UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC COMPUTATIONS TO AID IN THE DESIGN OF STATION 1097+60 WASTEWAY FOR THE HEART MOUNTAIN CANAL

SHOSHONE PROJECT, WYOMING

Hydraulic Laboratory Report No. Hyd.-221

BRANCH OF DESIGN AND CONSTRUCTION
DENVER, COLORADO

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Branch of Design and Construction
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Laboratory Report No. 221
Hydraulic Laboratory
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Subject: Hydraulic computations to aid in the design of Station 1037+00 Wasteway for the Heart Mountain Canal--Shoshone Project, Wyoming.

Introduction. Hydraulic computations were made in the Denver Hydraulic Laboratory during April of 1938 to aid in the design of Station 1037+00 Wasteway for the Heart Mountain Canal, Shoshone Project, Wyoming. The report was not completed at that time due to more urgent work in the laboratory.

The original design of the wasteway is shown in Figure 1. Station 0+00 of the wasteway is on the canal centerline at Canal Station 1037+84. The wasteway flow is controlled by two rectangular gates. An open chute carries the flow to the stilling-pool downstream. The purpose of the wasteway is to prevent overtopping of the canal banks during unusual flow in the canal.

The following recommended dimensions were determined by comparing the hydraulic properties of this structure with those of approximately similar structures for which the best designs have been determined by model experiments in the hydraulic laboratory. In the appendix, discharge curves are presented for several gate sizes. These curves may prove useful in future designs.

Wasteway Culvert. The experimental discharge rating curve for the culvert inlet was determined by test on the model culvert of the wasteway of the Gravity Main Canal, Station 169+80, Gila Valley Project, Arizona, which was under test at the time in the laboratory. The arrangement of the two culverts and the approach channels were essentially the same except that the roof entrance was square for the Heart Mountain Culvert and rounded for the Gravity Main Canal Culvert. The entrance was rectangular in the model. The model represented culvert openings 6 by 7.5 feet to a 1:24 scale for the Gravity Main Canal and openings 4 by 5 feet to a 1:16 scale for the Heart Mountain design. Figure 2 shows the experimental wasteway discharge rating curve converted to prototype quantities for the Heart Mountain Wasteway. From this curve, the wasteway will carry 560 second-feet for the maximum canal water surface elevation of 5171.19, or for the design canal discharge of 554 second-feet, the canal water surface elevation will be 5171.05. This discharge curve may be used to determine the canal water surface elevation when any stated flow is checked and diverted by means of the wasteway. The experimental values of the coefficients of contraction and discharge are plotted against head in Figure 3 for the Heart Mountain Wasteway.

Computation for mean cross-sectional velocity at Station 2+48.01. The velocity in the chute at Station 2+48.01 was computed by the Design Section by starting with the known velocity at Station 0+33.13 and treating successive short reaches as is customary for flow in steep chutes. As a check, the assumption is made that the flow will be non-accelerating at Station 2+48.01. From Manning's equation, a velocity of 37.7 feet a second was obtained, which compares very favorably with 37.4 feet a second as computed by the first method.

Computation for vertical curve, Station 2+48.01 to 2+74.45. Derivation of the equation for the trajectory in the chute, beginning at Station 2+48.01, is as follows:

$$y = x \tan \phi + \frac{g}{2 V_o^2 \cos^2 \phi} x^2 \qquad (1)$$

where x is the horizontal coordinate, y the vertical coordinate, ϕ the

slope angle of the chute (referred to the horizontal) at the beginning of the curve, V_o the mean cross-sectional velocity for the maximum discharge condition at the beginning of the curve, and g the acceleration of gravity. The Vallecito Dam Spillway, which was tested in the laboratory, possessed a trajectory near the end of the chute very similar to the proposed Heart Mountain design. The original trajectory of the Vallecito Spillway was based on Formula (1). This trajectory was found to be excessively steep, since the water sprang free of the chute floor downstream from this vertical curve. The computed mean velocity, V_o, considering all losses in the chute, at the beginning of the trajectory was computed to be 59 feet a second for the Vallecito Spillway. A flatter trajectory, based on the theoretical mean velocity of 82.5 feet a second, which was computed by neglecting all losses, proved satisfactory. The equation for the satisfactory trajectory proved to be:

$$y = x \tan \phi + \frac{g}{2V^2 \cos^2 \phi} x^2 \qquad (2)$$

where V is the theoretical velocity at Station 2+48.01.
The ratios of the squares of the two velocities are:

$$V^2 = \left(\frac{82.5}{59}\right)^2 V_o^2 = 2 V_o^2 \text{ (approximate)} \qquad (3)$$

Substituting (3) into (2)

$$y = x \tan \phi + \frac{g}{4 V_o^2 \cos^2 \phi} x^2 \qquad (4)$$

Inspection shows that if half the value of g is used in Equation (1), the results are identical with those of Equation (4). In the proposed design of Heart Mountain Wasteway, 1/2 g was used in Equation (1) so the trajectory is sufficiently flat to prevent any springing from the chute bottom at the vertical curve.

The velocity used in Equation (1) was the mean velocity of the cross-section. Since the maximum velocity is considerably greater than the mean velocity, a trajectory designed for the mean velocity will be too steep for the mass of water flowing in the area of maximum velocity, as was verified by the Vallecito Spillway tests. For this reason, it seems logical that the trajectory should be designed for the maximum rather than the average velocity of the cross-section at the beginning of

the trajectory. The ratio of maximum to mean velocity for the following three structures is:

(a) Vallecito Spillway (model measurement) $V_{max.}/V_{mean} = 65/55 = 1.18$

(b) Bull Lake Dam Spillway (model measurement)

$V_{max.}/V_{mean} = 68.9/59.4 = 1.16$

(c) South Canal Chute, Uncompahgre Project (field measurement)

$V_{max.}/V_{mean} = 34.8/28.1 = 1.20$

From these measurements, indications are that the velocity to use in Equation (1) should be about 20 percent greater than V_o. Equation (1) then becomes:

$$y = x \tan \phi + \frac{g}{2.88\ V_o^2 \cos^2 \phi} x^2 \qquad (5)$$

According to this analysis, Equation (1), using the mean velocity and 70 instead of 50 percent of g, may be used in computing the trajectory. Experiments should be conducted to support this statement. Entrainment of any air will add a safety factor, since velocities will be lower.

