DISCHARGE CHARACTERISTICS OF VERTICAL DROP FALLS

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by

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SYNOPSIS

A comprehensive and rational solution is proposed for the discharge characteristics of vertical drop falls. A vertical drop fall being essentially a broad-crested weir with vertical downstream face and horizontal crest, the previous studies on broad-crested weir discharge have been briefly reviewed.

The proposed solution of the discharge characteristics is based on a simple equation of discharge obtained by theoretical considerations and dimensional analysis. Thus, the discharge coefficient of the vertical drop falls is defined as a function of dimensionless ratios that describe the geometry of the fall, the approach channel and the relative influence of the factors that determine the flow pattern. The effect of these various dimensionless ratios on the discharge coefficient is determined from original experiments covering a wide range of the significant ratios. On the basis of the form of water-surface profiles over the experimental models, the broad-crested weirs forming the vertical drop falls have been classified as short, normal and long.

This study includes the discharge characteristics of vertical drop falls both in the free flow and submerged flow conditions.

1. INTRODUCTION

1. INTRODUCTION

1.1 General.-

Galileo remarked - " I had less difficulty in the discovery of the motion of heavenly bodies, in spite of of their astonishing distances, than in the investigations of the movement of flowing water before our very eyes." Indeed, viewing perspectively the history of the development of hydraulics, the statement appears to be truer than ever, today. For centuries, investigators made attempts to measure and predict the mechanics of flowing water precisely. Some of the investigations, perhaps, have been successful; but only to the extent of nearing precision and not attaining it to the full degree, in spite of the great advancements made in the field of instrumentation, in recent times. Thus, there exists, today, not a single method of measuring, accurately and satisfactorily, the flow of water in its great variety of forms - in open channels in irrigation and sewage systems, pipe lines or conduits, rivers and oceans.

However, in spite of the highly complex nature of the factors involved in the mechanics of flowing water, attempts were made and will have to be made to gauge the flow to the highest degree of accuracy possible. For, a proper knowledge of the rate of flow of water is a basic requisite for the effective design of hydraulic structures and utilisation of water resources.

Accurate and convenient water measurement from the smallest field-canal to the largest river is an essential feature of any planned irrigation system. This factor assumes an increasing importance, as an irrigation project approaches full development, when the demand for water equals the designed maximum supply.

1.2 Methods of Flow Measurement in Open Channels.-

Irrigation engineers employed weirs of various types for the gauging of water flow in open channels for many years. The rectangular sharp-crested weir with Francis' formula for determination of discharge over it, has been quite popular. Cipoletti weir is, in some cases, employed obviating the necessity of making corrections for end-contractions. For accurate determination of low discharges, triangular or V-notch weir has been used. All of these, while giving discharge with a reasonable degree of accuracy, suffer from a main disadvantage for their employment in irrigation channels. That is, the nappes of water flowing over the weirs will have to be aerated or ventilated, if the head-discharge relationship is to remain unaffected. This, again means, a clear drop in water level across the weir, which is greater than the upstream head. In irrigation canals, it is often required to gauge the discharge with only a small drop of head across the weir. Under such a demand the above types of weirs fail to serve the purpose satisfactorily.

Attempts were made, therefore, to overcome this difficulty. For channels, "rating flume" , consisting of a short rectangular section of fixed dimensions built in the channel, was employed. In this, the stage-discharge relationship would be established with the help of current-meter. This has its limitations. Firstly, the backwater effects from down-stream would affect the stage-discharge relationship. Secondly, any sitting in the flume would change the area of the section and affect the discharge measurement.

"Venturi flume", with a constricted section, is some times used in the canals. At the throat of the flume, there will be a local drop in water level, in accordance with the Venturi principle. The application of continuity and energy equations to the known sections and observed water levels will yield the value of discharge. This method, though free from the backwater effect, suffers from the disadvantage of requiring two readings always.

In the "Parshall flume", that was developed subsequently, the sections were so designed as to cause the flow through critical depth, so that the discharge will be independent of depth at throat and therefore could be computed from the upstream observed reading above. Ofcourse, if the flume is submerged deeply, two readings will have to be taken in the Parshall flume also. The Parshall flume consists of a short rising floor to an inlet section which has a level floor and side walls converging at 5:1 ; a throat section with parallel side walls and floor dropping at 2.67 to 1; and an outlet section with rising floor and side walls diverging at 6 to 1.

It may be observed, however, that the geometric design of the Parshall flume is largely arbitrary. Further, even the discharge formula for the flume is also of a purely empirical nature. When the flume was first proposed (4) by Parshall, Lindley and Hinds criticised it, expressing a preference for a form which could be analysed rationally. Thus, it would seem, the popularity of the Parshall flume is not due to any intrinsic merits, but due to fact that it has been thoroughly rated and discharge tables are available

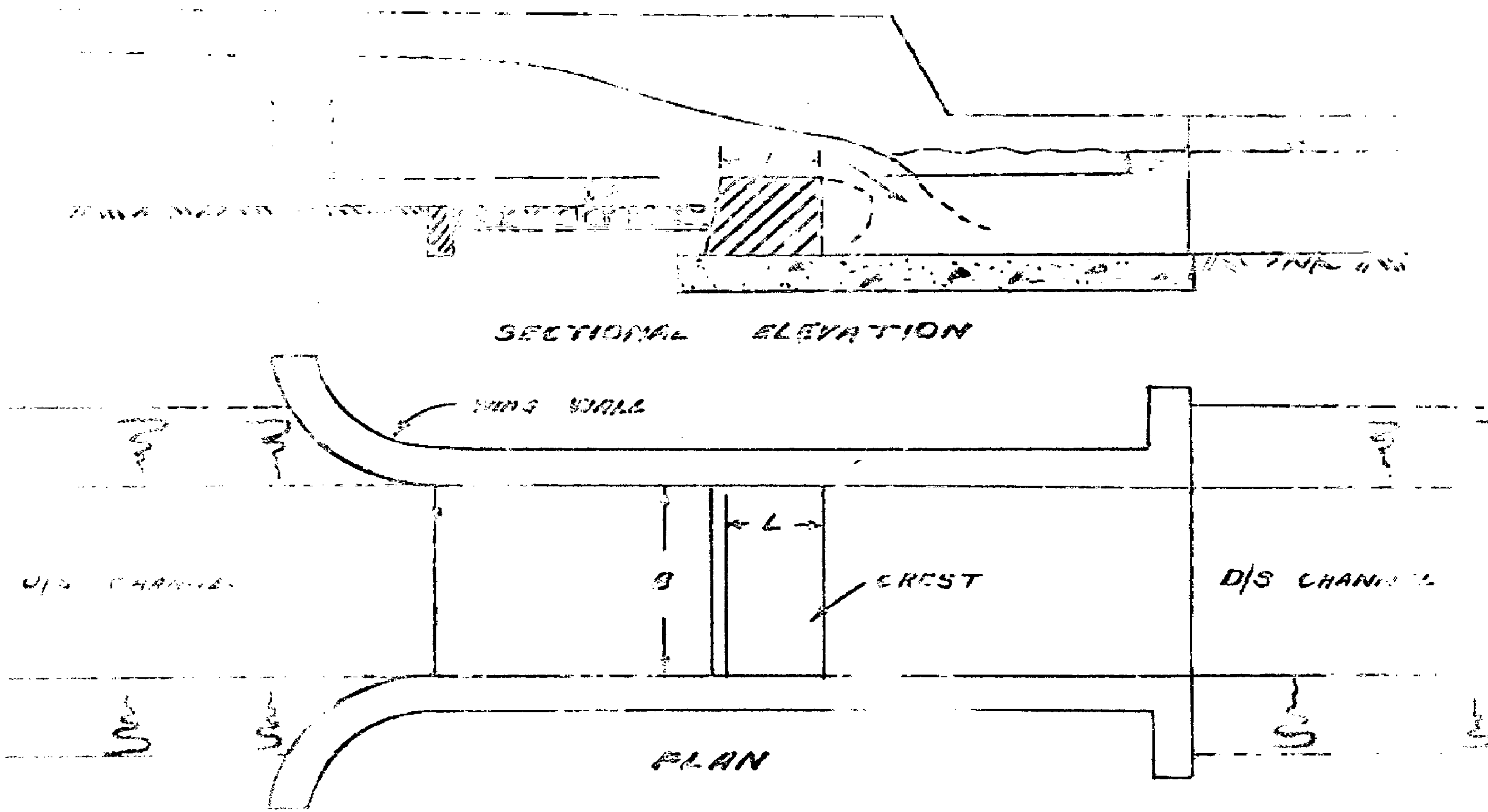


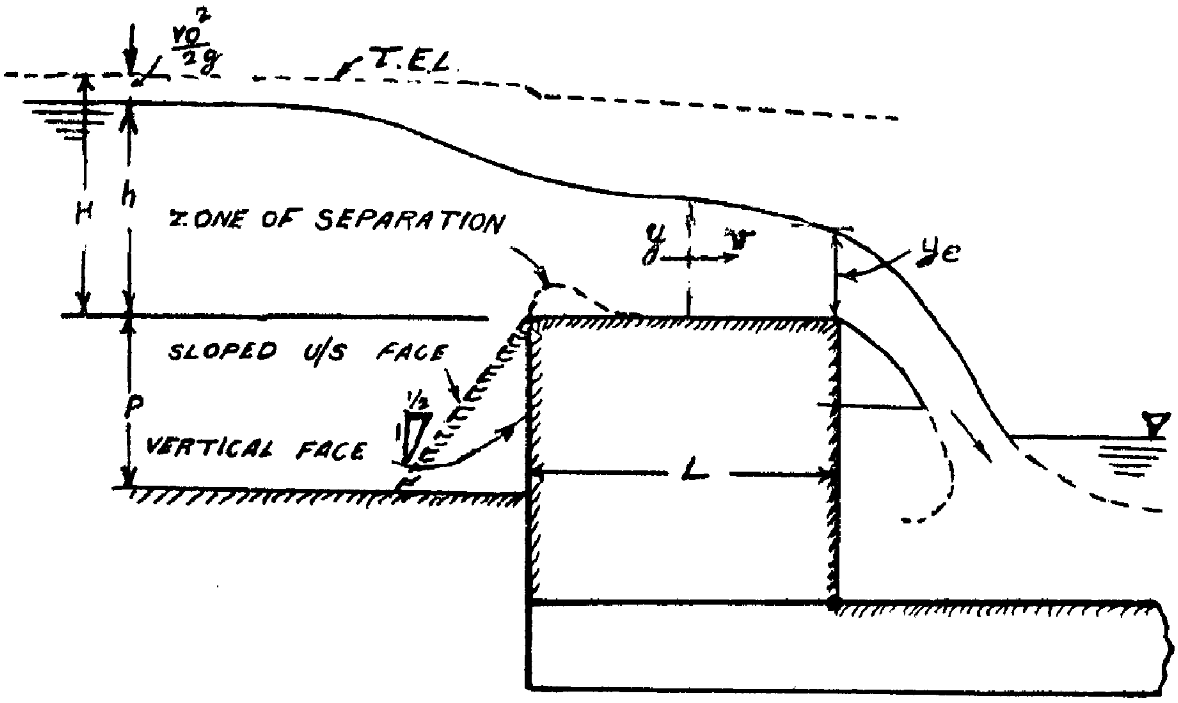
FIG. 1. DEFINITION SKETCH OF VERTICAL DROP FALL

for various sizes.

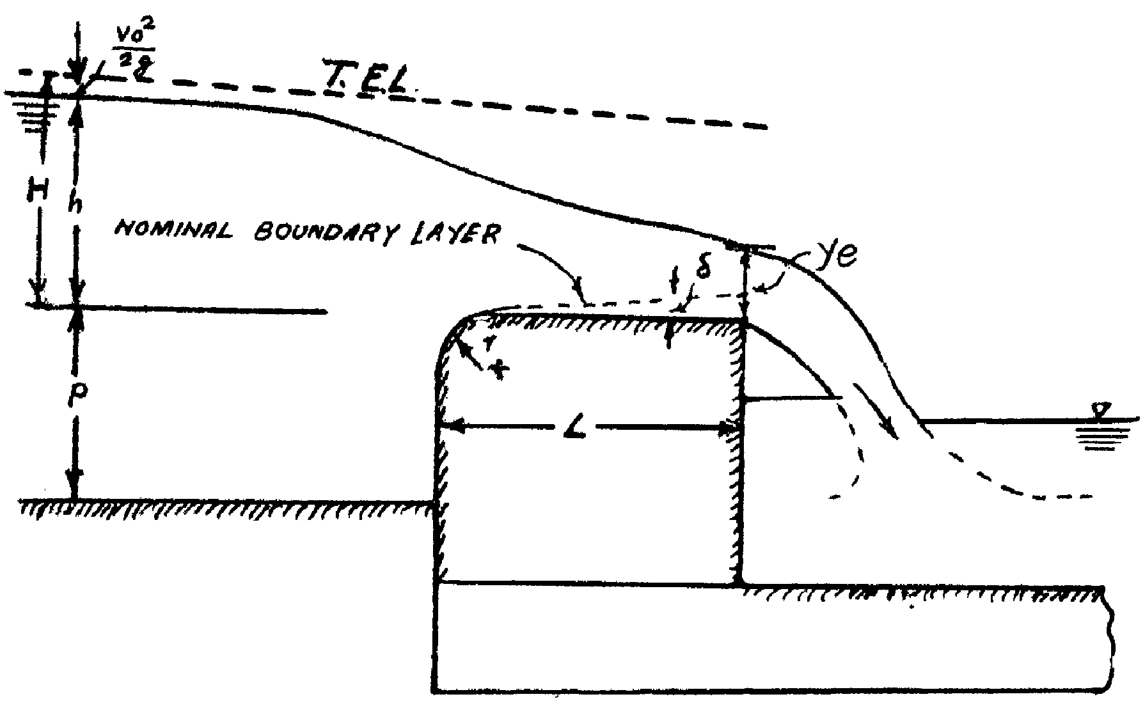
Obviously, then, it is more advantageous to eliminate completely such gauging structures, if the measurement of flow of water in the irrigation system can be accomplished to the same degree of accuracy with the help of structures that form a part of the system itself. Therefore, full advantage of the orifice and weir discharge characteristics of falls, checks, head-regulators on canal system and weirs across rivers which form control sections for the water courses, should be exploited for gauging the water flow; and this aspect of the problem needs fuller investigation.

1.3 Purpose and Scope of the Investigation.-

In this thesis an attempt has been made to study the "discharge characteristics of vertical drop falls", that exist in irrigation systems. By vertical drop fall is meant a hydraulic structure of the form of a suppressed broad-crested weir with a horizontal crest and vertical down-stream face, over which water flows and falls down to a lower elevation, as shown in Fig.1. Instances of such structures are the "Sarda Type" canal falls in the irrigation systems in India and weirs (or "anicuts") with a vertical downstream face,



(a) FREE FLOW; SHARP U/S CREST CORNER



(b) FREE FLOW, U/S CREST CORNER ROUNDED

FIG. 2- DIFINITION SETCHES OF CREST OF VERTICAL DROP FALLS (FREE FLOW)

across rivers. The discharge characteristics of the vertical drop falls were proposed to be studied on the basis of their form being that of a "broad-crested" weir. A proper knowledge of the discharge characteristics of vertical drop falls, of which the "Sarda Type" fall is typical, would obviate the necessity of constructing special gauging structures like meter falls and Parshall flumes, resulting in economy. For instance, the discharge in a reach of a canal can be estimated by the measurement of the head over the crest of a vertical drop fall that may exist in that reach of the canal. Thus a Sarda Type (vertical drop) fall on a canal will serve the dual purpose - that of creating a drop in the canal as well as that of a metering station. Similarly, a weir with a vertical downstream face will not only surplus water from the pond above the weir, but will also enable the discharge passing over it for the different stages to be estimated, from a knowledge of the head of water over the weir.

The suitability of the vertical drop fall as a metering device lies in the fact, that the crest across which the water falls, is in the form of a broad-crested weir. Though, no greater accuracy than of the other weirs is claimed for the broad-crested weir, from the point of view metering flow in irrigation channels, it has got one chief advantage. That is, the modularity of the broad-crested weir remains unaffected even with high degrees of submergence (of the order of 60% to 80%). Thus, only low values of head drop being permissible in irrigation canals, it is a positive advantage. With the crests of the falls often submerged, the discharge coefficient will not be altered and therefore the upstream water level will not be affected, even upto a fairly high degree of submergence due to water level in the canal downstream of the fall.

The typical design of an ordinary vertical drop fall of Sarda type (13) has a raised crest of the 'hybrid' or narrow flat-topped type with generally both the faces sloping to give a trapezoidal shape. In the case of smaller falls, the section always has both the faces vertical. The corners of the top of crest are invariably rounded. The crest functions as a broad-crested (or flat-topped) weir with suppressed end contractions, the leading channel confined between the vertical side walls having the same width as the length of crest. The formula suggested (13) for the computation of discharge does not contain any factor that takes into account the effect of the rounding of the upstream corner of crest, the amount of rounding also having not been defined. It is felt, as in fact it has been qualitatively established by previous investigations, that the rounding of the upstream corner of the crest has an appreciable effect on the discharge coefficient. Apparently, there has been, so far, no systematic study of the effect of rounding of the upstream edge of the crest on the discharge coefficient of a broad-crested weir. One of the objects of the present investigation is to study this aspect.

In many of the vertical drop falls, the upstream face is usually inclined, the magnitude of the inclination being arbitrarily fixed by the designer. Invariably, the formula suggested for the discharge does not include any factor that accounts for the effect of the slope of the upstream face, which evidently depends upon the degree of inclination also. There have been some experiments, usually made on models of individual structures to study this aspect of the problem. The results of these experiments are not tallying and sometimes even the trends of the results are in disagreement. Therefore a study of the effect of the inclination of the upstream face of the fall is the second object of this investigation.

It is well known that when water flows over a broad crested weir, the flow undergoes a change from sub-critical (or tranquil) state to supercritical(or shooting) state and critical depth will be established somewhere on the crest. The classical theoretical derivation by "maximum discharge" theory would give a value of 3.087 for the discharge coefficient C in the discharge equation,

$$Q = C\,B\,h^{3/2} ,$$

where Q = Discharge

B = Length of crest normal to flow direction

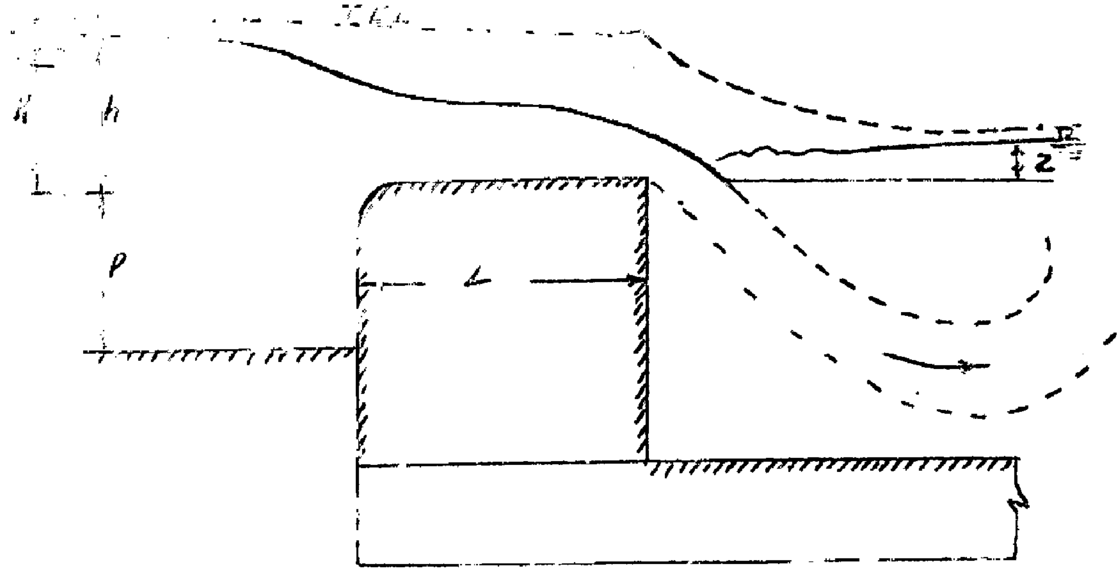
h = Observed head over crest.

However, earlier researches have shown that actual observed values of C have a variation within a very wide range, depending upon the length of crest L (parallel to flow direction) relative to the head on the crest. In other words, the value of C was observed to increase with the value of the ratio of head to length of crest (h/L). There have been experiments on broad-crested weirs showing the variation of the discharge coefficient C with the head on the crest.

