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WATER RESEARCH LABORATORY

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REPORT No. 89

By-wash Spillways for Farm Dams

by

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K. K. Lai, D. M. Stone and R. T. Hattersley

JUNE, 1966

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BY-WASH SPILLWAYS FOR FARM DAMS

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Report submitted to the Water Research Foundation of Australia Ltd.

June, 1966.

Preface

The School of Civil Engineering of the University of New South Wales has followed an extensive programme of research to improve methods for the design and construction of farm dams. This programme has operated continuously since 1957.

As part of the programme, the Water Research Laboratory undertook the work summarised in this report. It was commenced in 1964 with the financial support of the Water Research Foundation of Australia and continued until early 1966. The whole of the work has been carried out under the general direction of the officer-in-charge of the Water Research Laboratory. The experimental work was initiated by Mr. D. N. Foster, Senior Lecturer in Civil Engineering, assisted by Mr. J. R. Ewers, Project Engineer. Subsequently the work was completed by Mr. K. K. Lai, Project Engineer, under the supervision of Mrs. D. M. Stone, Project Officer, of the Water Research Laboratory staff.

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June, 1966.

Summary.

Model studies formed a basis for investigating the hydraulics of bywash spillways for farm dams. When constructed in natural earth such spillways are trapezoidal in cross section. The surface of the spillway needs protection from scour and this protection is usually in the form of grass.

These tests extended to variations in spillway geometry and variations in surface roughness. The effects of these variations on head discharge relationships were noted. The variation of discharge with head over the range of spillway surface lengths and the range of roughnesses tested was less than 10 pc. of the average discharge.

The report contains recommendations for the design of new spillways and examples indicating the design procedure are given.

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Notation.

A_c	-	Cross-sectional area at critical section (ft^2)
B	-	Width of weir (rectangular cross-section) (ft.)
B_o	-	Base width of spillway (trapezoidal cross-section) (ft.)
C_d	-	Discharge coefficient
f	-	a function
g	-	Acceleration due to gravity (ft/sec^2)
H_o	-	Difference between still water in reservoir and spillway crest (ft.)
H_c	-	Critical depth on the spillway crest measured above the ground surface (ft.)
h_b	-	Bend loss (ft.)
K	-	Bend loss coefficient
L	-	Length of spillway (ft.)
n	-	Manning's roughness coefficient ($sec. ft.^{-1/3}$)
Q	-	Discharge (cfs)
T_c	-	Surface width of flow at critical section (ft.)
R	-	Hydraulic radius
r	-	Subscript indicating ratio of prototype to model
S	-	Slope
v	-	Velocity (ft/sec.)
v_c	-	Critical velocity (ft/sec.)
x_r	-	Horizontal scale
y_r	-	Vertical scale
δ_*	-	Displacement thickness of boundary layer (ft.)
ϕ	-	a function

1. Introduction

The engineering design of small dams suitable for the retention of surface run-off from farmlands is subject to the limitations of the relatively small capital investment associated with the economy of construction of farm facilities. Consequently, earthen cut and fill operations are common. Earth fill embankments naturally form the downhill retention barrier of the pond. Notwithstanding improved compaction techniques, earth fill is notoriously weak in its resistance to scour from surface water flows. Artificial sheet protection in the form of concrete or similar materials is too expensive for most farm dam situations and excess flow must be diverted from the ~~spillway~~ EARTH FILL. Naturally bedded undisturbed earth is more resistant to scour than freshly placed earth fill and to provide for spillage ofexcess run-off it is advantageous to excavate a channel at the end of the earth fill retention bank in undisturbed material. To achieve this, the channel may be excavated at the side of a natural gully or against a hillslope forming part of the boundary of the dam.

Diversion channels of this kind conform in horizontal geometry to roughly a semi-circular shape, centred on the extremity of the earthfill embankment. Figure 1 illustrates the general layout of a farm dam with a circular type spillway. The approach section of the channel may be constructed of fill material if necessary because the velocity of approach of the water is strictly under control up to the line of the crest of the dam. Beyond the crest, which is flat, it is essential to limit the velocity of flow and also to protect the spillway surface by turf or other low cost means, at least to the point where scouring effects of spillage are controlled or harmless.

The design of a bywash spillway which results from the above-mentioned conditions is broad and shallow, curved in plan, with moderate approach and downstream slopes. Notwithstanding the vast literature on the hydraulics of weirs (mostly empirical) the special geometry and the nature of the surface of farm dam spillways necessitate the use of an hydraulic model to evaluate the head discharge relationships. In this report a theoretical analysis is made from existing information and the theoretical material is applied to the results of the model experiments conducted as part of this investigation to derive the head discharge characteristics.

2. Theoretical Considerations

A farm dam spillway is a form of a broad crested weir. For a broad crested weir, if the approach flow is tranquil, the flow attains a critical depth at some cross section on the actual crest itself. If the point at which a critical depth occurs can be ascertained, the theoretical discharge for the weir can be calculated. The theoretical discharge calculated in this manner is for a straight crested spillway.

2.1 Head Discharge Formula

The head-discharge relationship for a broad crested weir of the kind described above is expressible in the form -

$$Q = C_d \, f\left(\frac{H_o}{B_o}\right) g^{\frac{1}{2}} \quad B_o^{5/2} \qquad (1)$$

where Q = Discharge in cu. ft. per second

H_o = total head above weir crest (ft)

B_o = base width of weir crest (ft)

f = a function

C_d = a discharge coefficient depending on shape and character of boundary roughness

Equation (1) is derived from the theory of straight broad crested weirs (see Appendix 1). Modifications to this equation and its expansion to apply the equation to farm dam spillways are discussed seriatim in the following paragraphs.

3. Factors Affecting the Discharge Coefficient

The effect of the viscosity of water is neglected in the derivation of equation (1). Although the viscosity of water is of low magnitude it is responsible for such effects as skin friction against the wetted boundary of the spillway surface and drag produced by flow around irregularities in the boundary surface. The irregularity may merely consist of surface roughness or it may be present as a result of relatively large scale geometrical features of the spillway boundary surfaces. Drag produced from these causes may be considered separately for study and evaluation on the assumption that the effects are independent.

3.1 Skin Friction

The principal effect of skin friction in weir flow is to increase

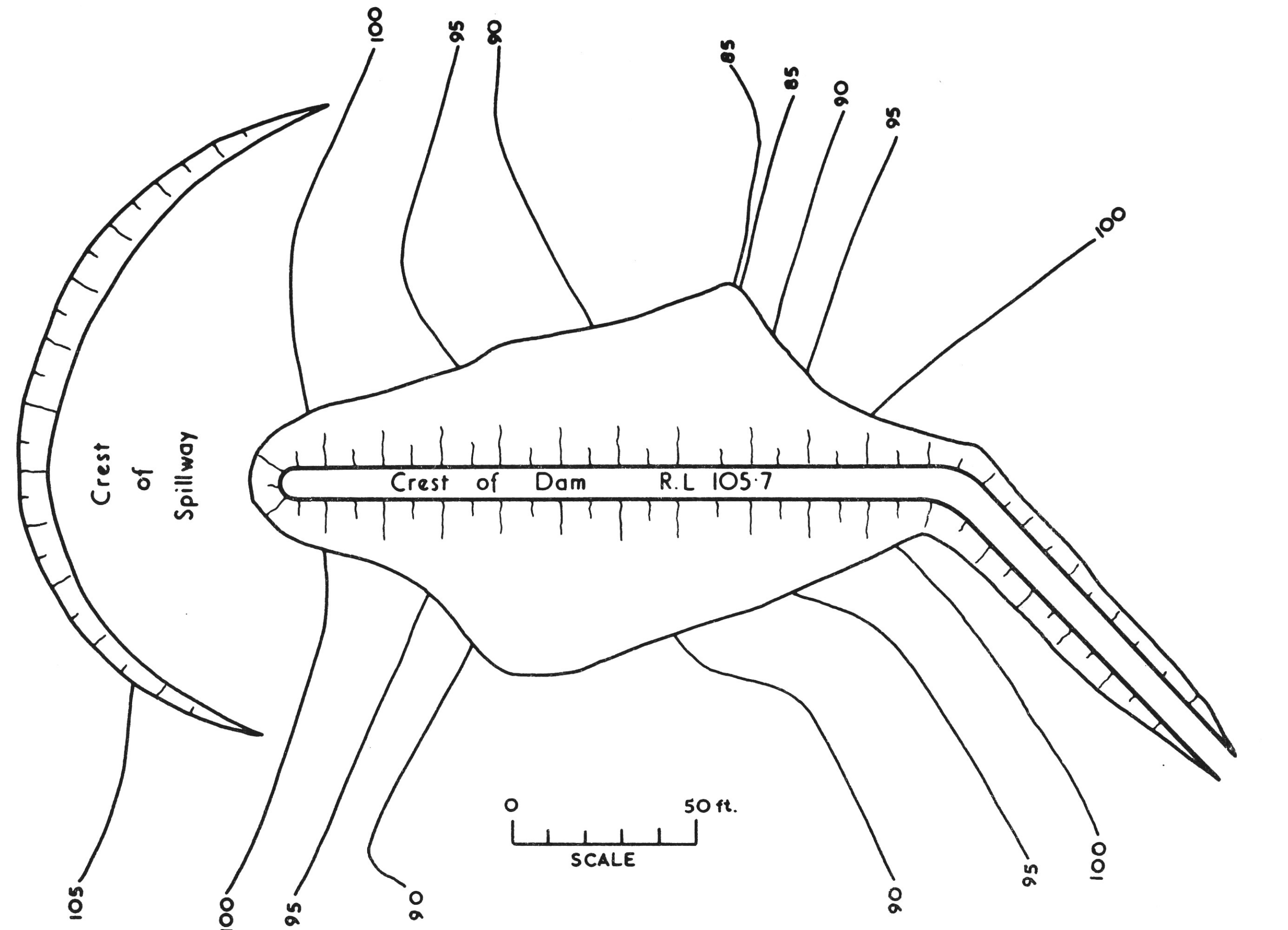

FIGURE 1: GENERAL LAYOUT OF A FARM DAM WITH SIDE-CHANNEL TYPE SPILLWAY
(Proposed Earth Dam in Marshall Mt., N.S.W.) CE-E-6714

the real cross sectional area required for a given flow by the equivalent of the displacement thickness (δ_*) of the boundary layer. In effect, the value of H_o reflects the displacement thickness. This is shown by the value being larger for a given flow than the predicted value obtained from non-viscous theory.

A modified form of the weir equation illustrating this effect is given in Appendix 2. The effect is also illustrated in Figure 2 for a width to length ratio ($\frac{B}{L}$) of 0. 20 as given by Rouse (1950). C_d is smaller for smaller lengths.

A further effect on the flow occurs if the boundary or surface of the spillway is rough. The flow in the boundary layer is retarded and a greater depth is required to carry a given discharge. In the case of flow over a spillway, critical depth is independent of the surface roughness. Because roughness retards the flow, increasing the mean depth, critical depth occurs further downstream for a rough spillway than in the case of a smooth spillway. A lower discharge coefficient results from increased roughness.

3. 2 Form Resistance

The geometrical form of a broadcrested weir may be varied by changing such variables as the length of the crest, the slope of the approach apron, the slope of the downstream apron, and the width of the cross section. King and Brater (1963) have summarised experiments by Bazin and the U. S. Deep Waterways Board which show that the discharge coefficient for straight weirs varies mainly with the elevation of the water surface upstream of the spillway crest.

For a 1:2(horizontal to vertical) approach slope, when the downstream face slope varies from 1 to 1 to 1 to 6, there is a difference of less than 7 pc. in the value of the discharge coefficients for the same $\frac{H_o}{L}$ ratio. The difference is generally less for a small $\frac{H_o}{L}$ ratio. On the other hand, if the downstream face slope is fixed at 2 to 1 there is a difference up to 10 pc. for a given $\frac{H_o}{L}$ ratio for a range of approach slope from 2 to 1 to vertical. Thus, there is a more marked effect on the discharge coefficient from different approach slope than from a different slope of the downstream apron.

Discharge coefficients increase with length of the spillway, as can be seen from the data shown in Figure 3. This will be discussed in more detail in Section 5. 3 with regard to scale effects.

3.3 Bend Loss

When the water flows around a bend, there will be some loss in energy called bend loss. If a bend occurs as part of a crest of a spillway, the elevation of the water surface upstream of the spillway above the crest will be higher for a given discharge than if the crest of the spillway is straight. Hence, a lower discharge coefficient will appear in equation (1) for a spillway with curvature in plan. Studies on a smooth channel with a semi-circular bend, by Allen and Chee (1962) indicate that, for a given depth to width ratio for a channel, a bend loss coefficient defined as

$$k = \frac{h_b}{\frac{v^2}{2g}}$$

is less for a higher Reynolds number.

In the above equation k = bend loss coefficient

h_b = bend loss in feet head of water

v = velocity of flow in feet per second

Figure 4 shows the relationship between the value of k and the depth to width ratio for various Reynolds numbers.

4. Model Investigation

A spillway which is designed specifically for a farm dam has geometrical features which are characteristic. These characteristics such as overall length, aprons, curvature in plan, all have influences, cited in the previous paragraphs, on the magnitude of the discharge coefficient. To evaluate the combined effect of these features and to study variations of them a model was built and tested. The scale selected was 1:15 and the materials used for the surfaces were painted plywood and galvanised iron. Details of the model design and construction are given in Appendix 3.