<u>Angle of flare of the chute, Station 2+94.45 to 3+65.88, entering the stilling-basin.</u> The economical widths of chute and stilling-basin are determined from cost considerations. The chute must be flared at its lower end such that there will be a uniform distribution of depth of flow entering the stilling-basin. There must be a spreading of the flow from the narrow chute section to the wider stilling-basin section at such a rate that the depth distribution remains relatively uniform. Such spreading under natural forces is probably affected by the several hydraulic properties involved, but its rate is known to be very closely related to the velocity. The theoretical mean velocity at the entrance to the stilling-basin for the Deer Creek Dam Spillway was 73.1 feet a second which is approximately the same as that for the Heart Mountain Spillway. Tests on the model of the Deer Creek Spillway showed that the maximum total angle of flare, in plan, to maintain a uniform depth distribution at the pool entrance was 11 degrees. Using 11 degrees for the Heart Mountain design places the beginning of the flare at Station 2+98.9. Unless there are other disadvantages, a reduction in cost and good pool action will result by beginning the flare at about

Station 2+95.

Slope of the chute leading into the stilling-basin, Station 2+94.45 to 3+65.88. General experiments have been conducted in the laboratory on chute slopes ranging from 1:1 to the horizontal. There has been little difference in the length of the jump or the tailwater required to maintain the jump for the several slopes tested. A steeper slope than 3:1, however, reduces the effectiveness of the jump in dissipating energy of the flowing water which was indicated by deeper scour when no blocks or sills were used. With blocks and a sill, or a dentated sill alone, the average depth of scour was about the same for all slopes. Hydraulic considerations would indicate a 1-1/2:1 slope. Economic considerations, however, show the steeper slope to be the more desirable. For a given elevation of the stilling-basin, the flatter slopes will require higher retaining-walls, a greater length of chute floor to be designed against uplift pressure, and more excavation, thus increasing the cost. The centrifugal forces at the junction of the slope with the basin floor are greater for steeper slopes, but the structure is usually excessively strong at this point.

Chute blocks. Small blocks placed on the chute at the entrance to the stilling-basin are effective in forming a more efficient jump. In such cases, the incoming jet is corrugated into a number of small jets, each producing its own system of eddies, thus resulting in a greater dissipation of energy. With the numerous jets entering the pool in a corrugated pattern, part of the high-velocity jet is lifted from the floor, which also adds to the effectiveness of the jump. Repeated experiments have shown chute blocks to be effective in reducing scour beyond the end of the stilling-basin, but that further reduction in scour is very slight with block heights greater than the theoretical D_1 depth entering the basin. D_1, in the case of Heart Mountain Wasteway, is equal to 0.38, which results in a block height of approximately 5 inches, prototype. It is very probable that this unusually thin sheet of water flowing at high velocity will contain a high percentage of entrained air. For that reason, the depth of water entering the pool may be as much as 100 percent greater than the theoretical depth, and so 9-inch high chute blocks are recommended. Tests have shown that a width of block approximately equal to the height is best. Blocks 12 inches wide, arranged as shown in Figure 4, are recommended.

Since the velocity is much lower at the sidewalls, blocks placed here have proved of little use.

Stilling-basin floor blocks. Any arrangement of baffle blocks on the stilling-basin floor will, of course, aid in the dissipation of energy in the pool. Blocks with vertical upstream faces are more effective than those with sloping upstream faces, so that smaller sizes of vertical face blocks may be used to produce the same results. However, the vertical upstream face blocks will receive greater impact as the high velocity carries through on the floor with little retardation. This calls for facing on the blocks, reinforcement, and anchorage. The results of model tests of three structures, with the maximum chute velocity approximately the same as for the Heart Mountain Wasteway, indicate that a row of blocks about one-sixth the theoretical D_2 height placed approximately at the downstream two-thirds point of the stilling-basin results in the best jump performance. Since the theoretical value of D_2 is 11 feet, a block height of 2 feet was chosen. It is recommended that row of basin floor blocks, arranged as shown in Figure 4, be incorporated in the design.

Stilling-basin floor elevation. The proper elevation of the stilling-basin floor will be determined by the use of results from model studies of two similar structures of the following hydraulic properties: (a) discharge of 106.6 second-feet a foot width, V_1 of 73.2 feet a second, D_1 of 1.457 feet; theoretical D_2 of 21.3 feet; (b) discharge of 160.0 second-feet a foot width, V_1 of 75.1 feet a second, D_1 of 2.13 feet and theoretical D_2 of 26.0 feet. The measured minimum D_2 depth for a good jump was 18.0 feet for (a) and 22 feet for (b), which are 84.5 and 84.6 percent of the theoretical, respectively. The possibility of having less depth than the jump theory indicates, without causing the jump to sweep out, is at least partly due to the influence of the blocks in maintaining the jump in the basin. The theoretical D_2 for the structure in question is 11.0 feet, and 85 percent of this is 9.35 feet.

The end of the transition at elevation 5089.50 is considered a control over which water flows at critical depth, which is approximately 2.25 feet. Considering the head at this point to be $3/2\ d_c$, gives a water surface approximately 3.35 feet for maximum discharge, and the required difference in stilling-pool floor and control elevations becomes 6.0 feet. From these considerations, the floor may be safely raised 2.09 feet to elevation 5083.50,

as shown on Figure 4. The jump for the tested structure swept out of the basin when the tailwater depth was reduced to approximately 75 percent of the theoretical D_2. On this basis, the elevation 5063.50 pool provides an excess of 1.1 feet against the jump sweeping out of the basin. In the tested structures, the difference in elevation between the stilling-basin floor and riverbed downstream was 3 feet. The 6-foot difference in the Heart Mountain design should give added safety against the jump sweeping out.