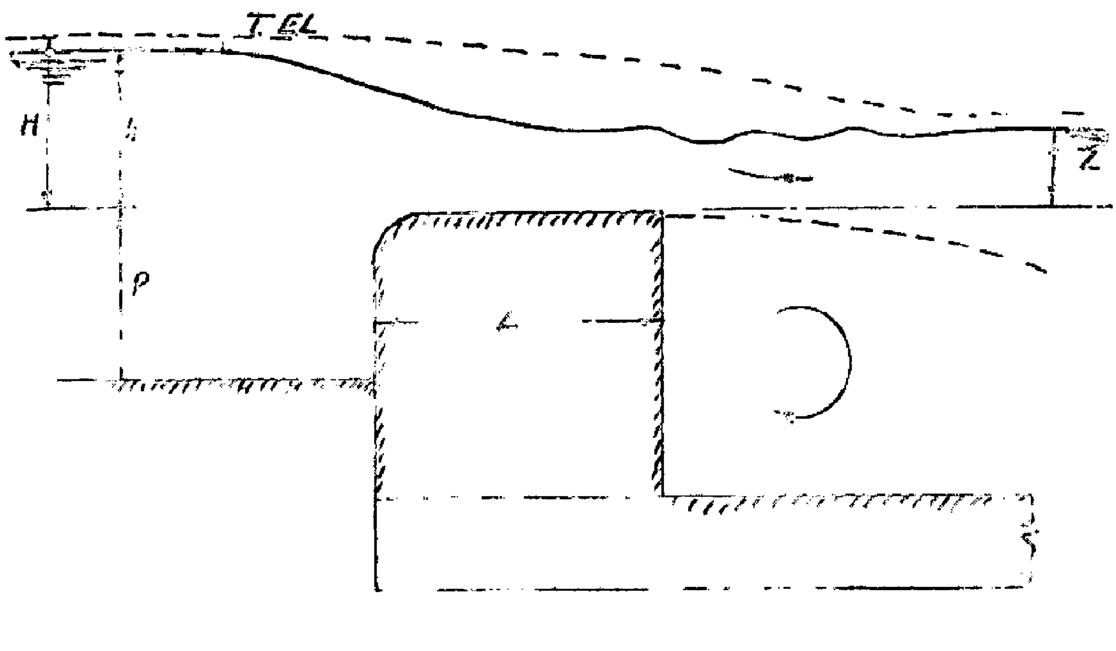
But these were mostly on very long weir crests, intentionally made so to ensure the establishment of uniform flow and hydrostatic distribution of pressure; the range of h/L was thus limited. Therefore, the third object of this study was to investigate the variation of the discharge coefficient, C, with an extended limit of h/L so as to cover the full range of flow over the crest functioning as a broad-crested weir.

The fourth aspect proposed to be investigated is the variation, if any, of the discharge coefficient C with the height of the crest of fall, P, above the bed of upstream channel in both the cases - when the upstream corner of crest is sharp and rounded.

In the operating range of vertical drop falls, the crest of the fall is often submerged by the water in the downstream channel. Upto a certain maximum degree of submergence, the rise of downstream water level will not affect, for the given upstream head, the discharging capacity of the crest of fall, which functions as a submerged broad-crested weir. In this range of operation, upstream water level, therefore, will not be affected by the variation of water level downstream. At a particular limiting degree of submergence, the modularity of the broad-crested weir is lost, when the upstream head will be affected by the variation of downstream depth of water over crest and consequently, the discharge depends upon both the upstream and downstream heads. Thus, in the 'non-modular' range of functioning of a broad-crested weir, for a given head upstream, the discharge coefficient C depends upon the degree of submergence. The limit between the 'modular' and 'non-modular' range is the 'modularity limit'. Although the 'modularity limit' is usually stated to occur when the degree of submergence reaches a value around 0.66, its precise value and the factors on which it depends are not known. Further, in the non-modular range, the variation of the discharge coefficient C with the degree of submergence, for a given head over a braod-crested is also not precisely known, although some experiments to determine this were done for individual weirs of a particular design. Hence, the practice (13) so far, has been to calculate the discharge over the submerged broad-crested weir (in non-modular range) as consisting of a flow above the downstream water level occurring as a free weir discharge and that below the downstream water level as orifice discharge. Experiments



(a) SUBMERGED FLOW, DIVING NAPPE



(b) SUBMERGED FLOW - UNDULATING NAPPE.

[illegible] SKETCHES OF [illegible]

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have, however, shown that the actual discharge does not tally with that calculated by the above procedure. Submerged discharge being of very common occurence in vertical drop falls, the fifth object and one of the chief objects of this investigation was to study the discharge characteristics of the fall in the submerged range. The scope of study of this aspect, proposed to be covered, was to investigate the precise value of the limiting degree of submergence at "modularity limit" and its variation, if any, under the influence of possible other factors and also the variation of the discharge coefficient C with the degree of submergence for different values of the ratio of upstream head over crest to length of crest (h/L).

It is stated earlier that the vertical drop fall has the characteristics of a broad-crested weir with a horizontal crest and vertical downstream face. The term "broad-crested weir" is, however, generally poorly defined. This is classified as such on the basis of the geometry of the structure rather than any rational considerations. Invariably, a weir is called "broad crested" if it has a nearly horizontal crest of finite length in the direction of flow. Experiments have shown that at sufficiently high head to crest length ratios, the nappe of overflowing water tends to spring clear of the weir crest, after which the structure no longer functions as a broad-crested weir. On the other hand, for very small head to crest length ratios, the weir crest becomes a reach of an open channel, in which the frictional resistance predominates, and for which the discharge is better evaluated by one of the open-channel flow formulas than by a weir formula. Therefore the definition of a broad-crested weir as commonly understood is inadequate. In this investigation, it is proposed to give a more logical definition of a broad-crested weir based on the flow pattern over the crest and the discharge characteristics.

To sum up, the objects and scope of this investigation are broadly, to evaluate the discharge characteristics of vertical drop falls from a study of the variation of the discharge coefficient with pertinent variables like

(1) The length of the crest in the direction of flow L.

(2) The height of the crest P

(3) Rounding of the upstream crest corner to different radii r.

(4) The slope of the upstream face.

(5) Submergence of the weir.

Notation.- The letter symbols adopted for use in this thesis are defined where they first appear, in the figures or in the text, and are arranged alphabetically, for convenience of reference, in Appendix. I.

2. REVIEW OF PREVIOUS STUDIES.

2. REVIEW OF PREVIOUS STUDIES

2.1 General.-

As the vertical drop falls have the hydraulic characteristics of a broad-crested weir with horizontal crest and vertical downstream face, it is pertinent to review briefly the studies, made earlier, on broad-crested weirs of the said form.

2.2 Blackwell's Experiments:-

The first recorded experiments on broad-crested weirs were those made in 1850 by Thomas E. Blackwell (2) in a side pond of the Kennet and Avon Canal, England. The crests experimented on were horizontal, 3 ft. long in the direction parallel to the flow and lengths of 3 ft., 6 ft. and 10 ft. perpendicular to the flow direction. The side faces were vertical. The discharge was volumetrically measured and the conditions were generally favourable for accuracy. The maximum head over the crest was 1 ft. The value of discharge coefficient C, without velocity of approach correction varied between 2.24 to 2.77. However, the variation of C with the length of crest L (in the direction of flow) was not observed to indicate any systematic trend.

2.3 Fteley and Stearns Experiments.-

Fteley and Stearns had given a formula for the discharge over broad-crested weirs (2) based on five series of experiments, made in the Sudbury River Conduit, Boston in 1877. The weirs were 'suppressed', 5 ft. long and 2, 3, 4, 6 and 10 inches wide. The discharge was measured volumetrically. The head ranged up to 0.894 ft. The results were given by Fteley and Stearns in the form of a table of corrections to be added algebraically to the measured head for the broad-crested weir to obtain the head on a thin edged weir that would give the same discharge. It is to be observed, in this connection, that in order to know the discharge over a bread-crested weir by the Fteley and Stearn's formula, a knowledge of the discharge coefficient for a thin edged weir for the given head is also needed in addition to the correction to the head obtainable from the tables given by them. Further, in order that the discharge calculated by the above procedure should be correct, the approach channel conditions for the given broad-crested weir and the hypothetical thin-edged weir, the discharge coefficient of which would be employed in the formula, should be the same.

Fteley and Stearns experimented somewhat on the effect of rounding the upstream crest corner on the discharge coefficient. Experiments were made by them on weirs with crest radii of $\frac{1}{4}$, $\frac{1}{2}$ and 1 inch. and a crest length of 4 inches in the direction of flow. On the basis of these tests, Fteley and Stearns had shown that the discharge coefficient increased with the rounding, and had given a correction formula for the head,

$$K = 0.41\ r$$

where 'K' is a correction to be added to the measured head before applying in the discharge formula and 'r' is the radius of rounding. This formula is stated to be applicable for heads of not less than 0.17 ft. and 0.26 ft. respectively on weirs with radii of $\frac{1}{4}$ and 1 inch. Fteley and Stearns had further shown that the effect of rounding the crest corner decreases with the length of crest L in the direction of flow and that it also decreases when expressed as a percentage, with the head. Thus, the effect of rounding of upstream crest corner resulting in increased value of the discharge coefficient was first established by them, though the formula given by them cannot be said to be applicable to all the cases of broad-crested weirs.

2.4 Bazin's experiments.-

Though a few hydraulicians like Blackwell and Fteley and Stearns investigated the effect of variation of the length of crest L, it remained for H.Bazin to elucidiate the whole problem of flow over broad-crested weirs in a series of masterly studies, conducted in the years 1888 to 1898, well annotated by G. W. Rafter (1) and Horton (2). These studies were remarkable, as regards detail and minute research.

Bazin investigated a large number of broad-crested weirs with different crest widths, varying slopes of upstream and downstream faces and curved profiles. He also studied of submerged weir behaviour in a general manner.

The weirs were tested by Bazin, in a long chamber, at the head of which a standard weir was installed. The discharges passing over the standard weir relative to the heads over it were established by actual measurement a sufficient number of times to ensure the error to be within 1 percent. The head over the broad-crested weir under test as well as the head over the standard weir were synchronously read. The discharge coefficients of standard weir having been established, the discharge coefficient of the experimental weir could be computed. Bazin's experiments on broad-crested weirs pertinent to problem in question included a series of 20 runs each for heads upto 1.4 ft. on weirs of 0.164 ft., 0.328 ft., 0.656 ft. 1.315 ft., 2.62 ft. and 6.56 ft. length in the direction of flow and of 2.46 ft. height. Other experiments made were for four narrower weirs with heights of 1.148 ft. and 1.64 ft. and other shapes having various upstream and downstream slopes. Bazin studied the effect of rounding of upstream crest corner by running two series of runs, with crest lengths of 2.62 ft. and 6.56 ft. both with radius of 0.33 ft. for the upstream crest corner.

Bazin studied and described in detail the nappes of overflowing water. He classified them into five categories.-

(1) Free nappes.- In this case, the nappe falls freely into the air, its lower surface always subject to the pressure of atmosphere.

(2) Depressed nappes.- The nappe remains detached from the face of the weir, imprisoning under it a volume of air at a sub-atmospheric pressure. The water in the space between the foot of the nappe and that of the weir rises to a level above that of the stream below

the weir. Depressed nappes are unstable, producing variations in the interior pressure.

(3) Nappes wetted underneath.- When the downstream water level rises and the air under the nappe disappears completely, the nappe is wetted underneath. The water enclosed by the overflowing nappe does not partake in the translatory movement of the vein of water.

(4) Undulating nappes.- When the level of water in the channel below the weir rises above the crest, the wetted nappe retains its form, though concealed by immersion, as long as the difference of water level on the upstream and downstream side does not descend below a certain limit. When the difference of level reaches the limit, the nappe springs suddenly to the surface, undulating in the meanwhile.

(5) Adhering nappes.- The nappe is in close contact with the downstream face of the weir, contrary to the nappe wetted underneath, the mass of turbulent water below which, does not partake in translatory movement.

Bazin had observed that the value of the coefficient of discharge of the weir is changed very little, as the nappe changes its form from 'free' to the 'depressed', 'wetted underneath' and 'undulatory' forms.

On the basis of tests made on 'beam weirs', Bazin observed that the overflowing sheet of water will be in contact with the horizontal crest only for values of heads below $1\frac{1}{2}$ to 2 times the crest length in the direction of flow. Between these two values, the nappe will be in a state of instability tending to detach itself from the horizontal crest under the influence of any external disturbance. Once the head exceeds the above limit, the nappe detaches itself completely from the horizontal crest and the weir will be functioning like a sharp-edged weir, the upstream crest corner acting as the sharp-edge.

Bazin gave formula for discharge coefficient of broad-crested weirs in terms of the discharge coefficient for sharp-edged weirs and the head to crest length ratio h/L .-

$$C = C_1 \left(0.70 + 0.185 \frac{h}{L}\right) ,$$

where C = Discharge Coefficient for broad-crested weir.

C_1 = Discharge Coefficient for sharp-edged weir.

He further observed that when h/L = 1.5 to 2, C/C_1 = 0.98 to 1.07 if the nappe adheres to the crest and C/C_1 = 1.00 if the nappe is detached and h/L is greater than 2.0 . Bazin considers that his formula gives accurate results for adhering nappes with lengths of crests up to 2 to 3 ft. For a crest 6.56 ft. long and h = 1.476 ft., he found the result by his formula 93.4% of that given directly by experiment.

It may be remarked, that Bazin's formula gives ratios C/C_1 which continually increase as h increases, L remaining constant and which continually decrease as L increases, h remaining constant. However, the formula gives a constant ratio for all widths or heads where h/L ratio is unchanged. In other words, it means, the discharge coefficient C of broad crested weirs depends upon h/L and increases in the same manner as the discharge coefficient of a sharp-edged weir, if h/L remains the same. Fteley and Stearns's experiments gave a discharge ratio which is less than unity, but which varies in an irregular manner, depending upon the head and length of weir in the flow-direction.

The results of the experiments by Bazin show (2) that except for very low heads, the discharge coefficient curves are simple linear functions of the head. Further, they show that the rate of increase of the coefficients as the head increases, becomes rapidly less as the length of the weir L increases, indicating that for a very long weir, the coefficient would be sensibly constant throughout the range of the stability of the nappe. For the narrower weirs, the coefficients tend to increase rapidly almost from the start towards the value for a sharp-edged weir with detached nappe.

From the experiments on weirs with rounded upstream corner with lengths of crest 2.624 ft. and 6.56 ft., Bazin found the effect of rounding to a radius of 0.33 ft. was to increase the discharge coefficient by 14% and 12% respectively. The experimental results of Fteley and Stearns are comparable, in this connection. They indicated the coefficient to increase in the ratio of $(h + 0.7\ r)^{3/2}$ to $h^{3/2}$ or nearly in the ratio of $(1+ r/h)$ to 1. The radius r in their experiments did not, however, exceed 0.039 ft. and it is clear that this approximate formula will not apply to cases where the radius is notably greater.

Bazin's experiments on weirs with inclined upstream face indicated the trend of increasing discharge coefficient (1) with increasing flatness of slope, other factors remaining constant. Further, it was observed by Bazin that rounding the upstream crest corner of such weirs resulted in increasing the discharge coefficient by 10 to 15 percent.

Bazin experimented upon the effect of submergence of the weir. He found that unlike the sharp-edged weirs, raising the water level on the downstream side of the weir does not commence to take effect on a broad-crested weir until after the level of water is considerably above the crest. In his experimentx on a crest of 6.56 ft. length, it was seen that the water on the downstream side of crest had its effect on the upstream water level, when the degree of submergence exceeded 5/6 .

2.5 Cornell University experiments:

An important contribution to the study of weirs was made by the experiments conducted in the Hydraulics Laboratory of Cornell University. The experiments were conducted by the United States Board of Engineers On Deep Waterways, in 1899 under the direction of George W. Rafter in conjunction with Prof. Gardner. S. Williams; and by the United States Geological Survey, in 1903, under the supervision of Robert. E. Horton and Prof. Gardner. S. Williams.

(a) U. S. Deep Waterways Board Experiments:- The experiments (1, 2) were made in a canal 418 ft. long, 16 ft. wide and 10 ft. deep. A rated standard sharp-edged weir fixed on a timber bulk-head situated 60 ft. from the upper end of the canal, was used to measure the discharge of water, which also passed over the experimental weir placed in a section of the canal flumed to 6.56 ft. width with an approach channel of the same width and 48 ft. length towards the lower end of the canal. The experimental weirs were about 4.5 ft. high and 6.56 ft. long perpendicular to the flow direction.

Among the many experiments made, four series of experiments (1, 2) were made on flat-topped crests. They were extensions on Bazin's weirs, the crests tested having lengths of 2.62 ft. and 6.56 ft. in the flow direction and 4.57 ft. high. These weirs were tested, both with square corners and corners rounded to a radius of 0.33 ft. The corresponding Bazin's weirs were 2.46 ft. high. The heads ranged from 0.5 ft. to 6.0 ft. in the Cornell University experiments, while in Bazin's series they were limited to 1.4 ft. In these experiments, some were made with upstream slopes; but these were on crests having width of only

0.66 ft.—too small for the heads tested to function as broad-crested weirs. No studies were made on the effect of submergence of weirs.

The results of the tests indicated (2) that the discharge coefficient varies linearly with the head, the values being generally higher for the weir of shorter length than for the weir of longer length. The effect of rounding of the upstream crest corner was to increase the discharge coefficient by about 7 to 8 per cent for the weir of length 2.62 ft. and 12 to 16 per cent for the weir of length 6.56 ft. in the range of their function as broad-crested weirs.

(b) U. S. Geological Survey Experiments.-

These experiments were carried out to know the flow characteristics of dams used at gauging stations by the U.S. Geological Survey. The experimental facilities available at Cornell University Hydraulic Laboratory were availed of. The weirs with a horizontal flat crest and vertical faces, that were experimented upon were all of height 11.25 ft. The widths of the crests were 0.479, 0.927, 1.646, 3.174, 5.875, 8.98, 12.239 and 16.302 feet. The nappes in some of these experiments were partly aerated.

The results of the experiments show (2) that for weirs of large length (5 to 16 ft.), there is no conspicuous tendency for the discharge coefficient C to change with the variation either in head h or length of crest L, the value of C being from 2.62 to 2.64. It was also observed, confirming Bazin's results, that the coefficient C slowly increased with the head and decreased as length of weir increased. The detachment of the nappe from the horizontal crest occured at head to crest length ratios of 1.5 to 2.0 as was also the case in Bazin's experiments.

Richard Lyman prepared and published a chart (3), giving the discharges over a series of suppressed broad-crested weirs all of height 11.25 ft. based on the experiments of the U.S. Geological Survey in Cornell University. On this chart, there are three series of lines. One gives the head on the crest, the second the discharge per foot width of weir and the third the length of crest in the flow direction in ft. For every point on this diagram, there is a definite discharge and also a definite head. It may be observed from this diagram, that for comparatively low heads, all these broad-crested weirs give the same result for the same head. At a particular point or head, however, for each width of crest, the quantity of discharge begins suddenly to increase much faster in proportion to the head, than it has done before and the line representing the discharge breaks away from the common line and extends across the diagram till it meets another common line representing the discharge over sharp-edged crests. The head shown by the point at which this common line is met represents the head at which the sheet of overflowing water jumps over the crest without touching it. For all higher heads, the sharp crest conditions prevail.

From the diagram, it may be observed that each line breaks away from the first common line at head to crest length ratio of 0.5 approximately.

Lyman also gave a table (3) giving coefficients with which to multiply the quantities of discharge over the broad-crested weir read from the chart, in order to give the discharge over similar weirs (for the same head) for various heights of such weirs. These tables were prepared on the assumption, apparently not correct, that the height of the weir in the case of broad-crested weir has the same effect as in the case of sharp-crested ones.

2.6 Woodburn's Experiments.-

J. G. Woodburn conducted experiments on broad-crested weirs (5) during 1928-1929 at the University of Michigan, with a view to design a weir which would produce flow at critical depth at the same point on the crest of weir for all discharges, so that the discharge could be determined with the help of a single measurement at the section of critical depth formation.

The experiments were carried out in a rectangular wooden flume 2 ft. wide and 47 ft. long and the discharge was measured with the help of a calibrated right angled V-notch. The nappes were aerated. The tests were carried out on weirs of length of crest (in the flow direction) from 10 ft. to 15.5 ft. The range of head was from 0.5 ft. to 1.5 ft. and the discharge from 2.0 to 11.0 c.f.s. Woodburn made tests on 10 ft. long level crests consisting of one series with sharp upstream corner and one series each with upstream corner rounded to radius of 2, 3, 6 and 8 inches. No studies were made on the effect of submergence of crest.

Observations made on the weir profiles showed that none of the weir models tested produced flow at critical depth at any fixed location throughout the entire range of heads.

The discharge coefficient C was observed to increase up to value of head of 1.1 ft. on the 10 ft. long crest, after which it was approximately constant. The increase of the coefficient for the smaller heads may be attributed to the relatively larger effect of weir surface roughness in that range.

It was found, that varying the curvature of the weir entrance from 2 to 8 inches radius had practically no effect on the discharge coefficient, the graphs for all these series lying within 0.5% of each other. The graph for the square cornered entrance was considerably lower showing values of the coefficients from 6% to 8% less than for rounded entrance.

Woodburn continued his study in the Hydraulics Laboratory of the State College of Washington, where he conducted three tests on broad-crested weirs with level crest 2 ft. long and heights of 4, 8 and 12 inches in a glassed-walled flume 1 ft. wide. The upstream corners of all the three weirs of different heights were rounded. It was observed that practically the same curve was obtained for all the three series, indicating that there is no variation of the discharge coefficient C with the height of weir. This was a very significant conclusion, dismissing the earlier assumption by Lyman (3) that it varies in the same manner as for a sharp-edged weir.

