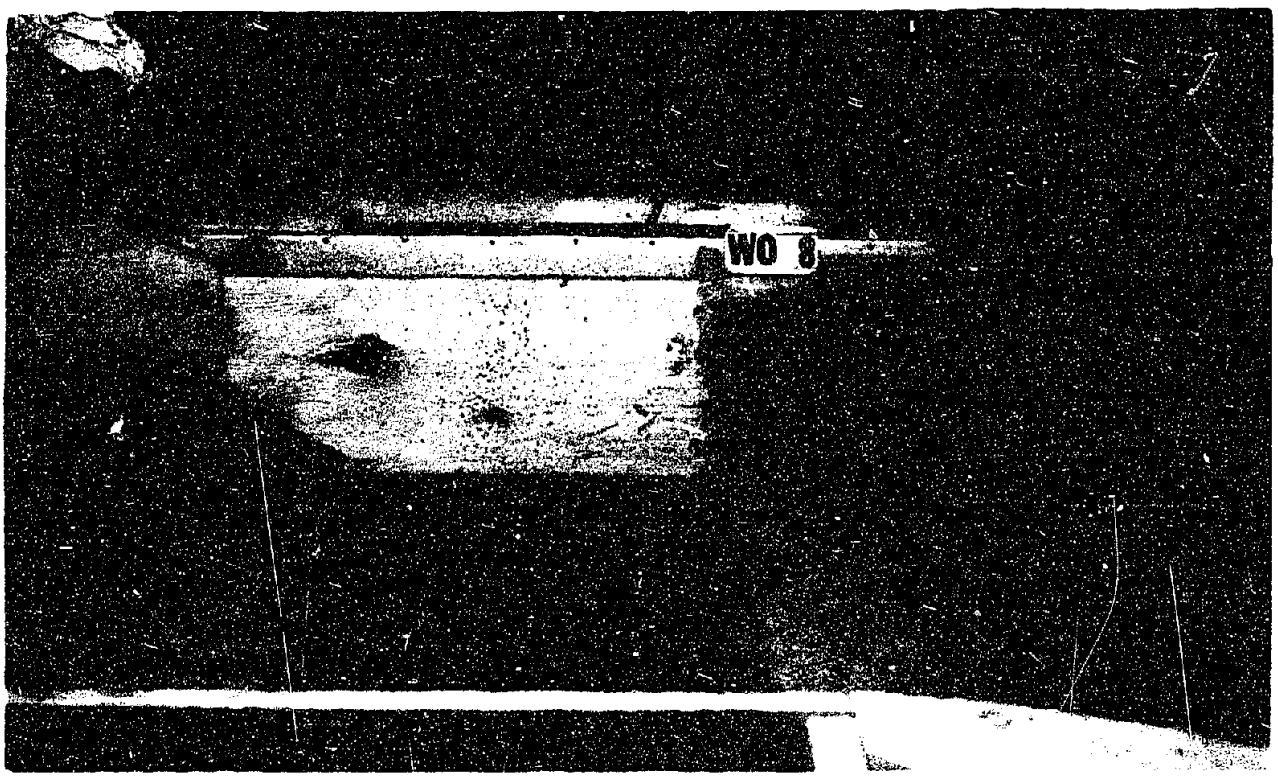
A. 1:16 SCALE MODEL OF OVERCHUTE STILLING BASIN. RECOMMENDED DESIGN. FLOOR ELEVATION 164.0. 2 FEET HIGH CHUTE BLOCKS, 2 FEET HIGH FLOOR BLOCKS 9 FEET FROM THE END OF THE BASIN AND 1 FOOT HIGH END SILL. DISCHARGE 2500 SECOND-FEET.



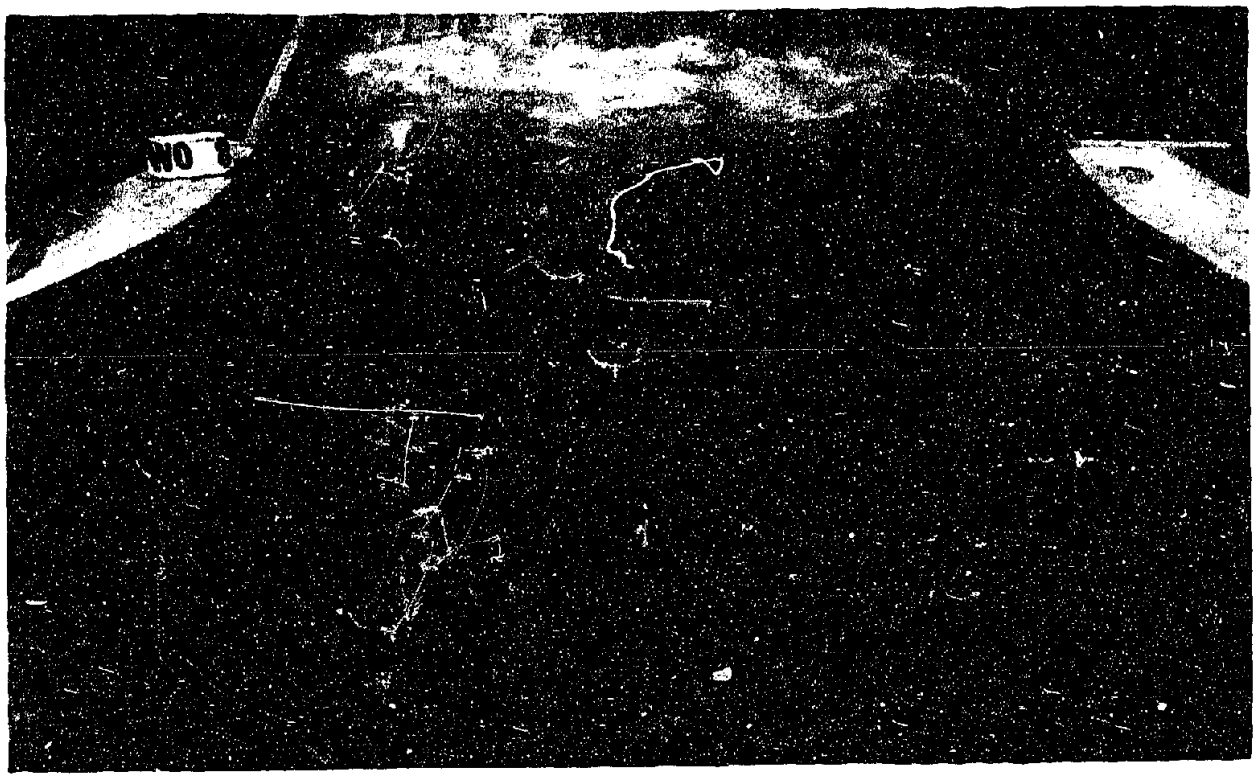
B. SCOUR OF PLATE 9-A.



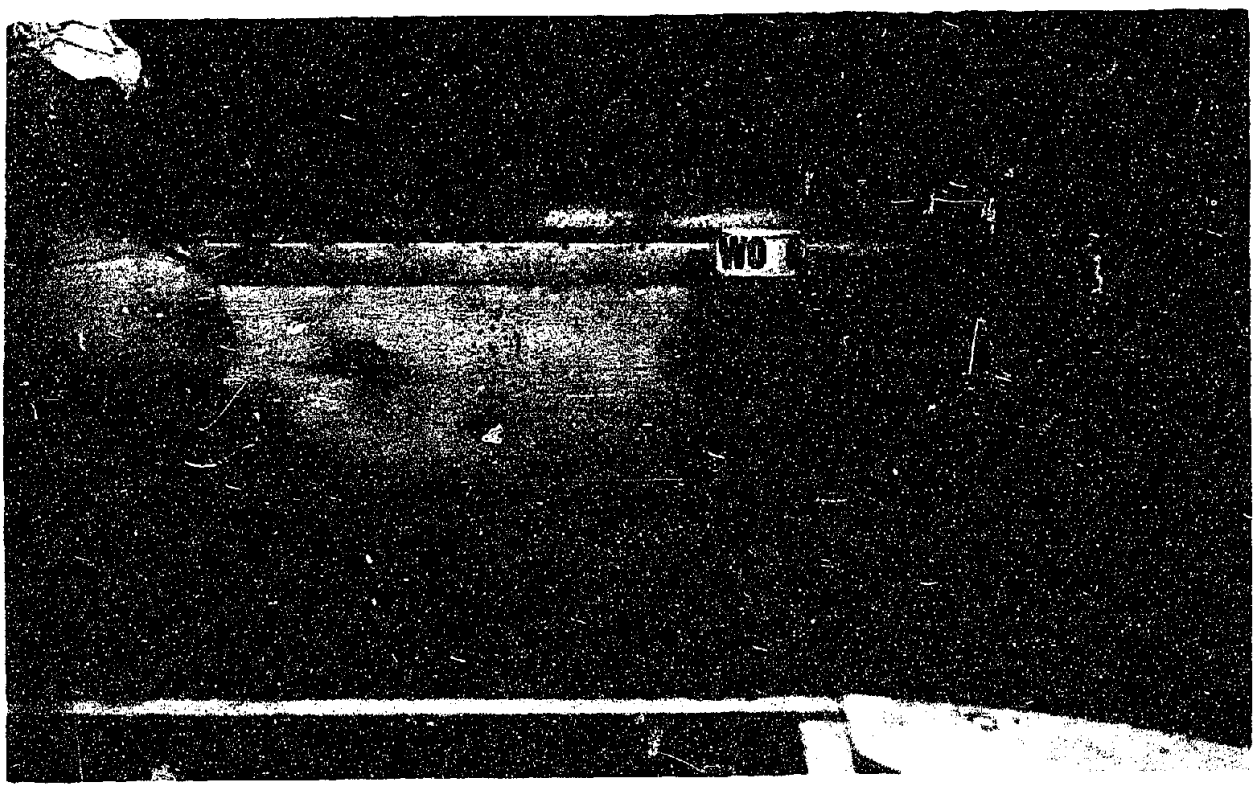
A. 1:16 SCALE MODEL ORIGINAL DESIGN OF OVERCHUTE STILLING BASIN. 2 FEET HIGH CHUTE BLOCKS AND 10-INCH HIGH RECTANGULAR END SILL. DISCHARGE 2500 SECOND-FEET.



B. SCOUR OF PLATE 10-A.



A. 1:18 SCALE MODEL ORIGINAL DESIGN OF OVERCHUTE STILLING BASIN. 2 FEET HIGH CHUTE BLOCKS AND 10-INCH HIGH RECTANGULAR END SILL. DISCHARGE 2500 SECOND-FEET.



B. SCOUR OF PLATE 10-A.

a sill, experiments showed that the depth may be reduced by 15 percent. Eighty-five percent of 8.9 is 7.5 feet and with a floor elevation 164.0 the tailwater elevation will be 171.5 and the energy gradient 171.8.

The first series of tests was with the floor at elevation 164.0 and a tailwater elevation 173.7, corresponding to a depth approximately equal to that indicated by the momentum formula. The second series of tests was with the floor 2 feet higher giving a depth equal to about 85 percent of the theoretical d_2. These conditions correspond approximately to the original floor elevation 164.0 and tailwater elevation 171.5.

Figure 12-B shows comparative tests for various pool arrangements with a tailwater depth of 85 percent of d_2. The best arrangement was found to be with the same chute blocks and end sill as for a tailwater depth equal to d_2, but with apron blocks 2.0 feet high and 9.0 feet from the end of the floor.

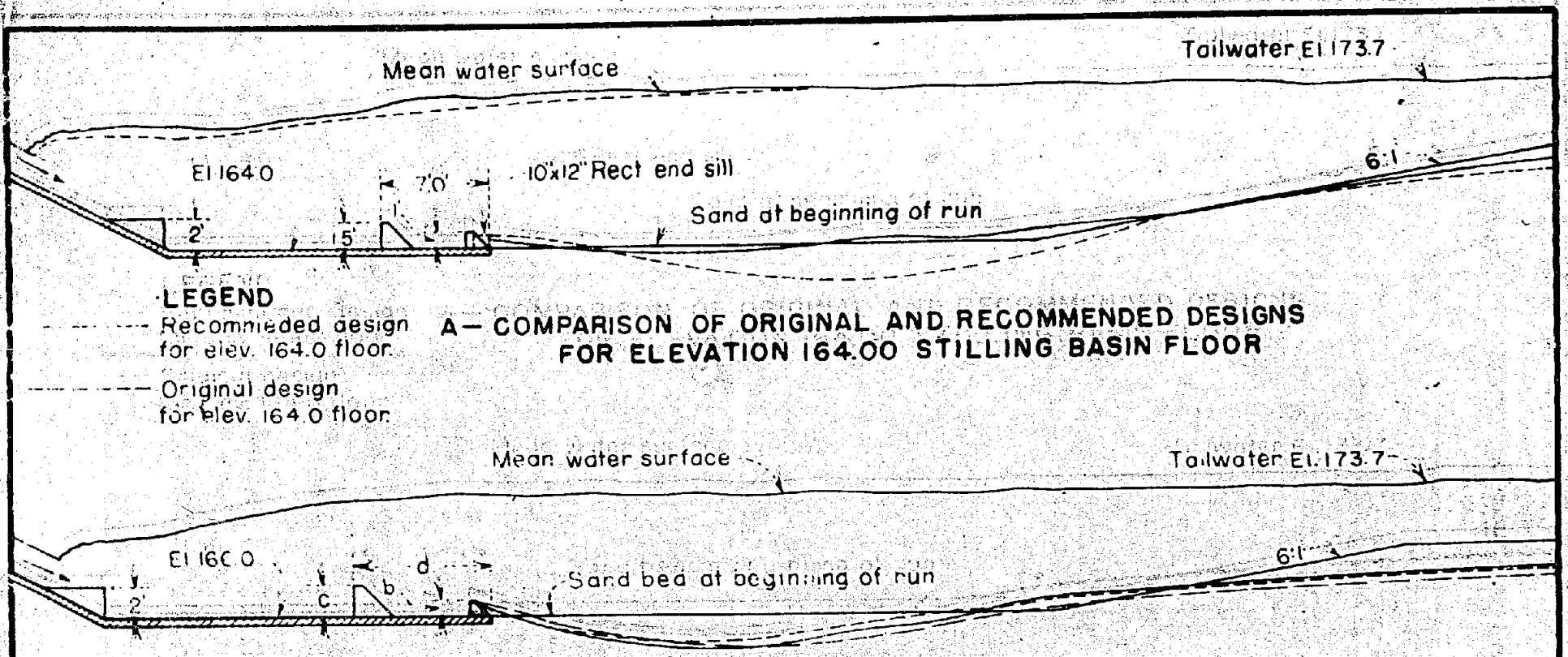
A very effective hydraulic jump formed with the decreased tailwater depth, and it did not appear to be partly submerged as did the jump with the depth equal to d_2. Further testing by lowering the tailwater indicated that the jump sweeps out of the pool when the depth becomes about 6.0 feet. However, the efficiency of the jump rapidly decreases, so the tailwater depth should be no less than 85 percent of the theoretical downstream depth.