Another factor effectively reducing the required D_2 depth is the influence of entrained air in the chute flow. With the high velocity and small depth entering the pool, available data on the absorption of air would indicate that the flow may contain as much as 50 percent air. Introducing this value in the jump momentum equation results in a reduction of D_2 by approximately 22 percent, which gives an added excess tailwater depth of approximately 1.7 feet against the jump sweeping out. The D_2 depth of 9.35 feet is, therefore, about 2.5 to 3.0 feet in excess of that at which the jump will sweep out.

Stilling-basin length. The length of the stilling-basin should be sufficiently long to prevent excessive erosion downstream. The length of basin is usually made equal to the length of the jump. The length of the jump and basin can be shortened when chute and apron blocks are used. The results on the two structures mentioned in the preceding section indicate that a basin length four times the tailwater depth for a good jump at maximum discharge extends the full length of the jump when blocks are employed. On this basis, the minimum length of basin is 37.4 feet, or, say 40 feet. The 15 feet of transition at the end of the basin is considered desirable and it will serve as an addition to the basin length. For economic reasons, it is recommended that the stilling-basin be shortened 10 feet over the original design, since satisfactory jump performance may be expected for the shorter basin. With the shorter basin and higher floor, some saving in excavation can be realized.

Training-wall heights. The sidewalls of the chute can be reduced at least a foot near the end of the chute, since the maximum water depth will not be more than 1 foot. The stilling-basin training-walls have about 4 feet freeboard when the flow is critical at the end of the transition. Since some scouring must occur downstream before the depth will be critical at the

end of the sloping floor, and to allow for some splashing in the pool, no reduction in training-wall height is recommended.

Anticipated scour. It is very probably that heavy, well-placed riprap at the end of the basin will result in no scour even with continuous operation at the maximum discharge. However, if velocities are sufficiently high to result in scouring, the current of the ground roller, formed under the stream leaving the sloped transition floor, will move material upstream giving protection to the cutoff wall rather than scour material away from this wall.

