UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES ON WASTEWAY ENTRANCES FOR THE FRIANT-KERN CANAL-- CENTRAL VALLEY PROJECT

Hydraulic Laboratory Report No. Hyd-139

RESEARCH AND GEOLOGY DIVISION

BRANCH OF DESIGN AND CONSTRUCTION
DENVER, COLORADO

FEBRUARY 17, 1944

Denver, Colorado, February 17, 1944.

MEMORANDUM TO CHIEF DESIGNING ENGINEER
(Fred Locher through J. E. Warnock)

Subject: Hydraulic model studies on wasteway entrances for the Friant-Kern canal - Central Valley project.

1. The Friant-Kern canal, under normal conditions, will obtain its entire capacity flow of 3,500 second-feet from the reservoir created by Friant Dam. Due to surface drainage into the canal during severe storms, the flow in the canal will be increased considerably and would overtop the banks unless some provisions were made for disposing of the excess. This is usually done by providing wasteways at critical points along the canal. These structures are designed so that an increase in the canal water surface above the normal water surface will automatically cause the wasteway gates to open and discharge the excess flow into a natural watercourse where it will no longer be a menace to the canal.

2. In the past, various types of entrances have been used in conjunction with the wasteway structures, but very little usable information pertaining to the relative merits of the designs was available. On the Friant-Kern canal two types of entrances were proposed for different structures where the design discharge was the same. In this instance the openings at the gate sections were identical, but the designs differed in that design 1 (figure 1) had wing walls flared 30 degrees from the center line of the structure, rounded sections at the intersections of the vertical and sloped plane surfaces, and a drop of 24 inches from the bottom of the canal to the floor under the gates; whereas, in design 4 (figure 4), there was no flaring at the entrance, no rounding of corners, and no drop in the floor of the canal. From the viewpoint of construction cost, design 4 (figure 4) was probably less expensive. However, it was highly probable that design 1 would produce better flow conditions. Because of these factors and the desirability of studying the flow conditions in these designs and their variations (figures 2 and 3), a 1:16 scale hydraulic model was constructed in the laboratory.

3. The first model consisted of a section of canal with the wasteway entrance and gate section as shown on figure 1A. The stilling pool downstream from the gates was not a part of the studies and was eliminated in all of the tests. The investigations of this design and its variations, designs 2 and 3, revealed that design 1 gave the most favorable surface flow conditions. The turbulence and the splash were at a minimum, and the maximum capacity of the opening under the conditions of the test was 152 percent of the design capacity. Design 2 was identical with design 1, except that the rounded corners on the flaring wing walls were replaced with square corners, and had the same maximum discharge as design 1. However, a surface boil replaced the swirling motion that existed

upstream from the gate section in the previous arrangement (figures 1B and 2B). This was not objectionable, and the performance of the two arrangements could be considered identical. Design 3 was a combination of designs 2 and 4 (figure 3). It consisted of the flared upstream wing wall in combination with a downstream wall normal to the center line of the canal and no rounded surfaces at the intersection of the entrance walls and the canal lining. Tests showed the same maximum discharge with this design as with the previous designs. The water surface upstream from the gates was more turbulent, resulting in some spray which might wet the gate operating mechanism above.

The recess in the canal bottom in the above designs was provided to serve as a catchment area for sand and boulders. It did not function as anticipated because the velocity in the region farthest from the gates was insufficient to scavenge the deposited material. Design 4 (figure 4), with the entrance walls normal to the center line of the canal, was the most unsatisfactory type of entrance tested. There was considerable evidence of vibration in the structure, and, in addition, the maximum capacity of the opening was 25 percent below that of the three previous designs (figure 5). This reduction in capacity was due partially to the difference in head at the entrance and partially to the unsatisfactory flow condition, as shown in figure 4B.

4. As the gates were opened gradually during the tests with design 4, the control section shifted rapidly from the area under the gates to the opening in the canal slope. The exact point at which the change occurred could not be determined accurately; however, it seemed to occur when there was a vertical space of 10 feet under the gates. With the three previous designs this condition was not obtained until the gate opening exceeded 14 feet. Because of this shift in the control with design 4, extreme care should be exercised in the use of the gate discharge coefficients shown on figure 6, and under no circumstances should any of these curves be extrapolated.

5. The combined gate coefficients of discharge (figure 6) obtained during the tests indicate that there is no appreciable difference in the capacity of the entrances as long as the opening under the gate is well submerged. The variation in coefficients for large values of D/h is probably due to the combination of experimental error and the difficulty in setting the gate exactly in the same position for each test. For the lower openings, a small error in setting the position of the gate would cause a considerably larger error in the computed results. Another method for comparing the merits of the designs was based on critical flow which occurred with free discharge through the wasteway. This method of comparison is questionable[1] with these particular designs because curvi-

[1] See appendix.

linear flow instead of parallel flow is obtained in the structure, thereby

creating a condition where the formulas for critical flow do not give accurate results. However, for the value they may have in future design purposes, the results of an analysis with critical flow as a criterion are listed in the following table.

Design No.	Ha	H	Ha-H	Q	$C = \frac{Q}{5.676\, b\, Dc^{3/2}}$
1	18.59	17.23	1.36	5304	0.894
2	18.53	17.09	1.44	5233	0.884
3	18.67	17.23	1.44	5304	0.887
4	16.03	14.30	1.73	4004	0.842

Ha = total energy available in the canal.

$$H = \frac{3}{2}\sqrt[3]{\frac{Q^2}{b^2 g}}$$ where Q = measured discharge.