If for some reason the tailwater downstream becomes insufficient to cause the jump to form in the stilling basin, there will be moderate scour on the stream bed. Severe scour will result on the sides of the earth channel downstream from the pool if the slopes tested are to be retained.

Figures 13-B and 13-C show the recommended designs for tailwater depths equal to and 85 percent of the theoretical downstream depth, d_2.

13. Stilling basin length.

The length of the stilling basin should cover the range of velocity

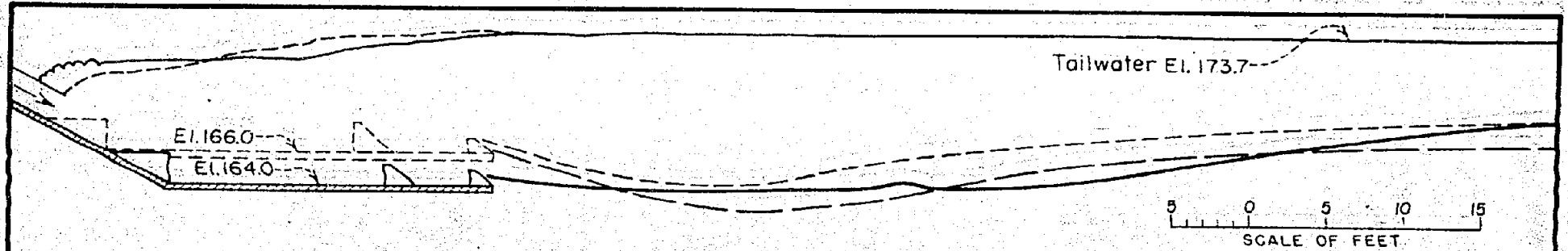


GILA GRAVITY MAIN CANAL

WASH OVERCHUTE
STILLING POOL STUDIES

EROSION AND WATER-SURFACE ELEVATIONS ORIGINAL AND RECOMMENDED POOL DESIGNS

FIGURE 12



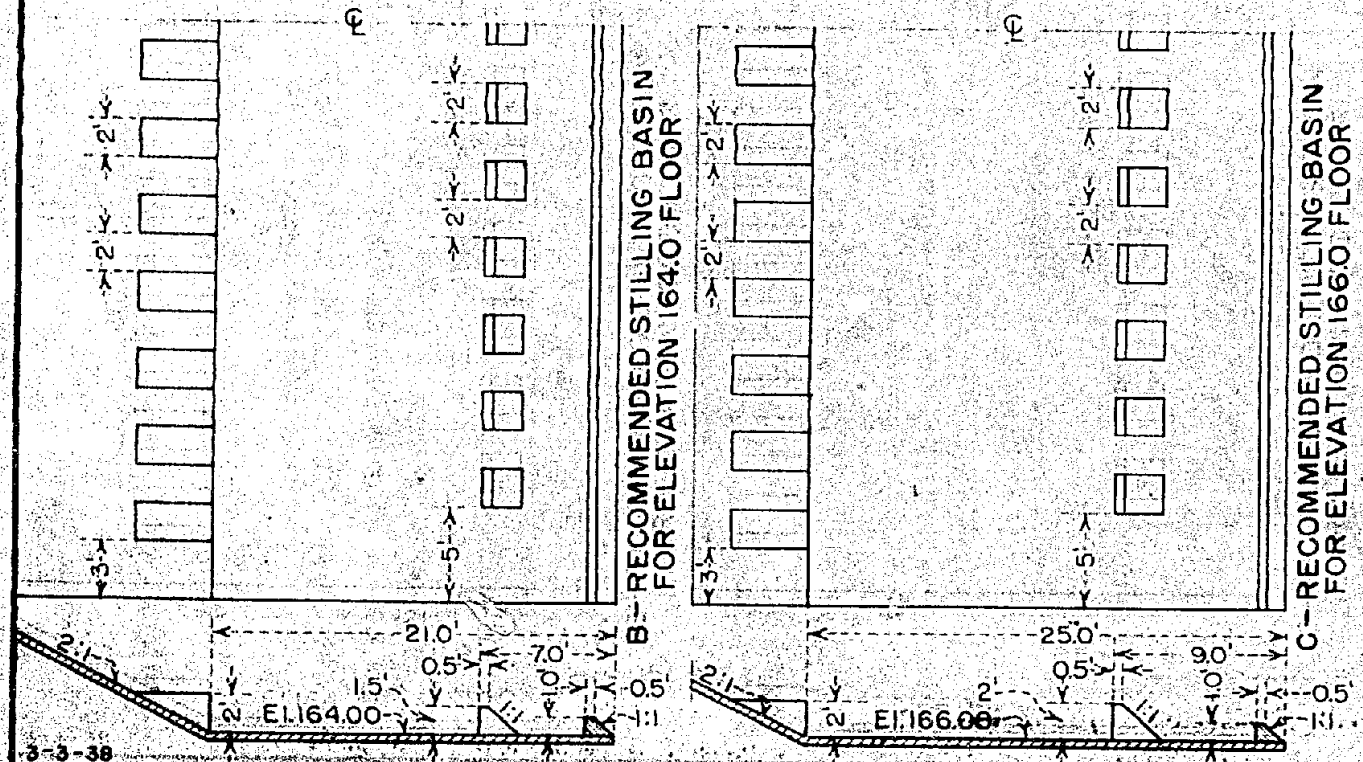
LEGEND

- Best design floor elev. 164.0
- Best design floor elev. 166.0
- Max. scour; jump out of basin; floor elev. 166.0

A- COMPARITIVE TESTS FOR FLOOR ELEVATIONS 164.0 AND 166.0

Conclusion:

Elevation 166.0 floor shows the least scour. Tests showed that the tailwater may be lowered 2 feet before the jump sweeps out.



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WASH OVERCHUTE
STILLING POOL STUDIES
EROSION AND WATER-SURFACE ELEVATIONS DIFFERENT POOL FLOOR ELEVATIONS

FIGURE 13

retardation until a point is reached where the velocities are low enough not to cause excessive erosion of the bed and banks downstream. The apron length may be shortened when baffle blocks, or an end sill, or both, are placed near the end of the basin to lift velocities just clear of the river bed until they are retarded in the natural course of the jump action. The length of jump, or distance downstream, required for full recovery varies as its magnitude, increasing with the discharge, thus the pool length should be that required for the maximum discharge.

No tests were made to determine whether the stilling basin could be shortened as it extended the full length of the jump at the maximum discharge. Any shortening of the basin would place the crest of the jump beyond the end of the pool. For the final design a floor length of 25 feet at elevation 164.0 and the sheet-piling weir crest at elevation 170.0 is recommended.

14. Rules for design of wash overchute stilling basins.

On the basis of the experimental data obtained for the wash overchute stilling basin model, a chart was constructed in which the several variables are given for the design of rectangular stilling basins employing chute steps, floor blocks and end sills (figure 14). By changing the model scale, several prototype values of each variable may be computed from the results of the one model. The chart was plotted by this method.

By studying the experimental data, a general rule for each of the variables was determined. It is believed that these rules are applicable for the range of values given in the chart. The rules follow:

1. The discharge, q, per foot at the pool entrance is equal to the design maximum discharge divided by the width at the pool entrance.

2. The theoretical depth, d_1, at the pool entrance is computed by dividing q by v_1.

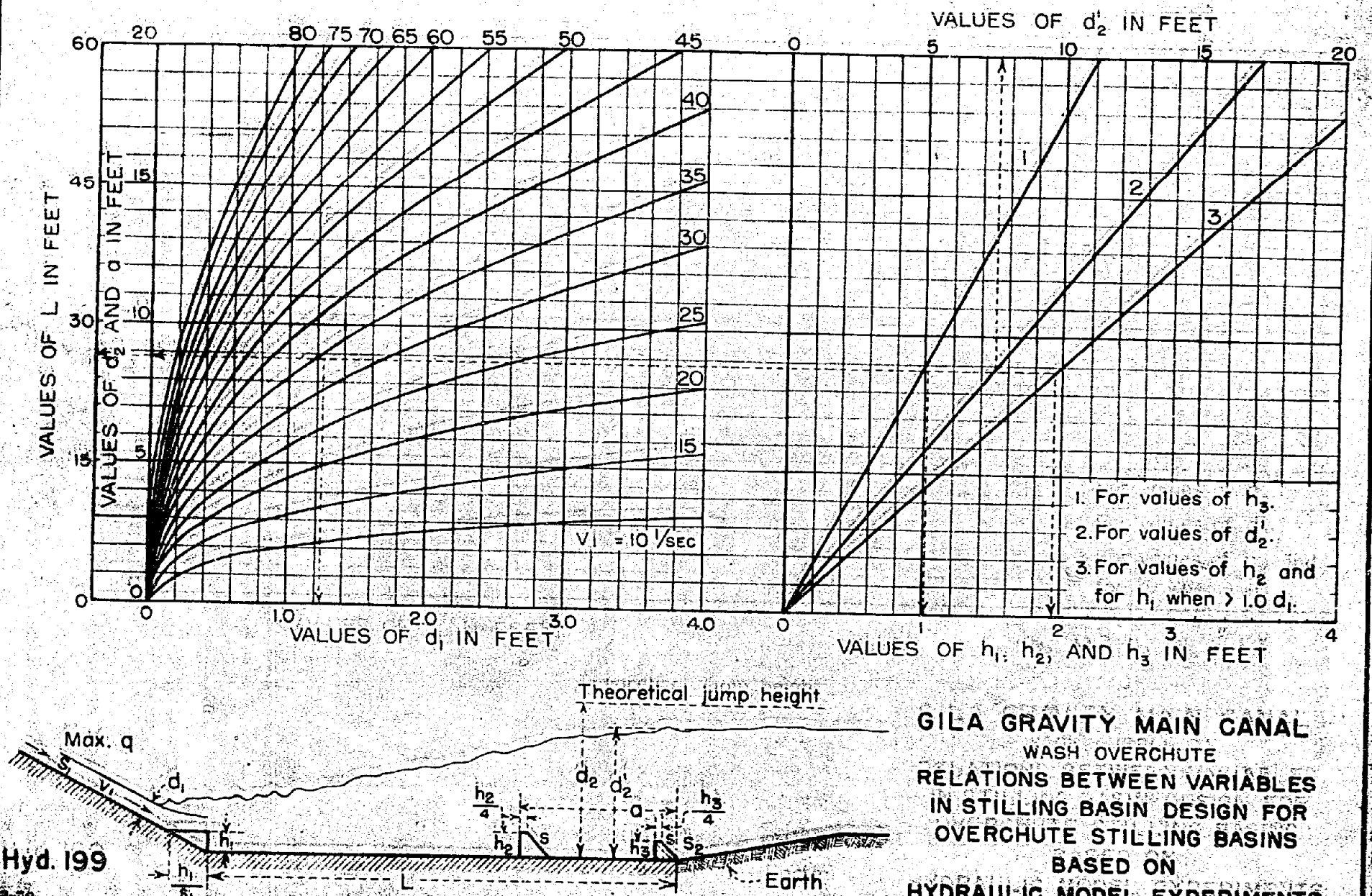


FIGURE 101 - STILLING BASIN DESIGN CHART

3. The theoretical velocity, v_1, at the pool entrance is computed by the formula for flow in steep chutes given by King.

4. The theoretical jump depth, d_2, is computed from the momentum formula.

5. The experimental jump depth, d'_2, is equal to 85 percent of d_2.

6. The required stilling basin floor elevation is equal to the maximum discharge tailwater elevation minus d'_2.

7. The required stilling basin length is equal to $3d_2$.

8. The height h_1, of the chute or toothed apron step is equal to d_1 or about $2/9\ d_2$, whichever is larger.

9. The height, h_2, of floor blocks is equal to $2/9\ d_2$.

10. The height, h_3, of solid end sill is equal to $1/9 d_2$.

11. The distance, a, from the end of the stilling basin to the vertical upstream face of the apron blocks is equal to 1/3L.

12. The top dimensions of the floor blocks and end sill parallel to the length of the basin are equal to $1/4 h_2$ and $1/4 h_3$, respectively.

13. The width of the teeth of the chute step and the apron blocks is equal to h_1 and their spacing is equal to $2h_1$.

14. The teeth of the chute step and the apron blocks should be staggered with no blocks or teeth against the side walls and one more floor block than tooth.

15. The back slopes of the floor blocks and end sill may be such as to be the most economical.

16. The slope of the downstream excavation may vary from the horizontal to 1:6.

17. The chute slope entering the pool may vary from the horizontal to 1:1.

15. Combined overchute and wasteway stilling basin.

The final design of the structure at Station 169+80 included a culvert wasteway from the main canal on each side of the overchute, discharging into the overchute stilling basin. With the maximum discharge of 700 second-feet per wasteway, the velocity entering the pool, v_1, is approximately 30 feet per second and d_1 is 3.0 feet. The corresponding downstream depth, d_2, is 11.5 feet which requires the basin floor to be about 3 feet lower than the adjacent overchute basin floor. For reasons of economy it was desirable to make the entire basin floor at one elevation.

The desired capacity through the wasteways fixed the wasteway bottom elevation at 166.5. To prevent opening of the radial gates by back-flooding in the wasteways when the overchute operates alone, the sheet-piling control weir was lowered to elevation 166.5. From the previous study of the overchute alone, a floor at elevation 160.0 was indicated. The recommended blocks and sill of the overchute model were employed and a test with the overchute operating alone showed good jump conditions to exist except on each side in front of the wasteways where there were reverse whirls. Operation of the wasteways alone resulted in a large whirl in front of the overchute.