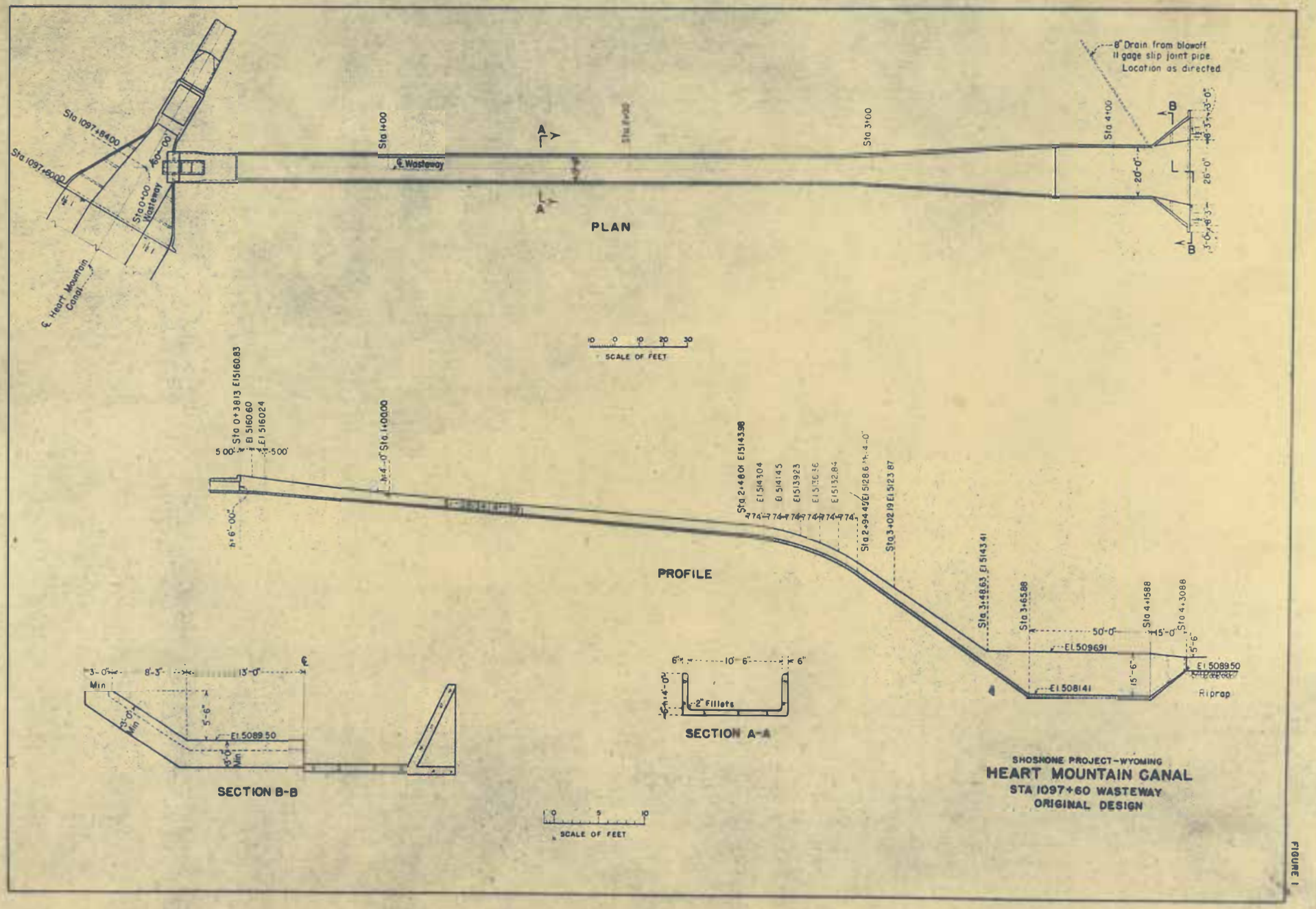
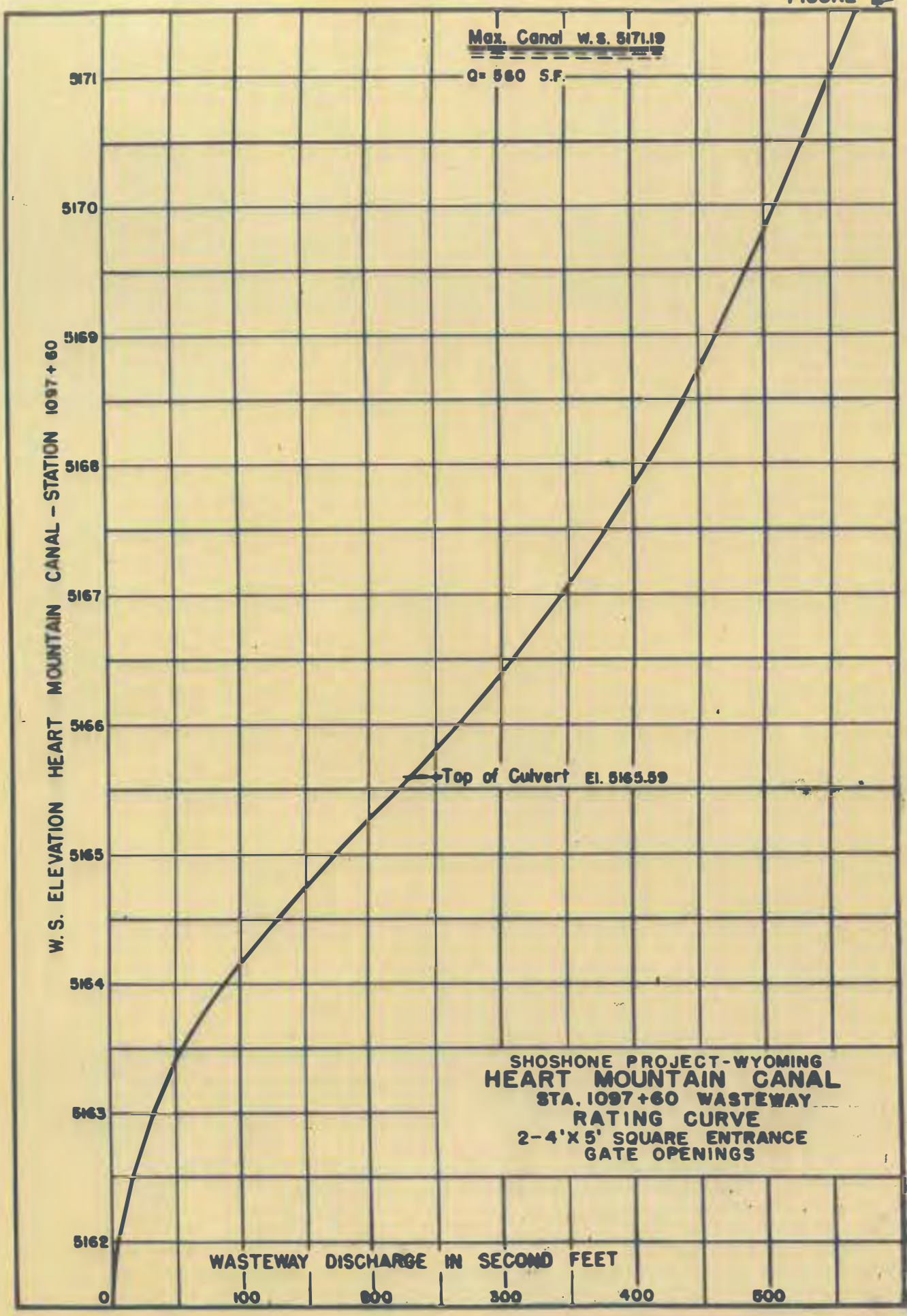
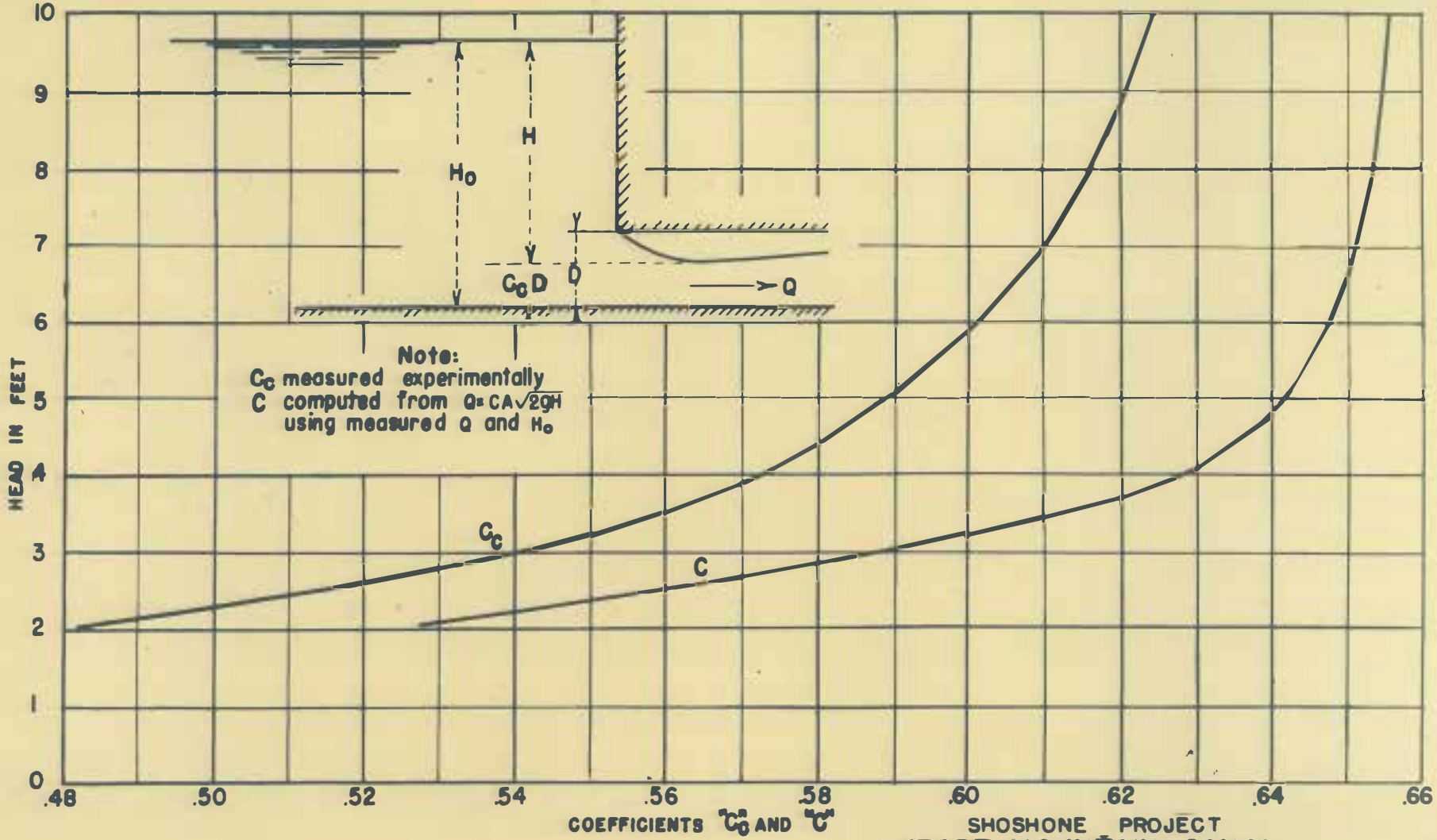