4.1 Discharge Coefficient

The head discharge relationship shown in equation (1) may be written in the form

$$Q = C_d \, Q_{th} \qquad (2)$$

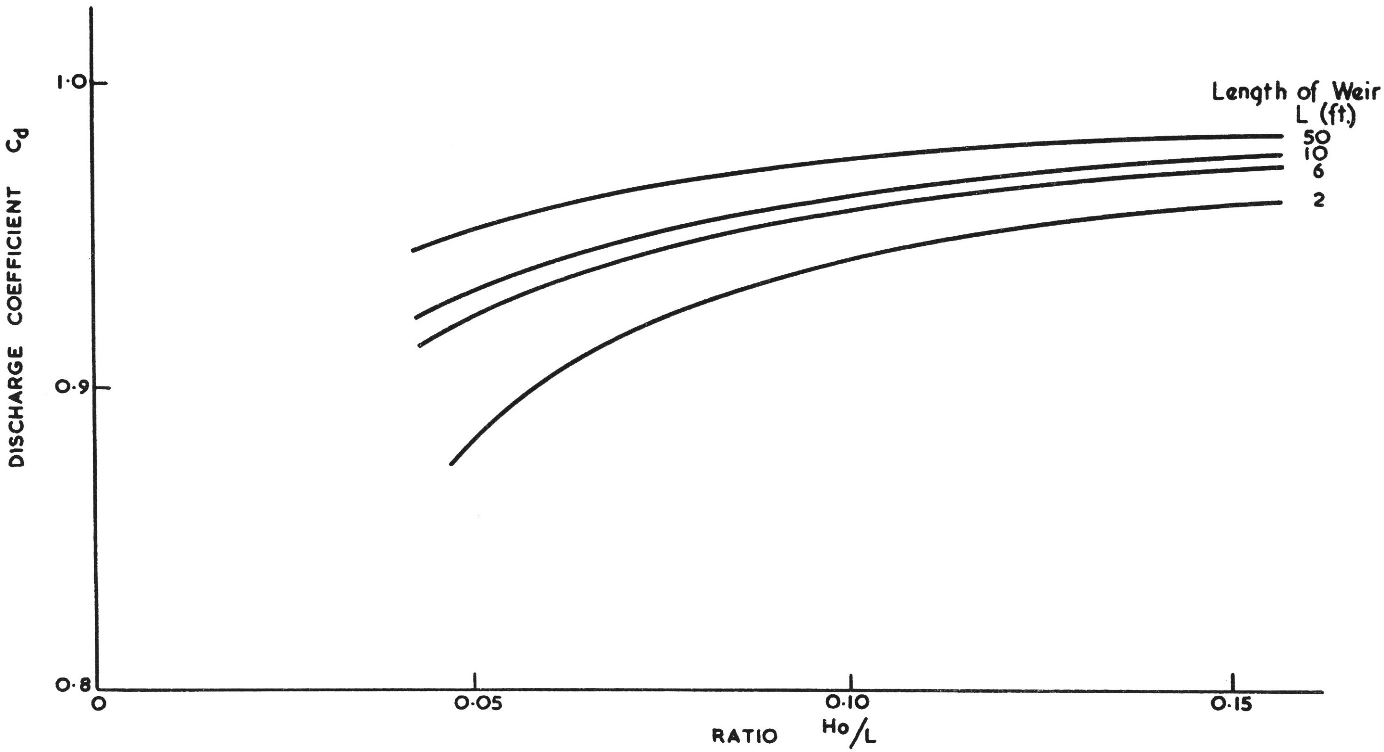

FIGURE 2: DISCHARGE COEFFICIENT FOR BROAD-CRESTED WEIRS WITH $B/L = 0{\cdot}20$

(After Rouse, p.527)

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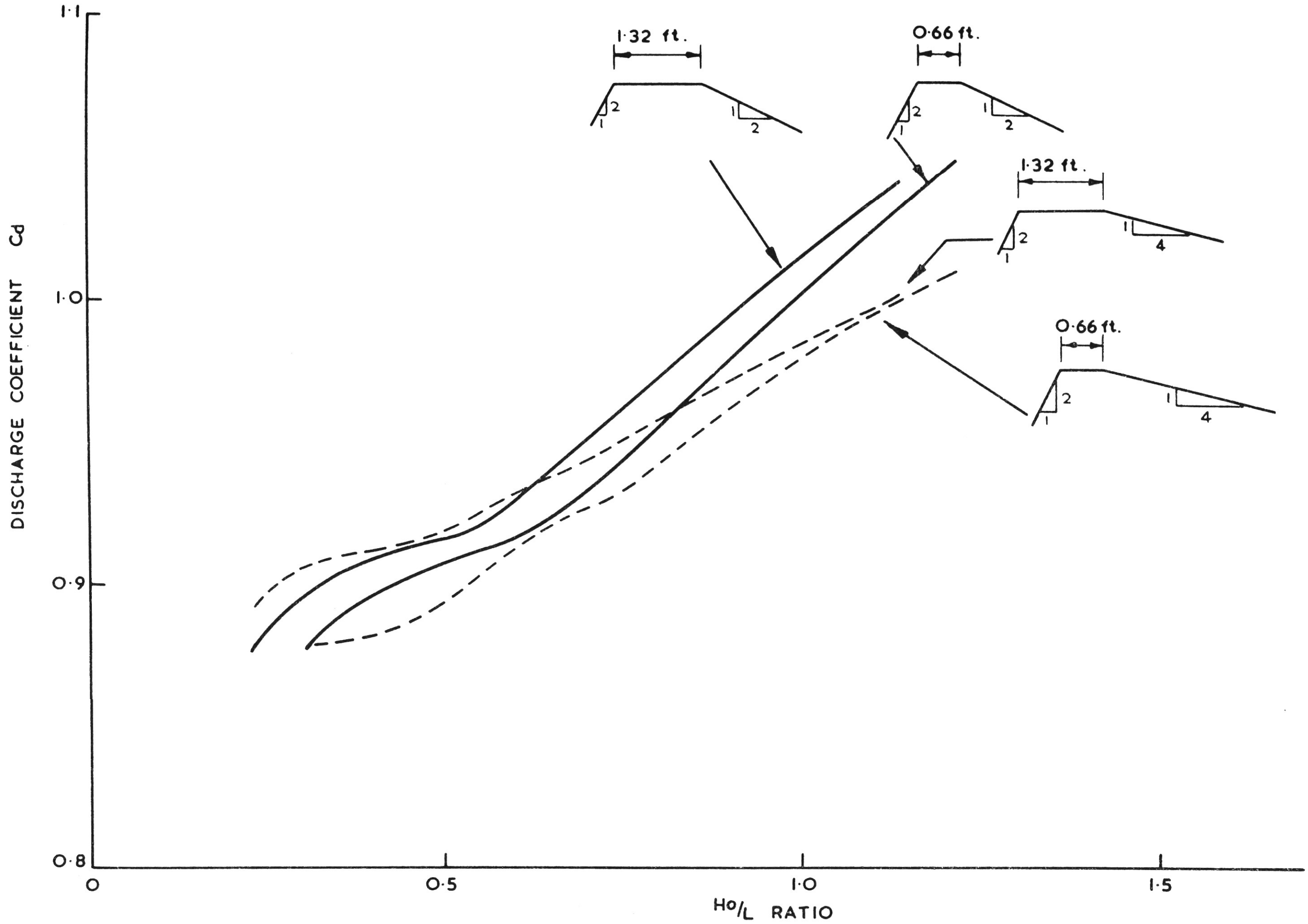

FIGURE 3: DISCHARGE COEFFICIENT FOR BROAD-CRESTED WEIRS WITH VARIOUS SHAPES AND Ho/L RATIOS. (After King, p. 5-49) CE-E-6716

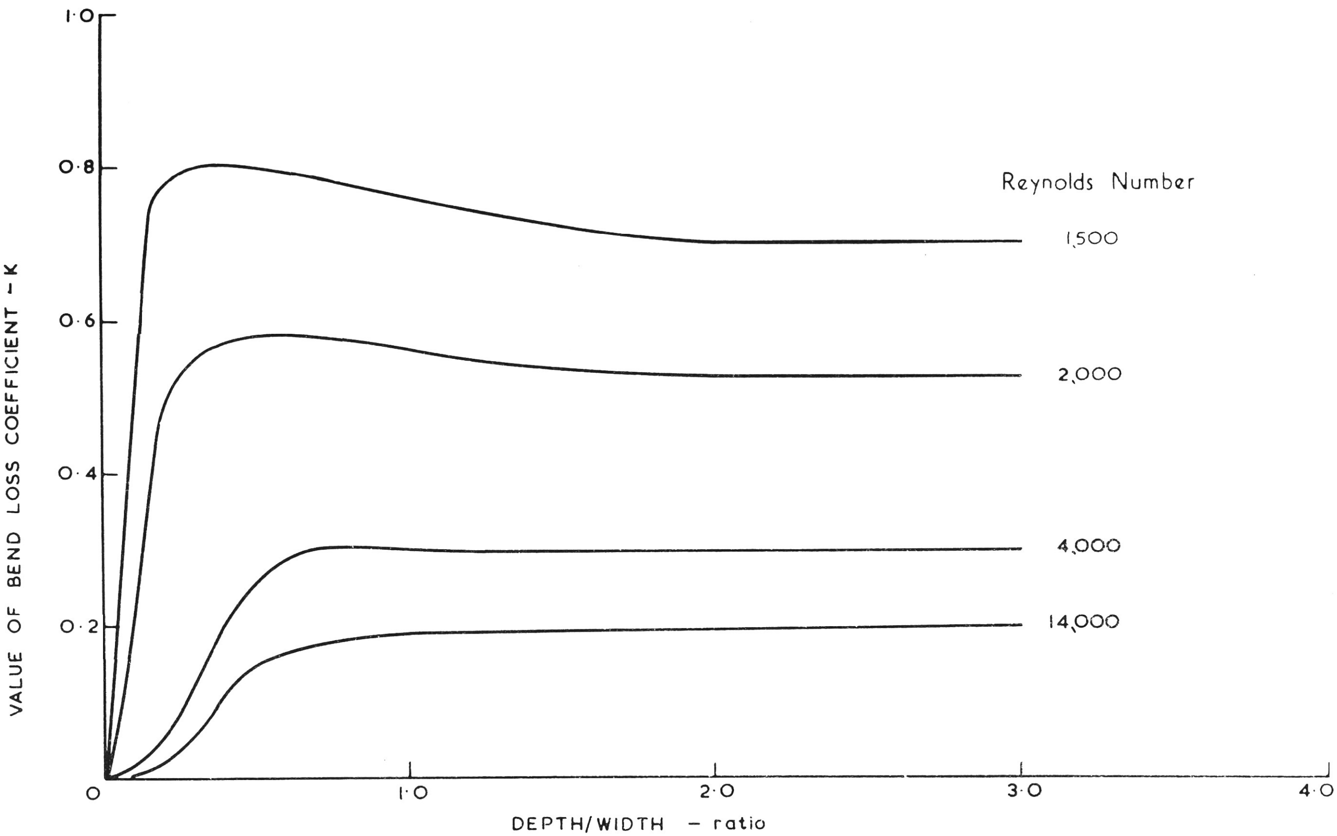

FIGURE 4: VALUE OF BEND LOSS COEFFICIENT AS A FUNCTION OF DEPTH/WIDTH-RATIO AND REYNOLDS NUMBER. (After Prof. Jack Allen and Sek-Por Chee)

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where Q_{th} is the theoretical discharge computed by using equations (5) and (7) in Appendix 1.

By comparing actual discharges (Q) measured in the model with the theoretical discharges (Q_{th}) values of C_d were obtained which implicitly contained corrections for the factors of form and skin friction.

For convenience in plotting the experimental results, equation (1) was used in the rearranged form

$$\frac{Q}{g^{\frac{1}{2}} B_o{}^{5/2}} = C_d f \left(\frac{H_o}{B_o}\right) \quad (3)$$

where C_d equals unity in the computation of the theoretical discharge.

4.2 Description of Model Tests

The first series of tests was carried out in a spillway model of 3 ft. base width. Both the upstream edge and the downstream edge of the crest made angles of 45° with the axis of the dam. The side slopes of the spillway were 3 to 1 (horizontal to vertical). The approach slope on the upstream end and the slope of apron on the downstream end of the crest were respectively 3 to 1. A range of flows from 0.3 to 1.2 cu. ft. per second was run over the model spillway and the corresponding heads were measured. Tests were repeated for two patterns of artificial roughness consisting of river gravel spaced 6 inches by 6 inches and 3 inches by 3 inches on the spillway surface.

The base width of the spillway was then reduced to 2 ft. and subsequently another series of tests was conducted with a base width of 1 ft. Tests were again conducted for three different types of roughnesses.

For the base width of 2 feet, some data were also obtained for a 5:1 upstream apron approach slope. It was found that the difference compared with a 3:1 slope, was so small that it did not warrant tests of other approach slopes.

In the second series of tests, the upstream edge was made 90° with the axis of the dam, that is, the upstream end of the crest of the spillway was perpendicular to the axis of the dam, with the downstream edge remaining at 45°. Two base widths (3 feet and 1 foot) and two roughnesses (painted plywood and 3 inches x 3 inches spacing of river gravel) were tested. In the third series of tests the downstream edge of the crest was also made perpendicular to the axis of the dam and the crest of the spillway became a semi-circle in plan. As before, the two base widths (3 feet and 1 foot) and the two roughnesses (painted plywood and 3 inches x 3 inches spacing of river gravel) were tested.

4. 3 Summary of Model Results

The double logarithmic plot of equation (3) in Figure 15 shows the experimentally derived points plotted above the theoretical curve of the equation

$$\frac{Q_{th}}{g^{\frac{1}{2}} B_o^{5/2}} = f\left(\frac{H_o}{B_o}\right)$$

All cases tested, covering variations in roughness, curvature in plan and approach slope, may be enveloped by a design curve such that

$$\frac{Q}{g^{\frac{1}{2}} B_o^{5/2}} = C_d \; \phi\left(\frac{H_o}{B_o}\right)$$

where C_d is a function of $\dfrac{Q_{th}}{g^{\frac{1}{2}} B_o^{5/2}}$

This design curve can be applied to a spillway which is semi-circular in plan covered by grass of dry standing height comparable with the water depth on the spillway. For a shorter spillway or one with short cropped grass, a design formulated on the curve in Figure 15 will be adequate but it will provide excess capacity of order reaching 10 pc.