An attempt was made to eliminate the whirls by placing dividing walls between the overchute and wasteway sections which essentially converted the single basin into three separate pools. Conditions were excellent for the overchute operating alone, but for the wasteway flow the jump was swept off the floor. This was expected since the floor was about three feet too high. Because of the additional cost of lowering the floor and the cost of dividing walls, the plan of separate stilling basins was abandoned.

The remaining alternative was to effectively spread the separate jets over the floor so that the whirls of separate operation were reduced

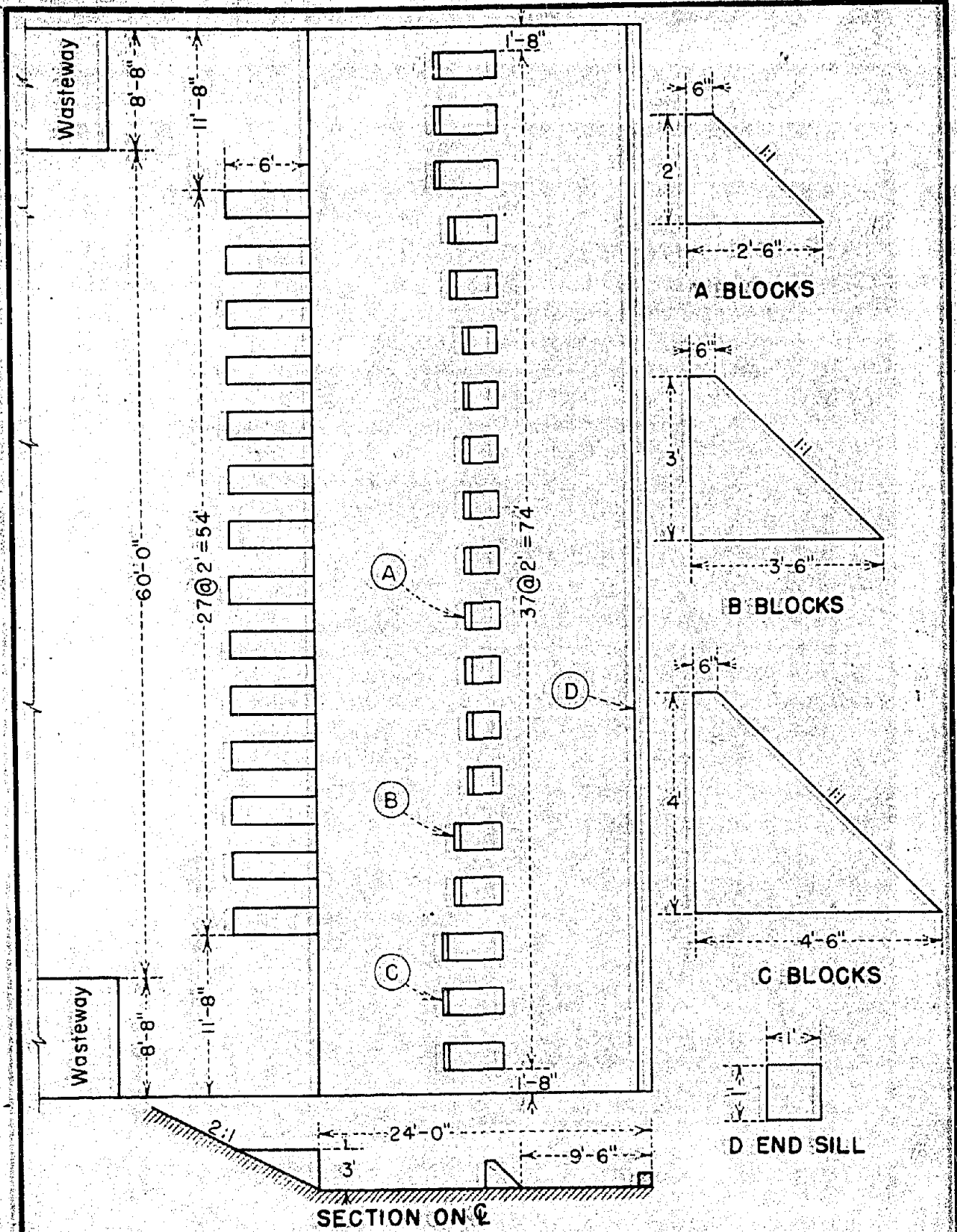
to a minimum. Several sizes of training weirs were placed at different angles on the slope in front of the wasteways in an attempt to spread the wasteway jets. Conditions were improved, but it was difficult to obtain a weir height and length suitable for all discharges.

A number of tests were next made using larger apron blocks in front of the wasteways. This scheme proved the best, resulting in the least scour and best whirl conditions. Figure 15 shows the recommended stilling basin design, resulting from about 60 different tests. It is the most economical design which will reduce scour to a minimum. For this design there was no scour along either of the transition walls at the end of the pool and there was some piling of sand against the full length of the end sill. The maximum scour about 15 feet beyond the end of the basin for any combination of flow was about 1.5 feet. This arrangement resulted in the best jump conditions for the three flows: (1) wasteways only discharging, (2) overchute only discharging, and (3) both wasteways and overchute discharging (plates 11 to 16, inclusive).

The best size of end sill was found to be the same as indicated by the original 1:18 scale model. The apron blocks in the center half of the basin width were also of the same size as those of the original model. The chute step height was increased to 3 feet to prevent excessive whirls on each side of the pool when the overchute operated alone. The larger size apron blocks in front of the wasteways were effective in spreading the wasteway flow over the floor when the wasteways operate alone. With larger chute steps, the smallest whirls and least scour resulted when the vertical faces of the "A" blocks were placed 12 feet from the end of the basin rather than 9 feet as found in the first model.

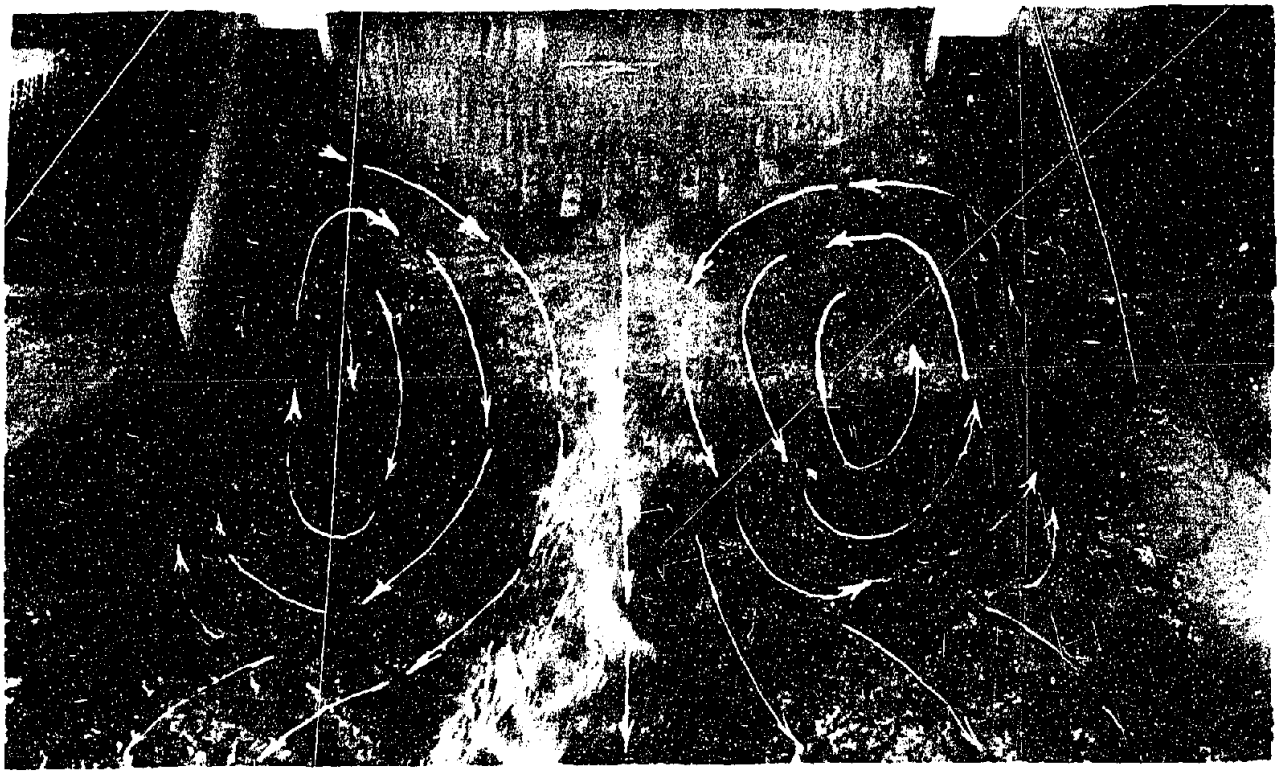
A low velocity whirl exists in the centerof the pool between the row of apron blocks and the 2:1 chute slope when the culverts operate alone. Low velocity whirls exist at either side wall when the overchute operates alone. It is considered impossible to economically eliminate

FIGURE 15

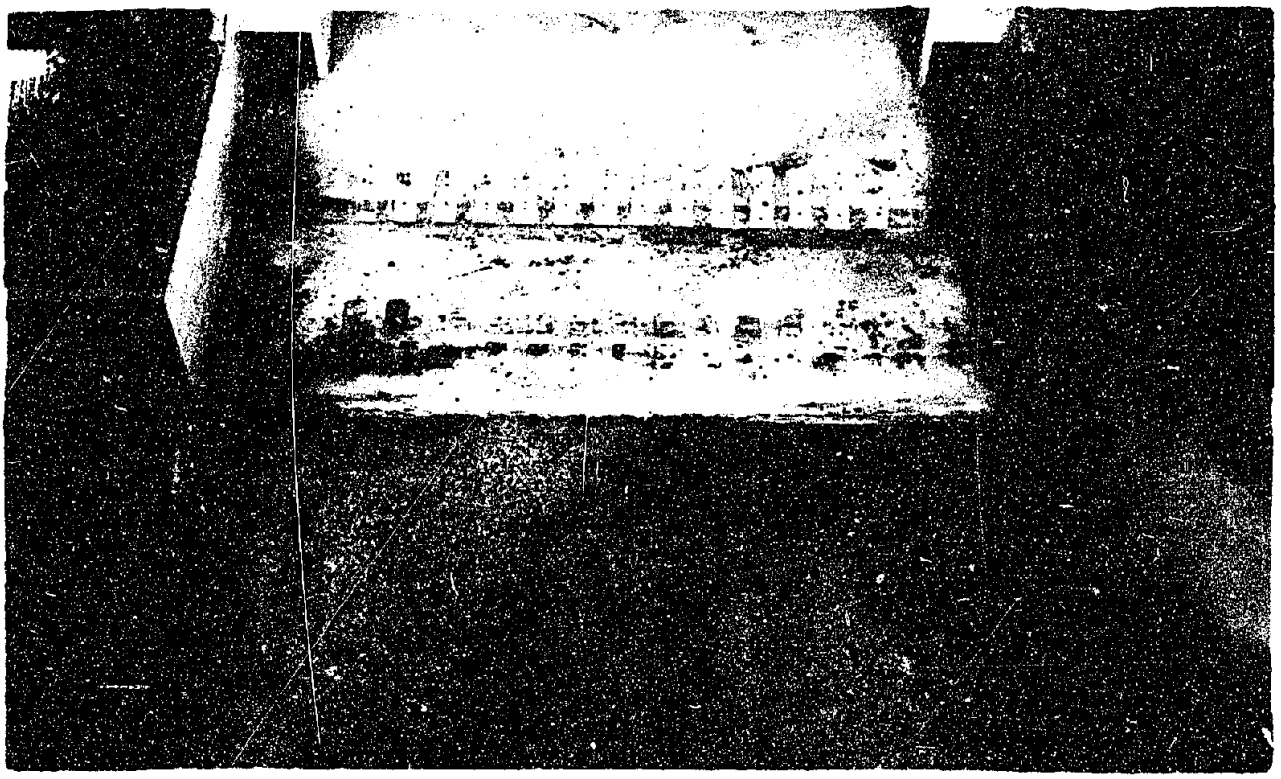


GILA GRAVITY MAIN CANAL
WASH OVERCHUTE
STILLING POOL STUDIES
DETAILS OF RECOMMENDED OVERCHUTE AND
WASTEWAY STILLING BASIN DESIGN