PLAN
Sta 1097+84.00
Sta 1097+60.00
Sta 0+00 Wasteway
℄ Heart Mountain Canal
Sta 1+00
℄ Wasteway
Sta 3+00
Sta 4+00
8" Drain from blowoff
11 gage slip joint pipe
Location as directed
SCALE OF FEET
PROFILE
Sta 0+38.13 El 516.83
El 516.60
El 516.24
Sta 1+00.00
Sta 2+48.01 El 514.38
El 514.04
El 514.45
El 513.23
El 512.84
Sta 2+94.42
Sta 3+02.19 El 512.87
Sta 3+48.63 El 514.41
Sta 3+65.88
Sta 4+15.88
Sta 4+30.88
El 509.91
El 508.41
El 508.50
Riprap
SECTION B-B
El 508.50
SECTION A-A
2" Fillets
SCALE OF FEET
SHOSHONE PROJECT-WYOMING
HEART MOUNTAIN CANAL
STA 1097+60 WASTEWAY
ORIGINAL DESIGN

FIGURE 1

FIGURE 2


Max. Canal W.S. 5171.19
Q= 560 S.F.
Top of Culvert El. 5165.59
W.S. ELEVATION HEART MOUNTAIN CANAL - STATION 1097+60
5171
5170
5169
5168
5167
5166
5165
5164
5163
5162
0
100
200
300
400
500
WASTEWAY DISCHARGE IN SECOND FEET
SHOSHONE PROJECT-WYOMING
HEART MOUNTAIN CANAL
STA. 1097+60 WASTEWAY
RATING CURVE
2-4'X5' SQUARE ENTRANCE
GATE OPENINGS