Should the features of an individual spillway be known to conform to the geometry described in the legend to the plot of equation (3) a closer estimate of the discharge coefficient is possible, though this is not generally warranted.

In Figure 15 it is obvious that the experimental points deviate more from the theoretical curve with increase in length of the spillway and increase in surface roughness, indicating a lower discharge coefficient in such cases.

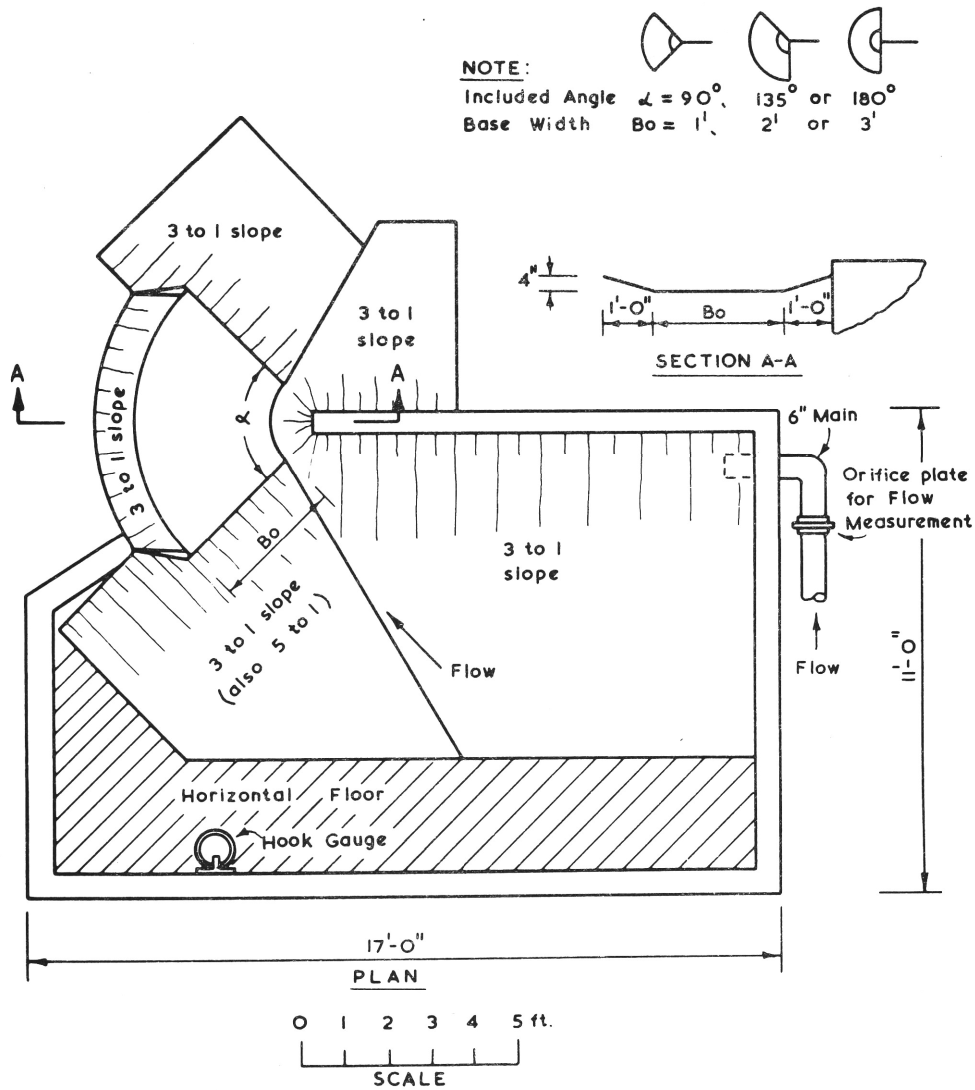

FIGURE 5: GENERAL LAYOUT OF MODEL SPILLWAY

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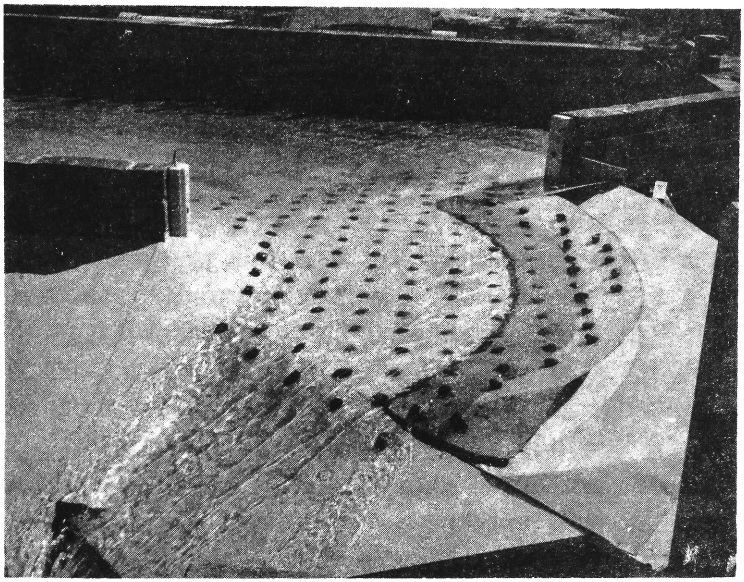

Figure 6: Model Spillway in Operation.

For a given value of $\frac{H_o}{B_o}$ there is a theoretical value of $\frac{Q}{g^{\frac{1}{2}} B_o{}^{5/2}}$ with a corresponding experimental value and the discharge coefficient can be obtained from these two values using equation 2 in Section 4.1 above. Due to the small range in magnitude of $\frac{H_o}{B_o}$ values in each series of tests, the discharge coefficient does not show the trend to increase with the $\frac{H_o}{B_o}$ as it would over a wider range of values for a broadcrested weir.

For practical purposes a design curve can be drawn which envelopes all the experimental points. This design curve can be applied to spillways having the shape of up to a semi circle in plan with surfaces protected by grass with an excellent stand and height up to 30 inches above the spillway surface. For a shorter spillway or a spillway protected by shorter grass, a design using this curve will be on the safe side if the curve is used to estimate the size of the spillway itself. Heights of 30 inches stand for grasses likely to be used on the farm dam spillways contemplated in Australia are not likely - a more reasonable maximum would be about 1 foot. The design will therefore be conservative.

4.31 <u>Depth of Flow over Crest of Spillway.</u>

After entering the crest region of the spillway the water surface drops with the distance downstream. The drop is partly due to the fact that with tranquil approach flow, the water attains a much higher velocity over the crest of the spillway and partly due to the headloss of the flow caused by the factors previously discussed. Because of the short length and the curvature of the spillway, the flow over the crest is not uniform. It is to be expected that the flow will be shallower and the velocity higher near the inside downstream corner of the spillway. Figures 10 and 11 show the depth of the water at various points on the crest of the spillway. It can be seen that the water has a greater depth near the upstream edge and on the outer perimeter of the spillway. Near the inside downstream corner the depth of water decreases rapidly and high local velocities occur. There is a danger of scour in this region which is close to the end of the dam and additional protection may be necessary to prevent damage to the embankment. Alternatively, the spillway may be enlarged to limit the velocity at this point.

Figure 12 also shows some longitudinal profiles along the centreline of the spillway.

4.32 Velocity Distribution

As stated in Section 2 of this report, critical depth will occur at some point on the crest of the spillway if the approach flow is tranquil. The observations on the model showed that the critical depth occurs near the downstream end of the crest. Upstream of the point where critical depth occurs the velocity is lower than the critical velocity. Downstream of the critical section the flow is supercritical. Velocity measurements were made at various points on the crest of the model spillway in two cases, one for a crest width of 3 feet with a discharge of 1.20 c.f.s. and the other for a width of 1 foot with a discharge of 0.8 c.f.s.

The measurements were taken with a miniature Ott meter and a DSIR miniature flowmeter. The measurements were taken at mid depth of the flow. Figures 13 and 14 show the velocity readings obtained at various points on the crest of the spillway. The figures shown are ratios of the measured velocity to the critical velocity. The critical velocity was computed by assuming the spillway to be straight and by making use of the relationship shown in equation (3) of Appendix 1, namely

$$\frac{Q^2 T_c}{g A_c^3} = 1$$

where
$$T_c = B_o + 6H_c$$
$$A_c = (B_o + 3H_c)\, H_c$$
$$V_c = \frac{Q}{A_c}$$

In Figures 13 and 14 it can be seen that the flow attains critical velocity (as shown by the 1.0 velocity contour) not very far from the upstream edge of the crest on the inner side of the spillway. On the outer side of the spillway the critical flow is near the downstream edge of the crest. Upstream of the 1.0 velocity contour, the flow is subcritical, and downstream of it, it is supercritical. Except very close to the inside corner of the downstream end of the spillway where the flow was too shallow to take velocity measurements with a meter, the maximum velocity ratio was about 1.6. Qualitative observations with dye injection show that velocities in this small region of shallow water were not much greater than about 1.6 times the critical velocity as found in the nearby areas where velocity measurements were possible. It can be expected that the maximum velocity near the downstream end of the spillway crest will have a smaller velocity ratio if the spillway surface is rough.

It may also be noted that these velocities were measured on a spillway with an included angle of 90°. With an increase in this angle up to 180°, the position of the critical section moves downstream, thus limiting the increase in velocity near the downstream end to less than the velocity for the 90° angle. Quantitative measurements were not made on the model, but qualitative observations of the model were that similar velocity ratios prevailed near the downstream end of a spillway of 180° angle to those seen for a spillway of 90° angle.

5. Scaling of Model Results

To apply the results of measurements taken on a model constructed to a smaller geometrical scale than the natural size spillway, recognition should be made of the possible sources of error due to scale effects between the model and the full scale spillway. The scale effect is appropriately considered by subdividing the consideration of the results under the headings previously referred to, namely Form Resistance, Bend Loss, Skin Friction.

5.1 Bend Loss

Since the Reynolds number of the flow on a spillway is higher than the Reynolds number attained in a model, the discharge coefficient will be higher for the full scale spillway because the value of K in general has been found to be high for low values of Reynolds number, assuming that the depth to width ratio of the discharge on the spillway is preserved.

The values of Reynolds number for the model are of the order of 10^4 whereas the full scale Reynolds numbers are of the order of 10^6. From Figure 4, it can be seen that the bend loss coefficient for both will be less than 0.2, being about 0.15 for the model and 0.1 for the full scale spillway. The total headloss caused by the bend is about 1 pc. of the head. Hence, the use of a model coefficient of discharge for the full scale design would result in an over design of about 1 pc. as far as the bend loss is concerned, when calculating the discharge from a known head.

5.2 Skin Friction

From Fig. 2 it can be seen that for a low $\frac{H_o}{L}$ ratio (in the range 0.05 to 0.1, the range likely to be applicable to grassed spillways and such as were also used in the model study), the value of the superficial local drag coefficient C_d for a spillway with a representative length of 50 ft. is seen to be 2-3 pc. higher than that for a spillway with a length of 6 ft. which is a representative length for the model spillway used in these tests. Hence if model C_d values are applied to full scale spillways, the design will be conservative by this amount, assuming the discharge is to be calculated from known values of head.

5.3 Form Resistance

In Figure 3, the effects of skin friction and form resistance have not been separated in the overall C_d values. The variation between C_d values can be ascribed largely to form resistance. For a given value of $\frac{H_o}{L}$ it can be seen that a full scale spillway designed by using a C_d determined from a half scale model will be about 2 pc. overdesigned for the range of $\frac{H_o}{L}$ values available. Values of $\frac{H_o}{L}$ below 0.2 are not available, whereas for the spillways under consideration $\frac{H_o}{L}$ values will be about 0.05 to 0.1. The best assumption is that similar variations will apply for these $\frac{H_o}{L}$ values. With spillways scaled from a 1:15 model, the variation in C_d will obviously increase above the 2 pc. quoted for the 1:2 scale model. As can be seen from Figure 2, the rate of increase of variation decreases rapidly, so that a variation of about 3 times that for a 1:2 scale model may be regarded as a reasonable estimate for a 1:15 scale model. Therefore, an overdesign of 6 pc. calculated on discharge may be contemplated as resulting from using model values of C_d for the full scale design.

Figure 3 also indicates that a semi circular spillway may be expected to yield a smaller value of C_d than a quadrant spillway, because the $\frac{H_o}{L}$ will be about one half that for the quadrant. The model tests have shown that the H_o value varies less rapidly than the L value. The model tests have borne out the hypothesis that the C_d values will be smaller for semi circular spillways than for quadrant spillways.

5.4 Combination of Various Scale Effects

If all the scaling errors were directly additive, it will be seen that an overdesign of up to 10 pc. could result. For small spillways such as those with H_o values in the range from 1 to 5 feet, a 10 pc. overdesign on discharge is not excessive, and in any event the roughness of the surfaces of the spillways are not likely to be determinate within a better range of accuracy. A further consideration is that the usual construction techniques applied in the case of farm dam spillways will not ensure location of the crest level with comparable accuracy. It is also to be noted that all the scaling errors are conservative when calculating discharge from a known head. Therefore, the prototype spillway will actually have a greater discharge capacity for a given head than would be calculated by using the model coefficients without adjustment.

6. Application of Model Results to Full Scale Spillway

The model spillway was constructed of painted plywood, the Manning's roughness coefficient, n, being about 0.015. More often than not full size spillways are grassed for protection. The height of the grass is usually from several inches to 1 foot or over. The Manning's n for such grassed surfaces depends on the kind of grass used, its height, density and the condition of growth, as well as the depth of flow and the velocity. The Manning's n values for different types of grass can be reasonably expressed as a function of grass height alone, as long as the grass is of good stand, and values derived from measurements made by the U. S. Soil Conservation Service and reported by Chow (1959) are shown in Figure 7 in relation to a parameter involving the velocity and depth of flow.