2.7 Rouse's Study of Free Overfall.-

Dr. Hunter Rouse conducted experiments on the depth at a free overfall (6) at the end of a long mild channel and had shown that the depth at the free overfall could be conveniently used as a flow-meter without calibration.

Based on theoretical analysis and experiments, he concluded that the depth y_e at the free overfall will be 0.715 of critical depth with parallel flow y_c , and as such a single measurement at the free overfall would yield the value of discharge. This finding has certain significance in the present problem, as the flow over broad-crested weirs with relatively long crests may be considered as the flow at a free overfall; and therefore the discharge may be found by a single measurement of depth at the downstream end of the crest.

However, the value of $y_e / y_c = 0.715$ given by Rouse is open to question due to different values observed by other experimenters. M. P. Obrien (18) appears to have the lowest value of 2/3, working from the impulse-momentum principle with an experimental value of 0.643. Thus, the value seems to be varying between the limits of 0.64 and 0.715 involving a rather generous spread.

2.8 Experiments on Some Individual Structures:-

In addition to the classical experiments described above, there have been several model experiments made (7, 10, 15, 17, 20, 21, 23) for the determination of the discharge coefficients of individual weir structures in their respective operating range of the discharge and head.

C.D. Smith reported (22) results of his experiments on the use of a broad-crested weir with rounded upstream corners as a gauging device. The discharge coefficients plotted in the graph gradually increases with h/L, whose range was limited to 0.7. He varied the crest corner radius in the range of ratio of head to radius from 0.7 to 4.5. There was no indication of the effect of radius of rounding in that range on the discharge coefficient.

Smith also gave the curve of variation of the discharge coefficient C with the degree of submergence on the basis of his experiments. He concluded that the modularity limit would occur around 70% degree of submergence for h/L less than 0.45 and less than 70% degree of submergence for ~~greater~~ values of h/L greater than 0.45. The range of experiments was, however, limited in his studies.

2.9 <u>General Review</u>.-

A survey of the previous experimental work will lead to some general conclusions regarding the flow over broad-crested weirs with horizontal crest and vertical downstream face.

Most of the work done and the results obtained therefrom are empirical in nature and apply to the particular weirs tested. No general characteristics of flow, readily applicable to other structures of the form of broad-crested weir were given from those tests. However, certain trends indicated by the experiments are noteworthy.

1. As regards the variation of the coefficient of discharge C with the length of crest L, it appears to have a constant value around 2.60 to 2.65 till a value

of head to crest length ratio h/L of about 0.35 and a gradually increasing value thereafter till the occurrence of sharp-edged weir conditions, which occurs when the head reaches a value of 1.5 to 2.0 times the length of crest. This was observed in the experiments of Bazin and Cornell University.

2. Rounding the upstream crest corner of a broad-crested weir has the general effect of increasing the discharge, as revealed by the experiments of Fteley and Stearns, Bazin and Woodburn. The range of radii used by Fteley and Stearns is too small to permit any generalisation of the effect. Bazin made experiments with only a single radius of 0.33 ft. In Woodburn's tests, variation of radius in the range of 2 to 8 inches had no effect on the discharge coefficient. From Woodburn's experiments (5) and similar experiments by C. D. Smith (22), it appears that the rounding of upstream crest corner has its effect on the discharge coefficient ~~has its effect on the discharge coefficient~~ only upto some degree of curvature, beyond which the effect is absent.

3. The height of the weir was shown to have no effect on the discharge coefficient C by Woodburn's experiments made with three different heights of crest

of 4, 8 and 12 inches and upstream crest corner rounded.

4. The experimental results regarding the influence of slope of the upstream face of a broad-crested weir are, generally, conflicting. Experiments conducted by C.W. P. Research Institute, Poona (20) to design a suitable section for Kosi Barrage led to the general conclusion that the effect of increasing the steepness of the slope of the upstream face is to increase the coefficient of discharge from a slope of 1 in 3 to an optimum slope of 1 in 1.167 and decrease thereafter upto vertical face. Earlier experiments done by the above Research Institute, in connection with Lloyd Barrage revealed an optimum slope of 1 in 2 for the best value of the discharge coefficient (20). Experiments done at the Punjab Irrigation Research Institute, in connection with Trimmu Barrage (10) showed that the coefficient increases with flatter upstream slope. Studies by the U. S. Bureau of Reclamation (11) in connection with the Boulder Canyon project inidcate, in a general way, that the coefficient decreases, though only slightly, on account of the steepening of the upstream slope with head to

crest height ratio (h/P) greater than 1 and vice versa.

5. Studies on the effect of submergence on the discharge coefficient have been few and inconclusive. However, these studies have shown that the broad-crested weir, unlike the sharp-edged weir, functions in the modular range upto large degrees of submergence. The degree of submergence at modularity limit in the different studies has been in large variance. Bazin's experiments (1) showed the modularity limit at a degree of submergence as high as 5/6. C. D. Smith(22) obtains a value of about 70% for h/L ratios less than 0.45 and much smaller values for h/L ratios greater than 0.45. Experiments by Punjab Irrigation Research Institute, Amritsar in connection with Hanumannagar barrage design (20) showed 80% degree of submergence at the modularity limit. P. K. Kandaswamy has reported (21) a limiting submergence of about 66% on the basis of tests conducted in connection with Kodiveri anicut.

In addition to the lack of any knowledge of the factors influencing the modularity limit, there is no available experimental data from which the flow characteristics of bread-crested weirs in the non-modular range may be studied. There have been a few investigations (21, 22, 23), mostly on individual structures of specific forms; but the data obtained from these experiments does not permit any generalised conclusions, especially in the range of high degree of submergence, on account of the limited range of experiments.

Thus, it may be noted, that a comprehensive study of the discharge characteristics of broad-crested weirs on rational lines, readily useful for application to any structure of the broad-crested weir form, has not been made in the earlier studies.

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3. THEORETICAL CONSIDERATIONS

3. THEORETICAL CONSIDERATIONS

3.1. Critical Depth on Weir Crest.-

When water flows over a broad-crested weir, which is the basic form of a vertical drop fall, the flow undergoes a transition from the tranquil (sub-critical) to the shooting (super-critical) state. Therefore, the vertical drop fall functions as a control section and the flow will take place with critical depth, somewhere on the crest of the broad crested weir forming the said structure. If the critical depth can be measured, the discharge can be computed by the traditional equation

$$q = \sqrt{g} \cdot y_c^{3/2} \qquad .. \quad (1)$$

where q = Discharge per unit width of weir

and y_c = Critical depth of flow.

This elementary method of discharge calculation, however, is not satisfactory, when accurate value of discharge is required to be determined. The flow depth does not correspond to the critical depth y_c everywhere on the weir crest. In fact, the precise location of the control section, where critical depth forms, is not known. Woodburn's studies (5) on broad crested weirs have revealed the fact that there is no single section on the weir-crest where the flow will take place with critical depth for all heads and discharges. It has since been realised that the critical depth section is not constant, but varies with discharge, weir geometry and crest roughness.

The traditional discharge equation for a broad-crested weir was derived on the basis of "maximum discharge" theory. Belanger first suggested, around 1849, that the discharge over a broad-crested weir will be the maximum obtainable for a given total head ("charge"). For a total head $H = \left(h + \frac{v_0^2}{2g}\right)$ consisting of static head h and head due to the velocity of approach v_0, and a crest sufficiently long in the direction of flow, the depth y (see Fig. 2) is supposed to form in such a manner that the discharge will be a maximum. Thus, the critical depth was understood, generally, as the depth of flow over the crest, such that the discharge is a maximum possible for the given total head. The deductions from the "maximum discharge" theory, however, are not found to hold in the face of experimental evidence. This will be evident from the following analysis.

Let K be the ratio of the depth of flow over crest y to the total head H, v the velocity of flow over the crest, and c_v a coefficient of velocity to take account of the hydraulic losses occuring as the water flows over the crest to the section of flow depth y on the crest.

Then,

$$v = c_v \sqrt{2g(H-y)} = c_v \sqrt{2gH} \cdot \sqrt{1 - y/H}$$

$$= c_v \sqrt{1-K} \sqrt{2gH} \quad .. \quad (2)$$

$$q = y.v. = K.H.v$$

$$= c_v K (\sqrt{1-K}) \sqrt{2g}. H^{3/2} \quad .. \quad (3)$$

Designating the combined weir coefficient as C_1,

$$q = C_1 H^{3/2}$$

By comparing equations (3) and (4),

$$C_1 = c_v . K \sqrt{1-K} \sqrt{2g} \quad .$$

Obviously, the maximum discharge is obtained when C_1 is maximum. For a given form of the entrance edge, c_v may be assumed to be unchangeable. For maximum discharge, therefore,

$$\frac{dC_1}{dK} = c_v \sqrt{1-K} - \frac{1}{2} . \frac{c_v K}{\sqrt{1-K}} = 0$$

The above equation gives $K = 2/3$ .. (5)

That means, if the maximum discharge theory were to govern, the critical depth occuring on the crest would be $y = 2/3\ H$ for any type of crest, irrespective of its length, roughness, or the form of the upstream crest corner.

This deduction from the maximum discharge theory is at variance with observed experimental facts. Woodburn's profiles (5) and experiments by U.S.Geological Survey (2) show K to be less than 2/3.

Recognising this discrepancy between the traditional "maximum discharge" theory and experimental facts, it was suggested by Boss in 1919 and Bakhmeteff in 1912 that the problem should be approached from energetic considerations. They proved, independently, that the flow in transition from tranquil to shooting state passes through a critical depth y_c for which the total head, for a given discharge, is a minimum. That is, the total energy line is at its lowest. Thus, for a given total head $H = \left(h + \frac{v_o^2}{2g}\right)$, the critical depth on the crest y_c and the resulting discharge q will be such as to make the contents of energy at critical depth section a minimum.

If a discharge Q flows with a uniform depth and with parallel stream filaments in an open channel of a given form such as in Fig. 4 and if the depth y varies, there is a certain "specific energy of flow" E, that corresponds to each depth of flow.

Specific energy E is the energy head referred to the bottom line of section

$$E = y + \frac{v^2}{2g} = y + \frac{Q^2}{2ga^2} \qquad .. \qquad (6)$$

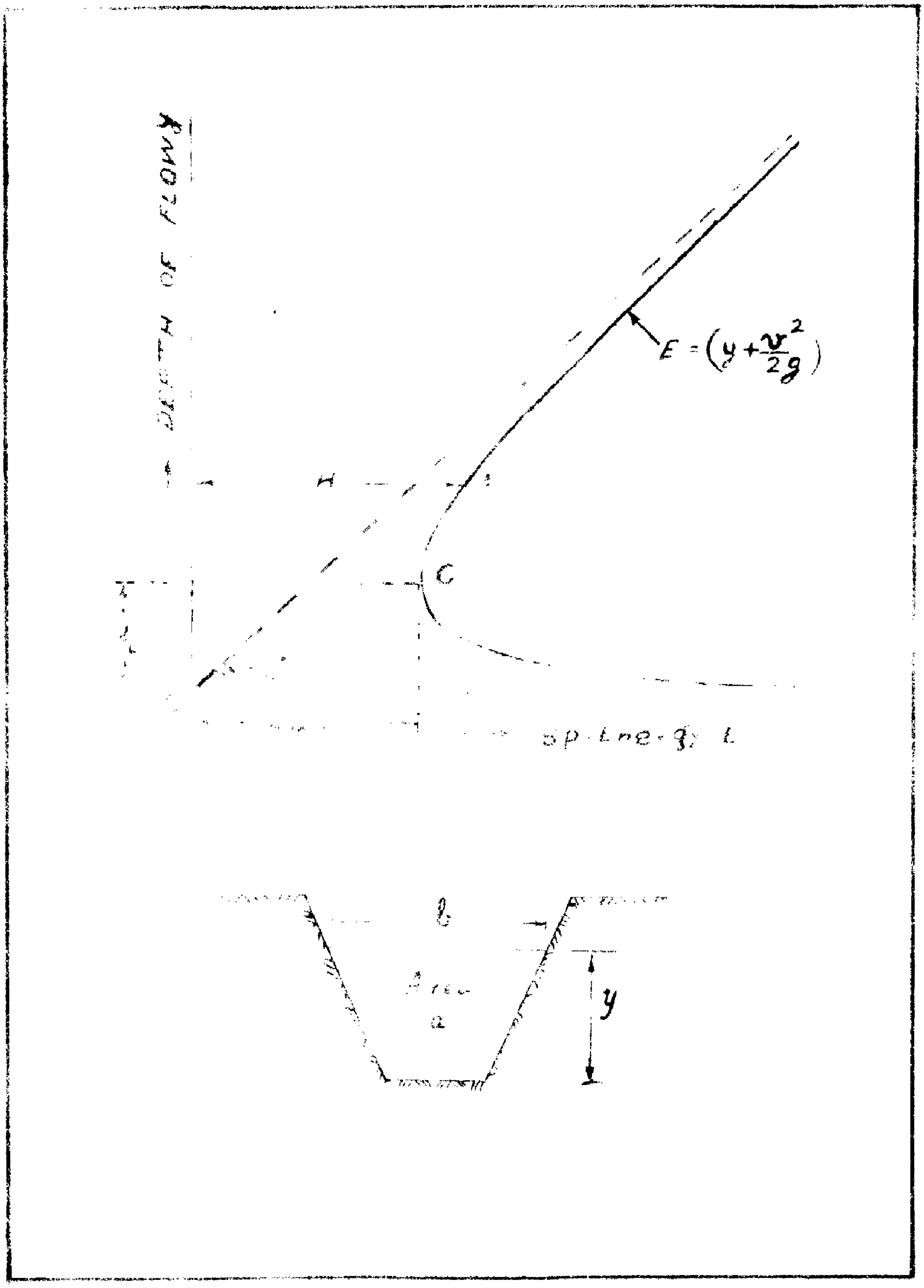
where a represents the cross-sectional area of channel at depth y.

The graph in Fig. 4 shows the contents of specific energy E, as it varies under different depths of flow y. This energy curve is tangent to the horizontal axis and to a straight line drawn through the origin at 45^o and passes through a minimum point at C. The ordinate y_c at point C is the critical depth, where the discharge in the channel flows with the smallest contents of energy per unit weight.

It is known that for a canal of any form the critical depth is determined by the equation

$$(a^3/b)_{cr} = Q^2/g \qquad .. \qquad (7)$$

where b is the width of water surface at the depth y_c. For a rectangular canal, the critical depth is given by



ECIFIC ENERGY DIAGRAM

the well-known formula

$$y_c^{\,3} = \frac{q^2}{g} \quad .. \quad (8)$$

On the energy diagram (Fig. 4), flow over a broad-crested weir is shown by the movement from an initial point A corresponding to the energy content H, down along the energy curve toward C. Only an addition of energy from extraneous sources, which is obviously impossible, can bring about a further lowering of depth below C. Thus, the critical depth corresponding to point C is the natural limit, to which the surface level may lower and which will establish itself at the end of the crest.

The analytical relation between the different parameters of flow may be determined as follows:

By equation (3),

$$q^2 = c_v^{\,2}.K^2(1-K).2g..H^3 \quad .. \quad (9)$$

The condition of minimum energy gives,

$$q^2 = g\ y_c^{\,3} = g\ K^3\ H^3 \quad .. \quad (10)$$

By equations (4) and (10), $C_1^{\,2} = g\ K^3$

$$\text{Or} \quad C_1 = 5.67\ K^{3/2} \quad .. \quad (11)$$

By equations (9) and (10),

$$c_v^2 \; K^2 \; (1-K) \; 2g.H^3 = g \; K^3 \; H^3$$

Or

$$2c_v^2 \; (1-K) \quad = \quad K \qquad .. \qquad (12)$$

From the above equation, $K = \dfrac{2c_v^2}{(2c_v^2 + 1)}$ .. (13)

and $$c_v = \sqrt{\frac{K}{2(1-K)}} \qquad .. \qquad (14)$$

It is clear from equation (13), that in contradiction to the "maximum discharge" theory (by which K is always 2/3), under actual conditions K is always less than 2/3, as c_v is always less than unity. The precise value of K, however, is unknown, as it is a function of the velocity coefficient c_v. The velocity coefficient, in its turn, is dependent upon weir geometry (like, for instance, the nature of the upstream crest corner) and crest roughness.

Thus, it is seen that the geometry of the inlet of the broad-crested weir can be seen to be of prime importance, both from the point of view of location of control section as well as the magnitude of critical depth for a given total head.

If the upstream crest corner is sharp (see Fig. 2), separation will occur just downstream of the entrance to the weir, and the critical section will coincide with the maximum elevation of the separation surface, where the specific energy attains its true minimum. Flow beyond this will be in super-critical state and hence the weir surface could have a downward slope without affecting the depths upstream. The critical section for a weir with sharp upstream corner, however, is in a zone of curvilinear flow, for which, for a given specific energy, the discharge is not necessarily the same as for parallel flow. Equation (1) applies only to rectilinear motion and will not be applicable to curvilinear motion.

If, on the other hand, the upstream crest corner is sufficiently well rounded, separation of the flow at the corner will be prevented. Consequently, the control section will be shifted downstream towards the end of the crest - again a zone of curvilinear motion. If the weir crest is of great enough length, a central region of essentially parallel flow will form, which is free from the curvilinear effects at both ends. If the fluid were frictionless, the depth of flow would be critical everywhere in this region. Actually, an appreciable boundary layer is formed on the crest, in which are concentrated the viscous shear losses which are mainly responsible for the slope of the total head line. This boundary layer growth, again, is subject to variation with weir size, form and discharge; the total energy line accordingly changes in slope. That this effect of boundary layer is subject to approximate evaluation in terms of the boundary layer theory was shown by A. T. Ippen (16), as follows.

For negligible boundary shear, the traditional discharge equation for a broad-crested weir is

$$Q = B\, y_c \cdot v_c \quad = 3.09\ B.\ H^{3/2} \quad .. \quad (15)$$

where B is the width of weir. With boundary shear along the walls and bottom, both B and H must be corrected for displacement thickness δ of the boundary layer. The equation then becomes

$$Q = 3.09\ (B - 2\delta)\ (H - \delta)^{3/2} \quad .. \quad (16)$$

Or

$$Q = 3.09\ B\ H^{3/2}\ \left(1 - \frac{2\delta}{B}\right) \left(1 - \frac{\delta}{H}\right)^{3/2} \quad . \quad (17)$$

The two terms in the parenthesis evidently represent a factor C_I smaller than unity, which can be calculated from the equations for boundary layer thickness.

Laminar boundary layers exist upto Reynolds numbers of $R_x = \frac{Va}{\nu} \approx 3 \times 10^5$, for which

$$\frac{\delta}{x} = \frac{1.73}{R_x} \quad .. \quad (18)$$

where x represents the length of crest surface up to the section at which the thickness of boundary layer is δ.

Turbulent boundary layers may be calculated for smooth boundaries above $R_x \approx 3 \times 10^5$, from the expression

$$\frac{\delta}{x} = 0.63\, c_f \quad .. \quad (19)$$

wherein the local coefficient of boundary resistance c_f may be obtained from a graphical representation of

$$\frac{1}{\sqrt{c_f}} = 1.70 + 4.15 \log_{10} (R_x\, c_f) \quad .. \quad (20)$$

For purposes of this calculation, it is sufficiently accurate to assume the velocity to be equal to the critical over the entire length of the weir.

From the above relationships, Ippen calculated the discharge coefficient characteristic for weirs of different length. For various ratios of head to length of the weir H/L, he established the general trend of the discharge coefficients, which is given in the table below. The middle line is taken from Woodburn's investigations (5) on a broad-crested weir of 10 ft. length. While the general tendency for the calculated value and the observed one is of similar nature, the magnitude of the coefficient itself differs nearly by 5 per cent.

Discharge Coefficients of broad crested weirs related to H/L :

H/L	0.05	0.07	0.075	0.09	0.10	0.11	0.125	0.13	0.15
$C_{obs.}$	0.902	0.909		0.916		0.922		0.925	0.925
C_{theor}	0.932		0.953		0.964		0.970		0.975

The above analysis is only approximate, in as much as the boundary layer displacement thickness can not be correctly evaluated. Few measurements of boundary layer in water have been made on account of instrumentation difficulties and therefore little progress in this direction has been made. However, this analysis shows qualitatively how the magnitude and position of critical depth is dependent on several unknown factors.