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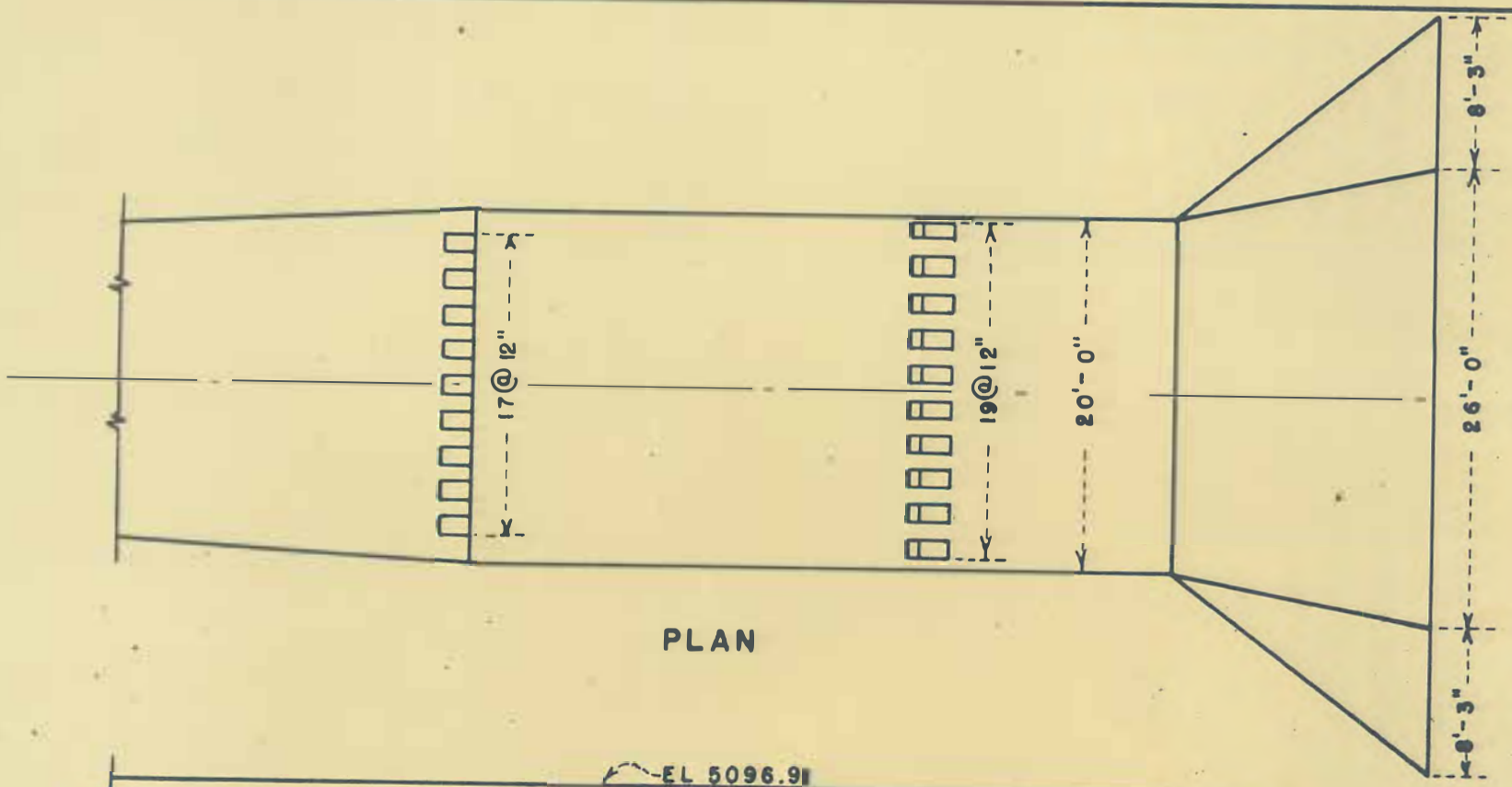
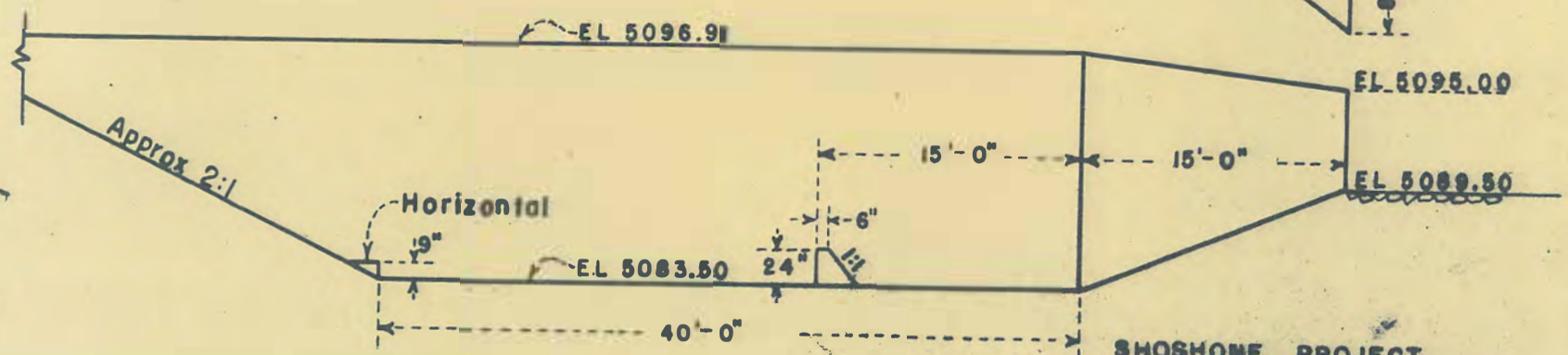
HEART MOUNTAIN CANAL

STA. 1097+ 60 WASTEWAY

COEFFICIENTS FOR SQUARE ENTRANCE CULVERT GATES WITH PARTIALLY SUBMERGED FLOW 2-4'x5' GATE OPENINGS

FIGURE 3





SHOSHONE PROJECT
HEART MOUNTAIN CANAL
STA. 1097+60 WASTEWAY
RECOMMENDED STILLING BASIN DESIGN

FIGURE 4

APPENDIX

According to Torricelli's theorem, water discharging from an orifice under a head, H, has a theoretical velocity equal to the velocity acquired by a body falling freely "in vacuo" through a vertical distance, H, that is:

$$V_t = \sqrt{2\,g\,H} \qquad (1)$$

or

$$H = V_t^2/2g \qquad (2)$$

The expression $V_t^2/2g$ is termed the velocity head.

Because of the effects of friction and viscosity, the mean velocity of a jet is always less than the theoretical velocity. Expressed by symbols:

$$C_v = V_o/V_t \qquad (3)$$

Therefore, from (1)

$$V_o = C_v \sqrt{2gH} \qquad (4)$$

If A' is the cross-sectional area of the jet at vena contracta and A is the area of gate opening,

$$C_c = A'/A \qquad (5)$$

C_c decreases as contraction is reduced and approaches unity for an opening with well-rounded corners.

The discharge from any gate opening is equal to the product of the cross-sectional area of the jet at the vena contracta, the mean velocity at the same section and a coefficient, C_a, which represents the effect of velocity of approach; that is,

$$Q = C_a\,V_o\,A' = C_a\,C_c\,C_v\,A\,V_t$$

or

$$Q = C\,A\sqrt{2\,g\,H} \qquad (6)$$

where $C = C_a\,C_c\,C_v$

The values of C_a and C_v are difficult to obtain experimentally, and these coefficients are of theoretical rather than practical value. The value

of C_c may be readily obtained by measuring the area of the jet at the vena contracta. Numerical values of C are obtained by measuring the discharge from an opening of known dimensions and the governing pressure head.