In the full size spillway, the grass on the surface not only offers a high resistance to the flow, but also has a height protruding above the spillway surface which is comparable to the depth of flow. The artificial roughness elements in the model should therefore be large in size compared with the depth of flow, and their arrangement should give the Manning's coefficient as required.

The roughness ratio between the prototype and the model can be approximated as follows. Assuming a uniform flow in a wide open channel, the velocity ratio between the prototype and the model can be written as

$$v_r = \frac{1}{n_r} \, y_r^{2/3} \, S_r^{\frac{1}{2}}$$

or, since for open channel flow velocity is scaled by the Froude criterion as the square root of the depth,

$$y_r^{\frac{1}{2}} = \frac{1}{n_r} y_r^{2/3} \frac{y_r^{\frac{1}{2}}}{x_r^{\frac{1}{2}}}$$

whence $$n_r = \frac{y_r^{2/3}}{x_r^{\frac{1}{2}}}$$

where v_r = velocity scale

y_r = vertical scale

x_r = horizontal scale

n_r = ratio of Manning's n in the prototype and in the model

For a 1 to 15 undistorted model, the roughness ratio between the prototype and the model from Equation 13 should therefore be

$$n_r = \frac{15^{2/3}}{15^{\frac{1}{2}}}$$

$$= 1.6$$

In order to obtain suitable artificial roughness elements for the model, natural river gravel was used (compare Herbich and Shulits 1964, also Mirajgaoker and Charlu, 1963). The gravel used was quite uniform, rounded at the corners and edges, with about 1 inch maximum horizontal diameter and an average height of 0.6 inches. The height of the gravel was hence comparable to model flow depths, the maximum depth of flow in the model being 0.3 ft. It was arranged in some patterns to give a Manning's roughness coefficient as required.

In order to obtain the Manning's n value of the artificial roughnesses, a flume 12 inches wide by 6 inches deep by 15 feet long was used. The bed of the flume was made of the same sort of painted plywood as was used in the model. The flume was set at a very mild slope and the depth of flow could be adjusted by a tailgate to give a normal depth for a given flow: that is, with a water surface slope the same as that of the bed of the flume. The Manning's n thus obtained for the painted plywood was 0.015 for a depth of 0.1 ft. As the surface of the board was smooth, it can be assumed that the Manning's n was substantially constant for greater depths of flow in the flume.

The river gravel was then laid on the surface of the painted

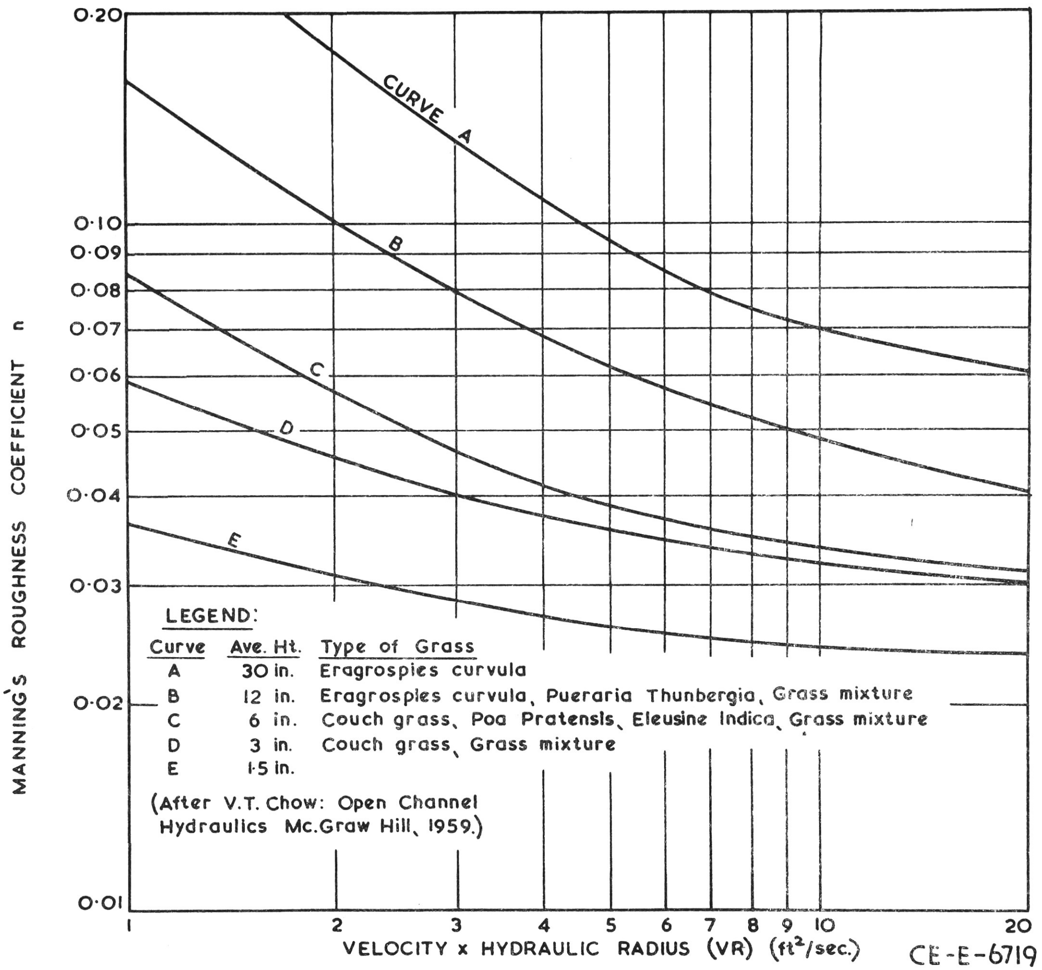

FIGURE 7: MANNING'S n FOR VARIOUS TYPES OF SURFACE ROUGHNESS

plywood and fixed in position with wax. Two patterms of gravel spacing were used, one 6 inches by 6 inches spacing and another 3 inches by 3 inches spacing, both staggered (See Fig. 8). As the size of the gravel was comparable with the depth of flow in the flume, standing waves were formed at the surface of the flow. The tailgate was adjusted to give a mean depth of flow (the mean of the crest and the bottom of the wave) approximately the same for a length of 5 feet in the middle of the flume. The Manning's n values obtained are shown in Table 1.

Table 1

Results of Flume Tests.

Roughness Spacing	V R	Manning's n
6" x 6"	0. 20 0. 15 0. 10	0. 027 0. 032 0. 036
3" x 3"	0. 20 0. 15 0. 10	0. 041 0. 043 0. 046

The prototype Manning's n values corresponding to these figures and corresponding VR values (prototype) are shown in Fig. 9 for the purpose of comparison. Comparing Fig. 7 to Fig. 9, it can be seen that the densely arranged river gravel (3 inches x 3 inches spacing) corresponds to grass of good to excellent stand, with an average height of 30 inches above the spillway surface (type A in Figure 7). The gravel spaced at 6 inches x 6 inches corresponds to the type B grass of good stand with an average height of 12 inches. Therefore, the results obtained in this investigation can be applied to a spillway with a surface roughness corresponding to Manning's n up to 0. 07 (30 inch high natural grass) if the prototype dimensions are approximately 15 times the model dimensions. Where prototype dimensions vastly different from 15 times the model dimensions are used, the design curve derived from the model can still be applied, the roughness scaling being computed from the equation

$$n_r = \frac{y_r^{2/3}}{x_r^{\frac{1}{2}}}$$

7. Spillway Design

Apart from considerations of the calculation of the discharge already described, other considerations such as the stability of the spillway surface under the erosive action of the water and the elevation of the spillway crest need to be taken into account. These are discussed in the following paragraphs.

7.1 Spillway Surface

Because farm dams are commonly constructed of earth, the circular spillway is required to discharge the surplus flow in such a manner that the earthen embankment of the dam proper will at no time be overtopped. The spillway is conveniently constructed at one end of the dam by excavating the natural surface down to the required level. The spillway surface is then composed of undisturbed natural soil which is erodible under swiftly flowing water. This soil may be seeded with grass or turfed to provide protection against erosion. Even if the surface of the spillway is grassed, the velocity of the flowing water over the crest should not be excessive. It should not in any case be higher than a prescribed velocity for the combination of soil and grass cover used.

The United States Soil Conservation Service has conducted experiments to determine the permissible velocities for different types of grass in channels with grassed surfaces. The results obtained by the United States Soil Conservation Service for channels with slope ranges from 0 to 5 pc. are shown in Table 2. Experimental extension of this work using selected Australian grasses is in progress at the Water Research Laboratory.

Table 2.
Permissible Velocities for Channels lined with Grass

Cover		Permissible Velocity (ft/sec)	
Name in U.S.A.	Corresponding Australian Grass	Erosion-Resistant Soil	Easily Eroded Soil
Bermuda Grass	Couch Grass	8	6
Kentucky Blue-grass	Poa Pratensis	7	5
Grass Mixture	Grass Mixture	5	4
Weeping Love-grass Kudzu Crabgrass	Eragrospies Curvula Pueraria Thunbergia Elusine Indica	3.5	2.5

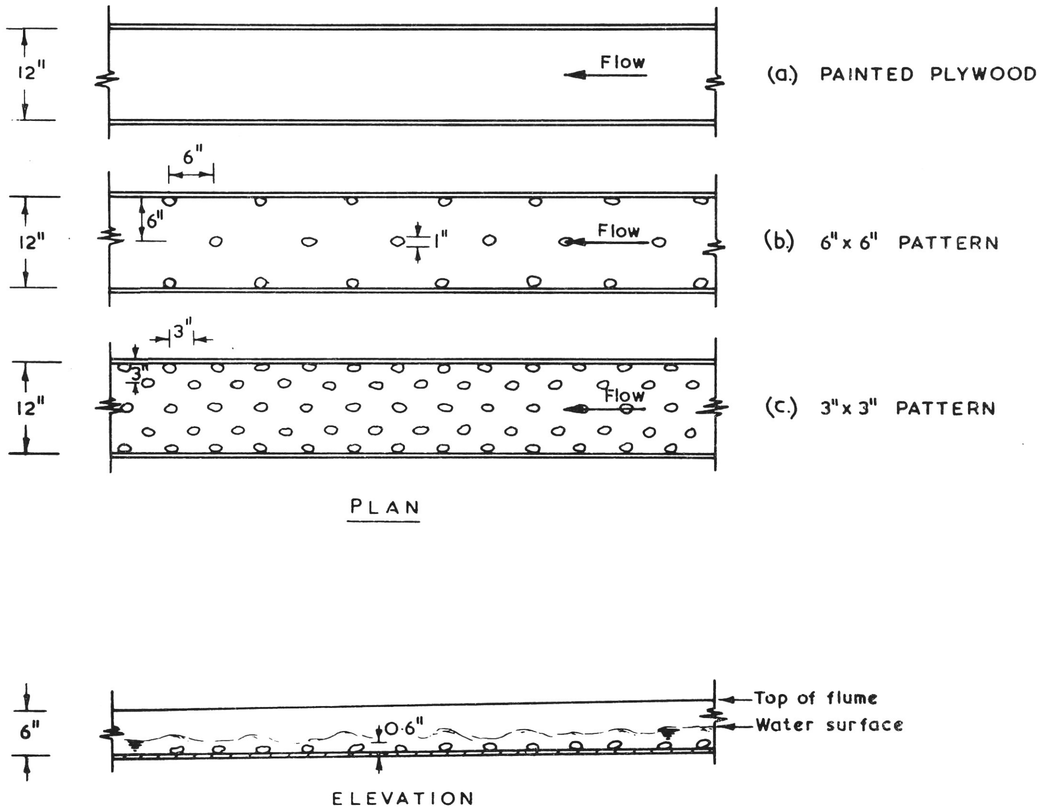

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FIGURE 8: ARRANGEMENT OF ARTIFICIAL ROUGHNESS

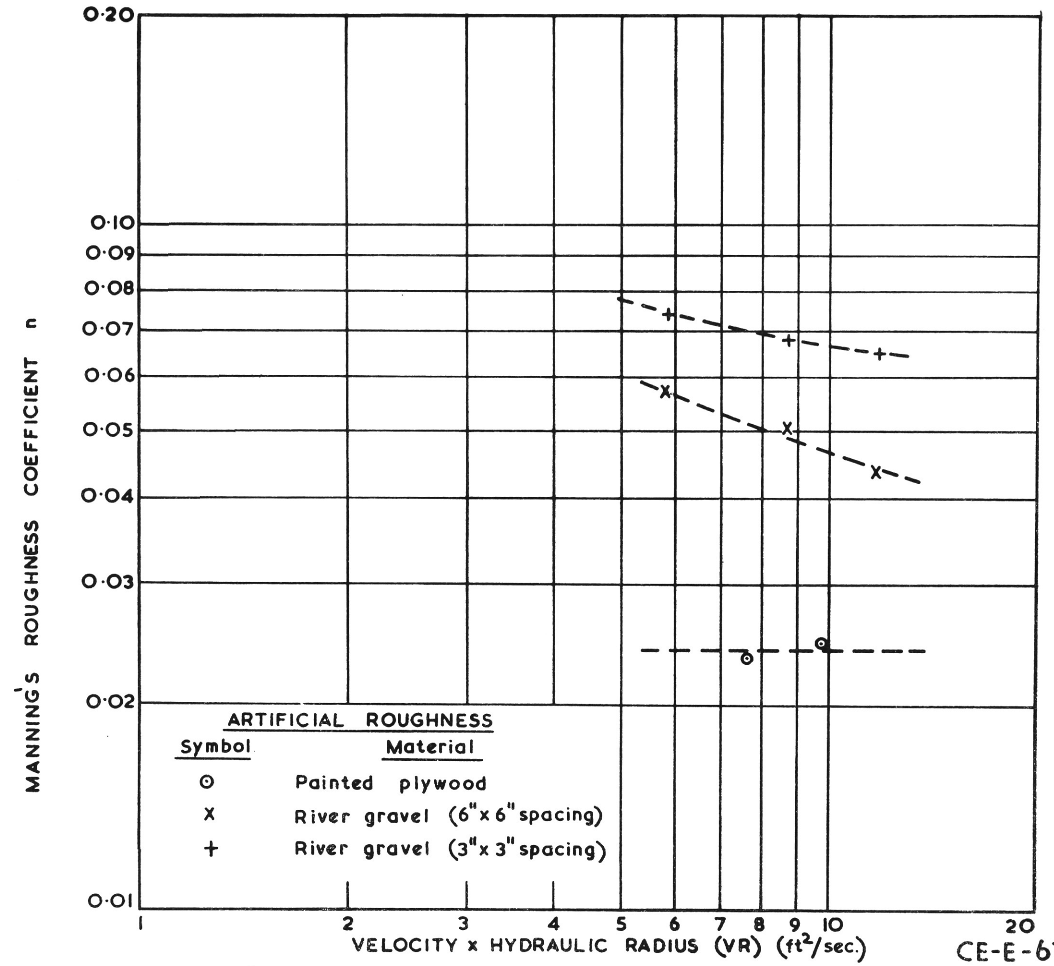

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FIGURE 9: MANNING'S n FOR ARTIFICIAL SURFACE ROUGHNESS (Prototype values)

The crest of the spillway for a farm dam is in most cases nearly horizontal and the permissible velocities listed in Table 2 may be adopted for design purposes. However, on the crest of the spillway, the flow changes from subcritical to supercritical and due to the curvature of the crest, the flow will not be uniform, and careful consideration must be given to the velocity distribution of the flow. The velocity distribution has been described in Section 4.32 of this report. Near the inside edge of the spillway at the downstream end of the crest, velocities are higher, and they could be as high as 1.6 times the critical velocity. Therefore, when considering the design of a spillway, either the high velocity is taken as the design criteria or it is necessary to strengthen portion of the crest surface where erosion under high velocity is likely to occur.