3.2 Curvilinear Flow on Crest:-

In addition to the factors enumerated above, there is another important factor affecting the magnitude and location of critical depth on the crest of a broad-crested weir. If the crest is relatively short, above certain values of head to length of crest ratios, the curvilinearity of flow at the two ends will merge into each other, there being insufficient length of crest for a region of parallel flow to form. In such cases, the flow takes place with curvilinear motion along the entire length of crest with the sheet of water in a convex curvature. The equation (8) for discharge, derived under assumptions of parallel flow will no longer be applicable. The requirements for the flow being "parallel", which underlie the whole theory of varied flow, were formulated by Belanger, as early as 1828. In this case and only in such a case the potential energy, $(d + p/w)$ in the Bernoulli's equation is identical for all and every point of the cross-section (Fig. 5(a) below) and is everywhere equal to the depth y, which accounts for the simple expression of specific energy given by equation (6). The physical characteristic of parallel flow is that the pressures within a cross section are distributed hydrostatically, the pressure diagram being the hydrostatic triangle abc (Fig. 5 (a))

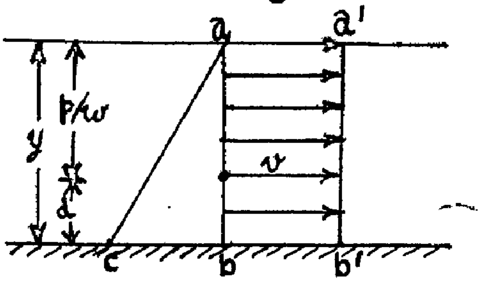


Fig. 5(a)

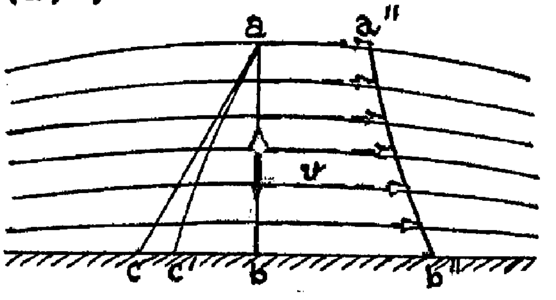


Fig. 5(b)

If the surface and the stream filaments are curved convex, as it happens over a broad-crested weir of relatively short crest, centrifugal forces bearing upward will come into action thus diminishing the hydrostatic pressure. The pressure distribution will now be given by the curve abc' in Fig. 5(b). The diminished pressures caused by the curvature of flow will result in corresponding increase in velocities. Therefore, the ideal rectilinear velocity diagram in Fig. 5(a) is modified into a" b" in Fig. 5(b), with increased velocities. Thus, it is seen, that the discharge will be increased above the value obtained with parallel flow. However, the magnitude of the increased discharge cannot be evaluated theoretically, as the curvilinear stream-line pattern is complex and is not subject to a mathematical analysis.

It is to be noted, in this connection, that the specific energy of flow varies from point to point in curved flow, the average contents per unit weight being given by

$$E = \frac{\int_0^y \left(p/w + d + \frac{v^2}{2g}\right) \quad v.\ dd}{\int_0^y \quad v.\ dd} \quad .. \quad (21)$$

Bakhmeteff, in his discussion on Woodburn's experiments (5) has shown by a theoretical analysis that curvature of stream-lines in water flow will result in the real value of minimum specific energy as well as critical depth being lower than the values in parallel flow. With convex curvature of flow, as on a broad-crested weir, flow takes place with smaller values of specific energy and critical depth than those corresponding to parallel flow.

This fact reconciles the apparent contradiction between the theory, according to which the minimum contents of energy were to be reached at the very end (downstream) of the crest and the observed fact shown, for example, in Woodburn's profiles A, B, C, (5) namely that the critical depth $y_c = \sqrt[3]{q^2/g}$ is found to lie at some distance from the end of the crest. Evidently, $y_c = \sqrt[3]{q^2/g}$ will be formed in the section, where the parallel flow is supposed to end; but that flow will continue with a further loss of energy till the end of crest, at the expense of a further reduction of energy made possible by the fact that the surface becomes convex with the curvature increasing as it approaches the end of the crest.

The above analysis clearly shows that for relatively large heads or relatively short crests, the flow of water will take place with convex surface and stream filaments, on account of which the discharge coefficient, for a given head, will be greater than for crests over which a zone of parallel flow is formed. However, the phenomenon of increased discharge is not subject to a rigorous mathematical analysis.

H. A. Doeringsfield and C. L. Barker (9) attempted to establish a rational discharge formula, involving no empirical constants, by the application of pressure-momentum theory to the broad-crested weir functioning in the modular range. In the resulting formula, the discharge coefficient C was expressed by an expression involving the total head on the crest H, height of crest P, and a ratio of head to flow depth on crest K.

Experiments conducted by Doeringsfield and Barker to evaluate the value of K showed that its value varies in a wide range from 1.73 to 2.12, and an average value of 1.93 was suggested.

It should be noted, in this connection, that the pressure-momentum theory was applied on the basis of certain assumptions.-(a) the flow on the weir crest is parallel (b) the pressure distribution on the upstream face of the weir is hydrostatically distributed and (c) the frictional losses from the upstream end of the crest to the section of parallel flow are negligible.

Except for the first assumption which can be true in the case of relatively longer crests, the other two assumptions are evidently not justified. Hence the results obtained from the application of pressure-momentum theory will not be correct. Further, the range in which the value of K varies is so great, that it is hardly justifiable to accept an average value of 1.93 suggested for application in an accurate expression for the discharge coefficient.

3.3 Effect of Crest Height.-

As the flow of water over a broad-crested weir is essentially a phenomenon associated with the formation of critical depth on the crest, it may be logically assumed that the height of the weir crest has no effect on the discharge coefficient except that, if any, on account of the change of flow pattern and the consequent curvature of stream filaments on the crest which may result in an increase in the value of the coefficient.

3.4 Effect of Slope of Upstream face.-

The effect of the slope of the upstream face (Fig. 1) is a part of the effect of the geometry of the weir, and as such the upstream face slope has considerable effect on the discharge coefficient. The fundamental physical characteristic of the slope of the upstream face of a broad-crested weir is to provide an effective transition to the bottom flow boundary, like in fluming of channels, and thereby reduce the energy losses occuring at the entry of flow over the crest. Thus, a larger effective head is available with sloped upstream face than with a vertical face in which case the loss of head at the contraction of flow is greater. However, the magnitude of slope for the minimum loss of energy should be, logically reasoning, dependent upon the stream-line pattern of flow immediately upstream of the crest, which again is a function of head on the crest h, the height of the weir P and possibly the length of crest L.

3.5 Submerged Flow.-

The functioning of the broad-crested weir is not affected by the increase in the downstream water level upto a certain degree of submergence. In this "modular" range, the discharge coefficient depends upon only the upstream head. Above a critical submergence, this independence of discharge coefficient on the submergence of crest ceases and in this "non-modular" range the discharge depends upon both the upstream and downstream water level. The limit between the above two ranges is the "modularity limit".

On the problem of submerged flow, P.K.Kandaswamy, N. Rajaratnam and T.R. Anand (24) make the following observations. Until the modularity limit occurs, critical depth of flow occurs somewhere on the crest. In fact, it is generally seen that with a steady increase in the tail-water level, the critical depth keeps on moving downstream over the crest up to the modularity limit, when it falls into the relatively deeper downstream portion of flow.

The theoretical considerations mentioned above clearly show that the phenomenon of flow over the broad-crested weir forming a vertical drop fall, is not subject to a mathematical analysis. Hence, the most direct solution for the discharge function must be based upon a combination of dimensional analysis and experiments to evaluate the discharge coefficient in relation to significant dimensionless parameters which describe the flow. Such an approach to the problem is used here.

3.6 Dimensional Analysis.-

The variables describing the flow characteristics of a vertical drop fall consisting of a suppressed level broad-crested weir with a vertical downstream face (see Figs. 2 and 3) may be grouped into the following three classes.-

(a) Those which define the geometry of the broad-crested weir: the length of crest in the direction of flow L, the height of crest P, the width of crest perpendicular to the flow direction B, the radius of rounding of the upstream crest corner r, and the mean height of boundary roughness projections k.

(b) Those which define the geometry of flow and the approach channel: piezometric head of water above the crest h measured in the uniform flow section upstream of the weir, the velocity of flow over the crest v, the width of approach channel B, which is also the same as the width of crest, the height of weir crest P. In submerged flow, piezometric head of tail water above the crest Z, measured in the uniform flow section downstream of the weir also enters the consideration.

(c) Those which define the properties of the fluid flowing over the crest: the mass density ρ, the specific weight w, and the dynamic viscosity μ. Except when the head is very small, the effect of surface tension of water is negligible and has therefore been omitted from consideration.

Thus, a complete statement of the discharge function can be formed as

$$f(L, P, B, r, k, h, v, Z, \rho, w, \mu) = 0 \quad \ldots (22)$$

There are 3 independent fundamental dimensions and 11 physical quantities involved in equation (22). By choosing h, v and w as repeating variables, and combining these in succession with the remaining quantities, 8 dimentionless ratios may be formed.

Thus, equation (22) becomes,

$$f\left(\frac{v}{\sqrt{gh}},\frac{vh\rho}{\mu},\frac{h}{P},\frac{h}{B},\frac{h}{L},\frac{r}{h},\frac{h}{k},\frac{h}{Z}\right)=0 \qquad \text{.. (2.3)}$$

The first of the above ratios is in the form of Froude number. However, the weir is a control structure, and under the assumed conditions, the Froude number is not an independent variable. That is so because v and h cannot be varied independently of each other. Therefore, this characteristic number is best described as a coefficient of discharge and rewritten in the form $c_d = Q/(B\sqrt{g}\,h^{3/2})$. In engineering practice, the gravitational acceleration is usually included in the combined discharge coefficient $C = c_d\sqrt{g}$, which is a more convenient definition for use, though it lacks in dimensional purity, having the dimensions of $\sqrt{g}$.

The second of the ratios in equation (23) is the Reynolds number.

Equation (23) can hence be written as.

$$C = f\left(\frac{h}{L}, \frac{h}{P}, \frac{h}{B}, \frac{h}{k}, \frac{r}{h}, \frac{h}{Z}, R\right) \quad .. \quad (24)$$

In the above equation (24), the coefficient of discharge is shown as a function of a number of independent length ratios involving the head and the physical dimensions of the weir forming the vertical drop fall.

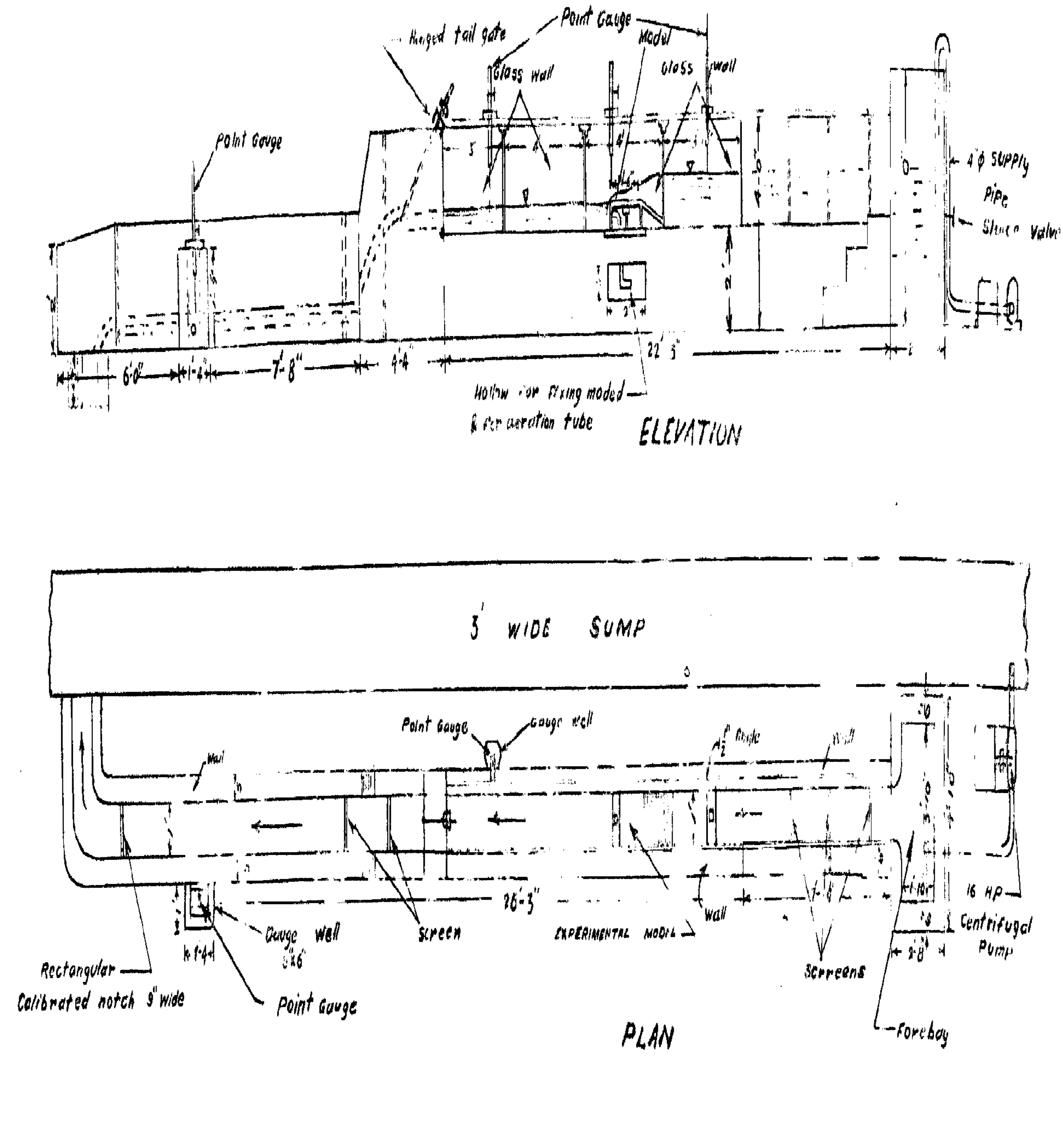
In the following sections of this thesis, the effect of each of these dimensionless parameters upon the discharge coefficient C will be investigated on the basis of original experiments conducted for this purpose, by the author.

4. EXPERIMENTAL EQUIPMENT AND PROCEDURE

FIG. 6.

1 FOOT WIDE FLUME FOR WEIR MODEL

SCALE 1" = 4'



4. EXPERIMENTAL EQUIPMENT AND PROCEDURE

4.1 Experimental Flume.-

The experiments were performed in a 41 ft. long, and 1 foot wide flume shown in Figs. 6, 7 and 8. The flume consists of two portions - the first one 25 ft. long with its bed level and at a height of 2 ft. above the laboratory floor, and in which the experimental models were fixed; and the second 16 ft. long with its bed level ~~and~~ at laboratory floor elevation, and in which the calibrated standard rectangular notch is installed for the measurement of discharge. The front side wall of the flume at the higher elevation is of sheet-glass for a length of 15 ft. to permit visual observations of the water surface profiles and other necessary details. The rear side of the flume is in 5 inch. brick masonry wall smoothly plastered and painted on the inside to render it hydrodynamically smooth. The experimental models were fixed on to a $\frac{1}{4}$ inch. thick brass plate forming the bed of the flume for a length of 2 ft. in the centre of the upper flume and which spans a hollow in the masonry of padestal below provided for fastening the models on to the plate.

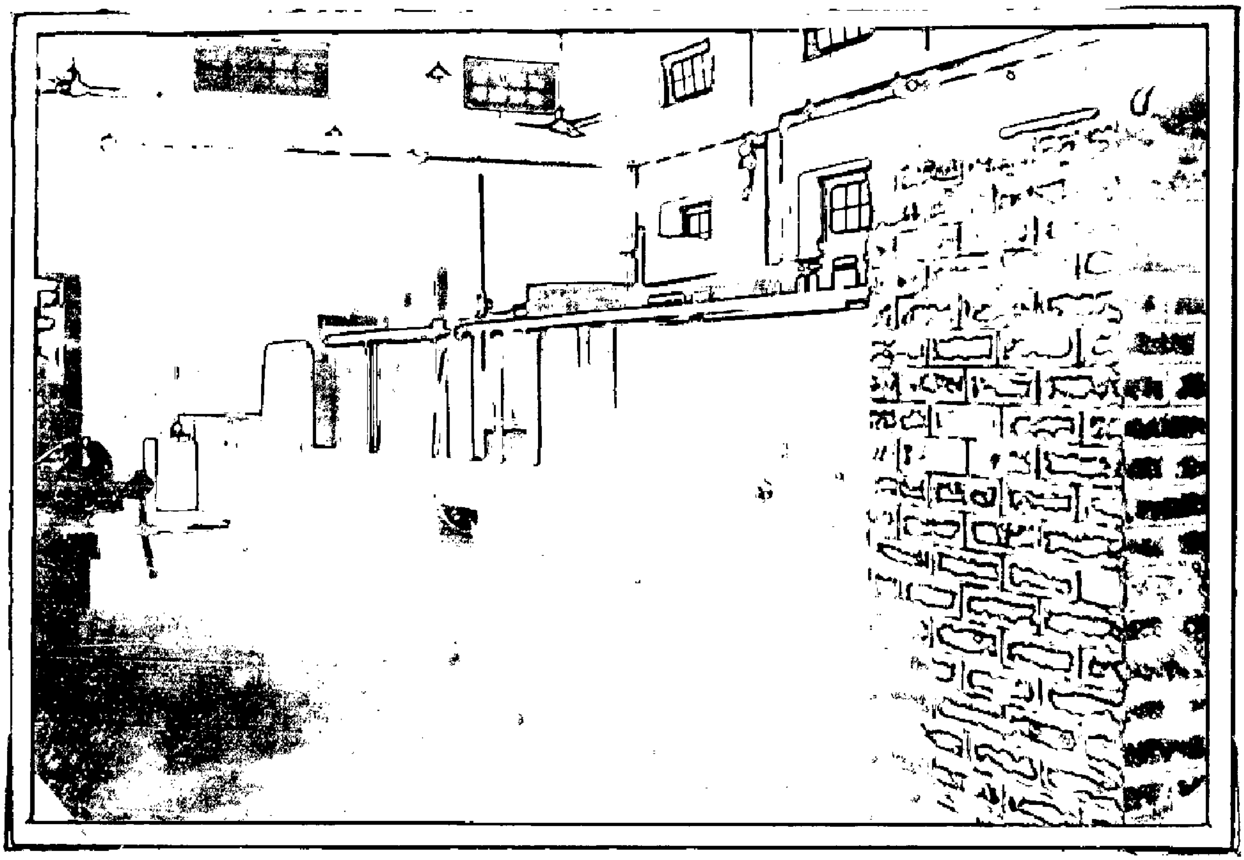


Fig. 7. View of the experimental set-up
from the upstream end.

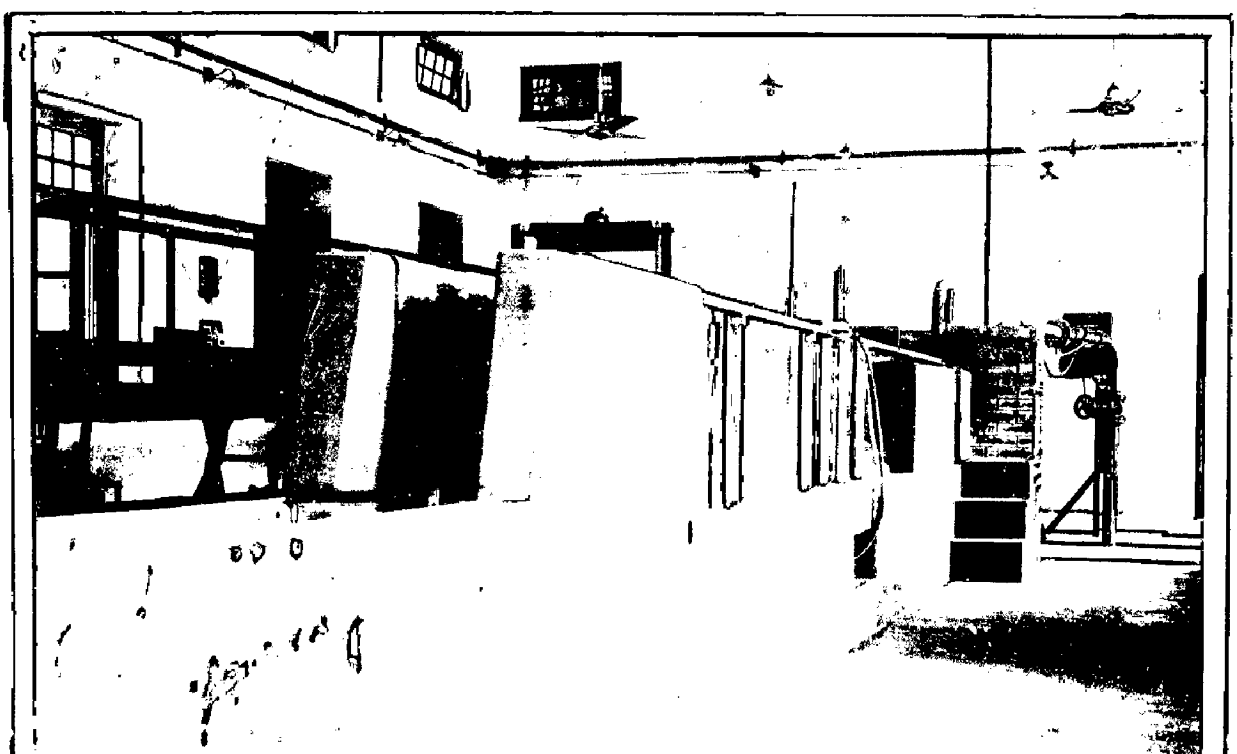


Fig. 8. View of the experimental set-up
from the downstream end.