Ha - H = loss between canal and point of discharge.

C = coefficient of discharge based on critical depth equal to 2/3 Ha.

b = horizontal opening under the gates.

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Fred Locher

APPENDIX

CRITICAL FLOW

Extracted from "Fluid Mechanics for Hydraulic Engineers"
by Hunter Rouse

The velocity distribution in a channel causes a definite variation in total head from the top to the bottom of the section, as well as from side to side. Since it is expedient to express the weighted mean total head in terms of the mean velocity, the quantity $\frac{V^2}{2g}$ must be multiplied by K_e, the velocity-head correction factor whose value becomes unity only when the velocity is constant over the section. The magnitude of K_e may be as low as 1.05 for very smooth boundaries and high velocities of flow, rising well beyond 1.20 if the relative roughness is great. Accurate investigations require due regard for this factor, particularly if the velocity distribution varies from one section to the next. In the development of methods of practical engineering value, inclusion of this variable as a function of flow characteristics introduces needless complexities not compatible with such inexact methods as are involved in the empirical expressions for resistance. Therefore, K_e is considered to have a value of unity in the following developments.

The total head over a given vertical section will be given by the expression

$$E = \frac{V^2}{2g} + \frac{p}{\partial} + h.$$

But since the potential head is constant over the section and equal to the depth y plus the elevation h_o of the lower boundary of flow, the elevation of the energy line above the lower boundary may be written as

$$E - h_o = H = \frac{V^2}{2g} + y \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

This quantity is commonly called the specific energy, H referring to a datum which coincides with the lowermost streamline. In two-dimensional

apparent that for every value of q two depths of flow would again be possible. The rate of discharge must then have a maximum value for the given specific energy, which may be found by differentiating q with respect to y and setting the result equal to zero,

$$\frac{dq}{dy} = \sqrt{2g}\,\frac{2H - 3y}{2\sqrt{H - y}} = 0$$

whence,

$$\left(\frac{2H}{3y}\right)_{q\,max.} = 1.$$

But since the minimum specific energy $\left(\frac{q^2}{gy^3}\right)_{H\,min.} = 1$, it is evident that the criterion for maximum discharge at a given specific energy is the same as that for minimum specific energy at a given rate of discharge. In other words, maximum discharge occurs when $y = y_c$ or when the specific energy of the flowing medium is a minimum. It follows that the maximum or critical discharge is seen to have the form

$$q_c = \sqrt{g}\; y_c^{3/2} = \sqrt{g}\left(\frac{2}{3}H\right)^{3/2} \quad \ldots\ldots\ldots\ldots\ldots(5)$$

The simplified foregoing equations of two-dimensional flow will apply to a channel of finite width only if the side walls are vertical for any other channel, since the rate of discharge Q is the product of V and the area A of the flow section $H = \frac{Q^2}{2gA^2} + y$. Since $dA = b\,dy$ in which b represents the surface width,

$$\frac{dH}{dy} = \frac{d}{dy}\left(\frac{Q^2}{2gA^2} + y\right) = -\frac{Q^2 b}{gA^3} + 1$$

whence, under conditions of critical flow

$$\left(\frac{A^3}{b}\right)_c = \frac{Q^2}{g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(6)$$

Probably no phase of open channel motion is so often misunderstood as flow under conditions of maximum discharge and minimum specific energy.

flow, the rate of discharge q per unit width of section is the product of depth and mean velocity. The equation of H then becomes

$$H = \frac{q^2}{2gy^2} + y \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The foregoing expression involves three variables, any one of which may be considered dependent upon the other two. In the most general case of flow, all three will vary; but in many types of motion one or the other may change slightly, if at all. Such types of flow may then be treated, at least as a first approximation, in terms of only two variables. If q remains constant, according to equation (2) there will be possible two depths of flow y for every value of H. At some depth the specific energy must reach a minimum value, which may be determined for any given q by differentiating equation (2) with respect to y and setting the result equal to zero. Thus,

$$\frac{dH}{dy} = -\frac{2q^2}{2gy^3} + 1 = 0,$$

whence,

$$\left(\frac{q^2}{gy^3}\right)_{H\,\text{min. value}} = 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The depth and the velocity corresponding to a minimum specific energy are designated as critical and are given the subscript c. In terms of a constant rate of discharge,

$$y_c = \frac{2}{3} H_c = 2\frac{V_c^2}{2g} = \sqrt[3]{\frac{q^2}{g}}$$

and

$$V_c = \sqrt[3]{gq} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

If H were taken as the constant term in equation (2), it is

So long as gradually varied flow is considered, the critical depth as previously described is a very useful parameter. But equation (4) applies specifically to rectilinear motion and can have only qualitative significance when otherwise used. Flow can pass from the tranquil to the rapid stage only as the result of surface curvature, under which circumstances it is seldom possible to assume hydrostatic distribution of pressure at the true critical section without introducing appreciable error. The true critical section means the section at which the actual specific energy is a minimum. It does not correspond to the nominal section determined from $y_c = \frac{2}{3} H$.