For a straight, wide, open channel with rectangular cross section and uniform flow, the critical velocity is proportional to the square root of the critical depth, that is

$$v_c = \sqrt{gH_c}$$

where v_c = critical velocity

H_c = critical depth

If the cross-section is trapezoidal with side slopes 3 to 1 the critical velocity, from Equation (3) and Appendix 1 can be expressed as

$$v_c = \sqrt{\frac{g\,A_c}{T_c}}$$

where $A_c = (B_o + 3H_c)\,H_c$, the cross sectional area at the critical section

$T_c = B_o + 6H_c$, the surface width of flow at the critical section.

For a design discharge Q with a given value of v_c, the required value of the base width of the spillway, B_o, can be obtained by solving Equation 5 and Equation 7 of Appendix 1 simultaneously.

Figure 16 shows the base width required, B_o, plotted against the critical velocity for a range of discharges. Therefore, if the crest is designed for a given velocity ratio and the spillway crest is strengthened in areas where the velocity ratio is higher than the design value chosen, the permissible velocity as shown in Table 1 may be used with the required factor applied to obtain the critical velocity. This leads to the determination of the base width B_o.

7.2 Height of Spillway Crest

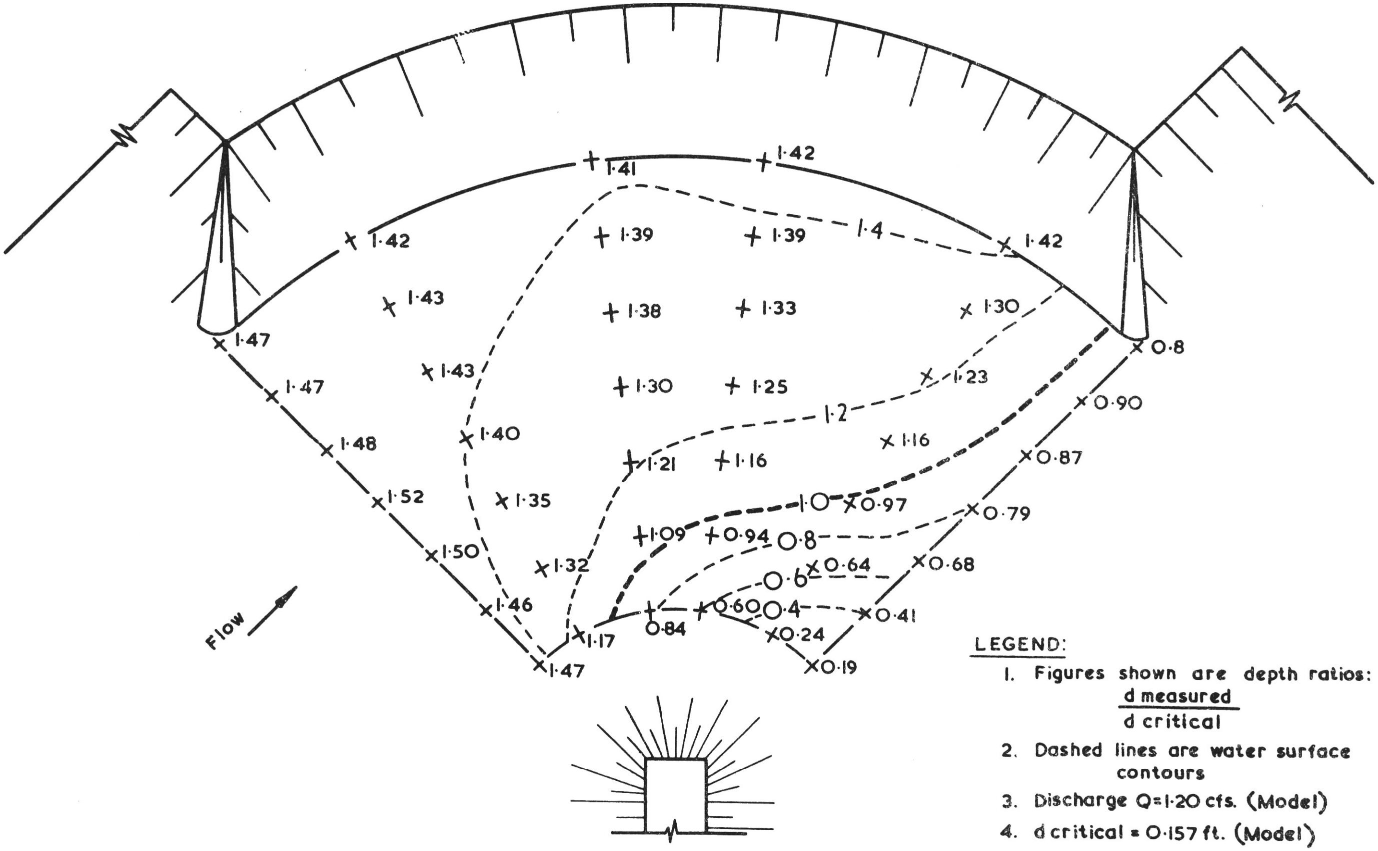
For a given design flow, the width of the spillway is governed by the permissible velocity at the spillway surface. The difference in level of the top of the dam and the crest of the spillway depends on the water surface level in the dam for a given flow. From equation (9), Appendix I, the elevation of the reservoir water above the upstream edge of the crest of the spillway can be computed if the discharge coefficient is properly chosen.

In this investigation, it has been shown that the discharge coefficient depends on the length of the spillway and the surface roughness, as well as the rate of flow over the spillway.

The design curve in Figure 15 gives the relationship of $\frac{H_o}{B_o}$ and $\frac{Q}{g^{\frac{1}{2}} B_o^{5/2}}$ as

$$\frac{Q}{g^{\frac{1}{2}} B_o^{5/2}} = C_d f\left(\frac{H_o}{B_o}\right)$$

for a spillway that forms on its outer edge an arc up to a semi-circle in plan and for surface roughness measured by Manning's n having a value up to 0.07. For a shorter spillway, or for Manning's n value less than 0.07, the discharge coefficient for a given flow will be higher and the $\frac{H_o}{B_o}$ value obtained from the design curve will be conservative. Because of difficulties in the accurate estimation of such variables as flow and surface roughness, it is not reasonable to refine the design by providing different curves for the different shapes and roughnesses. The curve given can be seen to cover all conditions without undue waste by overdesign. The difference in level of the top of the dam and the crest of the spillway should therefore be the H_o value obtained from the design curve plus the freeboard determined by such considerations as allowance for wave height and a general margin to prevent overtopping of the dam from contingent causes. The maximum wave height that could be generated on a farm dam is limited by the size of the dam, a figure of 1 foot being a reasonable design figure for dams up to 1 mile square.



LEGEND:

1. Figures shown are depth ratios: $\frac{\text{d measured}}{\text{d critical}}$
2. Dashed lines are water surface contours
3. Discharge Q=1·20 cfs. (Model)
4. d critical = 0·157 ft. (Model)

CE-E-6722

FIGURE 10: DEPTH OF FLOW ON CREST OF 3 FT. BASE WIDTH MODEL SPILLWAY

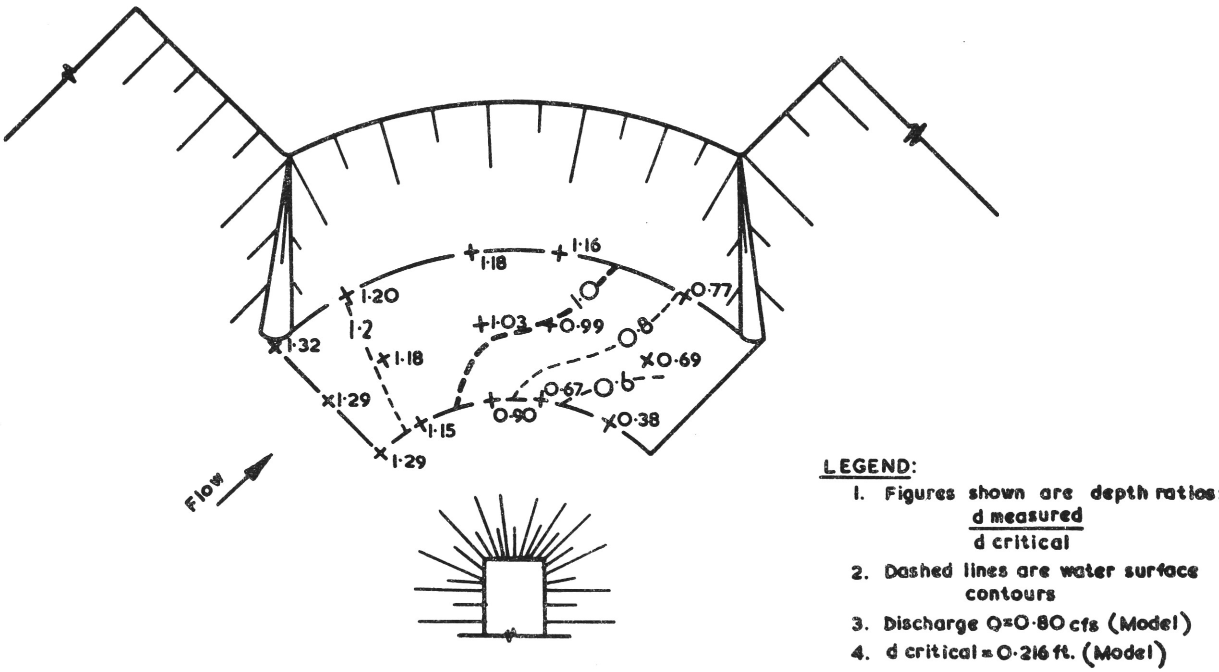
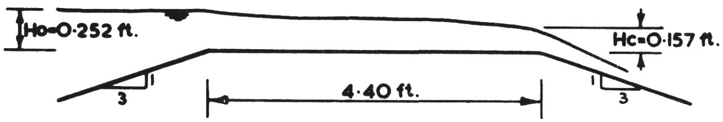
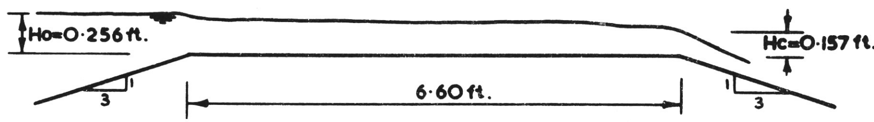


FIGURE II: DEPTH OF FLOW ON CREST OF 1 FT. BASE WIDTH MODEL SPILLWAY

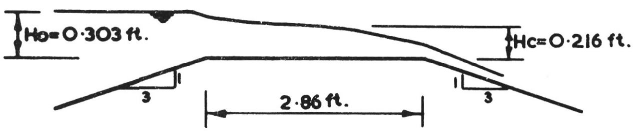
CE-E-6723



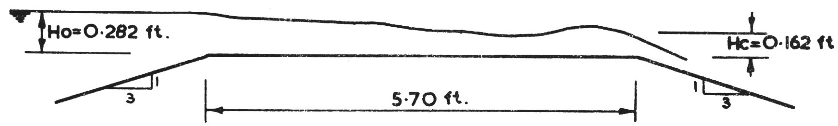
(a.) 3 FT. BASE WIDTH, SMOOTH SURFACE
Q=1·20 cfs., $\alpha=90^{\circ}$
(Model)



(c.) 3 FT. BASE WIDTH SMOOTH SURFACE
Q=1·20 cfs $\alpha=135^{\circ}$
(Model)



(b.) 1 FT. BASE WIDTH, SMOOTH SURFACE
Q=0·80 cfs. $\alpha=90^{\circ}$
(Model)



(d.) 1 FT. BASE WIDTH, SMOOTH SURFACE
Q=0·65 cfs. $\alpha=180^{\circ}$
(Model)