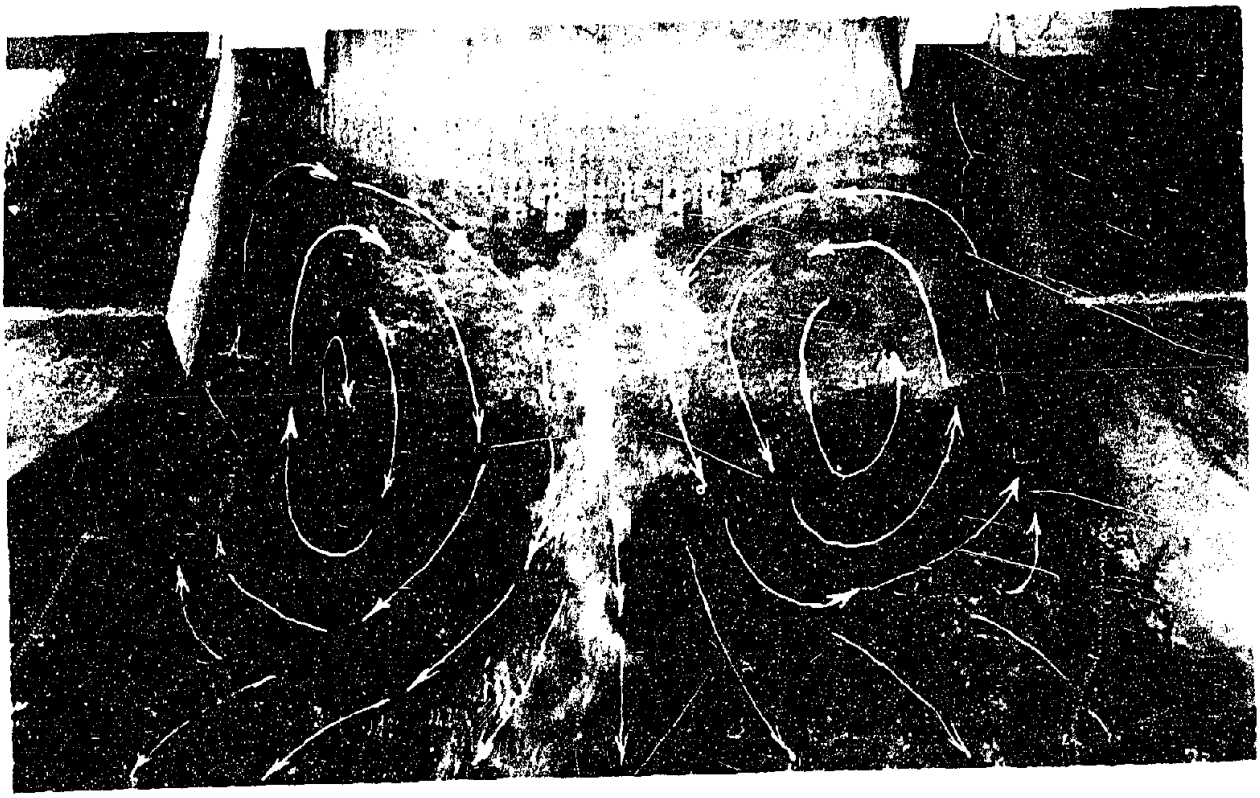
3-3-38



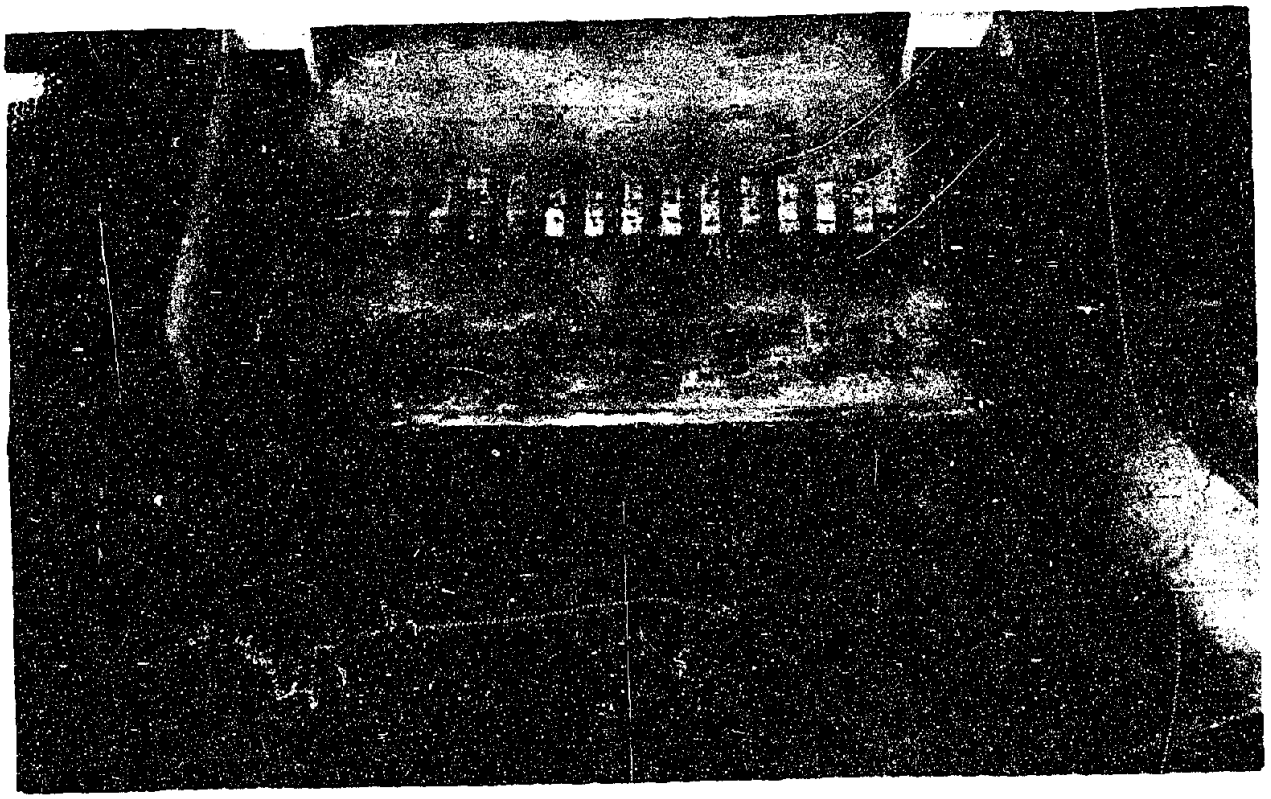
A. 1:24 SCALE MODEL OF OVERCHUTE AND WASTEWAY STILLING BASIN. NO BLOCKS OR END SILL. DISCHARGE 2500 SECOND-FEET.



B. SCOUR OF PLATE 11-A.

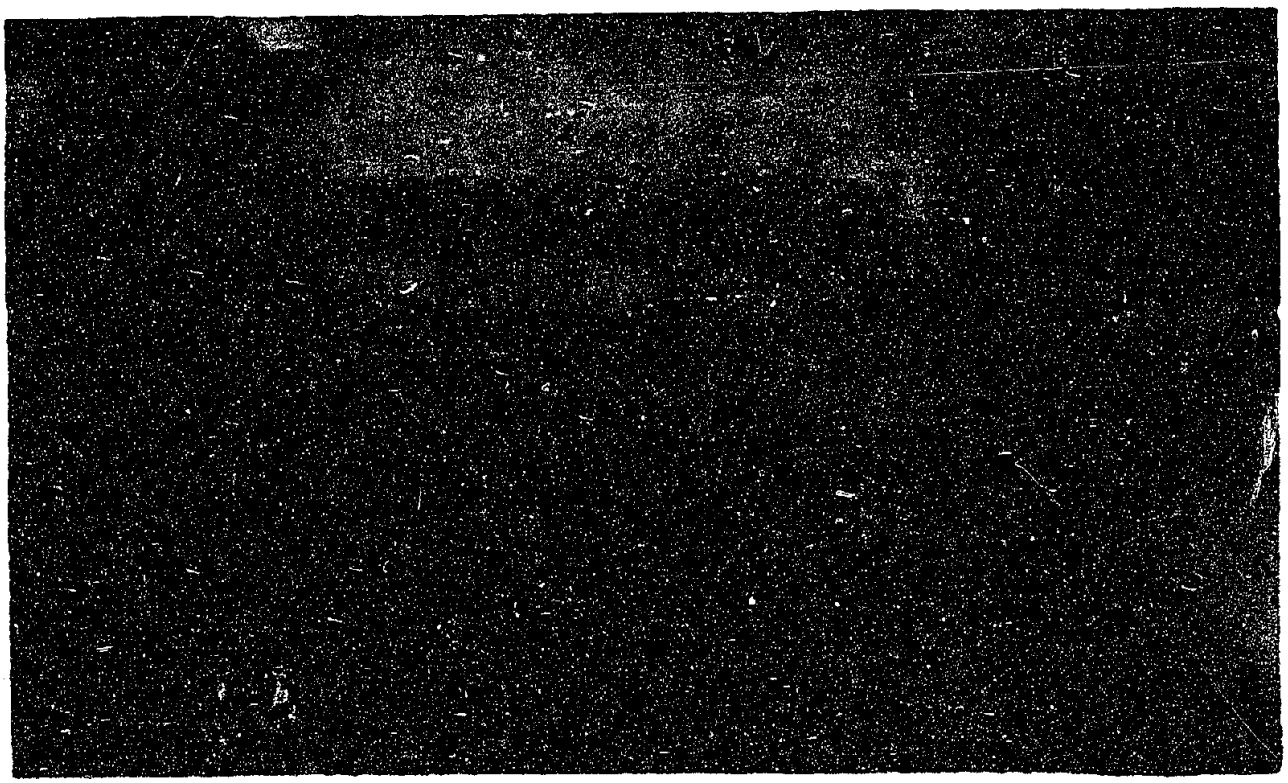


A. 1:24 SCALE MODEL OF OVERCHUTE AND WASTEWAY STILLING BASIN. NO BLOCKS OR END SILL. DISCHARGE 2500 SECOND-FEET.



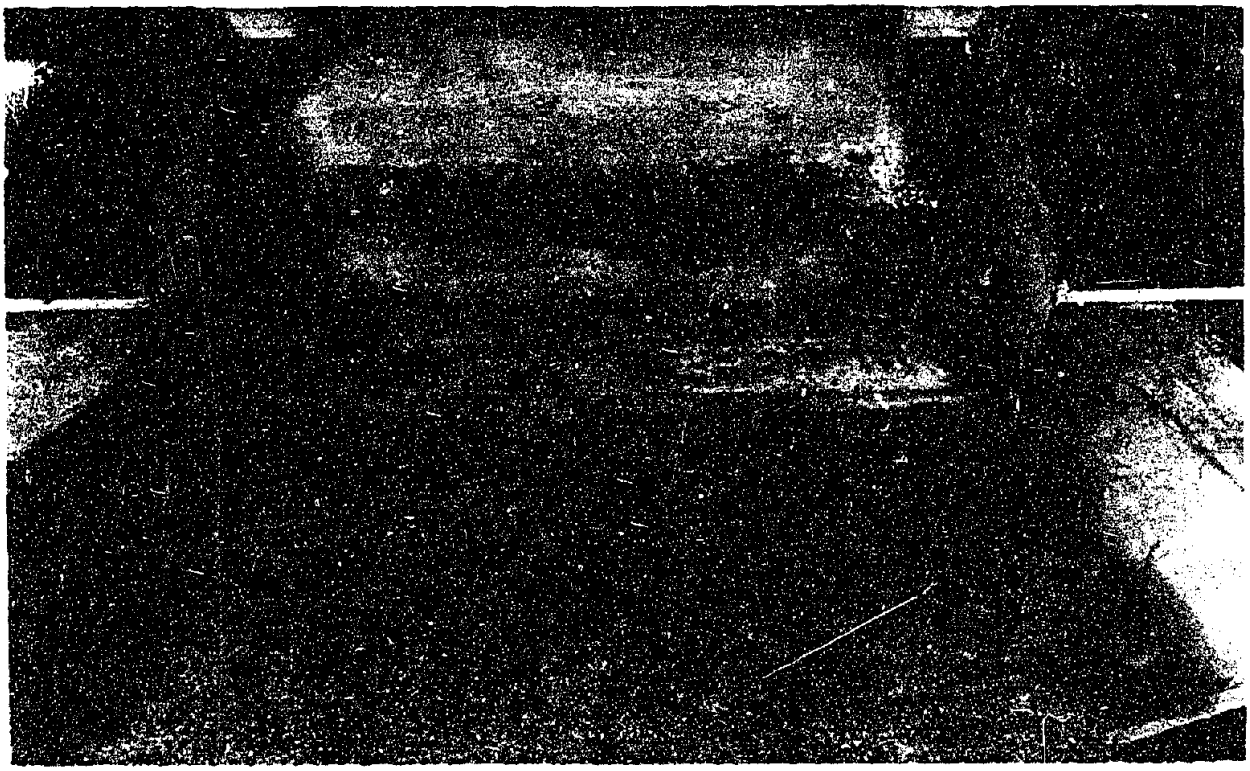
B. SCOUR OF PLATE 11-A.

A. 1:24 SCALE MODEL OF RECOMMENDED OVERCHUTE AND WASTEWAY STILLING BASIN. OVERCHUTE OPERATING ALONE. DISCHARGE 2500 SECOND-FEET.