Flow of water in the flume is created by a closed-circuit system. A centrifugal pump supplies water through a 4 inch diameter pipe to the forebay 3 ft. wide and 2 ft. long at the head of the flume. Discharge was regulated by a sluice valve on the supply line. The water flows from the forebay to the model through three damping screens used to reduce the large scale turbulence of water in the forebay. Thus uniform flow conditions were established upstream of the experimental model.

A tail gate hinged at its bottom and whose position could be adjusted by an attached screw device provided for the adjustment of water level to any desired elevation on the downstream side of the experimental model.

The water in the upper flume dropped into the lower flume over the tail gate and after flowing through the lower flume and over the calibrated notch installed at the tail of the lower flume, was conducted into the sump situated alongside the flume and from which the water was again lifted by the pump. The turbulence of water falling from the tail-gate was reduced by the damping screens in the lower flume, before the flow reached the calibrated notch, so that uniform flow conditions were established in the flow upstream of the notch.

4.2 Experimental Models.-

Experimental models were made of seasoned teakwood planks 1½ inches thick, jointed together to form the required shape. The surface of the models was sandpapered and painted to make it hydrodynamically smooth.

The model was fixed in position by means of M. S. bolts, the top ends of which were fixed to the underside of the model and the bottom threaded ends pass through holes provided in the ¼ inch thick brass plate forming the bed in the test bay. The bolts are fastened down to the brass plate by means of fly nuts on the bolts below the bed plate. The crest of the models was made perfectly horizontal with the help of a sensitive spirit-level. Leakage of water between the side-ends of the models and the walls of the flume was prevented completely by watersealing the joints with plasticene.

Aeration of nappes over the experimental models was provided by means of three holes $\frac{1}{2}$ inch diameter drilled through the downstream face of models, in which were fitted snugly the ends of three aeration tubes supplying air at atmospheric pressure. The tubes passed through the hollow of the model and holes drilled in the brass plate forming the bed of flume and then to the outside through the hollow in the masonry padestal below.

The models tested in this investigation had three heights - 1.0 ft. 0.75 ft. and 0.50 ft. The lengths of the crests were 1.25 ft., 1.00 ft. and 0.333 ft. The slopes of the upstream face ranged from vertical to 1 in 4. The experiments were made with upstream corner of crest sharp as well as rounded to radii of $\frac{1}{2}$, 1, $1\frac{1}{2}$, 2 and $2\frac{1}{2}$ inches.

4.3 Experimental Observations.-

Before recording observations, each experimental run was begun by establishing the desired head on the weir by controlling the discharge gradually with the help of the sluice-valve on the supply line. The establishment of flow regimen was verified after running the water in the flume for about 15 to 20 minutes for each experimental run. In the experiments on the submerged crest, submergence to the required degree was accomplished by adjusting the tail-gate provided at the downstream end of the flume. Thus, in the range of experiments above the modularity limit, both the sluice-valve on the supply line and the tail-gate had to be operated simultaneously to establish the required head and tail-water elevations. This involved a trial and error procedure of operation of the above said controls requiring, in most of the runs, 30 to 40 minutes' time in each run for the flow regimen to be established.

After the flow regimen was established, observations on the instruments were made and recorded.

The head of water upstream of the crest was measured by means of a point-gauge reading upto 0.001 ft., located centrally in the flume and situated at a distance of 4 ft. upstream of the experimental model. The head on the experimental model was set at ten stages in increments of 0.05 ft. in the range of 0.05 ft. to 0.50 ft. The observations of other variables were made after establishing the head in one of the stages, in each experimental run.

A point gauge, reading upto 0.001 ft., mounted on a platform supported on the rails at the top of the walls of the flume, and set in line with the downstream end of the crest of the experimental model enabled the observation of the depth of flow at the said section.

The tail-water elevation was measured by means of a point-gauge reading upto 0.001 ft., installed in a stilling well situated at a distance of 7 ft. from the downstream end of the crest, and uniform flow was found to be practically established, at that section in the flume.

The discharge of water was measured with the help of a sharp-edged rectangular notch 9 inches wide and with its crest 4 inches above the bed of the flume at the laboratory floor level. Before the experiments were started, the notch was calibrated by volumetric measurement of water for each head over its crest up to a head of 0.55 ft. and discharge of 1.1 cubic feet per second. The head of water above the crest of the calibrated notch was measured with the help of a point gauge reading upto 0.0005 ft. mounted in a stilling well situated at a distance of 4 ft. upstream of the notch and 10 ft. below the outfall of water over the tail-gate of the experimental flume. The water approaches this section with practically uniform flow conditions after passing through two damping screens in the lower flume forming the approach channel for the calibrated notch.

The surface profile of flow over the crest of models in the experiments were observed with the help of a point-gauge reading upto 0.001 ft., mounted on a trolley moving on rails provided on the side walls of the flume. These readings were plotted on a graph paper and the surface profiles were drawn.

4.4 Scope of the experiments.-

The experiments in the investigation involved in all 60 series, each series consisting of 10 to 14 runs.

The study of discharge characteristics with free overfall covered 48 series of experiments. Submergence studies were made in 12 series of experiments. The free overfall experiments had the upstream crest corner sharp, as well as rounded to radii of ½, 1, 1½, 2 and 2½ inches. The slopes of the upstream face in the different experiments were vertical, 1 in $\frac{1}{4}$, 1 in 1/3, 1 in $\frac{1}{2}$, 1 in 1, 1 in 2, 1 in 3 and 1 in 4, covering the entire range obtaining in actual structures. The models were tested with three different lengths - 0.333 ft., 1.00 ft., and 1.25 ft. The heights of crests were 0.50 ft., 0.75 ft. and 1.00 ft.

Submergence studies were made by 4 series of experiments on a model 1.00 ft. long, 1.00 ft. high with sharp upstream corner, 4 series of experiments on the above model with the upstream crest corner rounded to 2 inches radius and 4 series of experiments on a model 1.00 ft. long, 0.75 ft. high with upstream crest corner rounded to 2" radius. The heads in the four runs of each series were 0.20 ft., 0.30 ft., ~~0.30 ft.~~, 0.40 ft. and 0.50 ft.

Surface profiles of flow over the crests of the experimental models were observed for all the runs with free overfall of nappe and for heads from 0.20 ft. to 0.50 ft.

4.5 <u>Accuracy of Observations</u>.-

In view of the shortcomings of the experimental arrangement, a consideration of the accuracy of some of the observations is necessary.

Point-gauge readings were read to the nearest 0.001 ft. except those in the case of point-gauge at the calibrated notch, where they were read to the nearest 0.0065 ft. At higher discharges, the wavy water surface made it difficult to take the readings accurately. However, then the depths were relatively greater and by taking the readings twice or thrice, satisfactory mean values, as evidenced by the fairly consistent data, were obtained. For smaller discharges and the corresponding smaller heads of water, errors in the readings were reduced by the smoothness of the water surface.

Despite all the care taken, the thin band of plasticene clay, which was used as a water-sealing compound and placed between the ends of the models and the side walls of the flume, was projecting slightly above the surface of models and was causing a slight distortion of the flow pattern. However, this was local at the side ends of the models and does not seem to have had any effect on the discharge.

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5. EXPERIMENTAL RESULTS AND ANALYSIS

5. EXPERIMENTAL RESULTS AND ANALYSIS

5.1 General.-

Model experiments were made with a view to study the physical characteristics of flow and to evaluate the effects of the dimensionless parameters describing the flow, as analysed in chapter 3, on the discharge coefficient of vertical drop falls, both in the free flow and submerged flow conditions. The results of these experiments are presented in the tables of Appendix II, arranged in the following order:

Free Flow.-

Tables 1 to 10:- Experiments on vertical drop falls with sharp U/S corner and vertical U/S face with lengths of crest L = 1.25', 1.00', 0.333' and heights of crest P = 1.00', 0.75', 0.50' .

Tables 11 to 25:- Experiments on vertical drop falls with sharp U/S corner and sloped U/S faces varying from 1 in ¼ to 1 in 4, with lengths of crest L = 1.00', 0.333' and height of crests P = 0.50' .

Tables 26 to 48:- Experiments on vertical drop falls with vertical U/S faces and upstream crest.

corner rounded with radii from ½" to 2½" and length of crests L = 1.25', 1.00' and heights of crests P = 1.00',0.75',0.50' .

<u>Submerged Flow</u>:-

Tables 49 to 52:- Experiments on vertical drop falls with sharp U/S corner and vertical U/S faces, length of crest L = 1.00', height of crest L = 1.00 and heads on crest h = 0.20', 0.30', 0.40', 0.50', throughout the submergence range.

Tables 53 to 60:- Experiments on vertical drop falls with upstream corner rounded to 2" radius, vertical U/S faces, length of crests L = 1.00', height of crests P = 1.00', 0.75' and heads on crest h = 0.20', 0.30', 0.40', 0.50', throughout the submergence range.

The above tables show the observed values of the variables as well as the discharge coefficients C computed from the formula $C = Q/Bh^{3/2}$. In these experiments, the width of the crests was 0.985 ft. C_{s1} and C_r represent the discharge coefficients of crests with sloped U/S faces and rounded upstream corners.

respectively, the formula applied for evaluating these being the same as for C mentioned above.

In appendix III are presented the observed surface profiles of the flow over the crests of vertical drop falls functioning under free flow conditions. These cover experiments with vertical upstream faces and upstream corners sharp as well as rounded to a radius of 2 inches, length of crests L = 1.25', 1.00', 0.333' and height of crests P = 1.00', 0.75', 0.50' .

The results obtained from these experiments have provided sufficient data to illustrate several significant points regarding the flow characteristics of vertical drop falls.

5.2 <u>External Flow Pattern</u>.-

Before proceeding to an analysis of the variation of the discharge coefficient as outlined in the previous chapter, it will be useful to consider certain significant aspects of the external flow pattern over the vertical drop falls, as revealed by the observed profiles of the water surface on the crests.

<u>Nappe form</u>:- In all the series of experiments involving free discharge, the nappes were aerated, in order to facilitate the determination of the flow depth at the

downstream end of the crests formed with free overfall, which has certain significance. However, for higher heads and discharges, the water surface in the downstream channel was high enough to submerge the holes in the downstream face of models provided for aeration of the nappes. Above this range, the flow depths at the downstream end of the crests were not observed.

The nappe forms observed in these experiments were in conformity with Bazin's observations (1,2):

(a) "Adhering nappe":- For very low heads, the nappe adheres to the downstream face of the fall. However, these heads were below 0.05 ft. which was the minimum value in the experimental range.

(b) "Free nappe": For a considerable range of heads and discharges, when the nappes were aerated as described in Chapter 4, free nappes were obtained with the space between the downstream face of the fall and the jet of falling water filled with air at atmospheric pressure. This type is illustrated in Figs. 11, 12 and 26.

(c) "Depressed nappe": For higher heads and discharges, on account of the higher tail-water levels, the nappes were depressed. That is, a part of the air under the nappe is entrained by the moving fluid

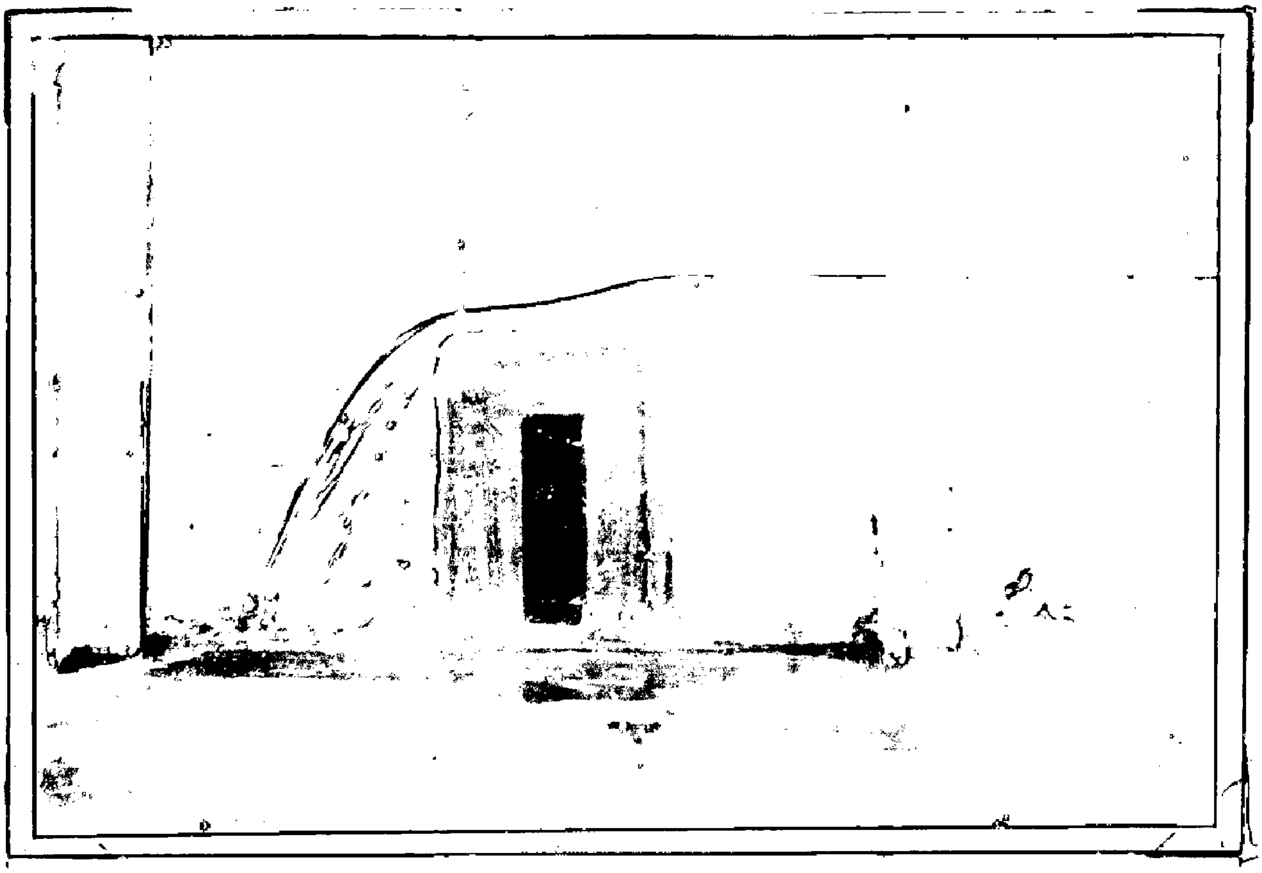


Fig.11. Free flow over crest 0.333 ft. long and 0.50 ft. high, under a head of 0.10 ft.

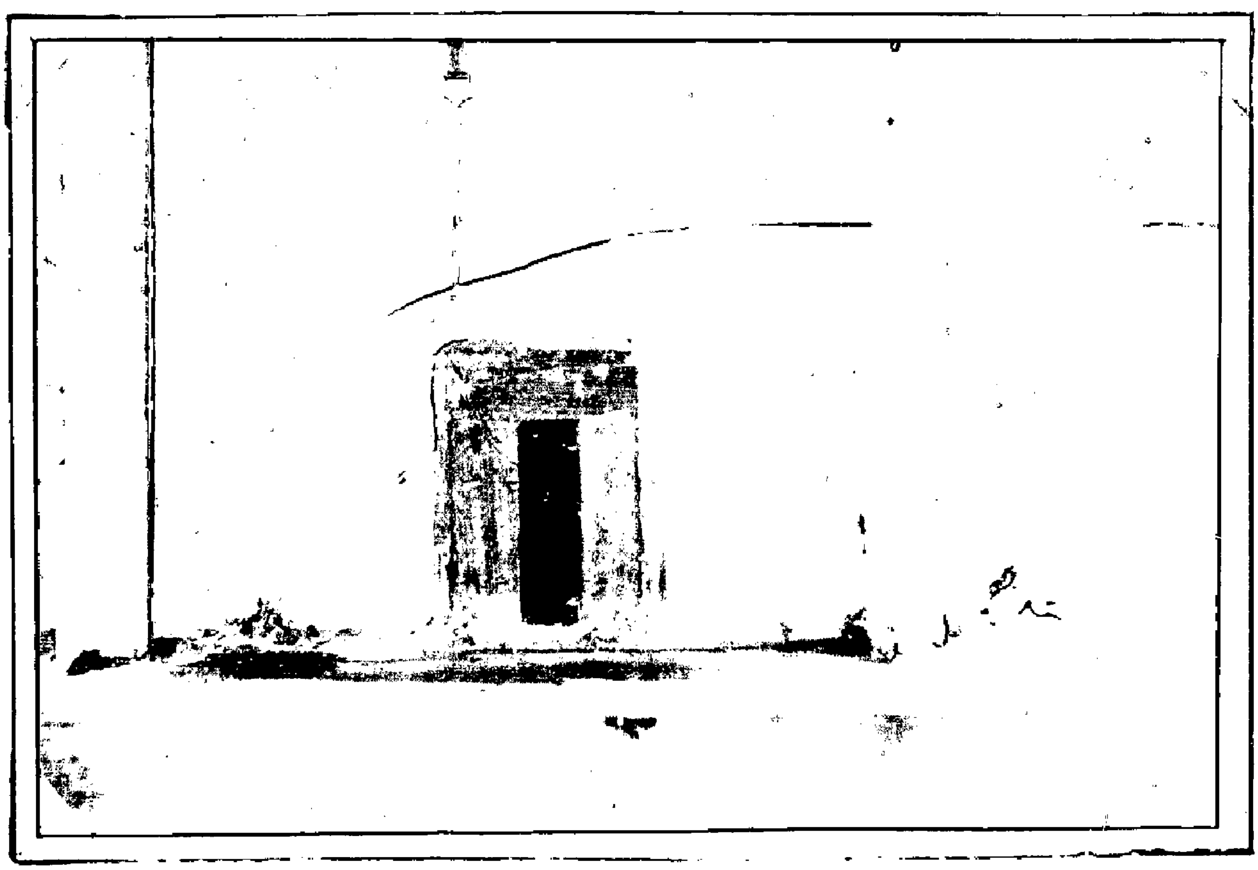


Fig.12. Free flow over crest 0.333 ft. long and 0.50 ft. high, under a head of 0.20 ft.

and replaced by water, leaving the remaining air at a pressure less than atmospheric.

(d) "Nappe wetted underneath": For still higher heads and discharges with correspondingly higher stages of tail-water downstream of the models, all the air underneath the nappe is entrained and replaced by water and the space formerly occupied by air is filled with eddying fluid. This type of nappe is illustrated in Fig. 13.

The experiments were performed with the maximum value of $h/L = 1.35$ only, due to limitations in the experimental setup. Therefore, "free" nappes, free altogether of the flat crest and in contact with only the upstream face of the weir forming the fall, were not observed in these experiments.