An example of traditional practice is seen in the case of the broad-crested weir. It is usually presumed that for a given reservoir level upstream from the weir the maximum possible quantity of water will be discharged per unit time. Still assuming parallel motion, the flow profile would have the form shown schematically in figure 1.

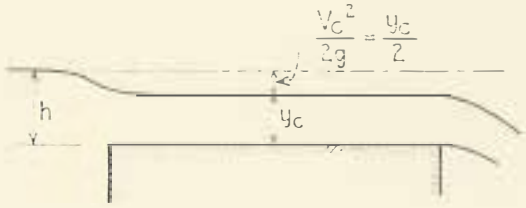


Figure 1

The rate of discharge should then be determinable from a single depth measurement,

$$q = \sqrt{g}\ y_c^{3/2} = \frac{2}{3\sqrt{3}}\ \sqrt{2g}\ h^{3/2}$$

Aside from the fact that this expression applies only to weirs of great height, four essential discrepancies are involved in this method

of attack:

1. The regions of curvilinear motion at either end of the weir extend a considerable distance in each direction. Unless the weir is very broad, at no section will the flow be truly parallel.

2. For a given specific energy, the maximum discharge for parallel flow is not necessarily the same as maximum discharge for curvilinear flow.

3. If the weir actually is broad enough to eliminate effects of curvature near the midsection, the energy line (and hence the water surface) will have an appreciable slope; the section at which $y = y_c$ will wander along the weir with changing discharge.

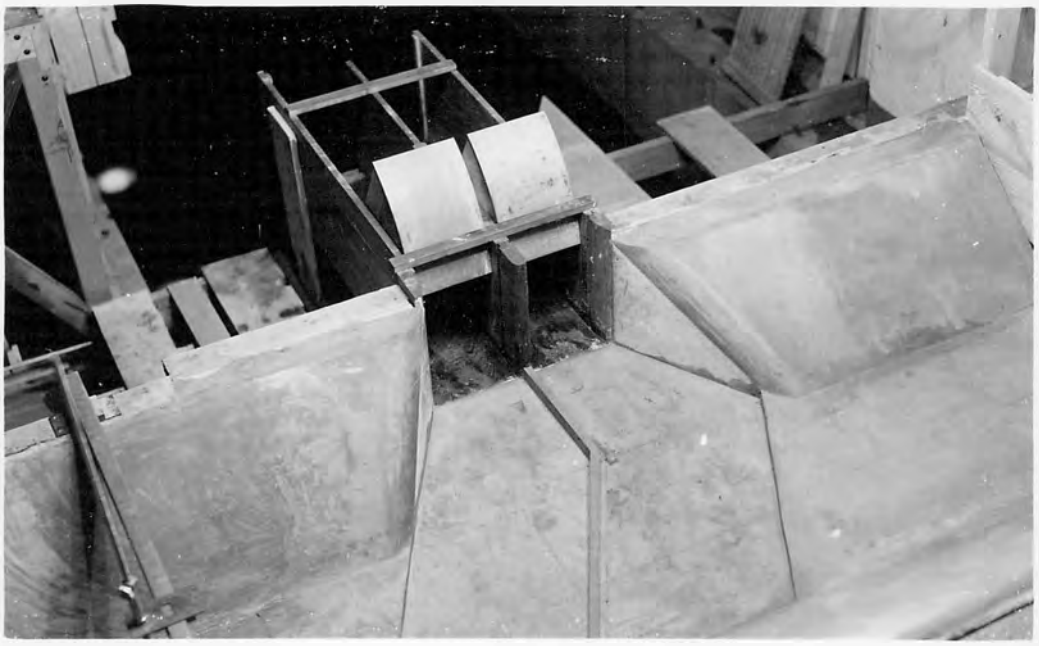
4. The true section of minimum energy does not then lie at some intermediate point along the weir, but at the end, which is a region of maximum curvature.

A true critical section for given boundary conditions must fulfill a two-fold requirement:

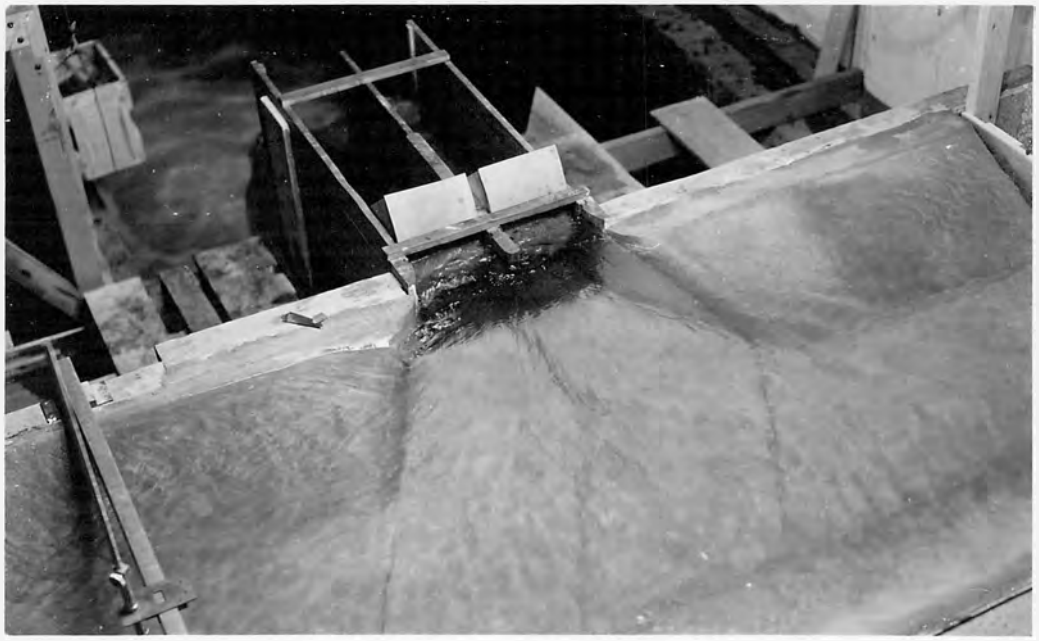
1. At the section, the distance between the sloping energy line and the lower boundary of the flow must be less than at any other point in the vicinity.