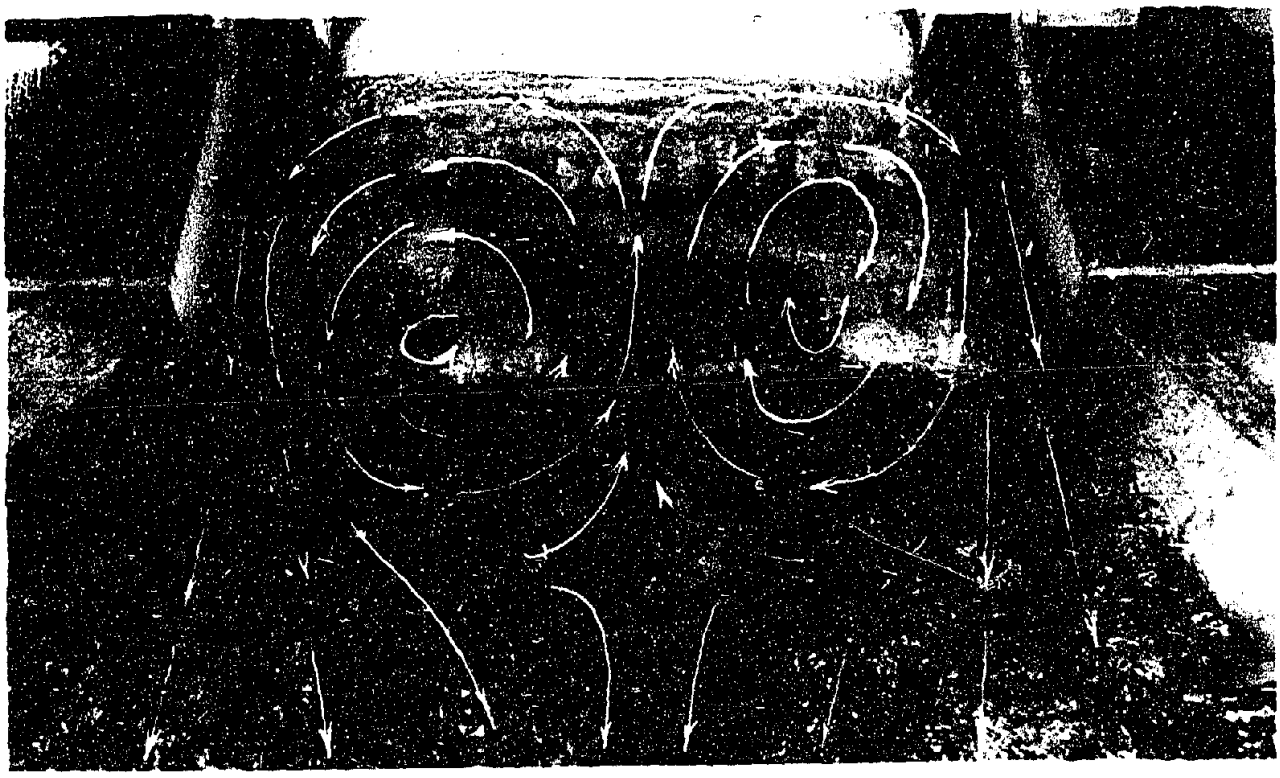


B. SCOUR OF PLATE 12-A.

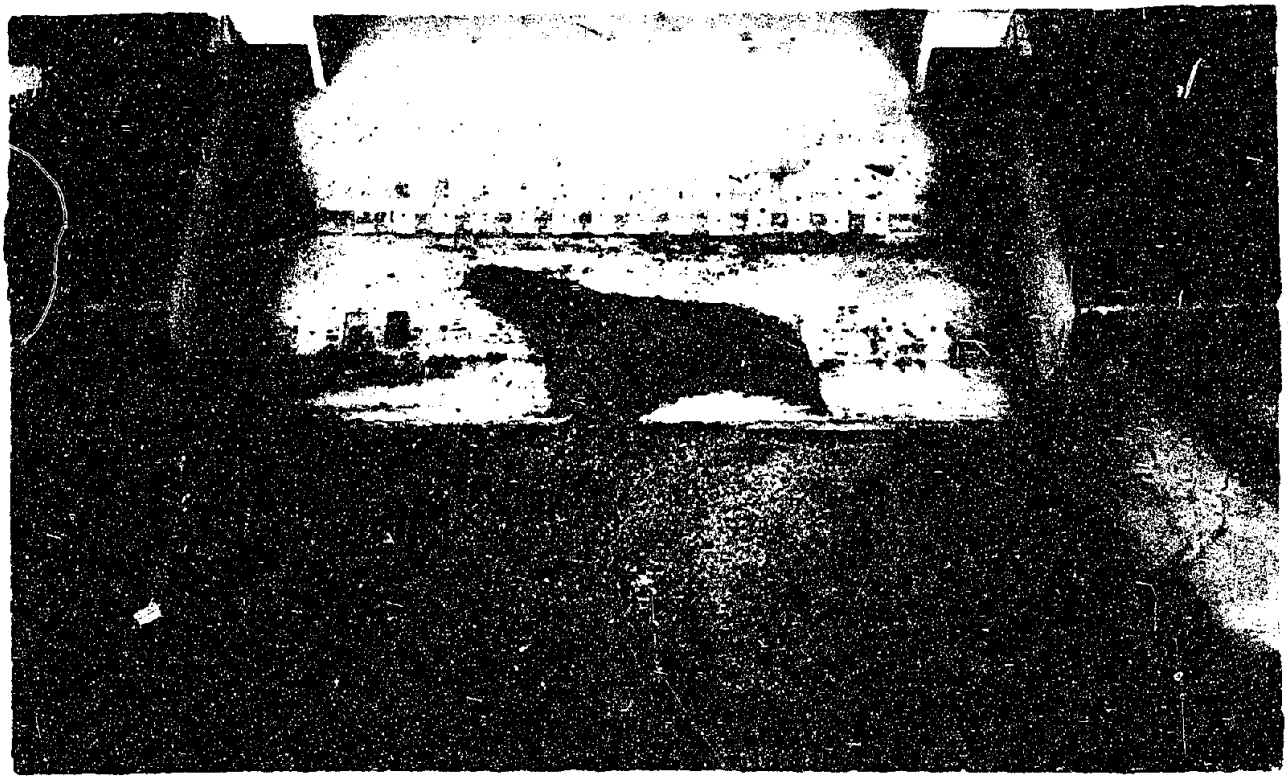
A. 1:24 SCALE MODEL OF RECOMMENDED OVERCHUTE AND WASTEWAY STILLING BASIN. OVERCHUTE OPERATING ALONE. DISCHARGE 2500 SECOND-FEET.



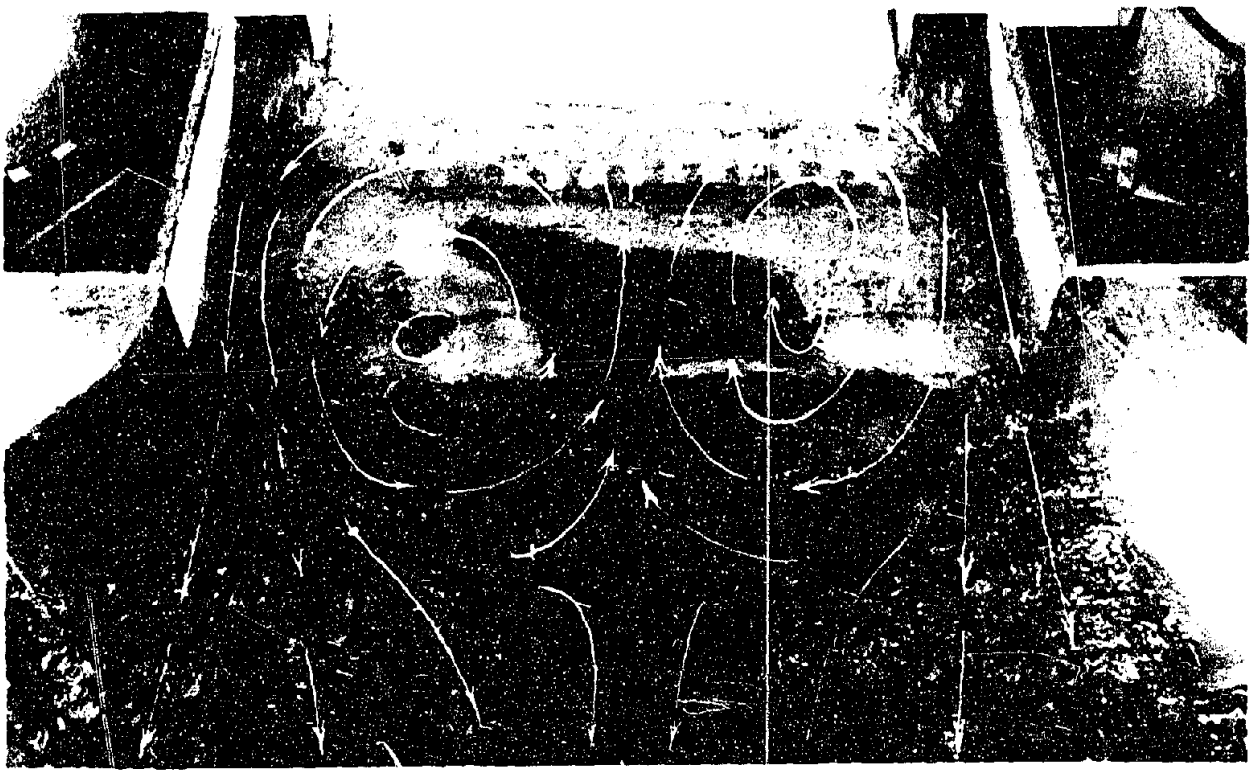
B. SAME AS PLATE 14-A.



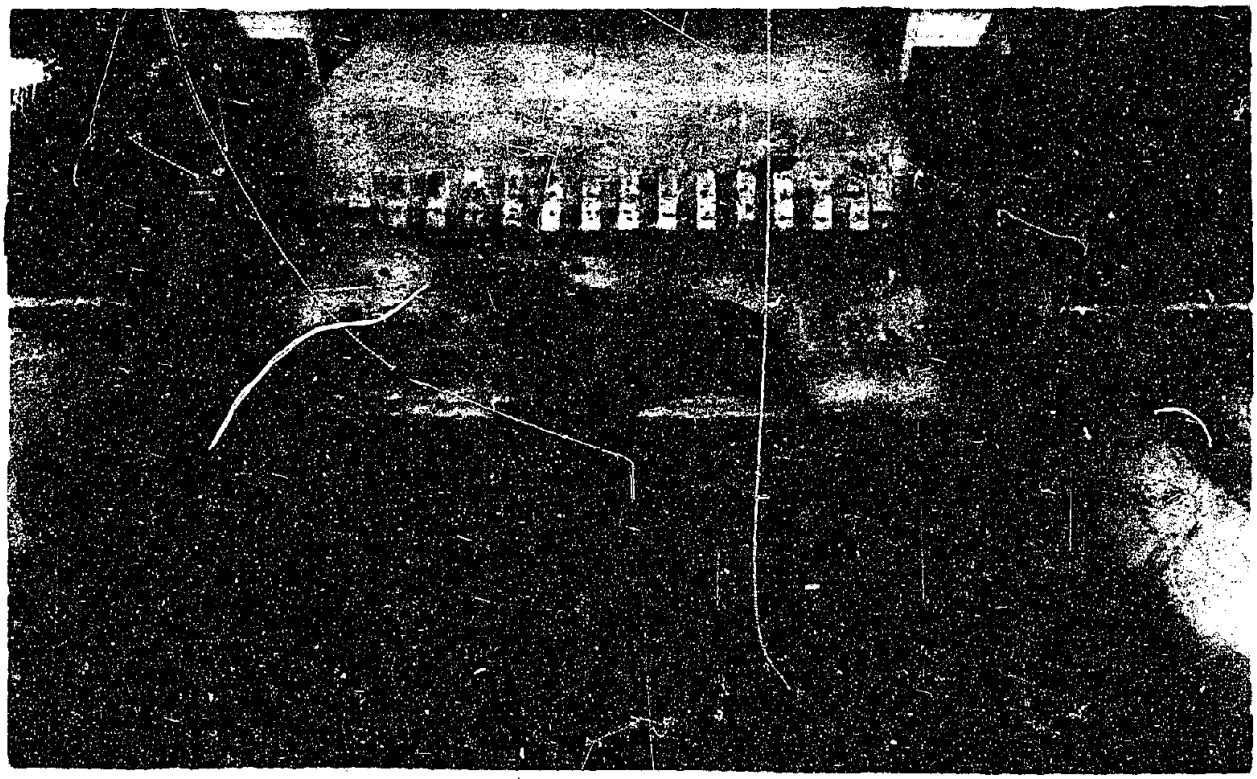
A. 1:24 SCALE MODEL OF OVERCHUTE AND WASTEWAY STILLING BASIN. NO BLOCKS OR END SILL. WASTEWAYS ONLY OPERATING. DISCHARGE 1390 SECOND-FEET.



B. SCOUR OF PLATE 13-a.



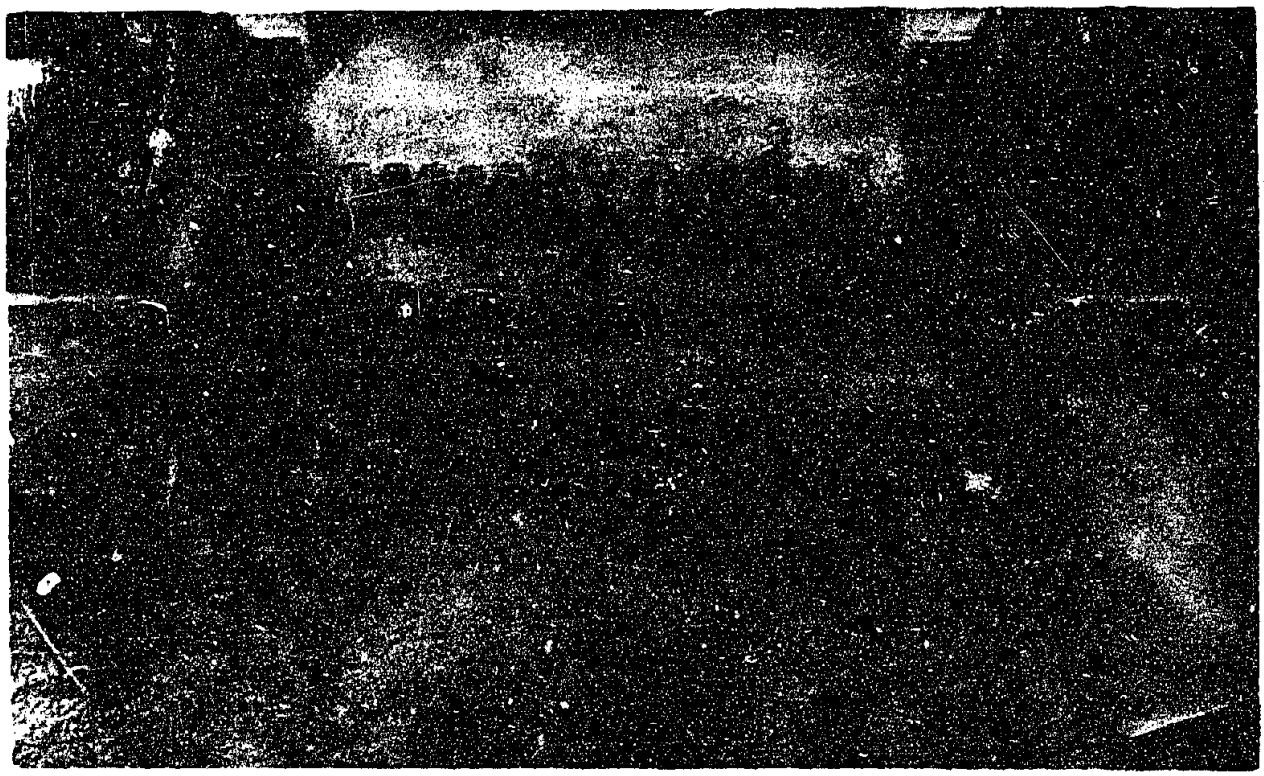
A. 1:24 SCALE MODEL OF OVERCHUTE AND WASTEWAY STILLING BASIN. NO BLOCKS OR END SILL. WASTEWAYS ONLY OPERATING. DISCHARGE 1390 SECOND-FEET.