FIGURE 12: LONGITUDINAL FLOW SURFACE PROFILE ALONG CENTRE LINE OF MODEL SPILLWAY CREST

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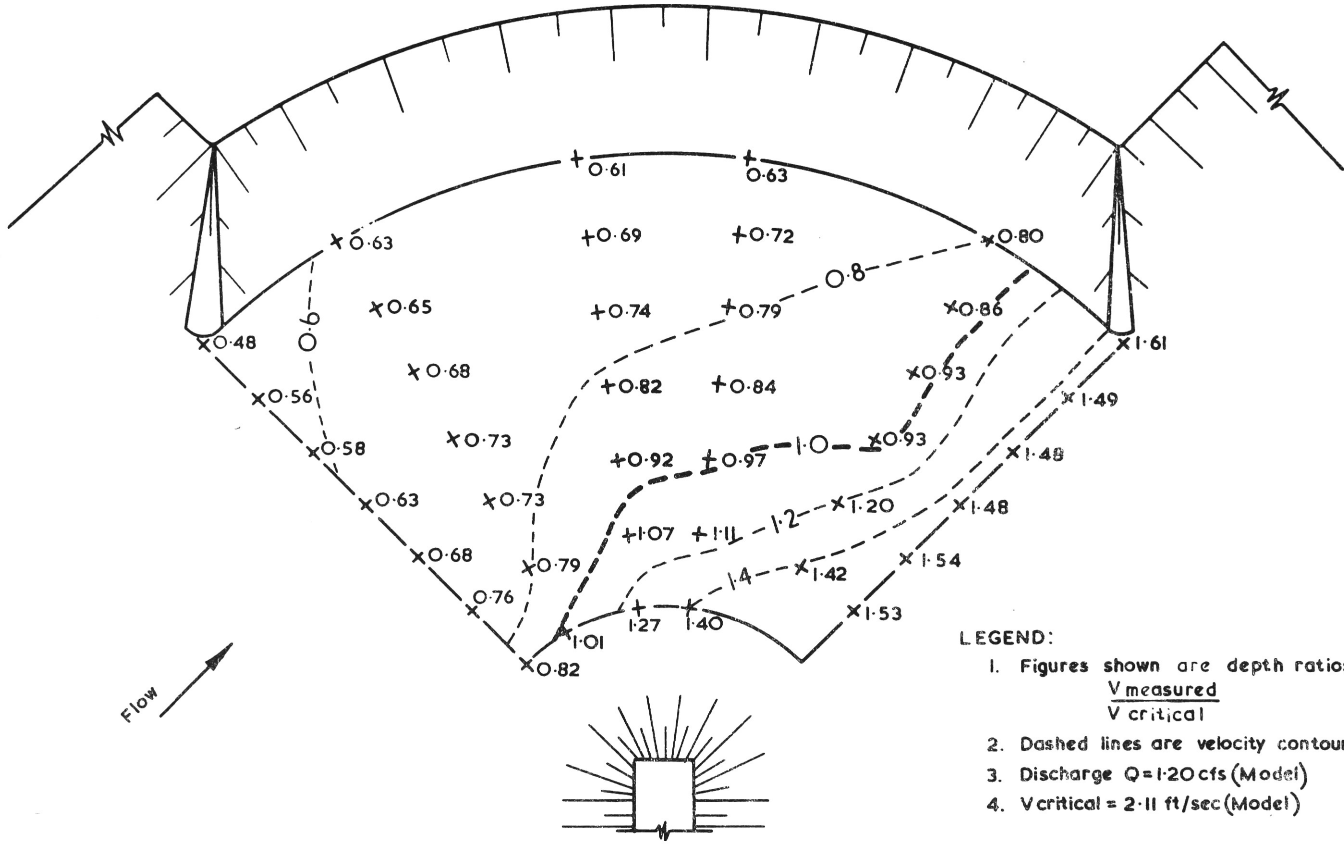
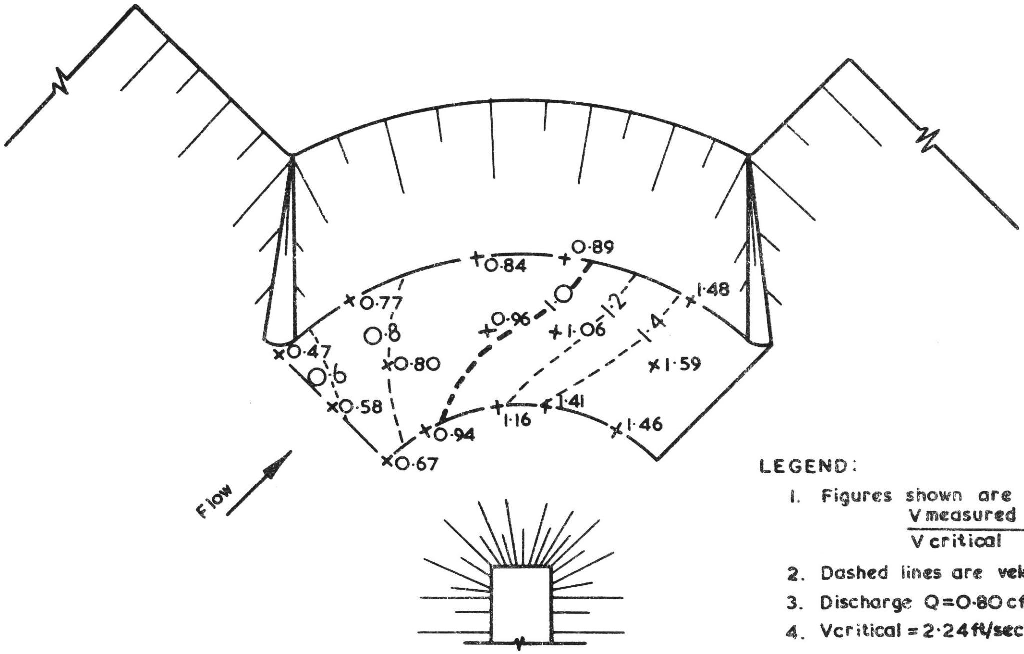


LEGEND:

1. Figures shown are depth ratios: $\frac{V\ measured}{V\ critical}$
2. Dashed lines are velocity contours
3. Discharge Q = 1·20 cfs (Model)
4. V critical = 2·11 ft/sec (Model)

CE-E-6725

FIGURE 13: VELOCITY DISTRIBUTION ON CREST OF 3 FT. BASE WIDTH MODEL SPILLWAY



LEGEND:

1. Figures shown are depth ratios: $\frac{V \text{measured}}{V \text{critical}}$
2. Dashed lines are velocity contours
3. Discharge Q=0.80 cfs (Model)
4. Vcritical = 2.24 ft/sec (Model)

FIGURE 14: VELOCITY DISTRIBUTION ON CREST OF 1 FT. BASE WIDTH MODEL SPILLWAY

CE-E-6726

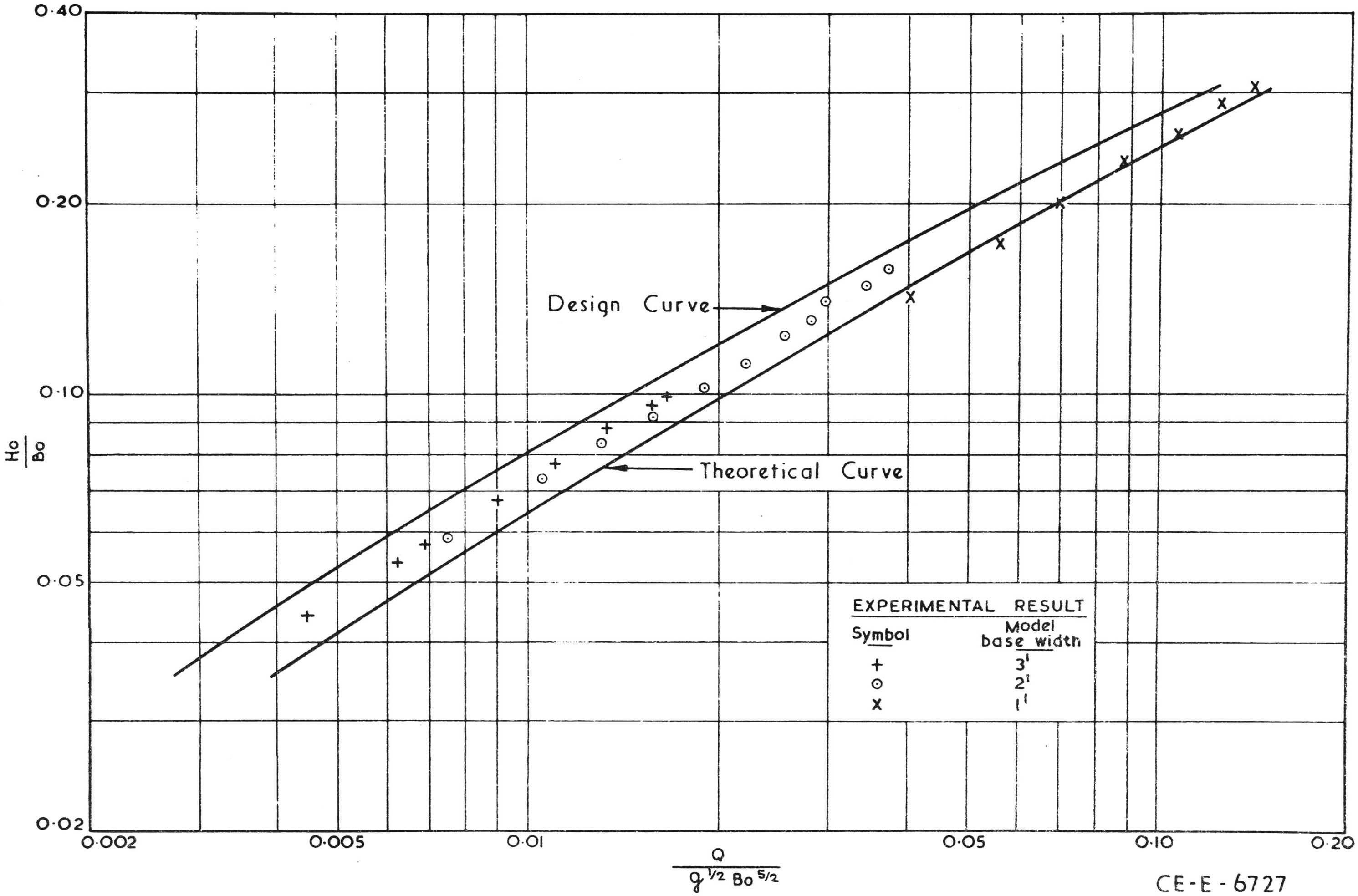


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FIGURE 15: HEAD-DISCHARGE RELATIONSHIP (a) 90° Included Angle; 3" x 3" Spacing River Gravel

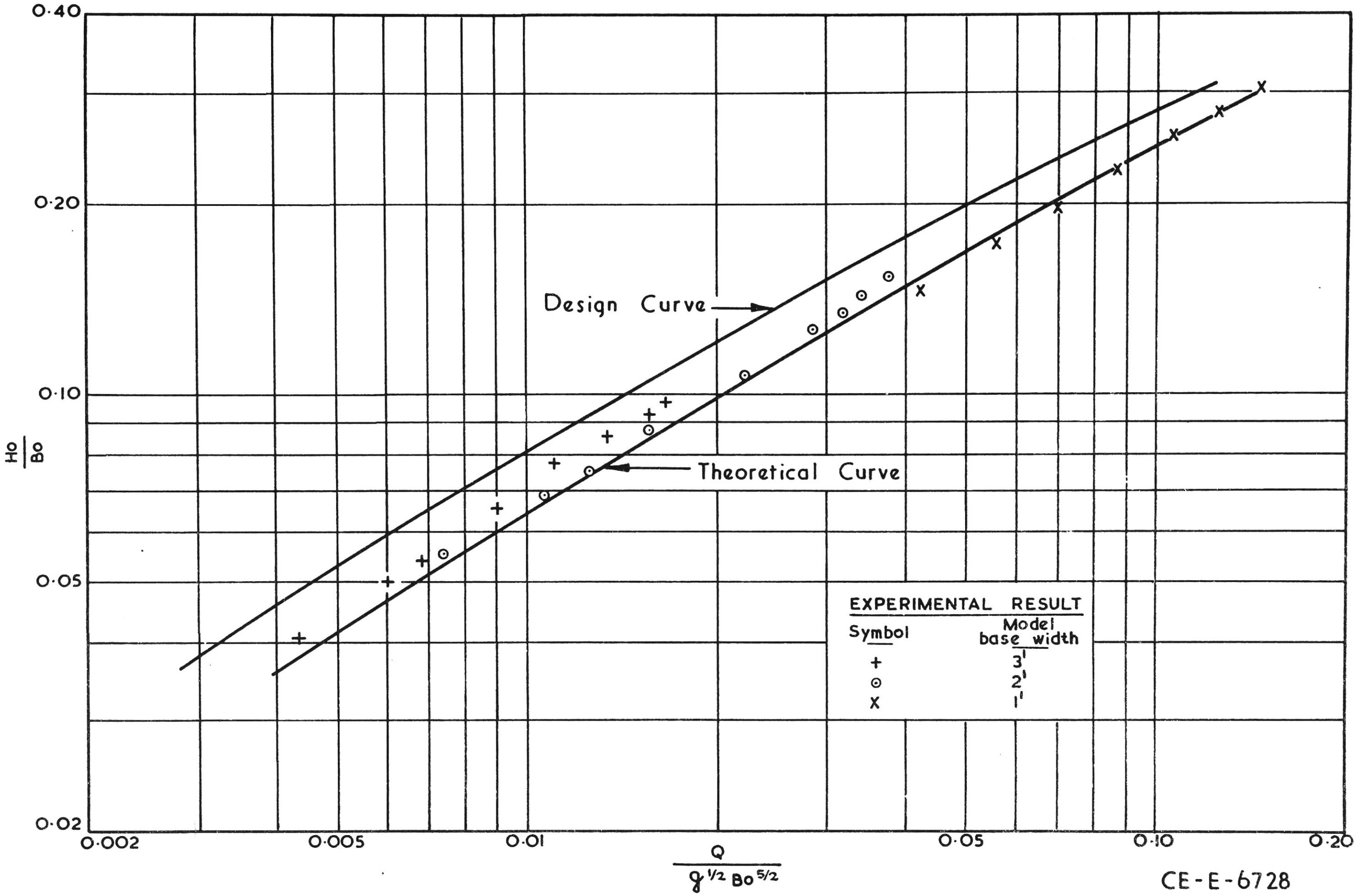

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FIGURE 15: HEAD-DISCHARGE RELATIONSHIP (b) 90° Included Angle; 6" x 6" Spacing River Gravel