It was of interest to observe, if any changes in the discharge coefficient and flow pattern would occur due to these different conditions of the nappe. Therefore, one series (Table 5) of experiments was conducted by varying the condition of nappe on a crest of length 1 ft. and height 1 ft., by suitably altering the upstream head h and the tail water level, and the surface profiles for these conditions were observed (see Fig.32). It was seen that the effect of the "depressed" and "wetted underneath" nappe forms on the flow surface

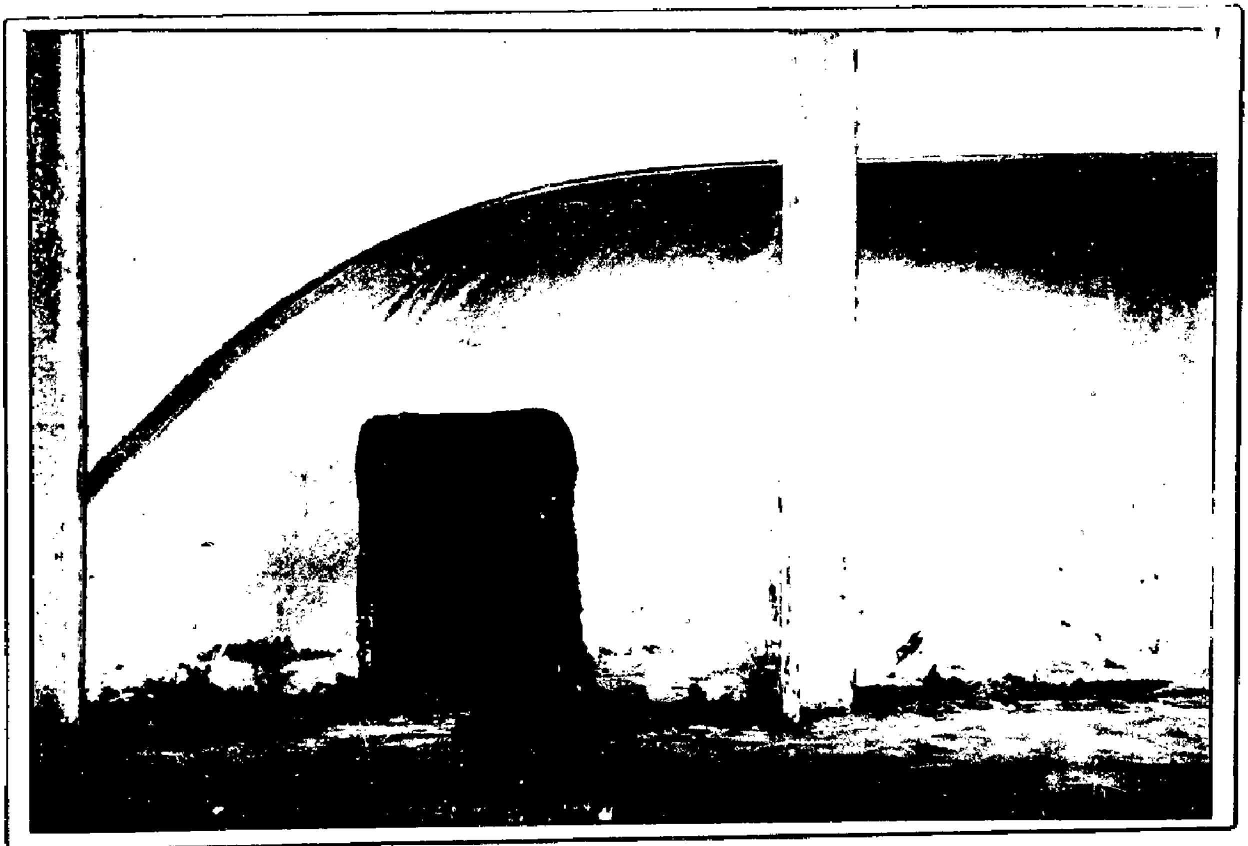


Fig. 13. Flow with nappe wetted underneath over crest 0.333 ft. long and 0.50 ft. high, under a head of 0.50 ft.

was purely local near the downstream end of the crest for a short length of crest. The surface profile for "depressed" nappe was slightly below that of the free aerated nappe; and the surface profile of the "wetted underneath" nappe was slightly below that of the depressed nappe. Further, the discharge coefficient of the weir forming the fall, was observed to have undergone no change in value, as the nappe form was altered through the above mentioned three types, as may be seen from their values in Tables 4 and 5. This, again, is in accord with Bazin's experiments (1).

This aspect of the flow pattern is of significance, as the nappes under field conditions are, usually, not aerated. Therefore precise discharge determinations in the field, on the basis of laboratory data, will not be affected.

However, it should be expected that if the change of nappe from one form to another can bring about a considerable change in the flow pattern and corresponding change in the curvatures of stream filaments, a variation, though usually negligible, may take place in the value of the discharge coefficient.

Surface Profile of Flow over the Crest.-

Some typical surface profiles of flow over crest are presented in the appendix III .

On the basis of the observed surface profiles, a dimensionless water-surface profile for flow over a vertical drop fall with a level crest and vertical upstream face is plotted as shown in Figs. 9 and 10. Fig. 9 shows the profile for a crest with sharp upstream corner, while Fig. 10 shows the profile for a crest with well rounded upstream corner. These profiles have been made dimensionless, for the sake of generality of study, by dividing both the distance x along the crest measured from its upstream corner and the depth of flow y measured as the elevation of water surface at the concerned section above the crest, by the piezometric head on the crest h. The right hand ends of these curves represent the dimensionless flow depth at the downstream end of the crest. Some points of significance are observed from these profiles.

The profiles for the crest with sharp upstream corner (Fig. 9) exhibit strikingly similar configuration characteristics. In the proximity of the upstream crest corner, where the flow accelerates, the flow

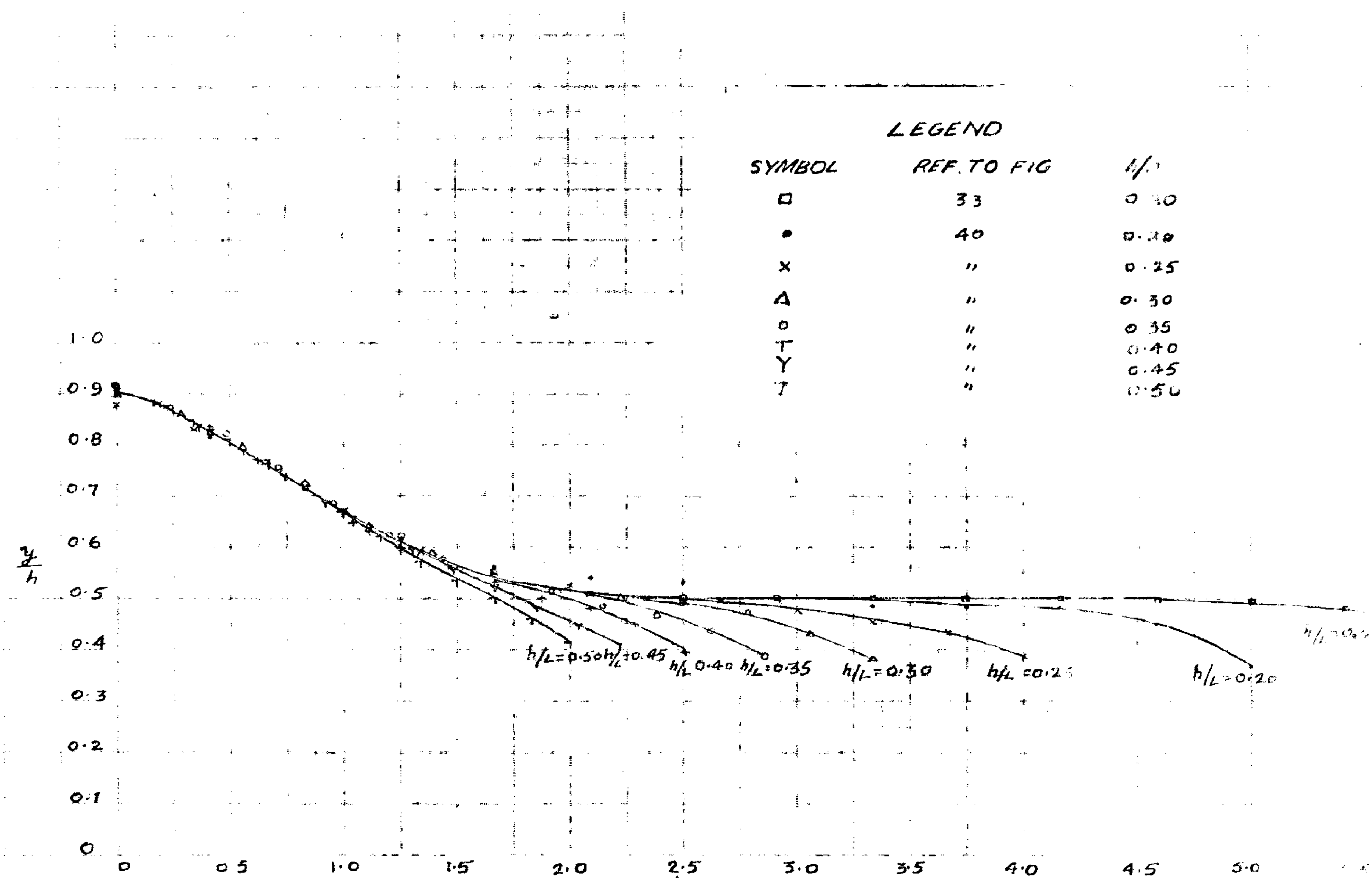
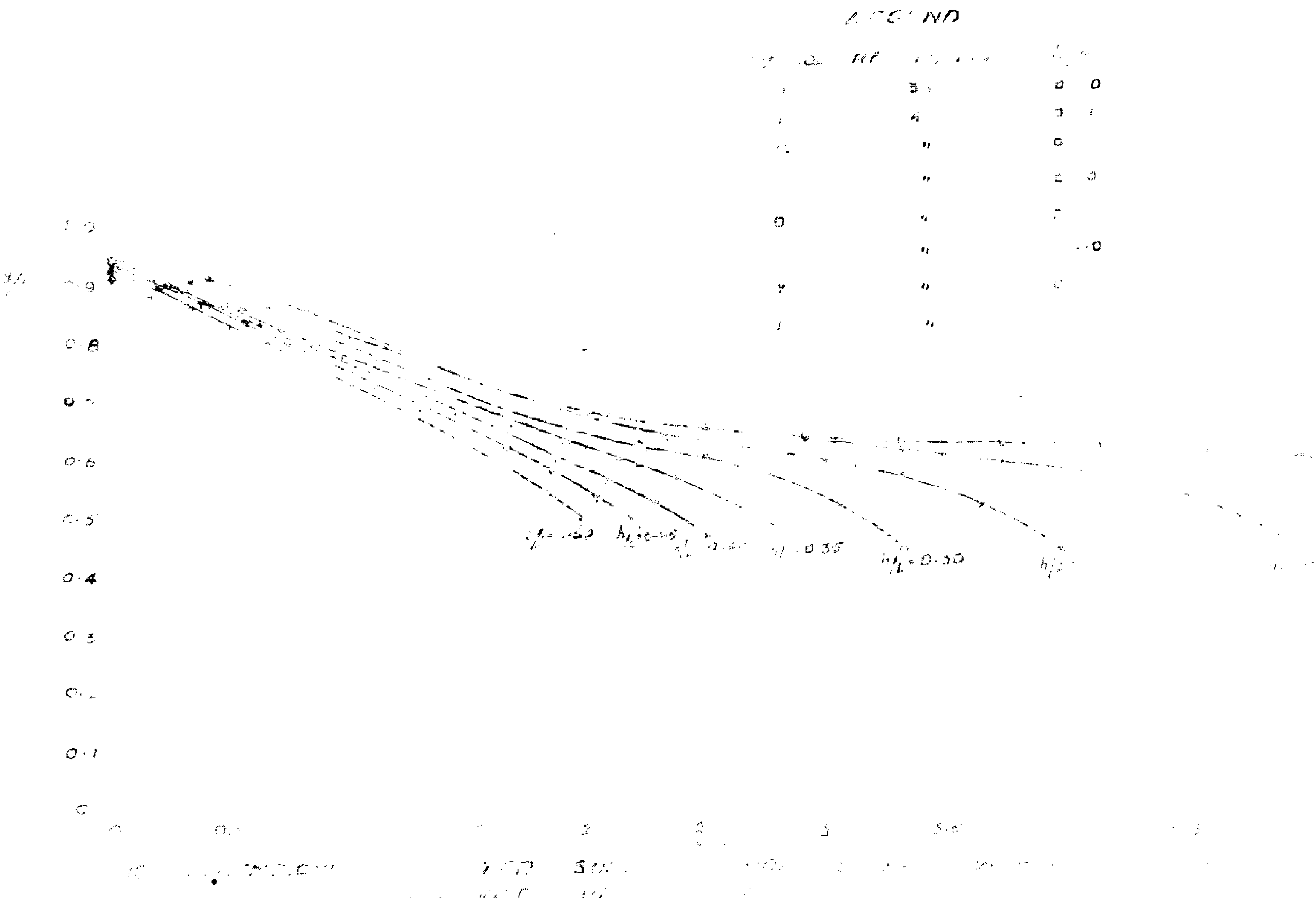


FIG :- 9. DIMENSIONLESS WATER SURFACE PROFILES FOR VERTICAL DROP FALLS WITH SHARP U/S CORNER depth to head ratio y/h decreases rapidly, its magnitude being practically constant for all the ratios of h/P . Separation of flow occurs at the upstream crest corner and the control section lies in the body of the fluid above this separation zone. The flow becomes supercritical beyond this control section. If the crest were rough enough to produce deceleration of the flow, a slowly rising water surface profile over the approximately parallel flow region of the crest should be expected.

Further, when the crest is relatively long, the flow depth must eventually approach, again, the critical depth on the crest as the flow continues to decelerate with increased distance along the crest. At this critical value, the specific head will be a minimum at which this discharge rate is physically possible. A further energy loss must cause an adjustment of depth by the formation of a standing-wave pattern of flow, which was observed in the tests, for very small heads.

It will be noticed from Fig. 9, that some distance downstream from the crest corner, the profiles for different values of h/P tend to superimpose, having an average value of $y/h = 0.50$ with parallel flow taking


0.8
0.6
0.5
0.4
0.1
0
h/L = 0.50

place under supercritical conditions.

The effect of rounding the upstream crest corner on the water surface may be observed by a comparison of Fig. 10 with Fig. 9. The profiles for both the sharp and rounded upstream corner have the same general shape. However, the profiles for the crest with rounded upstream corner of Fig. 10 are displaced vertically upward. Assuming that the flow is critical at a point just upstream from the drawdown zone over the downstream end of the crest for crests having a zone of parallel flow on it, the critical depth y_c for weirs with rounded upstream corner may be observed to be around 0.64 h, as the average y/h value in the region of parallel flow (Fig. 10) is around 0.64. This value is in general accord with the theoretical equation 13 for K = y/h derived earlier in Chapter 3. The difference in the actual value of 0.64 and the theoretical value of 2/3 may be ascribed to the energy loss due to the resistance offered by the crest surface.

Classification of broad-crested weirs.-

On a study of the pattern of the surface profiles of flow observed in the tests, the broad-crested weirs in general, may be classified according to the flow characteristics.

It is stated earlier, that when the crest length is relatively long, standing-wave type of flow occurs on the crest. This type of flow configuration was observed to occur for heads upto about 0.075 ft. on the 1.25 ft. long crests and 0.06 ft. on 1 ft. long crests. That is, the standing wave type of flow takes place for h/L ratios up to about 0.06.

If, on the contrary, the broad crest of the vertical drop fall is relatively short or the head on the crest is relatively large, the flow over the crest of the fall will be entirely curvilinear, as may be seen in the Figs. 9, 10, 12, 13, 20, 21, 26 and surface profiles of Appendix III. It is further observed from these profiles that this curvilinear profile is somewhat lower vertically than that for the crest of length large enough to permit a zone of approximately parallel flow to form. This entirely curvilinear profile first appears when the h/L ratio becomes about 0.35 for crests with sharp upstream corner and when the h/L ratio becomes about 0.25 for crests with upstream corner well rounded. This value varies for small degrees of rounding of the upstream crest corner. The above mentioned bounding figures are not precise and vary slightly with the height of the crests; or in other words with h/P values, As the h/L value exceeds the said values, it is evident that the water surface profile must eventually approach that of the upper water surface of a sharp-edged weir.

Thus, broad-crested weirs forming the main element of a vertical drop fall may be defined with respect to the flow pattern and classified as follows:-

(a) Short weir ; with entirely curvilinear flow on the crest

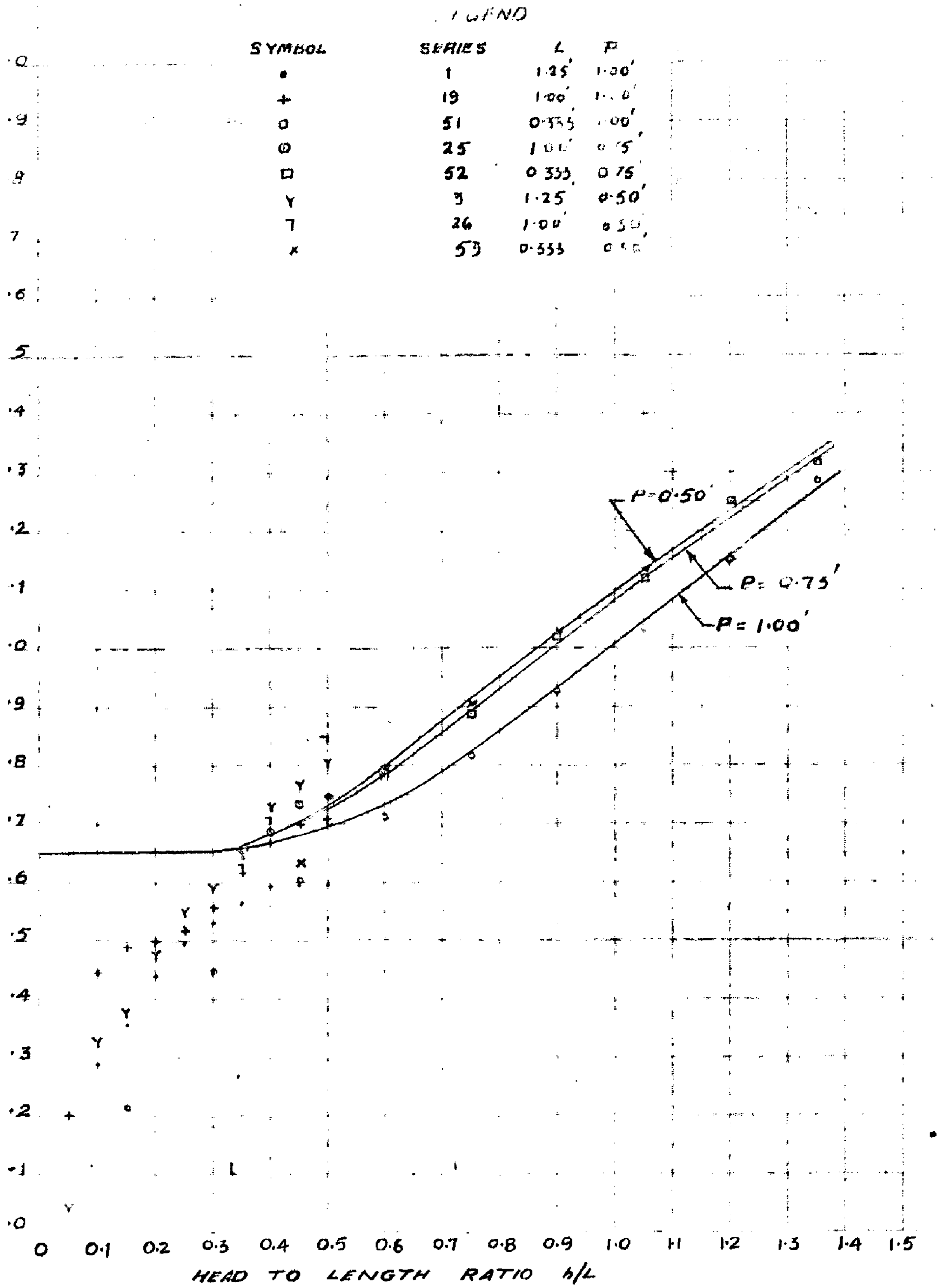
(b) Normal weir; with curvilinear flow zones at the beginning and at the end of the weir crest, and a central zone of essentially parallel flow. and

(c) Long weir; with a standing wave pattern of flow on the crest.

The approximate limits for these weirs are: the weir is short if h/L ratio is greater than 0.35 for weirs with sharp U/S corner and 0.25 for weirs with well rounded U/S corner; the weir is long if h/L ratio is less than about 0.06; the weir is normal between these two limits.

Flow depth at the downstream end of crest.-

It is quite evident that flow over vertical drop falls will be critical at a point near the downstream end of the crest at the transition between a normal and a long crest for crests with sharp upstream corner and at some point in the zone of parallel flow on the crest for normal and long crests with well rounded upstream corner. Hunter Rouse (6) established that the ratio of depth at a free overfall to the critical depth will be 0.715. Then, the critical flow depth on vertical drop falls may be obtained by dividing the depth at the downstream end of crest by 0.715. The depth at the downstream end of normal and long crests are observed (see Fig. 9, 10 and surface profiles in Appendix III) to be about 0.39 h for crests with sharp upstream corner and about 0.46 h for crests with well rounded upstream corner. This, therefore, means that the critical flow depth in terms of the head h is 0.545 h for crests with sharp upstream corner and about 0.643 h for crests with well-rounded upstream corner. This value for crests with well rounded upstream crest corner is in good agreement with the maximum depth of the profiles in the zone of parallel flow, in Fig. 10. Critical depth was not formed, in the range of experiments made, near the downstream end of crest, for crests with sharp upstream corner.