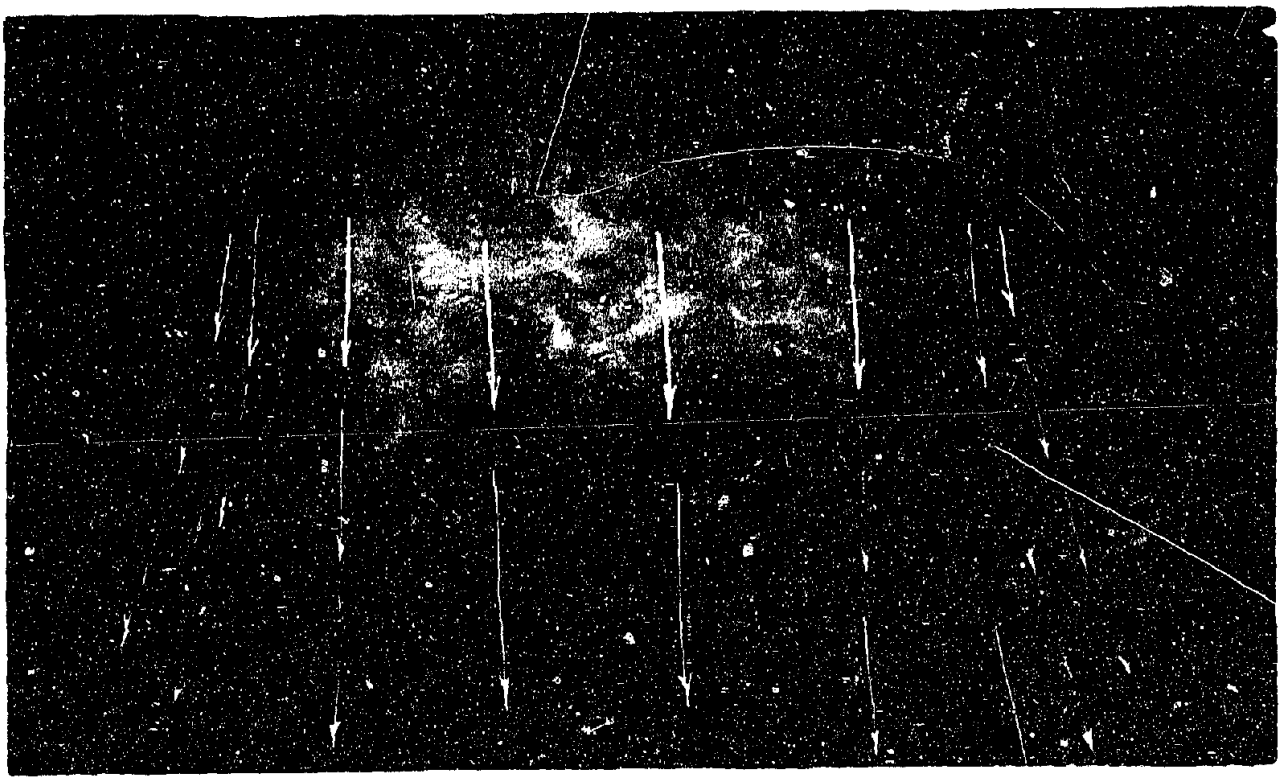


B. SCOUR OF PLATE 13-A.

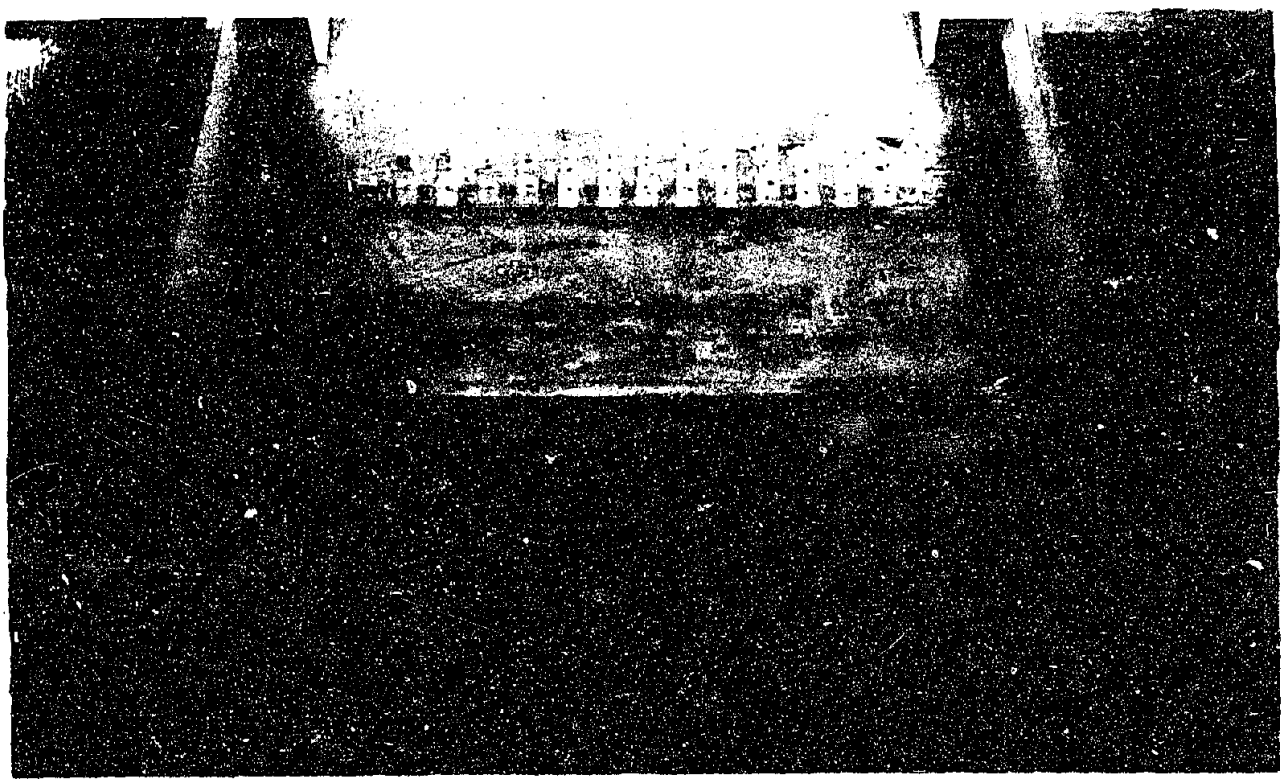
A. 1:24 SCALE MODEL OF RECOMMENDED OVERCHUTE AND WASTEWAY STILLING BASIN. WASTEWAYS ONLY OPERATING. DISCHARGE 2500 SECOND-FEET.

A. 1:24 SCALE MODEL OF RECOMMENDED OVERCHUTE AND WASTEWAY STILLING BASIN. WASTEWAY ONLY OPERATING. DISCHARGE 2500 SECOND-FEET.

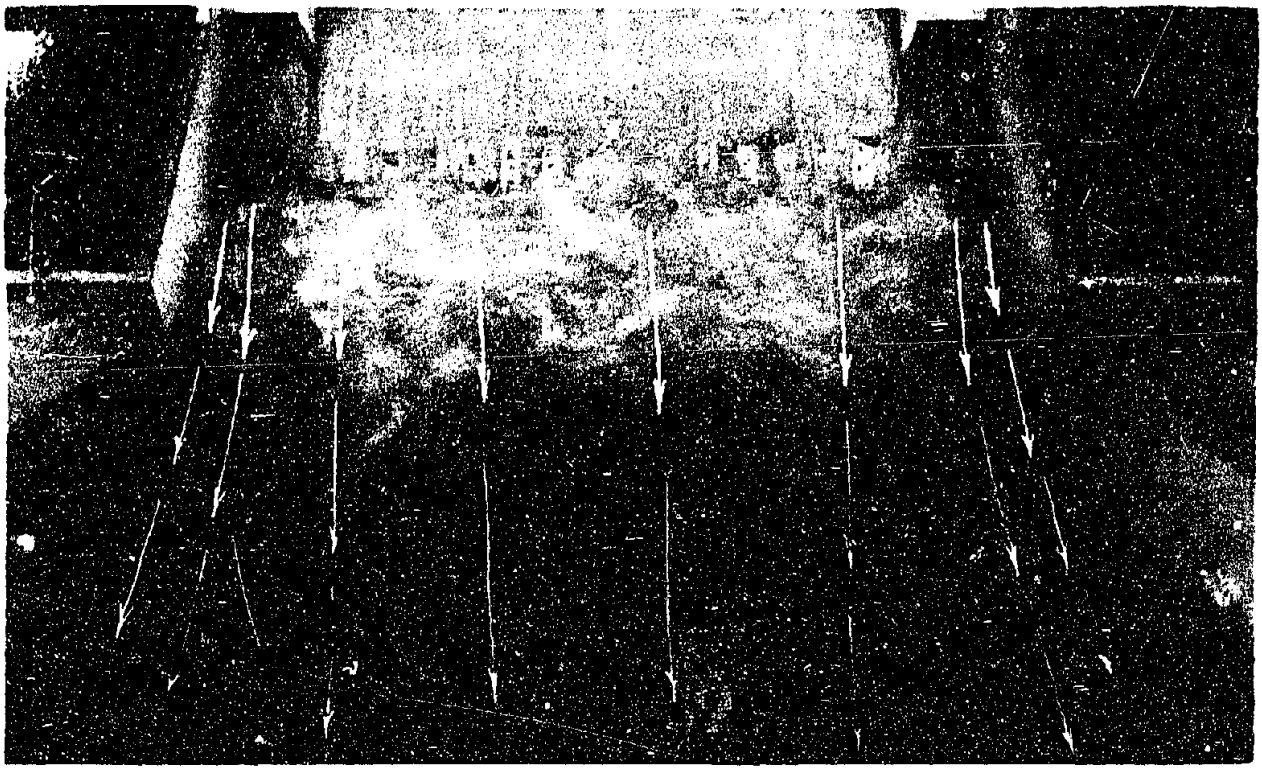




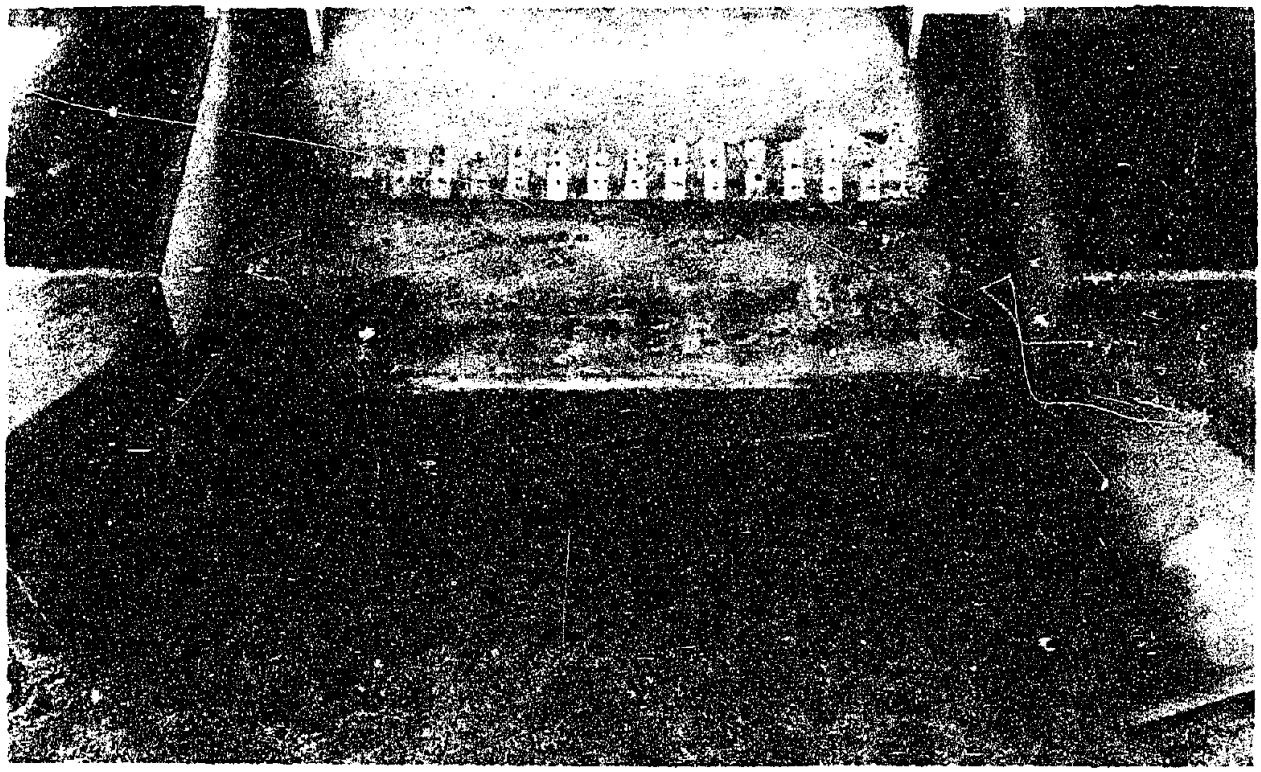
A. 1:24 SCALE MODEL OF OVERCHUTE AND WASTEWAY STILLING BASIN. NO BLOCKS OR END SILL. OVERCHUTE DISCHARGE 2500 SECOND-FEET AND WASTEWAYS DISCHARGE 1390 SECOND-FEET.



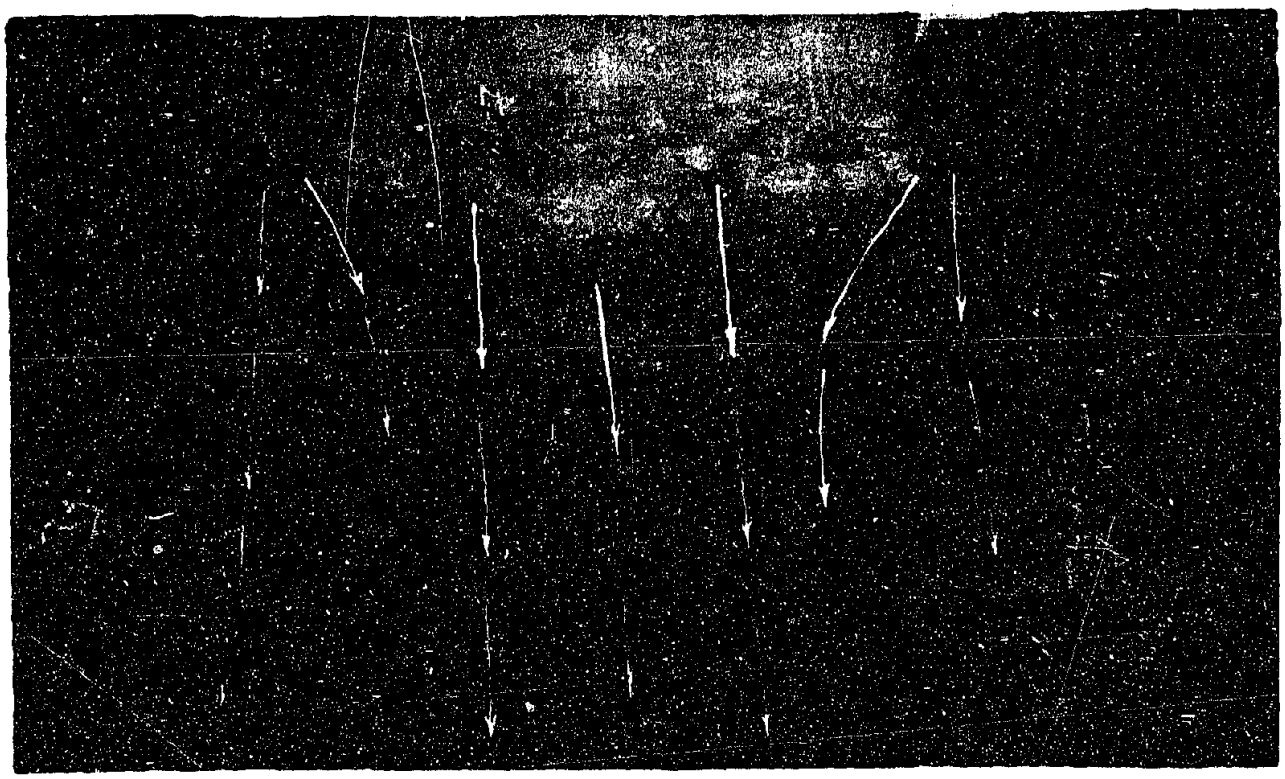
B. SAME AS PLATE 15-A.