2. For this magnitude of the specific energy, the rate of discharge must be the greatest that is dynamically possible under the given boundary conditions.

The free overfall at the end of a long, mild slope (figure 2A), a slope which will not sustain critical flow, is an example of boundary conditions which satisfy these conditions. Since the curvilinear flow at the crest is marked by nonhydrostatic pressure distribution, it is evident that critical depth for parallel flow will be found a short distance upstream. The actual location of this section is indeterminate, for it will move upstream with increasing discharge and downstream with

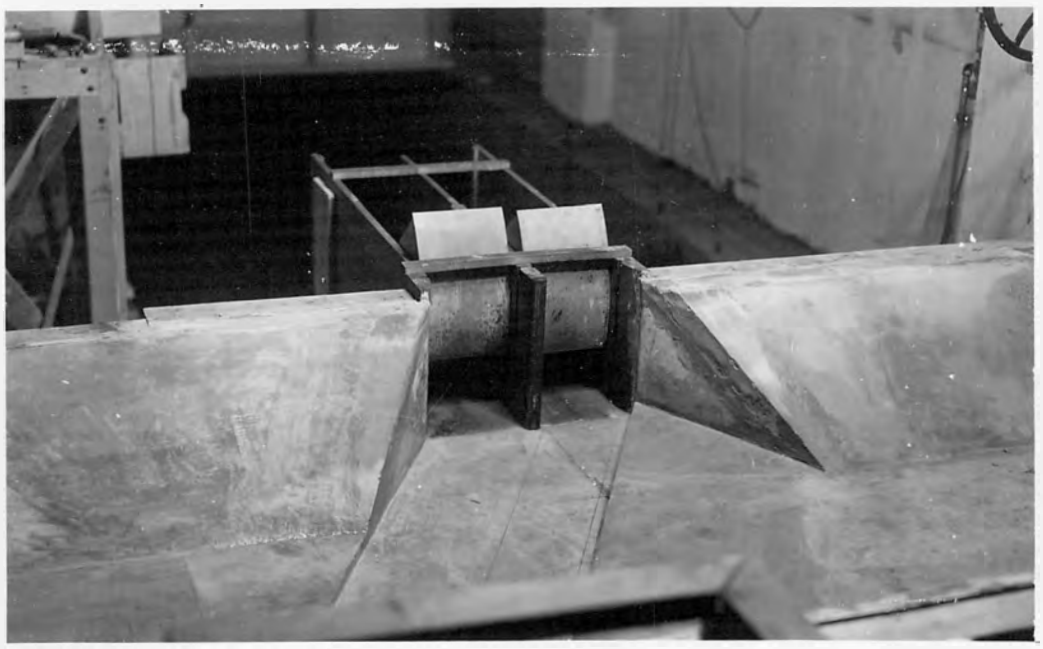


A. Model with round edges on transition.

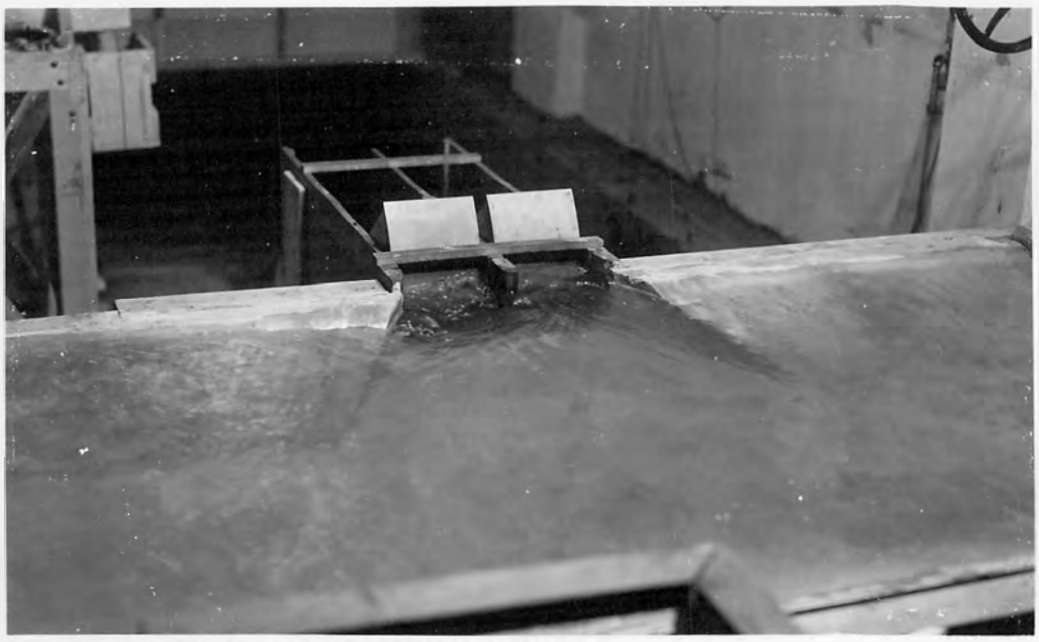


B. Flow conditions at maximum discharge.

DESIGN 1