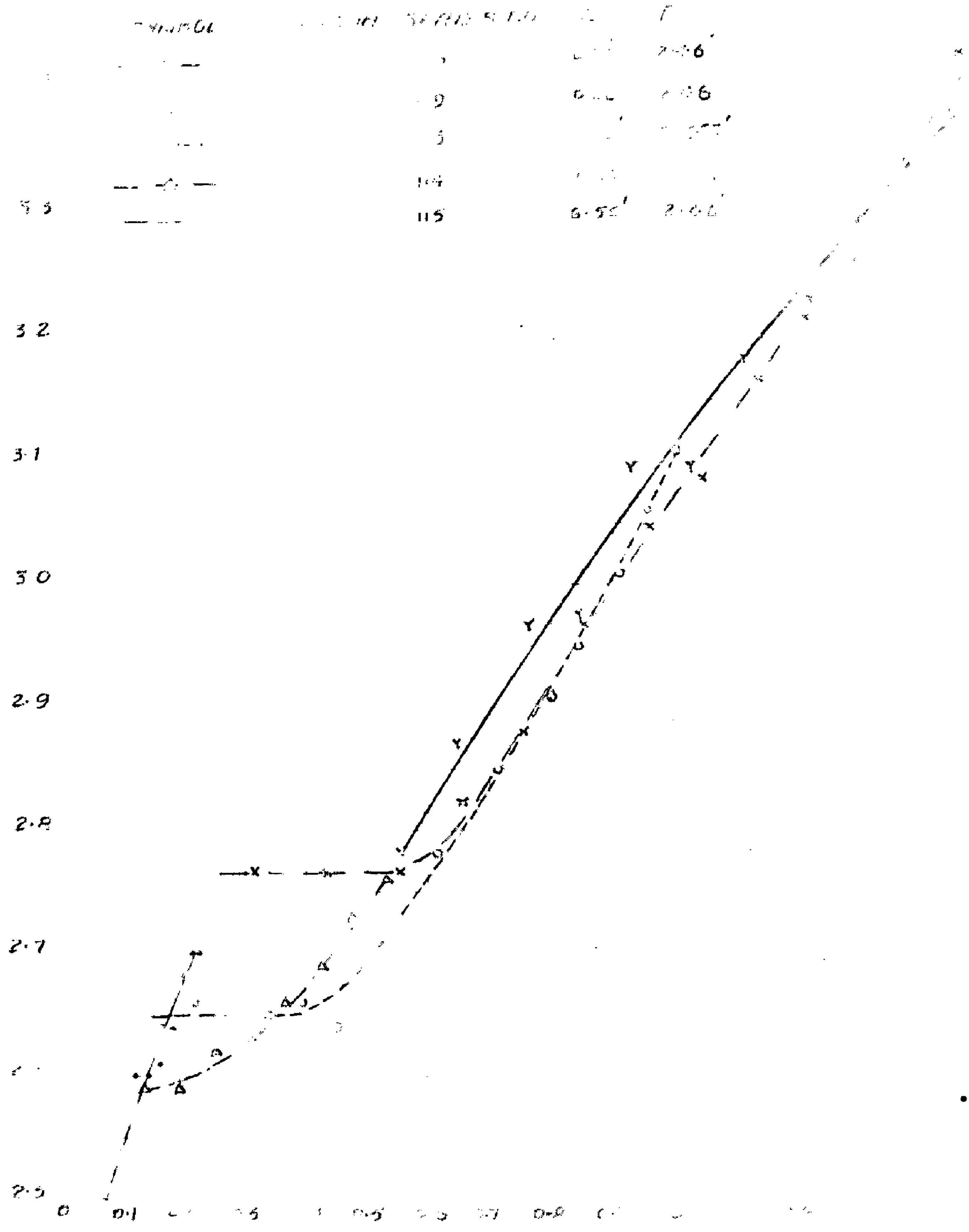
DISCHARGE COEFFICIENT FOR VERTICAL DROP FALLS WITH VERTICAL FACES AND SHARP UPSTREAM CORNER.

5.3 <u>Effect of Crest Length on the Discharge Coefficient</u>.-

The dimensionless parameter most important in fixing the value of the discharge coefficient C of vertical drop falls is the head to length ratio h/L. The significance of this parameter is in the fact that it governs to a major extent, the flow pattern on the crest. As already remarked, curvilinearity of flow sets in from h/L ratio of 0.35 for crests with sharp upstream corner and 0.25 for crests with well rounded upstream corner. The curvilinearity of the flow continues to increase in degree with still higher values of h/L . It was shown in article 3.2, that the discharge coefficient will be increased on account of convex curvature of stream filaments, which are obtained in the flow over a vertical drop fall with large values of h/L . Thus, it should be expected that with increasing values of h/L, the discharge coefficient goes on increasing for the range of x functioning of the vertical drop fall as short weir.

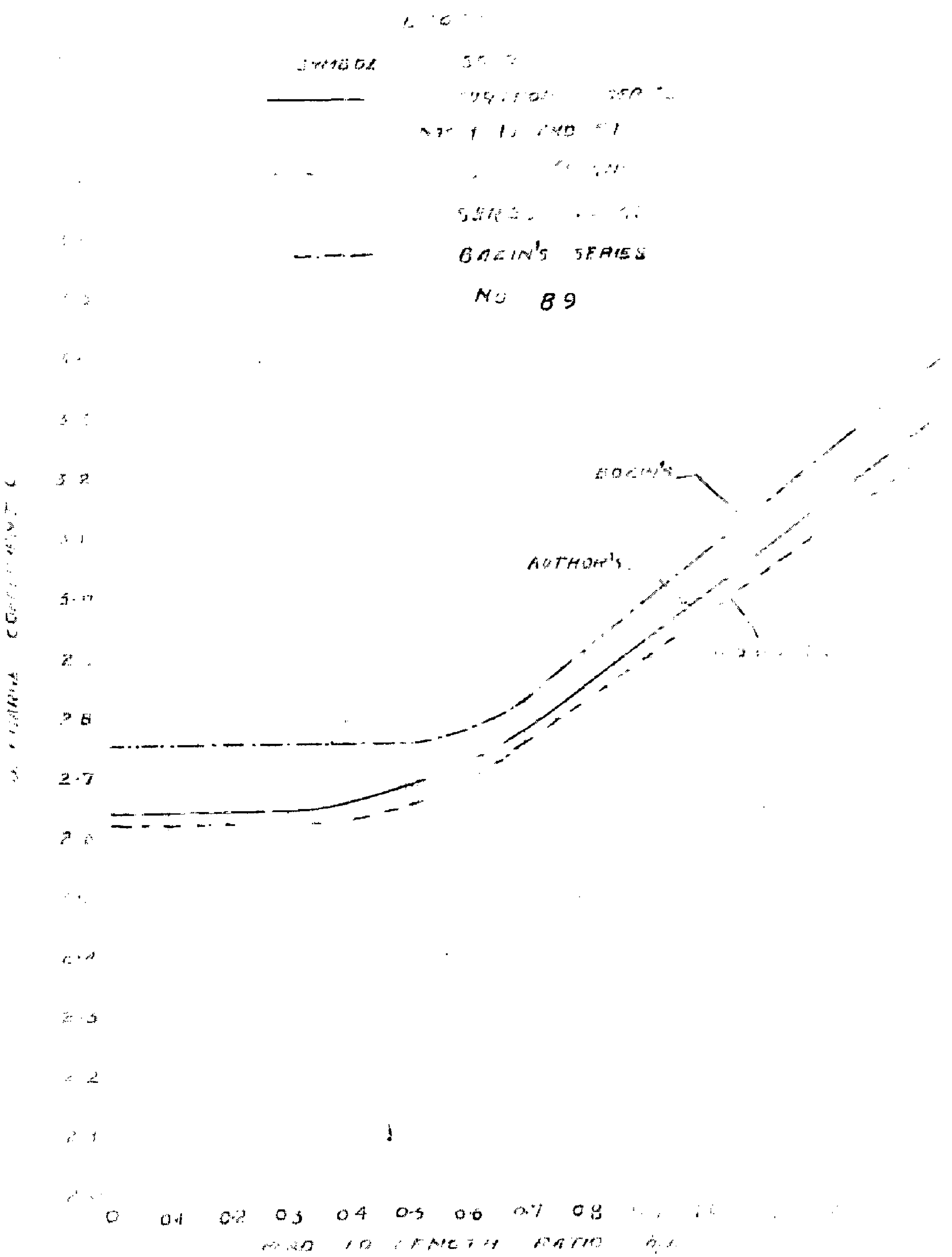
The discharge coefficient C is shown as a function of h/L ratio in figure 14 for vertical drop falls with vertical faces and sharp upstream crest corner, in figures 16 and 18 for vertical drop falls with vertical faces and well rounded upstream crest corner and in figures 22 and 23 for vertical drop falls with sloped upstream face and sharp upstream crest corner. In all these figures, the general trend of increasing values of the discharge coefficient with increasing values of h/L ratio may be observed.

It may be noted that in figures 14 , there is a wide scatter of the experimental points for h/L upto about 0.50. These points represent data obtained from experiments involving a sufficiently large range of values of length and height of crests. The scatter of the points may be ascribed to the scale effects of the models tested. If a mean curve were to be drawn through these points, that would show increasing trend of discharge coefficient with h/L ratio in this initial range of h/L upto 0.50. However, the following consideration will show the inconsistency of this trend. The normal crest (h/L less than 0.35) profiles show almost complete geometric similarity (see figs. 9 and profiles of Appendix III), suggesting clearly that the discharge coefficient should be constant in the range of h/L = 0.35 for the normal weir. This appears even more reasonable when one considers that the control


3.2
3.1
3.0
2.9
2.8
2.7
2.5
0
0.1
115

section is just downstream from the upstream crest corner and over the separation zone, and that, because of this fact, the crest length does not affect the discharge. Thus, it may be seen that the discharge coefficient should remain constant for h/L ratios upto 0.35 . For the above said reasons, the curve of discharge coefficient is shown in the figure 14 as a straight line representing a constant value and passing through the experimental point of maximum value corresponding to h/L = 0.35. This constant value may be seen to be 2.65, which is in general conformity with Bazin's results (1, 2) and Woodburn's results (5) of tests on broad-crested weirs of large crest lengths in which the scale effects are a minimum on account of large absolute values of heads on crest. A short transition connects this constant value portion of the curve with the remainder of the curve .

The curve representing the variation of the discharge coefficient C with h/t on falls with sharp upstream crests is compared with curves plotted from data of Bazin's experiments and U.S.G.S.(Cornell University) experiments in Figure 25.



[illegible] DISCHARGE COEFFICIENT [illegible] VERTICAL [illegible] FALLS [illegible]
[illegible] FACE AND SHARP UPSTREAM CORNER

Figure 17 shows the curve representing the variation of the discharge coefficient C with h/L for vertical drop falls with well rounded upstream corner, compared with that from Bazin's data.

5.4 Effect of crest Height on the Discharge Coefficient.-

In order to determine the effect of the crest height P on the discharge coefficient of a vertical drop fall, experiments were made with three different heights of crests.- 1.00 ft., 0.75 and 0.50 ft. These tests covered three different lengths of crests - 1.25 ft., 1.00 ft. and 0.50 ft. The results of these tests are given in the tables of Appendix II.

The results of these tests indicate that the effect of the height of the crest on the discharge coefficient is very slight for vertical drop falls with sharp upstream corner and nil for vertical drop falls with well rounded upstream corner. The discharge coefficient curves obtained for the three crest heights of 1.00 ft., 0.75 ft. and 0.50 ft. for vertical drop falls with sharp upstream corner are shown in Fig. 14. It is quite evident from the results in tables 29, 34, 39, 43, 45 and 47, that the height of crest P has no effect on the discharge coefficient of vertical drop falls with well rounded upstream corner.

The very slight variation in the discharge coefficient with the height of the crest P for vertical drop falls with sharp upstream crest corner may perhaps be explained by the fact that while the surface profile of flow over the crest near the upstream crest corner is not affected by the height of crest as evidenced by the dimensionless surface profiles in figure 9, the profile of the separating fluid surface at the upstream crest corner is affected by the height of crest, thus creating a slight variation in the flow area at this control section.

As the flow depth at the critical flow section is not affected by the crest height P for vertical drop falls with well rounded upstream corner, as evidenced by figure 10, the discharge coefficient C is not affected by the crest height.

Thus, while the effect of h/P ratio is negligibly small for vertical drop falls with sharp upstream crest corner, it is absent for vertical drop falls with well rounded crest corner. This is very significant from the point of view of selecting the proper shape of crest for the design of vertical drop falls and such other structures.

5.5 Effect of the Shape ratio on the discharge Coefficient.-

The ratio h/B in the equation 24 of article 3.6 is a measure of the shape of the discharging water stream. This parameter is seen to be of no significance, as shown by Blackwell's experiments on broad-crested weirs with widths B of 3 ft., 6 ft. and 10 ft. No tests were made in the present investigation to study this effect, all the tests having been done in the same flume of 1 foot width. Even from a theoretical consideration, the only effect of the width B of the vertical drop fall is in the reduction in the effective width of flow channel by an amount equal to the displacement thickness δ of the boundary layer formed along the side walls. This reduction in the effective width, however, is of negligible magnitude in all of the practical structures on the relatively wide channels.

5.6 Effect of Rounding the Upstream Crest Corner on the Discharge Coefficient.-

The rounding of the upstream crest corner of a vertical drop fall causes the removal of the flow separation at the corner, which is the primary source of energy loss. Hence, the rounding of the upstream crest corner results in an increase in the discharge coefficient above that with a sharp corner. This variation in the discharge coefficient is signified by the r/h ratio in the discharge equation 24 of article 3.6 .

Tests were made on crest lengths of 1.25 ft. and 1.00 ft., with the radius of the rounding of the upstream crest corner of ½", 1", 1½", 2" and 2½" . The results of these tests are shown in the tables 26 to 48 of Appendix II.

In figures 16 and 18 are shown the curves representing the variation of the discharge coefficient C_r with h/L for different degrees of rounding represented by r/h values of 0, 0.08, 0.16, 0.25, 0.33 and 0.42 from the data obtained from the experiments. It may be observed that the coefficient of discharge C_r increases with r/h ratio upto a value of r/h = 0.33, and thereafter remains constant for higher values of r/h. Further, the rate of increase in C_r decreases, generally, with increasing values of r/h, finally coming to zero for r/h = 0.33. The values of C_r for r/h = 0.42 do not differ from those of C_r for r/h = 0.33. Thus,

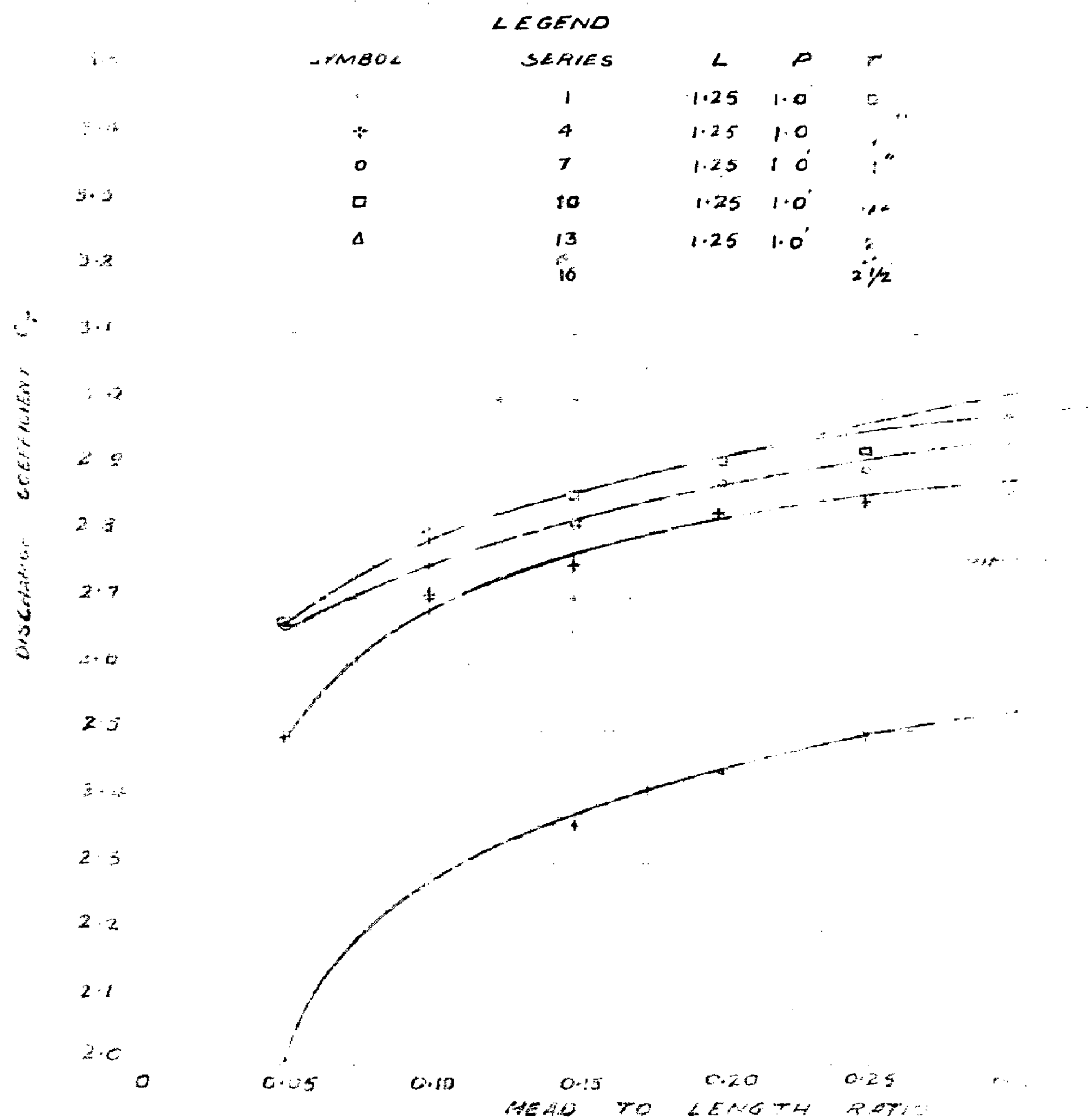
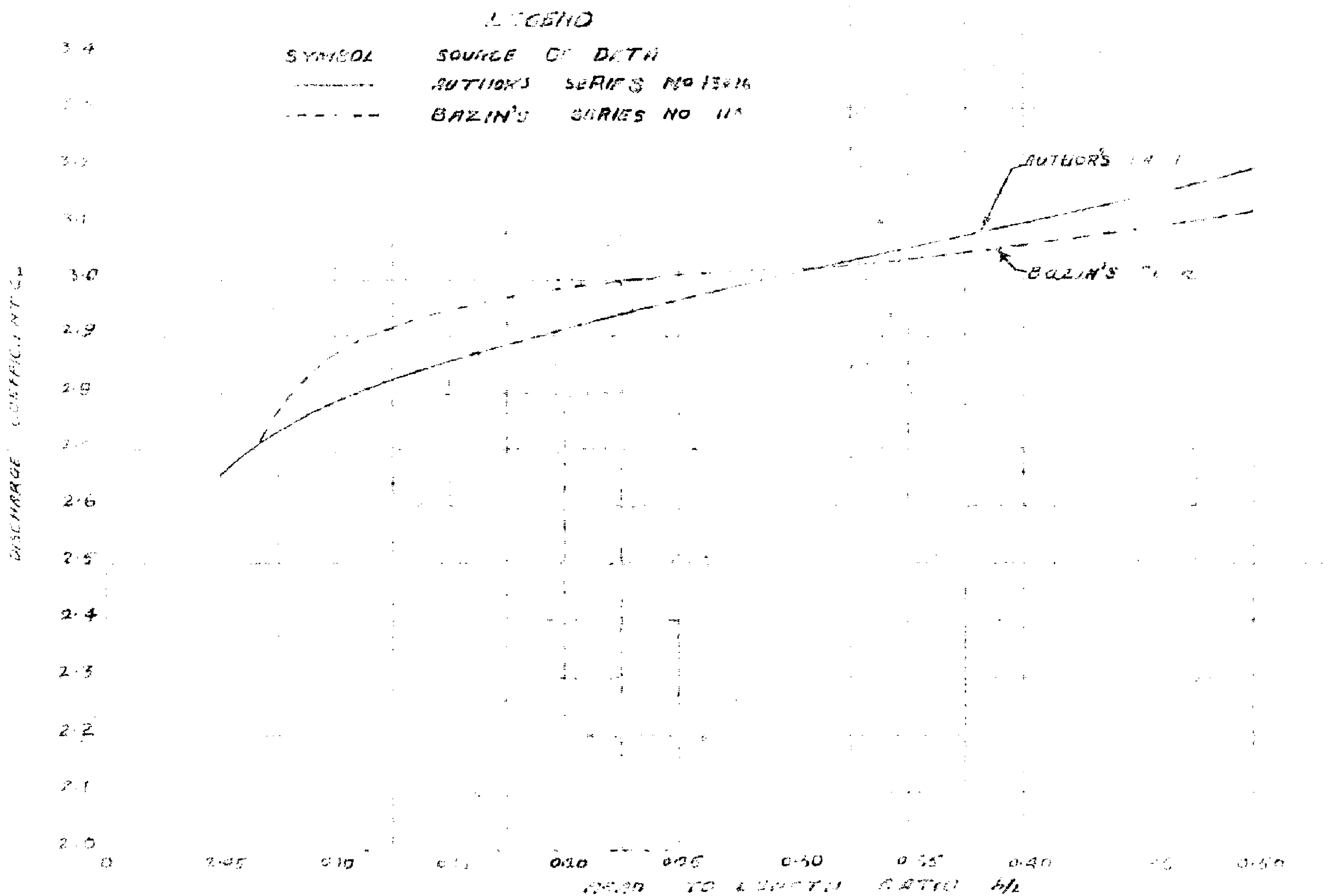


FIG. DISCHARGE COEFFICIENT FOR VERTICAL DROP crests of vertical drop falls with r/h greater than 0.33 may be said to be "well rounded", as no increase in C_r occurs with further increase in the rounding. Figure 17 shows the curve of discharge coefficient for the "well rounded crest" (corresponding to r/h = 0.33) compared with similar curve from Bazin's data.