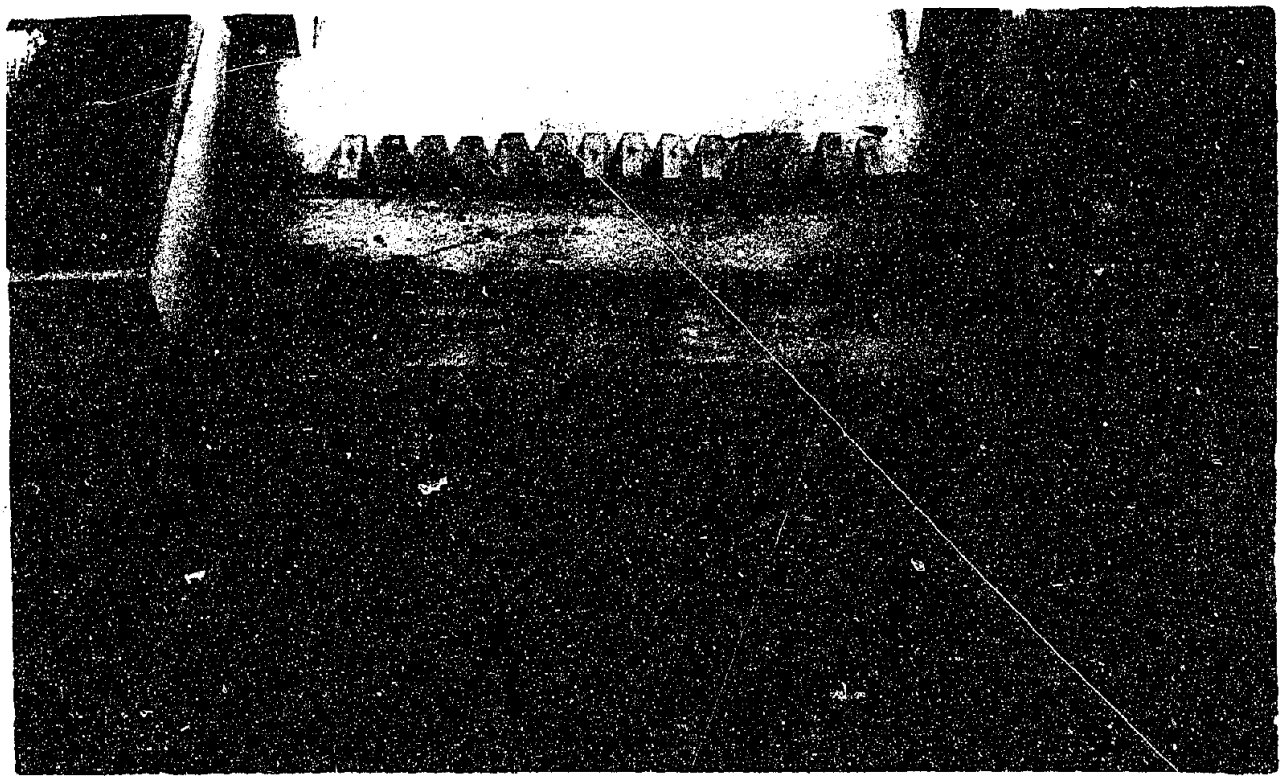
A. 1:24 SCALE MODEL OF OVERCHUTE AND WASTEWAY STILLING BASIN. NO BLOCKS OR END SILL. OVERCHUTE DISCHARGE 2500 SECOND-FEET AND WASTEWAYS DISCHARGE 1390 SECOND-FEET.



B. SCOUR OF PLATE 15-A.

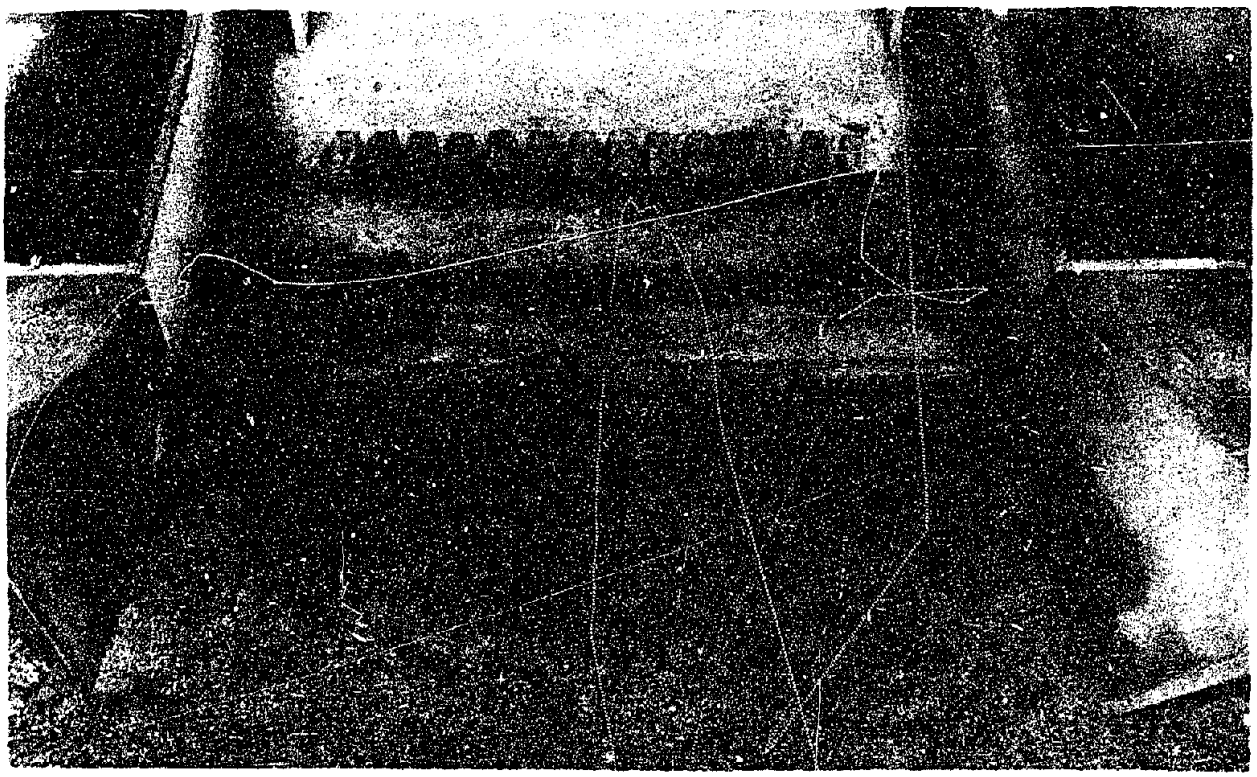


A. 1:24 SCALE MODEL OF RECOMMENDED OVERCHUTE AND WASTEWAY STILLING BASIN. OVERCHUTE DISCHARGE 2500 SECOND-FEET AND WASTEWAYS DISCHARGE 1590 SECOND-FEET.



B. SCOUR OF PLATE 16-A.

A. 1:24 SCALE MODEL OF RECOMMENDED OVERCHUTE AND WASTEWAY STILLING BASIN. OVERCHUTE DISCHARGE 2500 SECOND-FEET AND WASTEWAYS DISCHARGE 1390 SECOND-FEET.



B. SCOUR OF PLATE 16-A.

the whirls completely. Since they are confined entirely within the stilling basin and are low velocity whirls, no serious objection to them should be raised.

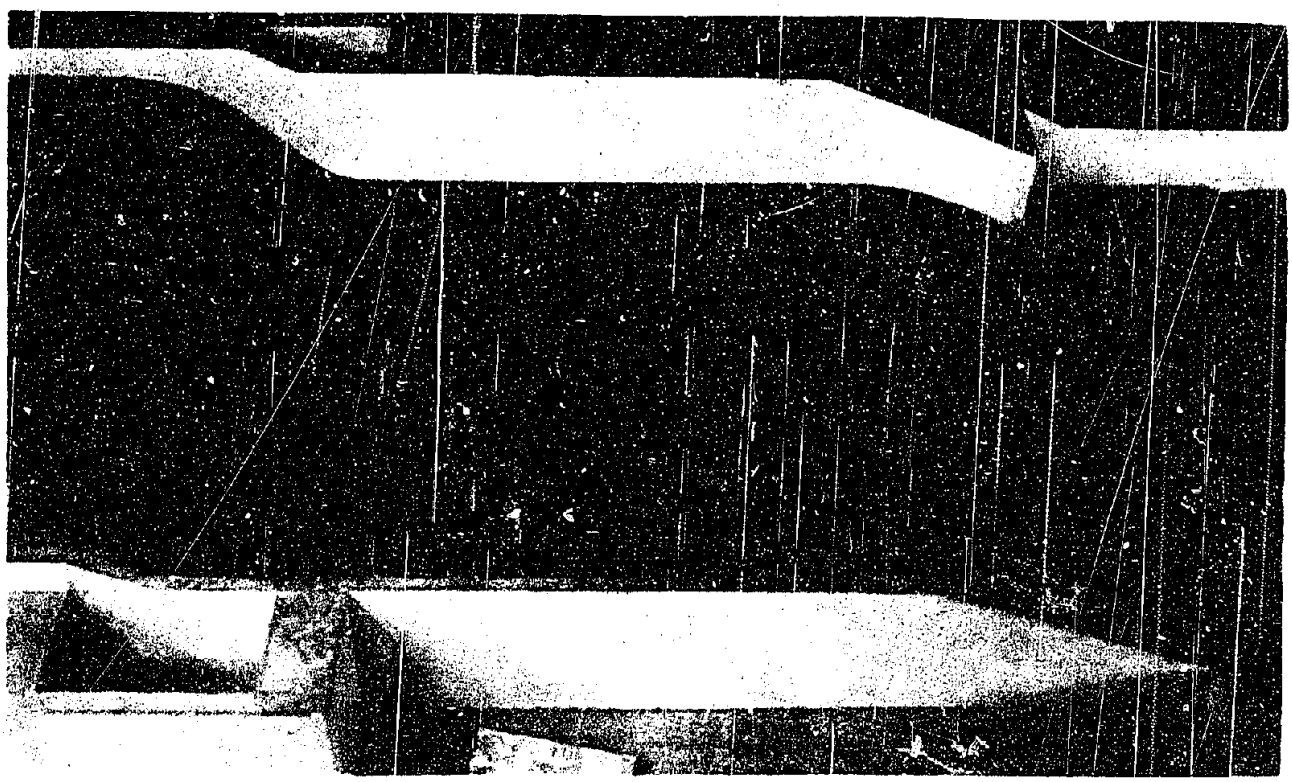
It is difficult to establish a set of rules for the general design of a combined overchute and wasteway stilling basin. In future designs, for which no model studies are to be made, the 17 rules outlined for the overchute basin design should be followed, making necessary modifications as are indicated by the results of the study on the overchute and wasteway basin. For conditions not too far removed from those of the structure studied, proportioning of dimensions should yield a satisfactory design.