The coefficient C as given by formula (6) includes the effect of velocity of approach, loss of head due to friction and viscosity, and contraction. The knowledge of the coefficient is not sufficient to justify the use of a formula which contains separate terms to correct for velocity of approach, loss of head due to friction and viscosity, and contraction.

Wasteway and sluice gates are usually of the rectangular type shown in Figure 1A for which the flow is termed partially submerged. It appears that for gates of this type conditions influencing side and bottom contractions have very little effect on C.

For partially submerged flow, a measurement at the point of minimum elevation at the vena contracta theoretically gives the proper d to use in formula (4). The distance C_cD was measured experimentally in the 1:16 model of two 4- by 5-foot gate openings for several values of H_o. The proper head for any stated discharge is then given by:

$$H = H_o - C_c D \qquad (7)$$

To make the experiments of this one gate size universally applicable to other gate sizes, a curve was drawn giving the experimental value of C_c for various values of the ratio of gate opening D and the head on the gate sill H_o. From this curve, shown on Figure 1A, C_c can be determined for any D/H_o ratio which allows the effective head to be computed by formula (7). The "C" curve for various values of D/H_o is also shown on Figure 1A. The discharge from any known gate opening and head of water above the gate sill may be computed by use of formula (6). Discharge curves for several double gate openings varying in size from 3 by 3 feet to 7 by 7 feet have been plotted on Figure 1A.

It is believed that the model data converted to prototype quantities can be assumed as quite reliable. F. W. Blaisdell reports, in Transactions of American Society of Civil Engineers, Vol. 102, 1937, that as a result of 1,500 experiments on six models compared with prototype results, the ratio of Q_p/Q_m varied as $l^{2.5}$, or that Froude's model law applies to discharge through sluices. The average departure of Q_p/Q_m from $l^{2.5}$ was

found to be 0.4 percent.

Some question remains regarding expansion of the experimental data for the one gate size to other gate sizes. Velocity of approach, entrance loss and contraction are probably all affected by the gate size and shape. However, King* has found that the combined influence of these factors on the discharge from an orifice is not more than about 5 percent of the total discharge, so any variation in discharge for the several gate sizes due to these factors will probably be no more than 1 or 2 percent of the discharge for the experimental gate opening. Further tests should be conducted on gate openings of several sizes and shapes to determine the correctness of this statement.

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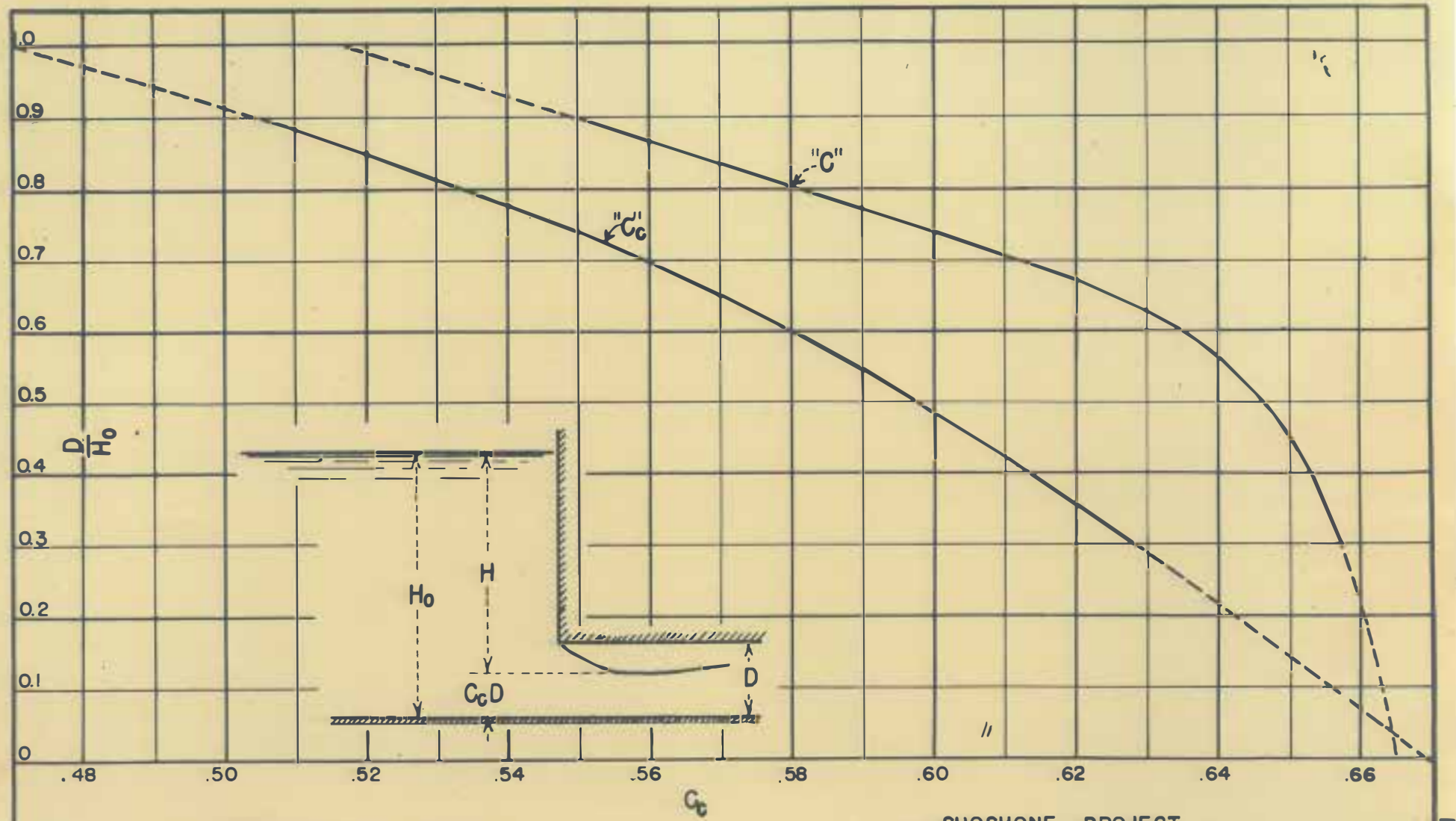
*King, H.W., Handbook of Hydraulics, Third Edition, pp. 40-55.