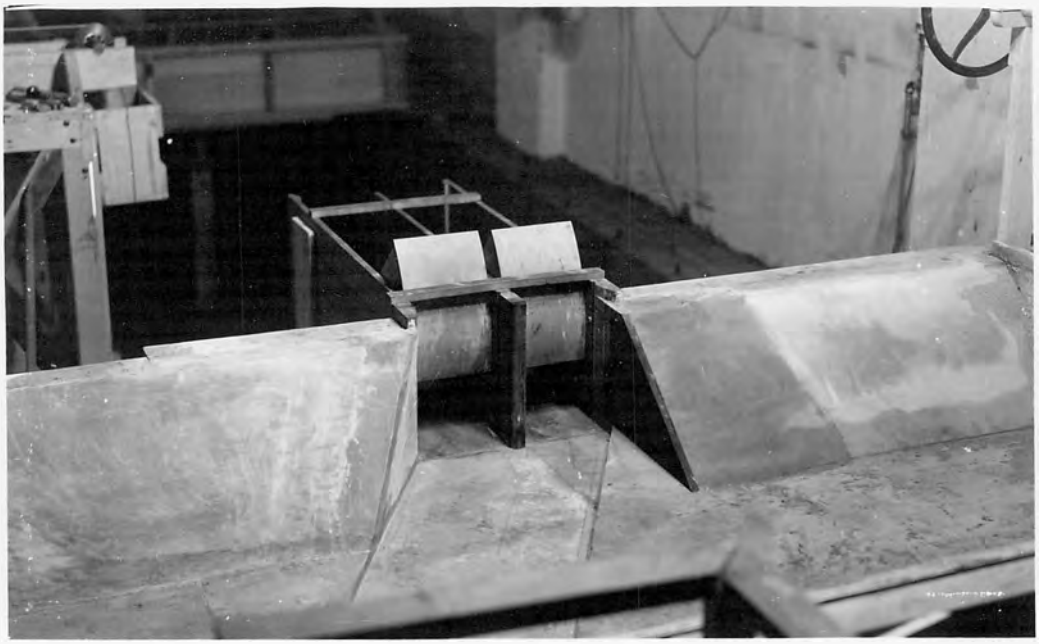


A. Model with sharp edges on transition.

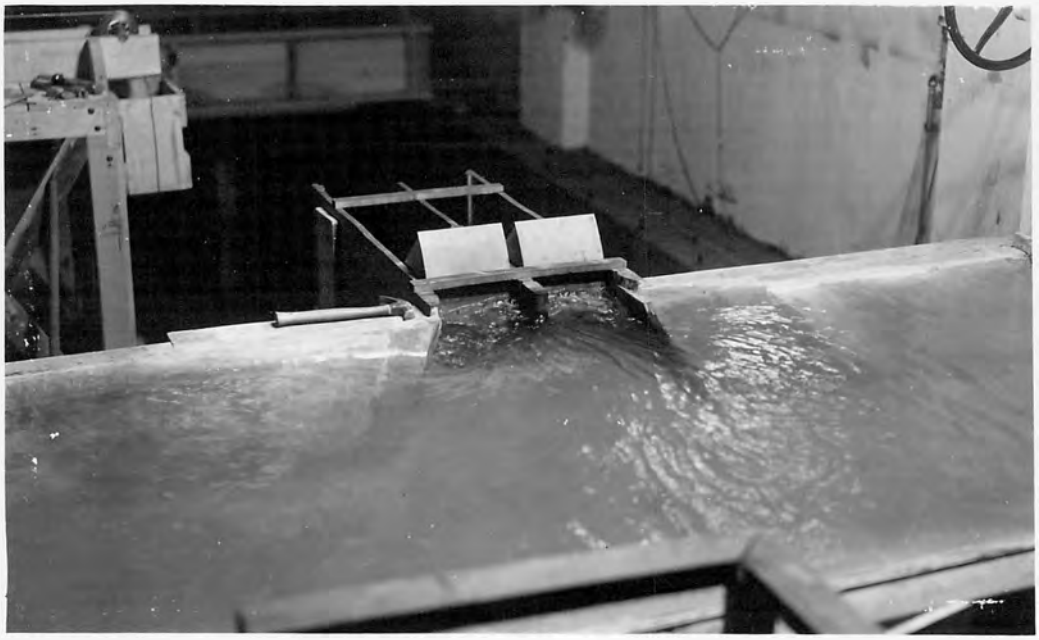


B. Flow conditions at maximum discharge.

DESIGN 2



A. Model with one flared wall and one wall normal to centerline of canal.



B. Flow conditions at maximum discharge.

DESIGN 3

increasing boundary roughness. The crest itself, however, is a true critical section. Experiments indicate that the ratio between the true critical depth at the crest and the nominal critical depth for parallel flow is 0.715, which happens to be the coefficient of contraction for a weir of zero height. A single measurement of the depth at the crest edge will permit immediate computation of the discharge

$$q = \sqrt{g}\ y_c^{3/2} = \sqrt{g}\ \left(\frac{y_{tc}}{0.715}\right)^{3/2} = 1.654\ \sqrt{g}\ y_{tc}^{3/2}$$