16. Piers on the elevation 172.00 overchute floor.

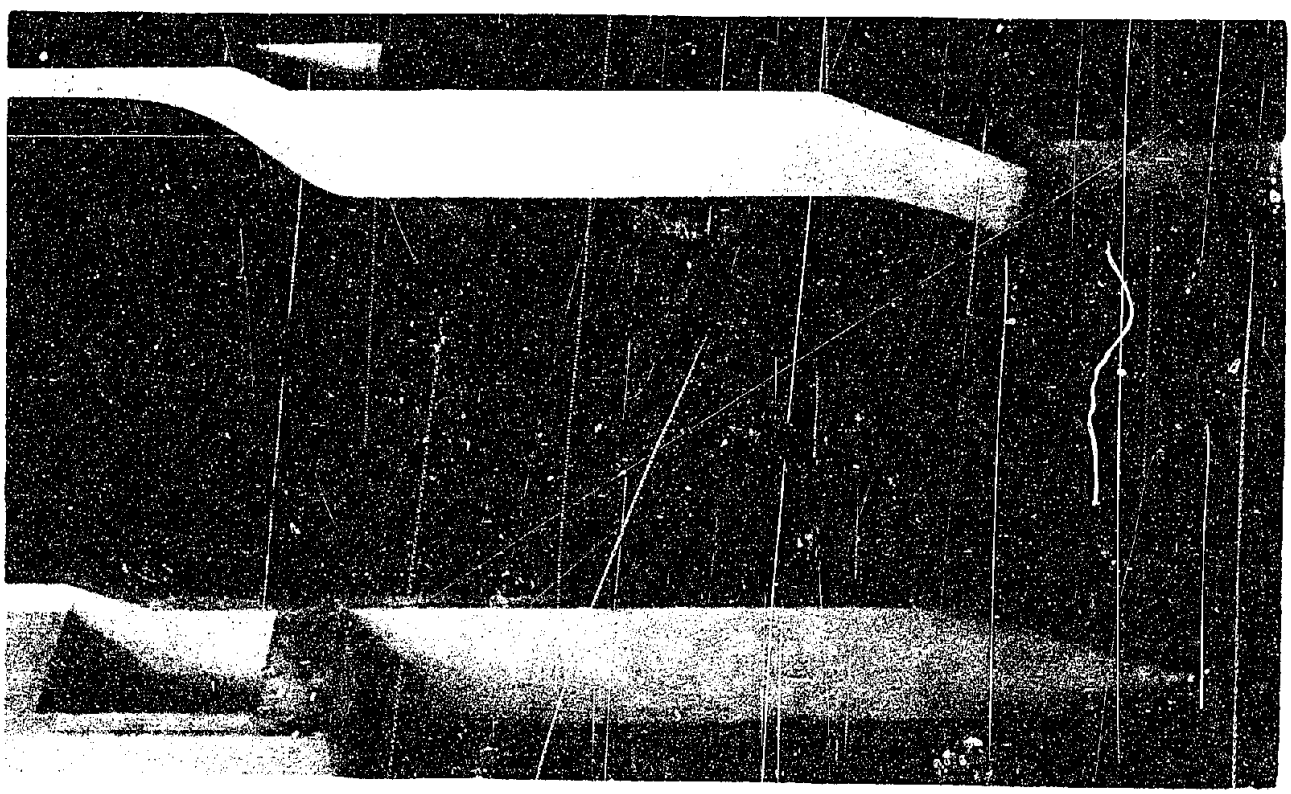
For the maximum discharge water climbs to elevation 178.5 against the original pier (plate 17-A). There should be no objection to this condition since the bridge is about 12 feet above the overchute floor. The pier nose must be made sharper should it be desirable to decrease the height of wave aginst the pier. The wave crest is lowered to elevation 176.5 for the pier of figure 16-B (plate 17-B). A very sharp pier is necessary to reduce the wave height to one foot. A sharp pier requires iron facing, so for economic reasons several feet of wave height will be acceptable for most designs. The tail end of the pier may be made square as shown in figure 16-B without having any noticeable influence on the flow conditions. While there is some disturbance of flow immediately beyond the end of the pier, the depth is nearly uniform at the pool entrance so there is no noticeable change in the hydraulic jump.

CONCLUSIONS AND RECOMMENDATIONS

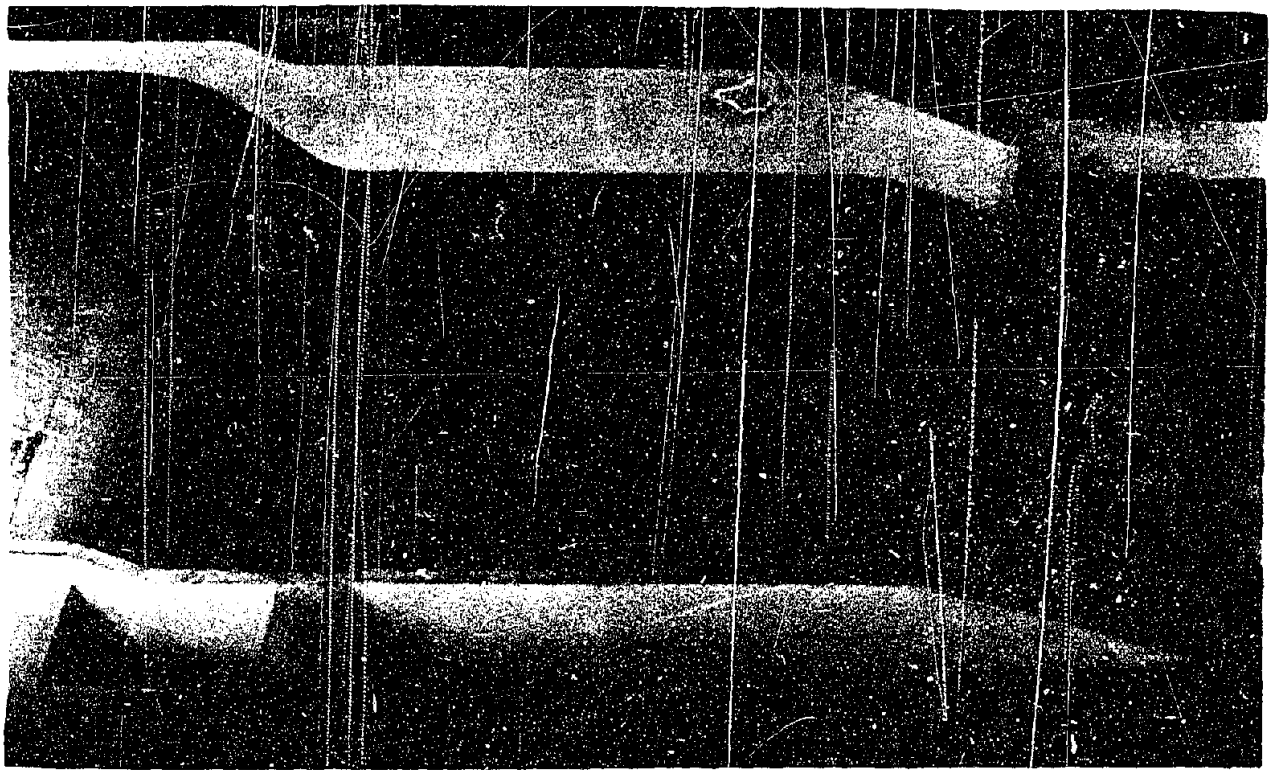
The original design overchute entrance will give satisfactory flow conditions and is recommended for use on the type of overchute tested.



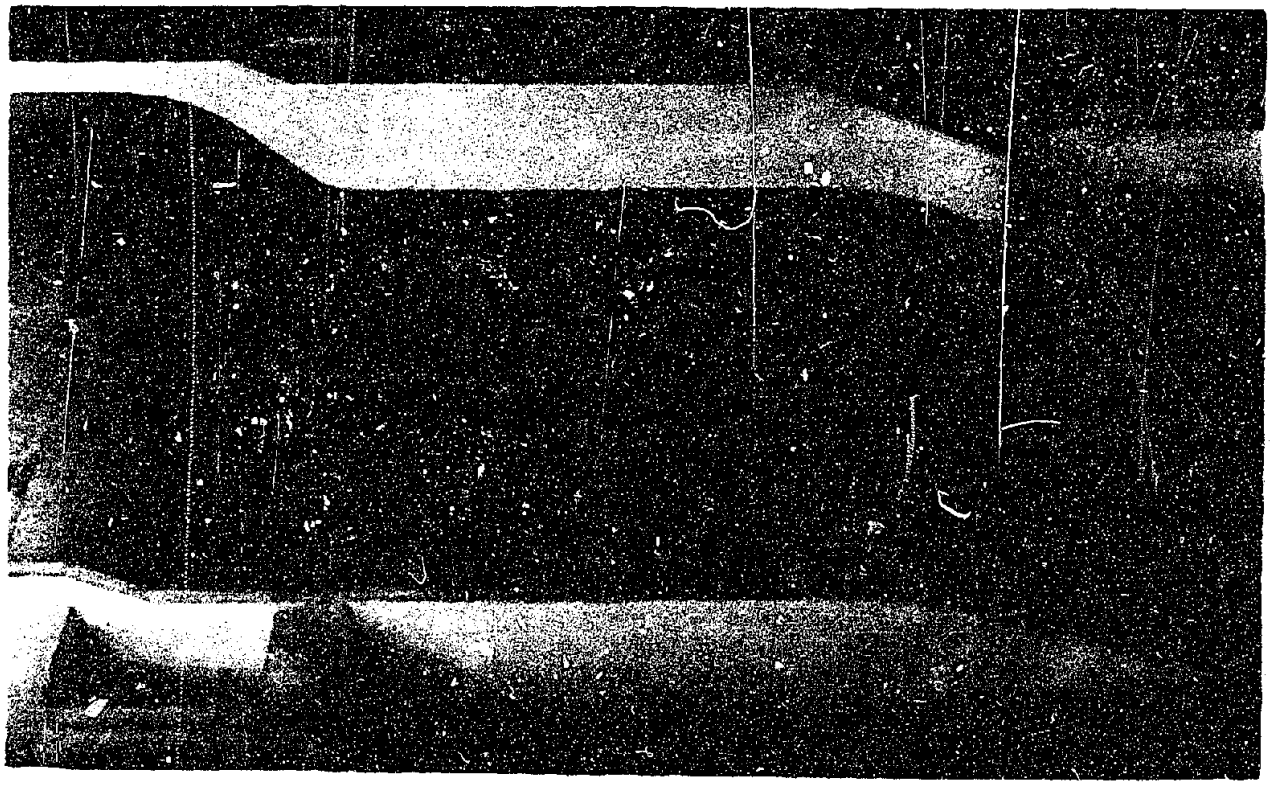
A. ORIGINAL BRIDGE PIER. 1:24 SCALE MODEL. DISCHARGE 2500 SECOND-FEET.



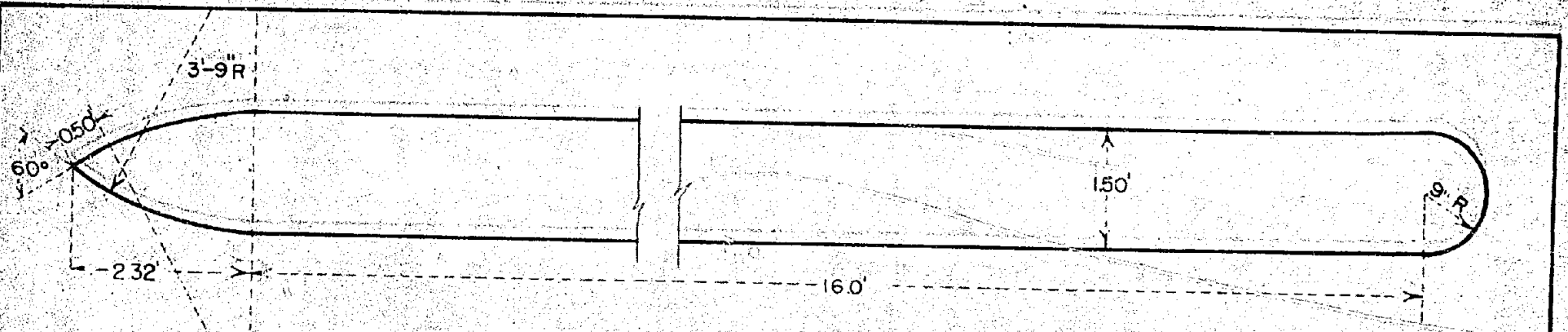
B. ALTERNATE BRIDGE PIER. 1:24 SCALE MODEL. DISCHARGE 2500 SECOND-FEET.



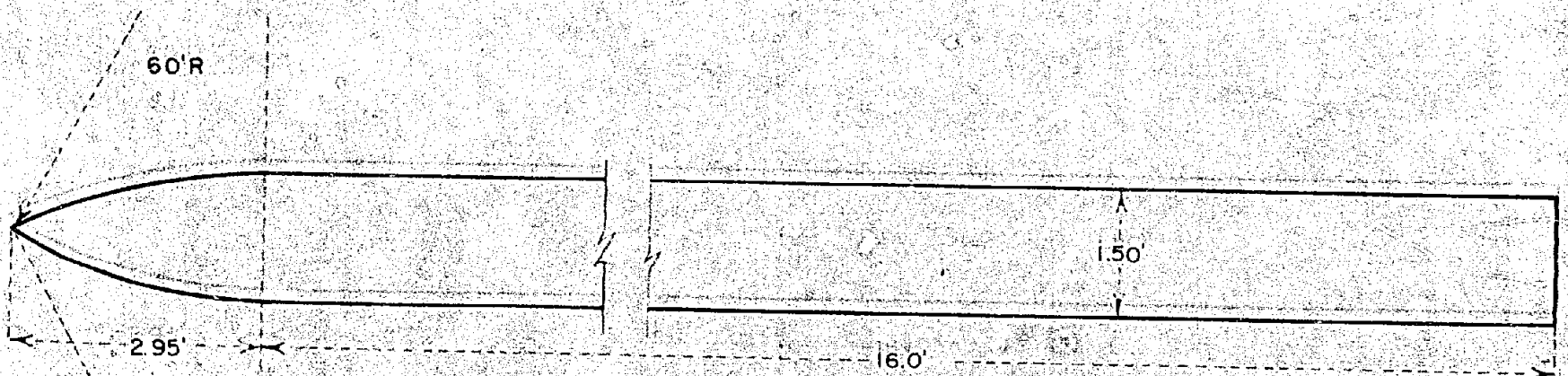
A. ORIGINAL BRIDGE PIER. 1:24 SCALE MODEL. DISCHARGE 2500 SECOND-FEET.



B. ALTERNATE BRIDGE PIER. 1:24 SCALE MODEL. DISCHARGE 2500 SECOND-FEET.



ORIGINAL PIER DESIGN



ALTERNATE PIER DESIGN

GILA GRAVITY MAIN CANAL
WASH OVERCHUTE
BRIDGE PIER FOR ELEVATION
172.0 OVERCHUTE FLOOR

FIGURE 16

3-3-38

Flow conditions were rough in the overchute with mud sills 12 inches high, spaced 19 feet 9 inches center to center.

A maximum spacing of 10 feet was indicated by the model as giving satisfactory results.

Considering structural characteristics, the height of 8 inches and spacing of 9 feet 10½ inches recommended for the prototype should prove satisfactory.

A protective layer of sand and gravel should be placed on the floor of the overchute when the structure is completed as a long period might elapse before it would be provided by nature. From available literature it was concluded that this material should not be smaller than 1½ to 3 inches.

Through the use of apron steps, blocks and sills it was possible to design a stilling pool for the overchute structures that required only 85 percent of the theoretical downstream depth to dissipate satisfactorily the energy contained in the overchute flow.

The characteristics of a stilling pool for similar structures can be obtained from the rules and curves presented in this report.

The stilling pool characteristics evolved for the combined overchute and wasteway structures should prove adequate for all similar arrangements. It was possible through use of the apron step and sill arrangement in this design to eliminate the construction of dividing walls to separate the wasteway and overchute pools.

The streamlining of the bridge pier in the overchute channel below the canal will give a minimum of disturbance in the flow at this point.

The original design wasteway gate section is adequate for the discharges for which it was designed. The discharge curves prepared from the model data and contained in this report should prove valuable in determining flood quantities during future operation of the wasteway structures.

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