C_c measured experimetally
C computed from $Q = CA\sqrt{2gH}$
using measured Q and H_o

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GATES WITH SQUARE ENTRANCES
COEFFICIENTS OF CONTRACTION IN TERMS OF D/H_o
PARTIALLY SUBMERGED DISCHARGE

FIGURE 1A

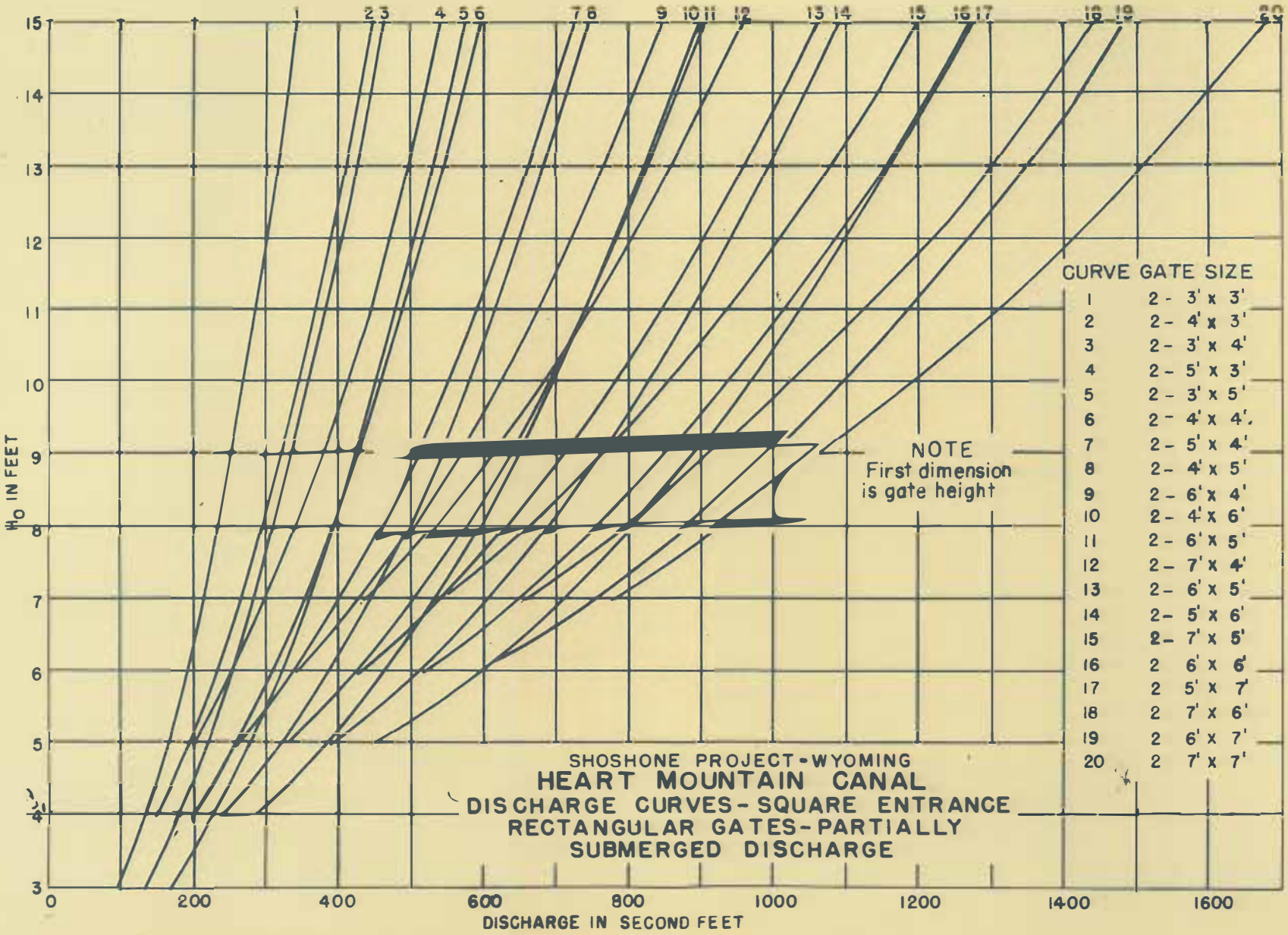
SHOSHONE PROJECT - WYOMING
HEART MOUNTAIN CANAL
DISCHARGE CURVES - SQUARE ENTRANCE
RECTANGULAR GATES - PARTIALLY
SUBMERGED DISCHARGE
NOTE
First dimension
is gate height
CURVE GATE SIZE
1 2 - 3' x 3'
2 2 - 4' x 3'
3 2 - 3' x 4'
4 2 - 5' x 3'
5 2 - 3' x 5'
6 2 - 4' x 4'
7 2 - 5' x 4'
8 2 - 4' x 5'
9 2 - 6' x 4'
10 2 - 4' x 6'
11 2 - 6' x 5'
12 2 - 7' x 4'
13 2 - 6' x 5'
14 2 - 5' x 6'
15 2 - 7' x 5'
16 2 6' x 6'
17 2 5' x 7'
18 2 7' x 6'
19 2 6' x 7'
20 2 7' x 7'
H_0 IN FEET
DISCHARGE IN SECOND FEET
0
200
400
600
800
1000
1200
1400
1600
3
4
5
6
7
8
9
10
11
12
13
14
15

FIGURE 2A