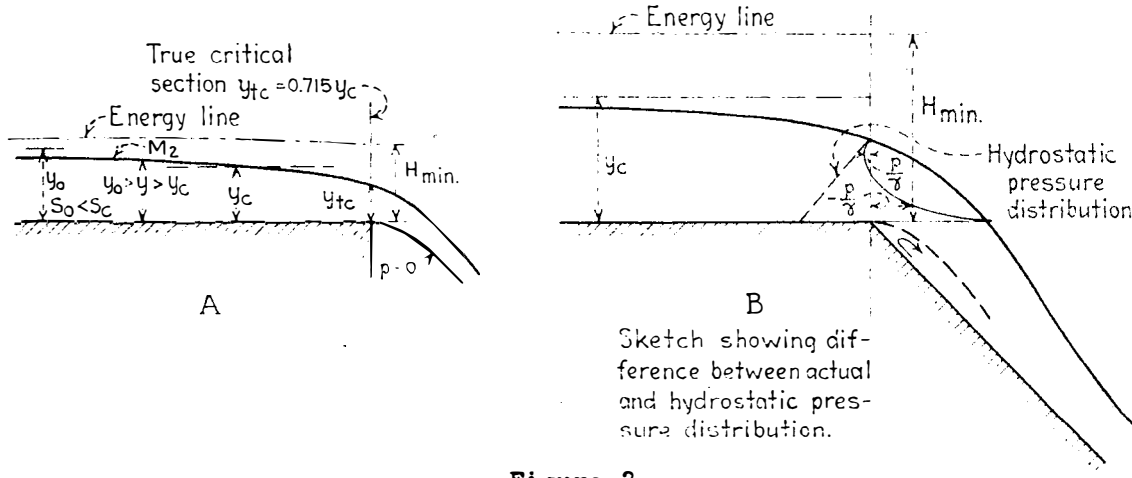


Figure 2

From a series of experiments conducted by M. P. O'Brien, it was concluded that $y = y_c$ was upstream from the crest a length $L = 11.6 y_c$. However, this is an approximation in that L is also a function of the roughness of the boundary. The discharge from this type of section may be computed from $q = \sqrt{g}\ y_c^{3/2}$ if the point at which critical depth y_c occurs can be located.

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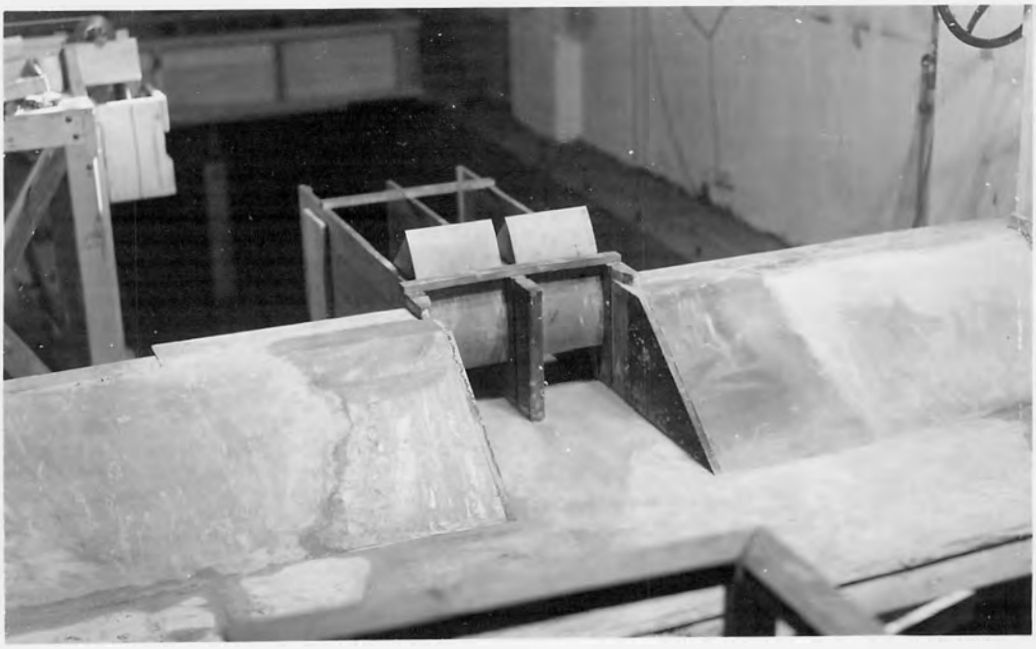
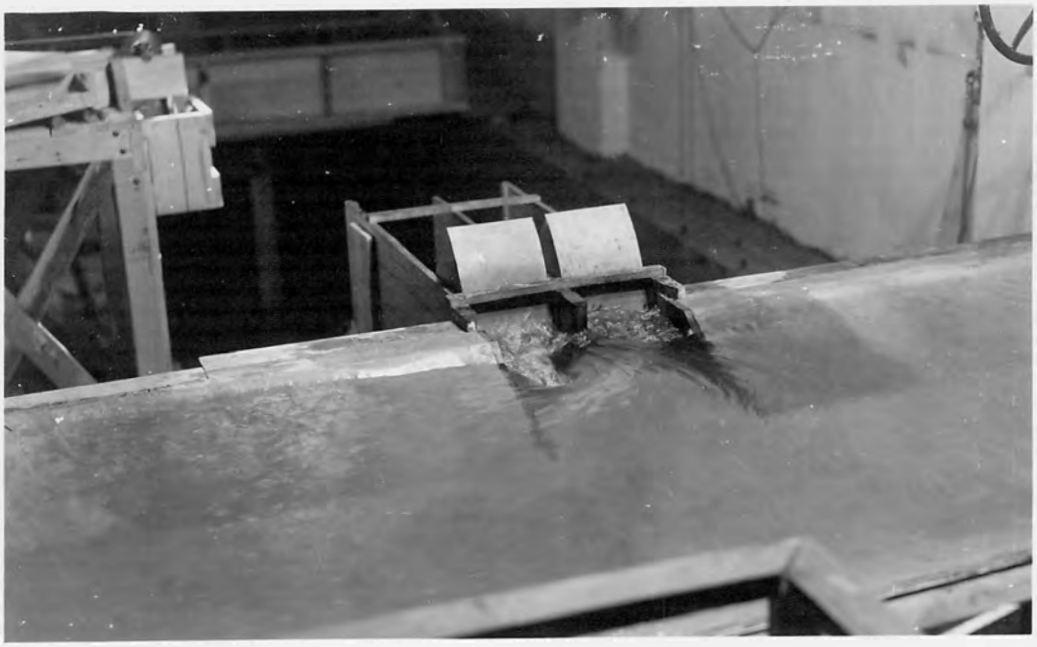
January 24, 1944.