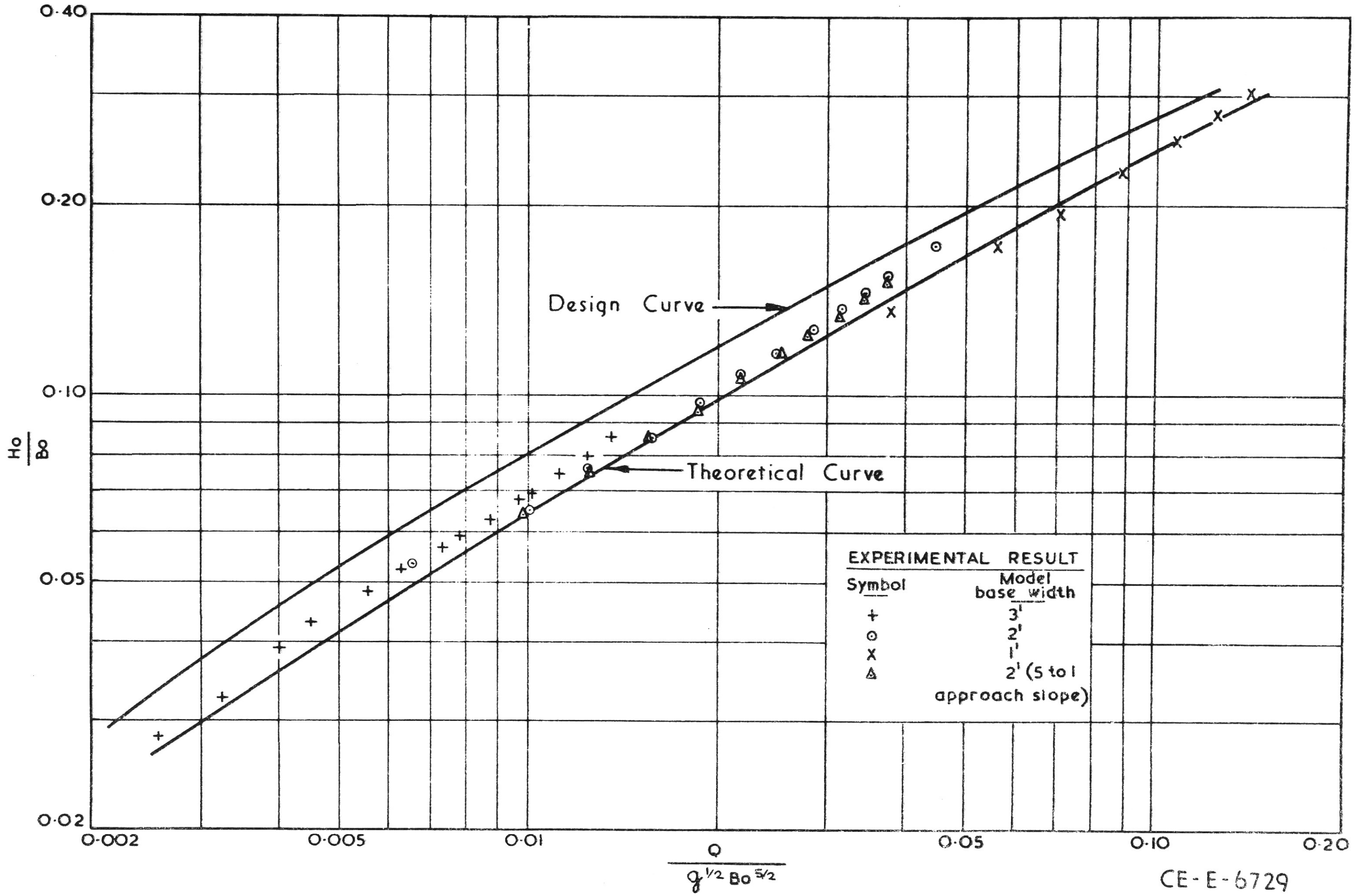

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FIGURE 15: HEAD-DISCHARGE RELATIONSHIP (c) 90° Included Angle; Painted Plywood Board

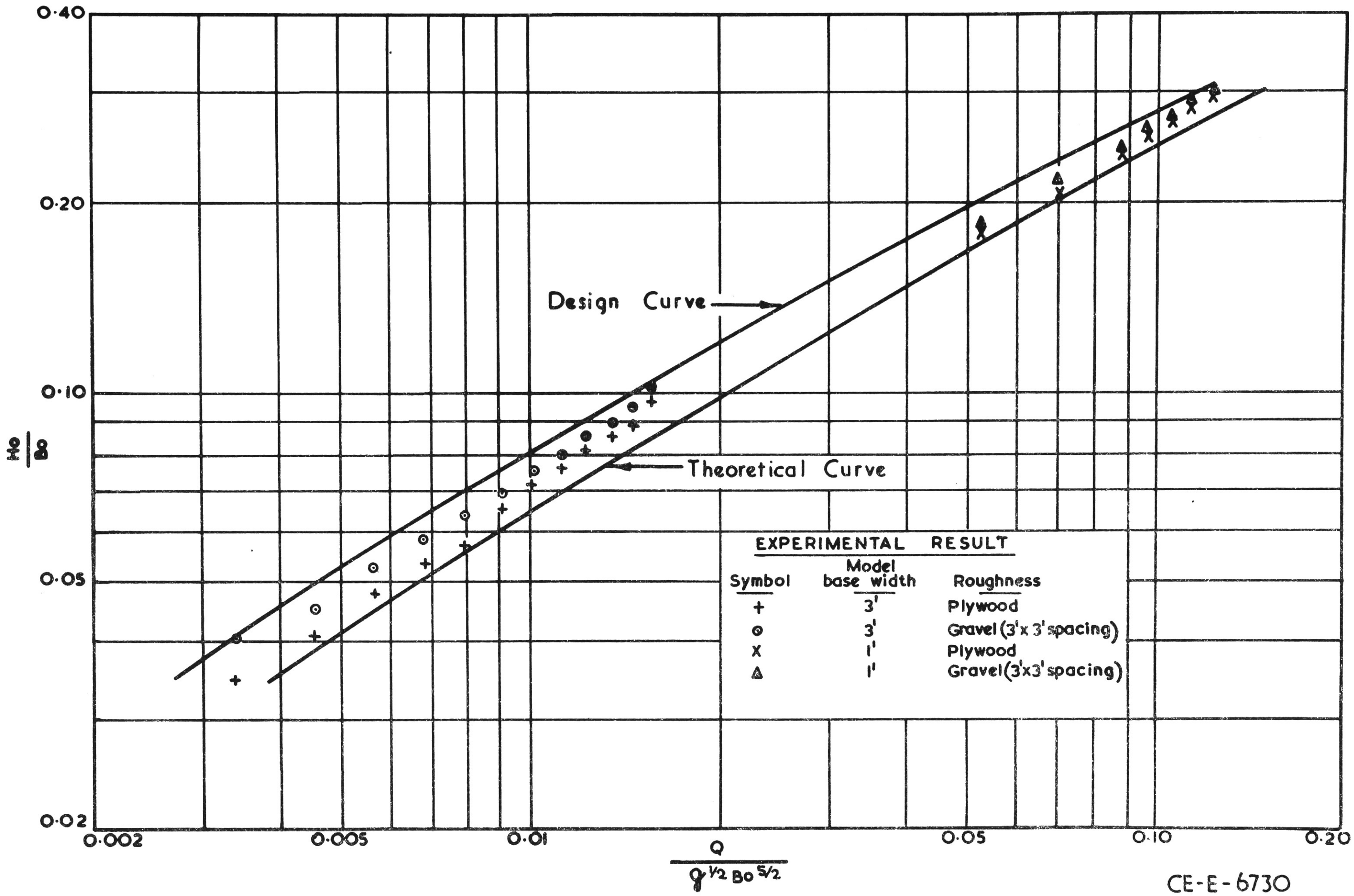

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FIGURE 15: HEAD-DISCHARGE RELATIONSHIP (d) 135° Included Angle

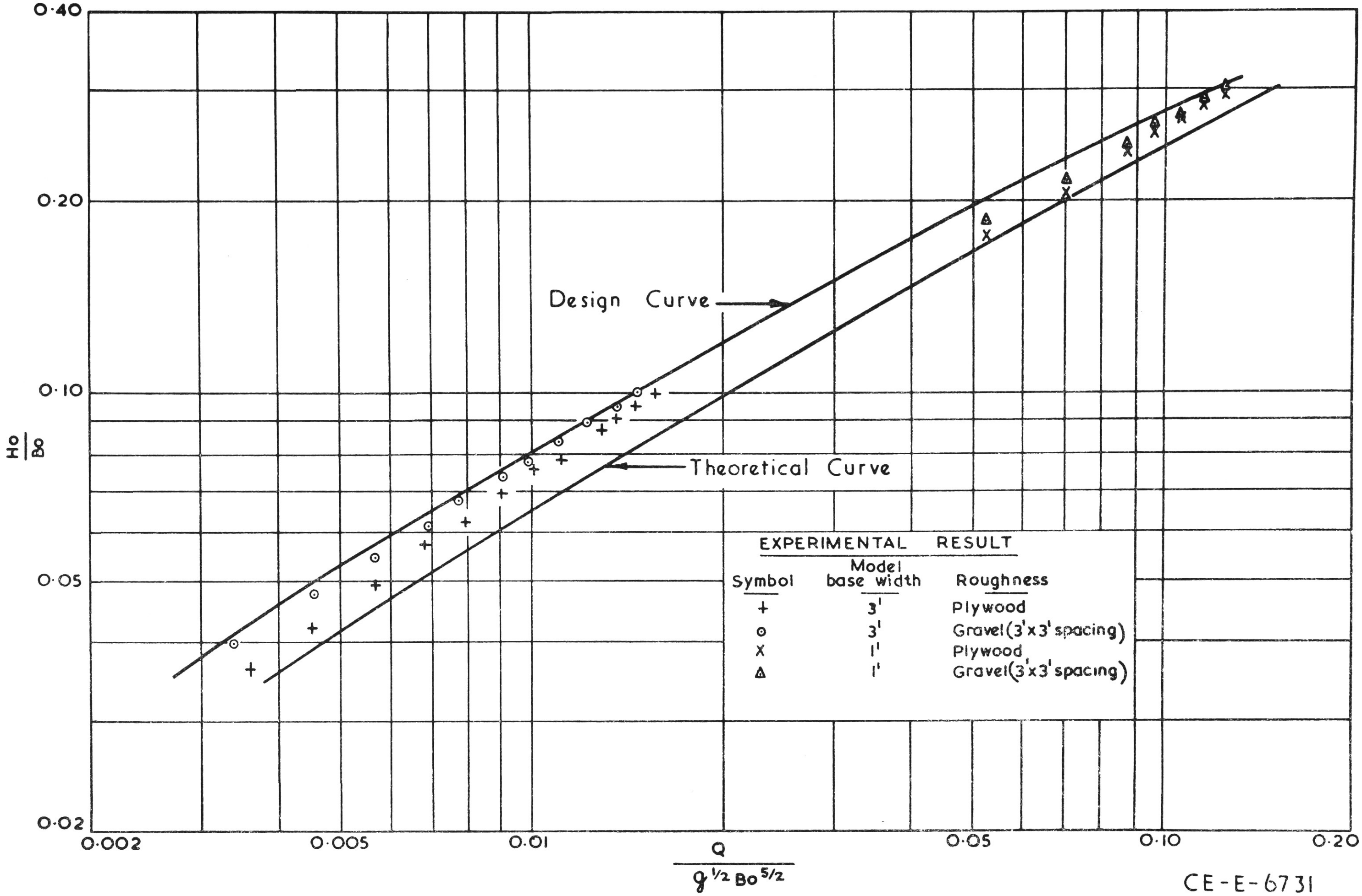

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FIGURE 15: HEAD-DISCHARGE RELATIONSHIP (e) 180° Included Angle