A. Model with walls normal to centerline of canal.



B. Flow conditions at maximum discharge.

DESIGN 4

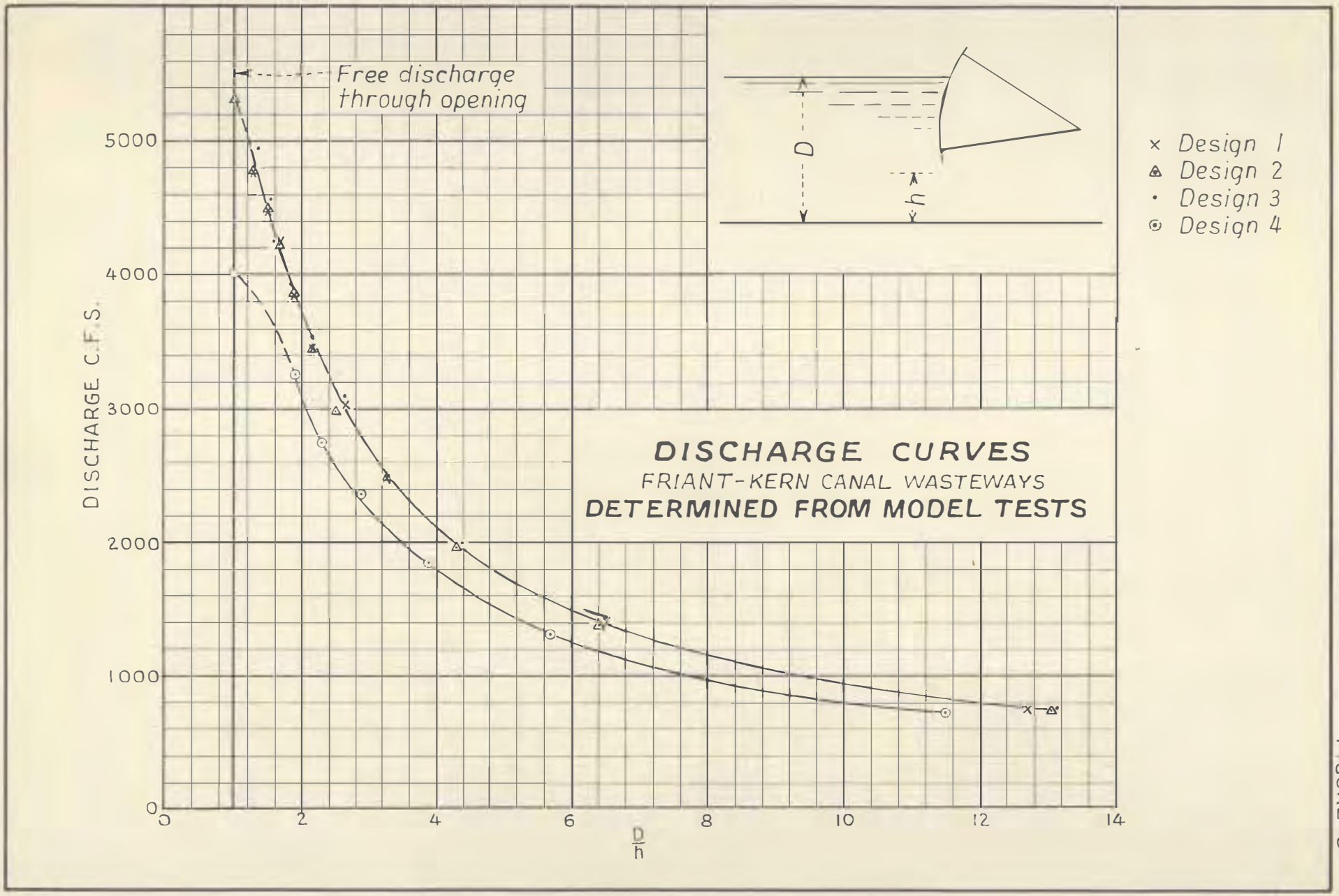

DISCHARGE CURVES
FRIANT-KERN CANAL WASTEWAYS
DETERMINED FROM MODEL TESTS
Free discharge through opening
D
h
× Design 1
Design 2
• Design 3
Design 4
DISCHARGE C.F.S.
0
1000
2000
3000
4000
5000
0
2
4
6
8
10
12
14
D/h

FIGURE 5

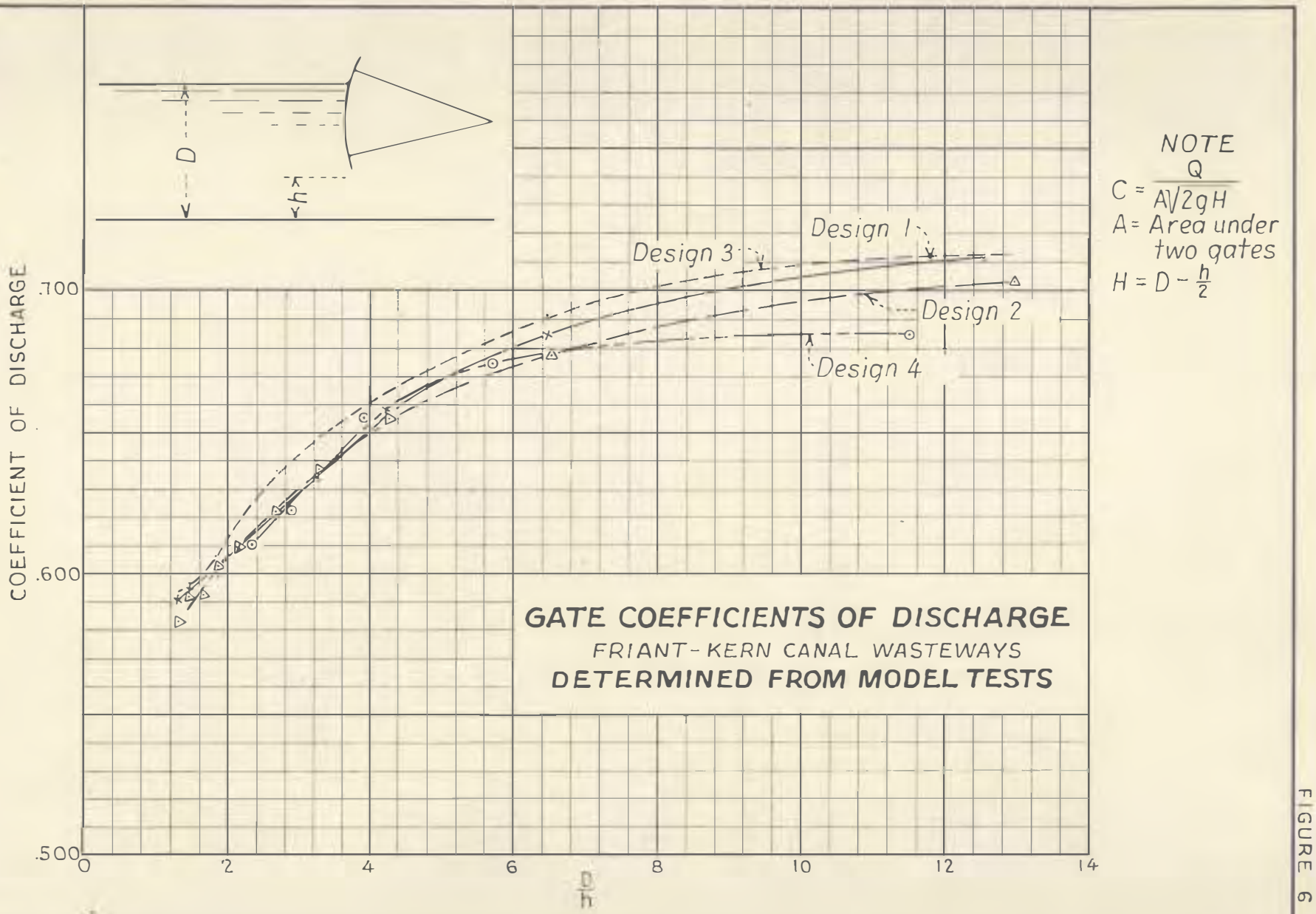

GATE COEFFICIENTS OF DISCHARGE
FRIANT-KERN CANAL WASTEWAYS
DETERMINED FROM MODEL TESTS
COEFFICIENT OF DISCHARGE
.700
.600
.500
0
2
4
6
8
10
12
14
D/h
D
h
Design 1
Design 2
Design 3
Design 4
NOTE
C = Q / A√2gH
A = Area under two gates
H = D − h/2

FIGURE 6