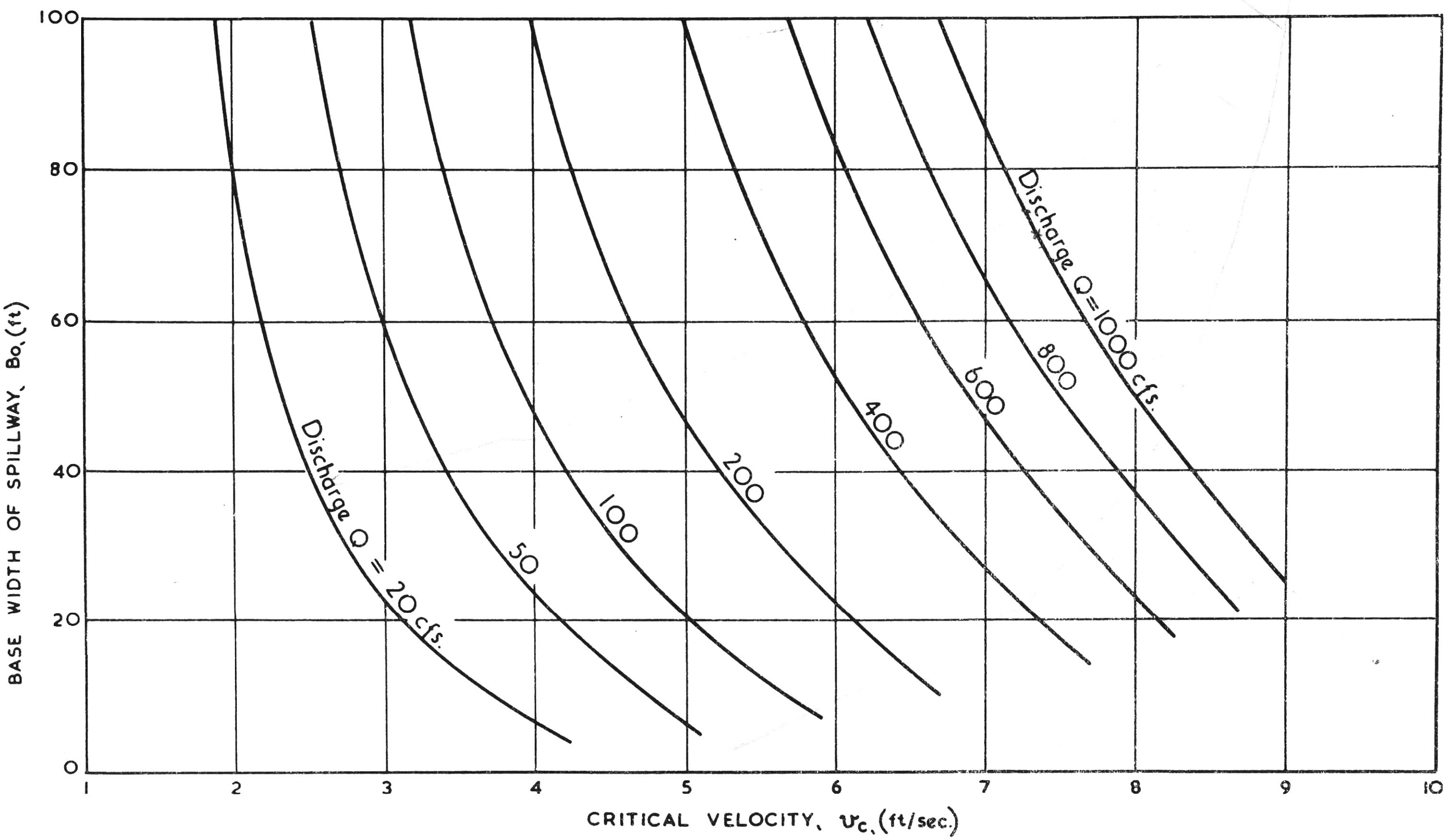

FIGURE 16: DIAGRAM FOR COMPUTATION OF BASE WIDTH WITH GIVEN DISCHARGE AND CRITICAL VELOCITY

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8. Conclusions

The general conclusions to be stated as a result of this investigation are as follows:-

8.1: For a spillway with a trapezoidal cross section and a horizontal crest the discharge coefficient depends on the selected slopes and the length and surface roughness of the crest. A design curve is given in Figure 15 for the determination of the crest level of the spillway. This can be applied to spillways which form an arc up to a semicircle in plan with grass of an excellent stand and an average height of up to 30 inches above the spillway crest. For spillways with less than a semi-circular arc or grass lower than 30 inches, the use of this curve will yield conservative but not unduly wasteful design.

8.2: Because of the comparatively short length of the spillway and its curvature in plan the flow over the crest will not be uniform. The flow has a greater depth on the outside of the spillway. At the inside near the downstream edge of the crest, the surface of the flow has a sharp gradient with high velocity. Local scouring of the crest of the spillway and the end of the dam is likely to occur. Unless this local scouring is harmless to the whole structure, either provision must be made to combat it by strengthening this part of the crest, or the higher velocity there must be used as the design criterion.

It is also feasible to have the crest sloping downwards slightly towards the outside of the spillway, thus tending to divert more flow towards the outside of the spillway and so reduce velocities and local scour on the inside downstream part of the spillway crest.

8.3: The level of the crest is determined from the discharge coefficient, while the selection of the base width of the spillway is governed by the velocity of the flow over the crest. The velocity of the flow at various points on the crest of the spillway can be expressed in the terms of a velocity ratio, the ratio of the velocity at a point to the critical velocity calculated by assuming a straight channel. The velocity ratio can be as high as 1.6. Permissible velocities for various types of soil and grass cover are given and these may be used with the appropriate velocity ratio for the type of design envisaged to obtain an allowable figure for the critical velocity and so determine the width of the spillway.

9. References

(1) Allen, Prof. J and Sek Por Chee "The Resistance to the Flow of Water Around a Smooth Circular Bend in an Open Channel" - Proc. I. C. E., Nov. 1962.

(2) Chow, V. T. "Open Channel Hydraulics" McGraw Hill 1959.

(3) Herbich, J. B. and Schulits, S. "Large-Scale Roughness in Open Channel Flow" Proc. A. S. C. E. Nov. 1964.

(4) King, H. W. and Brater, E. F. "Handbook of Hydraulics" 5th Ed. McGraw Hill, 1963.

(5) Mirajgaoker, A. G. and Charlu, K. L. N. "Natural Roughness Effects in Rigid Open Channels" Proc. A. S. C. E. Sept. 1963.

(6) Rouse, Hunter "Engineering Hydraulics" John Wiley and Sons, 1950.

Appendix 1.

Derivation of Head Discharge Relation

For a straight spillway with a rectangular cross-section, assuming ideal fluid flow, the discharge can be expressed as

$$Q = 3.09\ BH_o^{3/2} \qquad (1)$$

where Q = discharge (cfs)

B = width of spillway (ft.)

H_o = difference between water surface elevation and spillway crest elevation, measured at a point far enough upstream from the spillway for the velocity to be negligible (ft.)

When the depth of flow above the crest is large compared with the length of the spillway, the spillway operation becomes akin to that of a sharp-crested weir, the theoretical discharge being given by:-

$$Q = 5.35\ BH_o^{3/2} \qquad (2)$$

For small farm dams, the spillway is usually formed by excavating the natural surface of a hill to the required level. The spillway is hence formed of erodible material usually with a grassed surface for protection and the slopes on both sides of the spillway should be such that stability can be maintained. For ordinary undisturbed natural soils, it is quite safe to assume a stable slope at 3 to 1 (horizontal to vertical). Therefore, for this investigation, the model spillway had a trapezoidal cross-section with cross slopes of 3 to 1. The theoretical discharge can be derived as follows:-

B_o = the base width of the spillway (ft.)

H_c = critical depth of flow over the spillway (ft.)

T_c = width of the flow surface at the critical section (ft.)

A_c = cross-sectional area of flow at critical section (ft.^{2})

Q = discharge (cfs)

g = acceleration due to gravity (ft/sec^2)

The discharge is given by

$$\frac{Q^2 T_c}{g A_c^3} = 1 \tag{3}$$

Since the side slopes are 3 to 1, $T_c = B_o + 6H_c$ and $A_c = (B_o + 3H_c)H_c$ Equation (3) becomes

$$\frac{Q^2 (B_o + 6H_c)}{g \left[(B_o + 3H_c)\ H_c \right]^3} = 1$$

$$\text{or} \quad \frac{Q^2 \left(1 + 6 \frac{H_c}{B_o} \right)}{g\, B_o^5 \left[\left(1 + 3 \frac{H_c}{B_o}\right) \frac{H_c}{B_o} \right]^3} = 1 \tag{4}$$

$$\text{or} \quad \frac{Q}{g^{\frac{1}{2}}\, B_o^{5/2}} = \frac{\left[\left(1 + 3 \frac{H_c}{B_o} \right) \frac{H_c}{B_o} \right]^{3/2}}{\left(1 + 6 \frac{H_c}{B_o}\right)^{\frac{1}{2}}} \tag{5}$$

Assuming a tranquil flow with negligible approach velocity at the point where H_o is measured

$$H_o = H_c + \frac{V_c^2}{2g} \tag{6}$$

where $V_c = \dfrac{Q}{A_c}$, the critical velocity.

Equation (6) can be written as

$$H_o = H_c + \frac{Q^2}{2gA_c^2}$$

$$= H_c + \frac{Q^2}{2g\left[(B_o + 3H_c)\, H_c \right]^2}$$

$$\text{or} \quad \frac{H_o}{B_o} = \frac{H_c}{B_o} + \frac{Q^2}{2gB_o^5 \left[\left(1 + 3 \frac{H_c}{B_o}\right) \frac{H_c}{B_o} \right]^2} \tag{7}$$

Obviously, when H_o and B_o are given, the theoretical discharge Q over the spillway can be obtained by solving Equations (5) and (7) simultaneously. A plot of the dimensionless discharge parameter $\frac{Q}{g^{\frac{1}{2}} B_o^{5/2}}$ against the dimensionless depth parameter $\frac{H_o}{B_o}$ is shown in Figure 1~~5~~6. From equations (5) and (7) it can be seen that the theoretical discharge parameter can be written as

$$\frac{Q}{g^{\frac{1}{2}} B_o^{5/2}} = f\left(\frac{H_o}{B_o}\right) \tag{8}$$

For real fluids, the viscosity and the geometry of the spillway have effects on the head-discharge relationship. These effects can be included in a discharge coefficient, C_d, and Equation (8) can be written as

$$\frac{Q}{g^{\frac{1}{2}} B_o^{5/2}} = \phi\left(\frac{H_o}{B_o}\right) \tag{9}$$

or $$Q = C_d \, f\left(\frac{H_o}{B_o}\right) g^{\frac{1}{2}} B_o^{5/2}$$

Appendix 2.

Skin Friction Effects.

For a spillway with a rectangular cross section, a boundary layer develops along the walls and bottom of the spillway. Both B and H_o in Equation (1) of Appendix 1 must be corrected for the displacement thickness δ_* of the boundary layer and the equation becomes

$$Q = 3.09 \, (B - 2\,\delta_*) \, (H_o - \delta_*)^{3/2}$$

or

$$Q = 3.09 \left(1 - 2\,\frac{\delta_*}{B}\right) \left(1 - \frac{\delta_*}{H_o}\right)^{3/2} B \, H_o^{3/2}$$

and the discharge coefficient expressed as a ratio of actual to theoretical discharge for a given head is given by:-

$$C_d = \left(1 - 2\,\frac{\delta_*}{B_o}\right) \left(1 - \frac{\delta_*}{H_o}\right)^{3/2}$$

The discharge coefficient can be calculated from the equations for δ_* either for laminar boundary layers or turbulent boundary layers.

Appendix 3.

Model Details.

For small farm dams, the width of the spillway usually ranges from 20-50 ft. The scale of the model was chosen to be about 1 to 15. The general layout of the model is shown in Fig. 5, and Fig. 6 shows a photograph of the model spillway.

The base width of a spillway, B_O, was 3 ft. in the first instance. It could be altered to 2 ft. or 1 ft. by simply arranging a new wall closer to the dam, to study the effect of different base width and various depth/width ratios. The height of the side walls was 4 inches, and hence a maximum depth/width ratio of 0.33 could be obtained.

The slopes of the dam itself, the slope of the apron at the downstream end of the spillway and the side slopes on the crest of the spillway were all 3 to 1 (horizontal to vertical). The approach slope upstream of the spillway was also 3 to 1, and an approach slope of 5 to 1 was also tested. Tests revealed that the 5 to 1 approach slope did not have a significant difference from the 3 to 1 approach slope on the characteristics of the flow and the latter was therefore used for all subsequent tests.

The crest of the spillway was made of smooth painted plywood. It was artificially roughened with natural river gravel to give a surface roughness corresponding to that of a grassed surface as in the prototype.

Water was supplied to the model by gravity through a 6 in. main with an orifice meter installed for flow measurement. A hook gauge was installed in a position far away from the crest to measure the water level above the spillway crest for a given flow, in order to obtain the discharge coefficient.

Appendix 4.

Examples.

Example 1: Design a spillway for a flow Q = 200 cfs. The included angle between the upstream and downstream edges of the crest is approximately 90°. The spillway is formed of erosion-resistant soil and on top of it is couch grass of excellent stand with an average height of 6 inches. The top of the dam is at RL. 105, and the freeboard is 1 ft.

Solution: From Table 2 the permissible velocity for erosion-resistant soil lined with couch grass is 8 ft/ sec. If a velocity ratio of 1.5 is used, then the allowable critical velocity

$$v_c = \frac{\text{Permissible Velocity}}{\text{Velocity Factor}}$$

$$= \frac{8}{1.5}$$

$$= 5.33 \text{ ft/sec.}$$

From Figure 16, $B_o = 37$ ft.

$$\frac{Q}{g^{\frac{1}{2}} B_o^{5/2}} = \frac{200}{32.2^{\frac{1}{2}} \times 37^{5/2}} = 0.0042$$

From Figure 15, $\frac{H_o}{B_o} = 0.047$

$$H_o = 0.047 \times 37$$

$$= 1.74 \text{ ft.}$$

$$\text{Use } H_o = 1.75 \text{ ft.}$$

$$H_o + \text{Free Board} = 1.75 + 1$$

$$= 2.75 \text{ ft.}$$

$$\therefore \text{Crest Level} = 105 - 2.75$$

$$= 102.25$$

Example 2: If, in the above example, the crest near the downstream inner edge is covered by large, closely-packed stones so that a velocity ratio of 1. 2 can be used (see Fig. 13), design the spillway.

Solution: Allowable critical velocity

$$v_c = \frac{8.0}{1.2}$$

$$= 6.66 \text{ ft/sec.}$$

From Figure 1~~5~~6, $B_o = 12$ ft.

$$\frac{Q}{g^{\frac{1}{2}} B_o^{5/2}} = \frac{200}{32.2^{\frac{1}{2}} \times 12^{5/2}}$$

$$= 0.071$$

From Figure 1~~4~~5, $\frac{H_o}{B_o} = 0.235$

or $H_o = 0.235 \times 12$

$= 2.94$ ft.

Use $H_o = 3.00$ ft.

H_o + Free-Board $= 3.00 + 1.00$

$= 4.00$

$\therefore$ Crest level $= 105 - 4.00$

$= 101$