In figure 19 are shown the curves of variation of the dimensionless factor C_r/C representing the increase in the discharge coefficient due to rounding over that with a sharp crest corner, with h/L ratio. The values of C_r/C are computed from figures 16 and 18. It may be observed from these curves, that for each minimum degree of rounding represented by r/h, the increasing discharge coefficient C_r/C first decreases with h/L and after reaching a certain minimum value increases thereafter for higher values of h/L . Further, the h/L value corresponding to the minimum value of C_r/C also has a decreasing trend with increasing values of r/h.

In this connection, it may be pointed out that in Woodburn's (5) test series on broad-crested weirs, rounding of the upstream crest corner by 2", 4", 6" and 8" roundings caused no significant variation in the discharge coefficient. Further the discharge



D... HARGE COEFFICIENT ... ROUNDED ... COMPA...

coefficient was about 9 percent greater than the corresponding coefficients for sharp upstream corner ($r/h=0$).

The lack of variation in the value of C_r above a certain degree of rounding ($r/h = 0.33$) may be interpreted by analogy with similar data for contracted openings. Kindsvater and Carter (25) showed that the discharge coefficient for contracted openings increased almost linearly with the values of r/b from $r/b = 0$ to $r/b = 0.14$, where b is the width of the opening; the coefficient remained constant after r/b exceeded 0.14. A similar variation in the discharge coefficient for orifices is seen in the literature.

This phenomenon may be explained by the fact that the flow separation at the upstream crest corner, which is mainly responsible for the energy loss of flow, disappears after a certain degree of rounding. Thus, the energy losses are reduced to the minimum possible and no further increase in the discharge coefficient is possible.

5.7 Effect of the Slope of Upstream Face on the Discharge Coefficient.-

As already explained in article 3.4, the sloped upstream face of a vertical drop fall acts like a

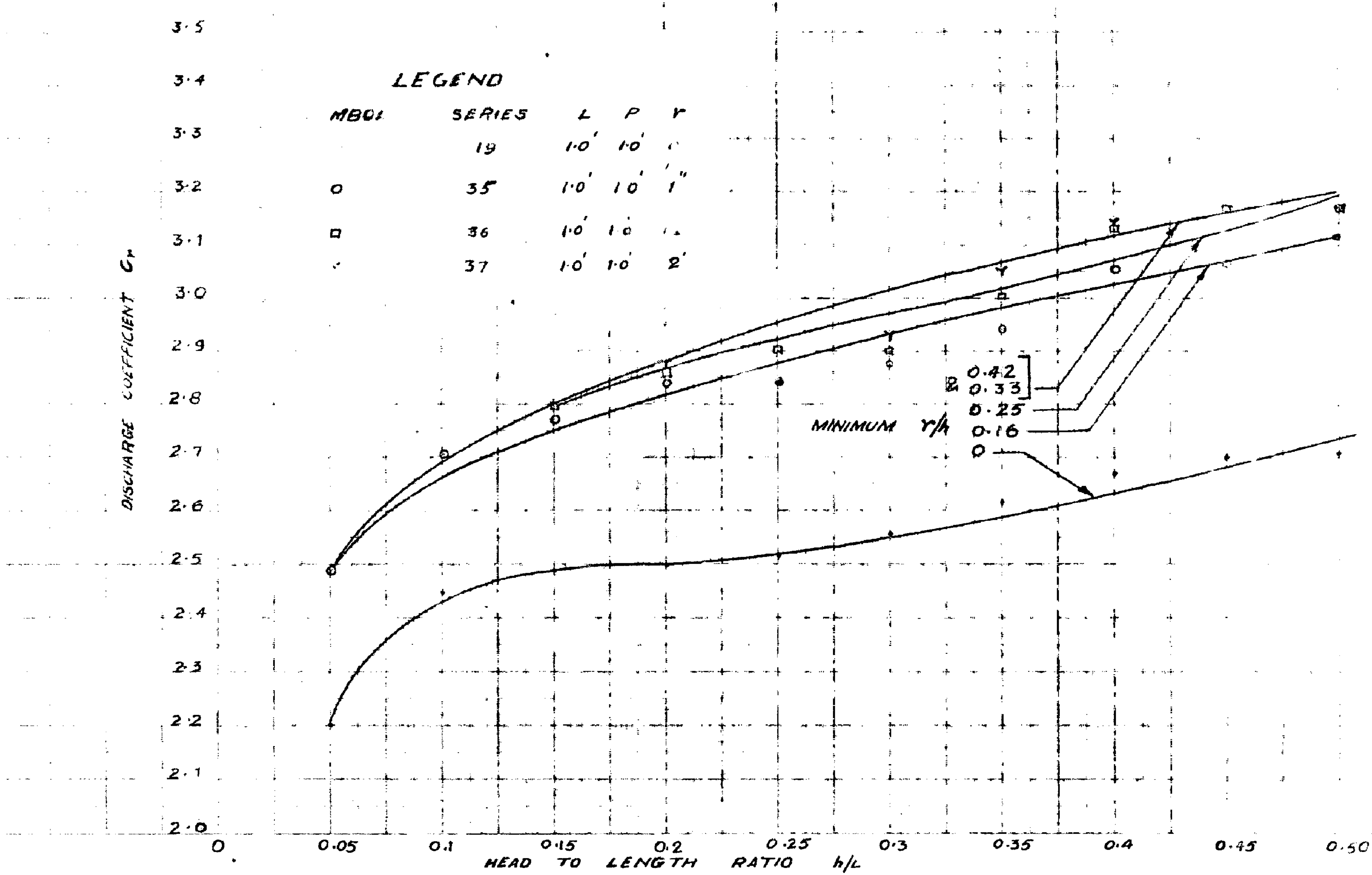


FIG - 18 DISCHARGE COEFFICIENT FOR VERTICAL DROP FALLS WITH ROUNDED UPSTREAM CREST CORNER ($L = 1.0'$)

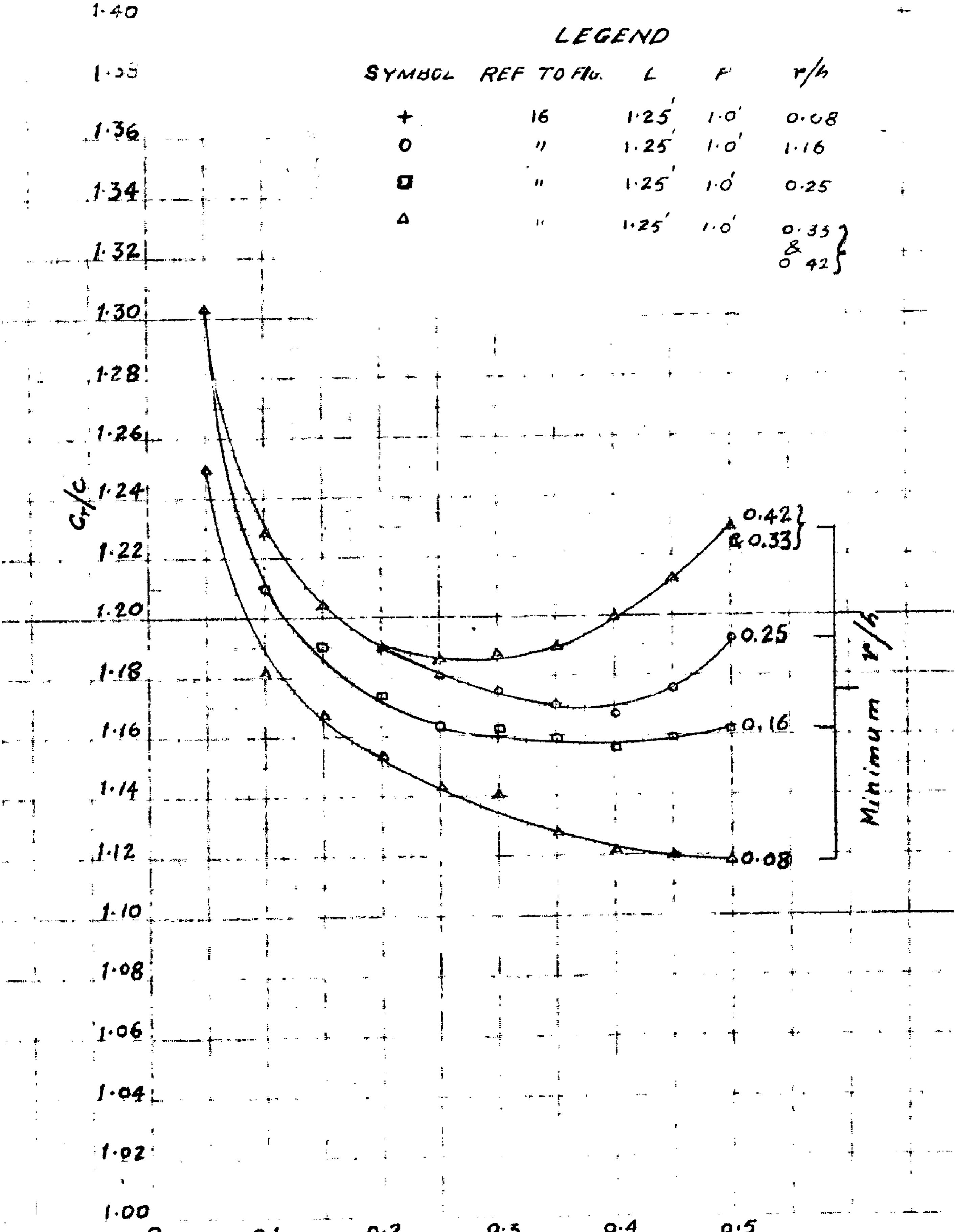


FIG:–19. VARIATION OF C_r/C WITH h/L FOR VERTICAL DROP FALLS WITH ROUNDED U/S CREST CORNERS

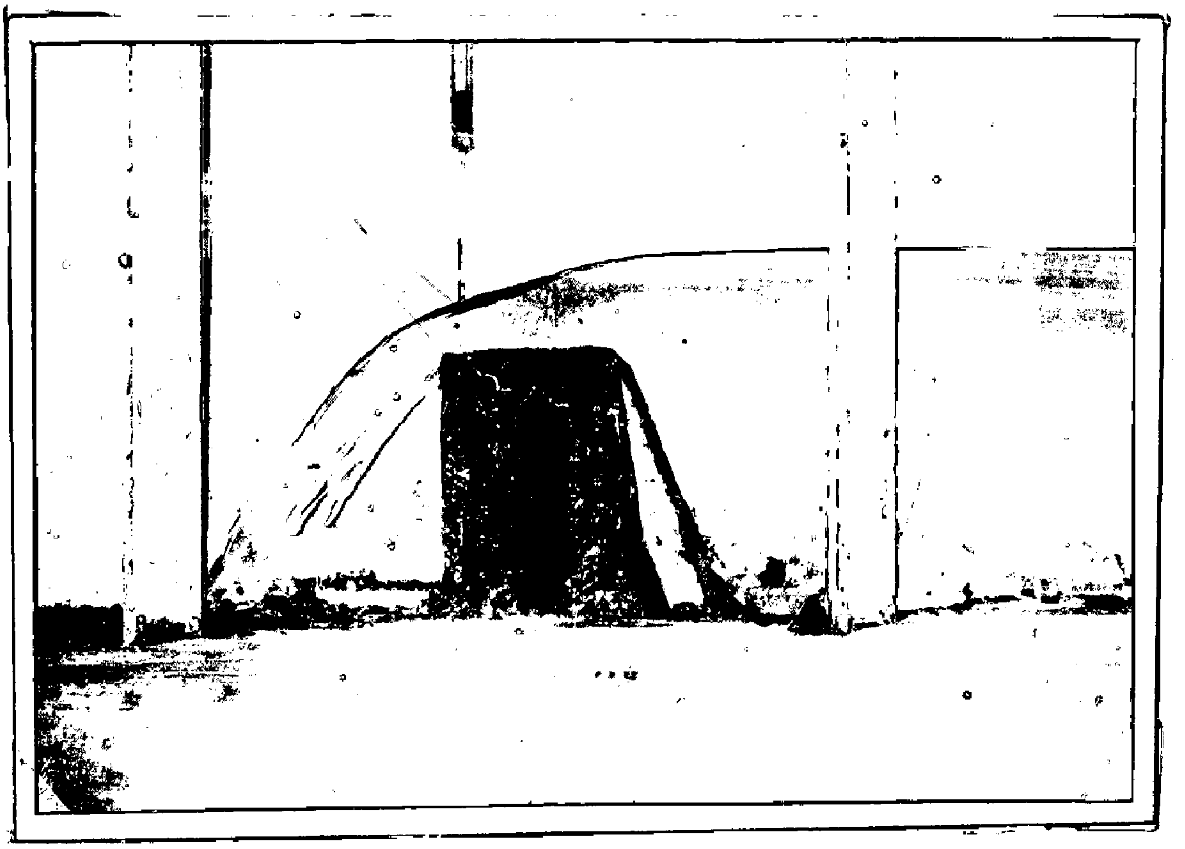


Fig. 20.Free flow over crest 0.333 ft. long, 0.50 ft. high and upstream face sloped at 1 in 1/3, under a head of 0.20 ft.

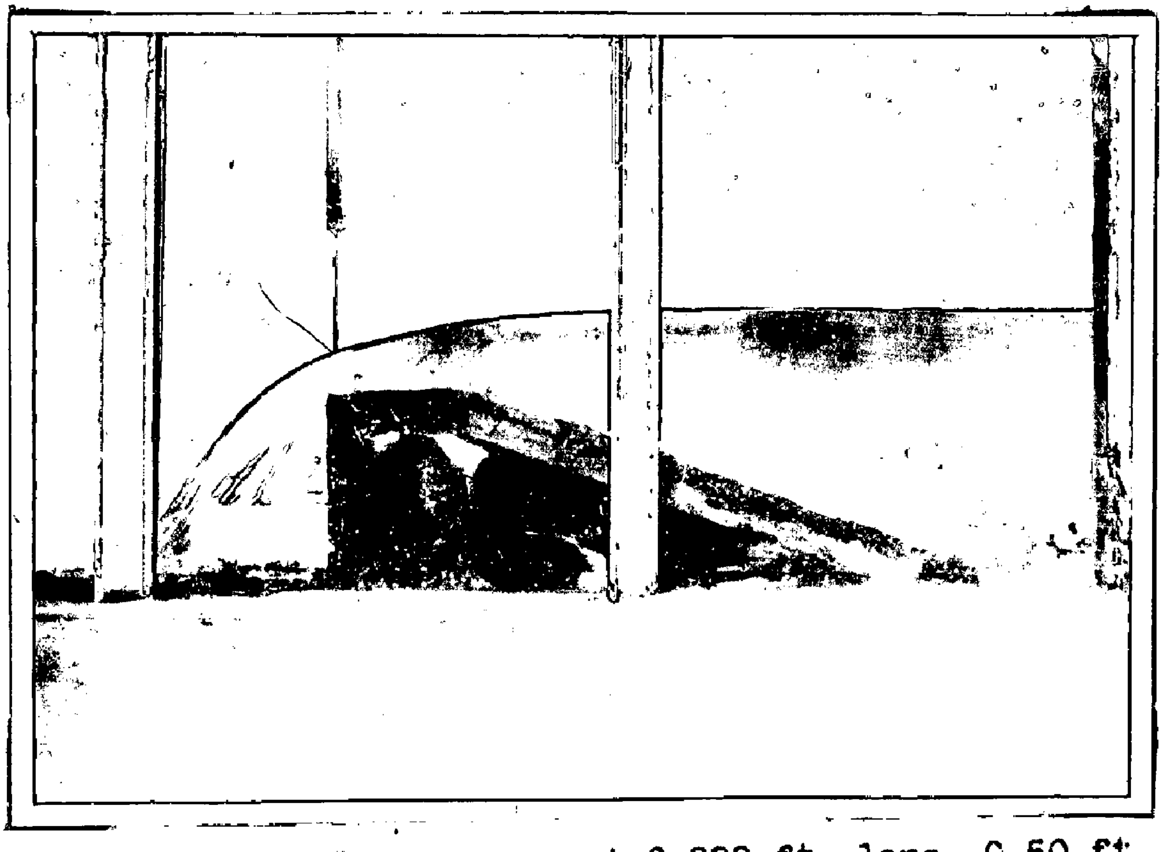
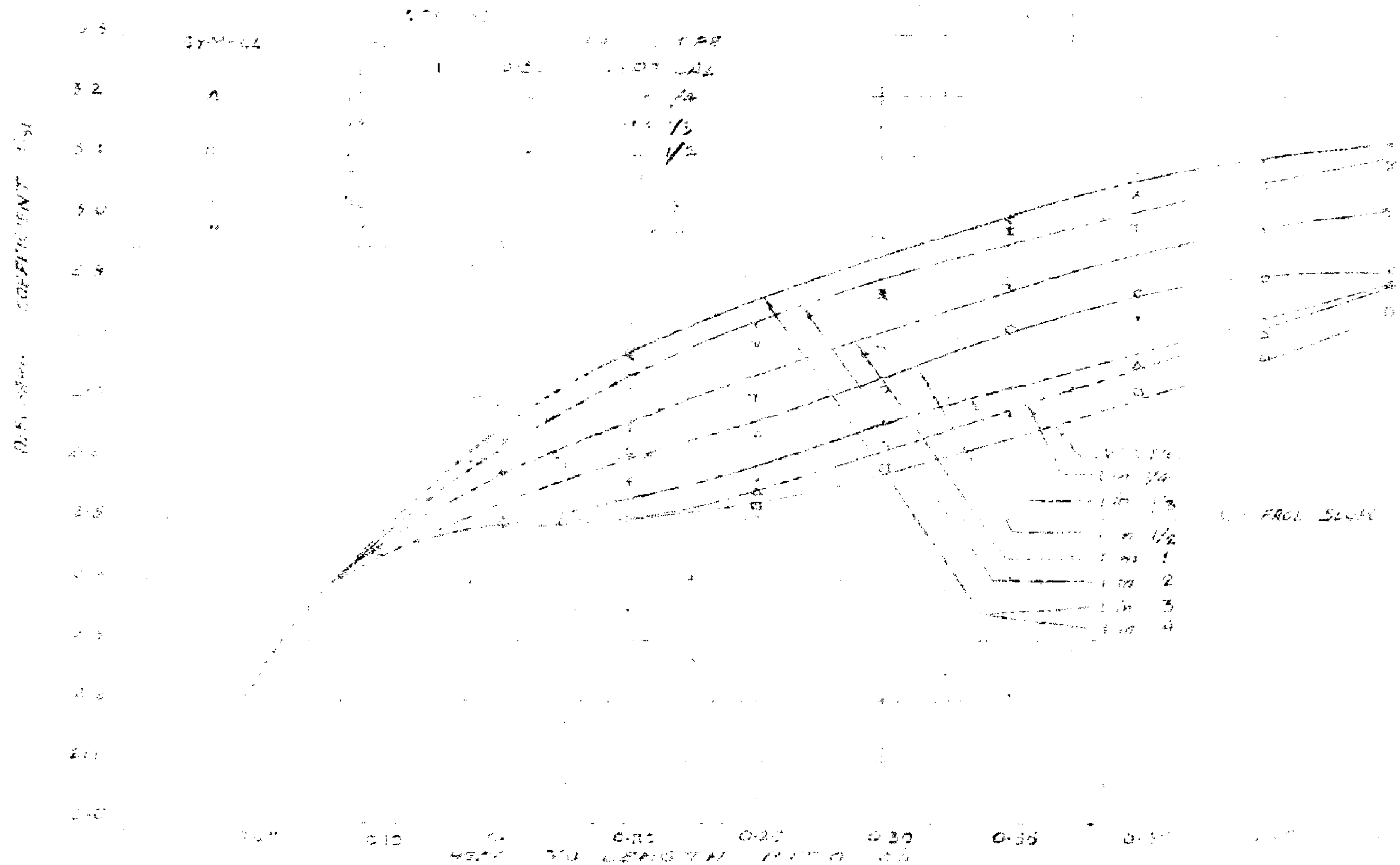


Fig.21. Free flow over crest 0.333 ft. long, 0.50 ft. high and upstream face sloped at 1in 3, under a head of 0.20 ft.

transition of flow boundary, thereby reducing the energy loss at the entry of flow over the crest at the upstream crest corner, resulting in increased discharge coefficient.

Experiments to determine this effect, were made on crests having lengths of 1.00 ft. and 0.333 ft. and heights of 0.50 ft. by varying the upstream face slope. In addition to the vertical face (zero slope) faces with 7 different slopes - 1 in ¼, 1 in 1/3, 1 in ½, 1 in 1, 1 in 2, 1 in 3, and 1 in 4 were tested. The results of these tests are presented in the tables 11 to 25 in Appendix II.

Figures 22 and 23 show the curves relating the discharge coefficient C_{sl} (for falls with sloped upstream faces) with h/L. It is evident from these figures that the effect of sloping of the upstream face is to increase the discharge coefficient with increasing flatness of slope in the range of slopes tested, namely from vertical upto 1 in 4 . Further, it may be observed, generally, that the rate of increase in the discharge coefficient decreases with increasing flatness of slope. A large part of the increase in the discharge coefficient is obtained even with a slope, of 1 in 1. This is a significant consideration



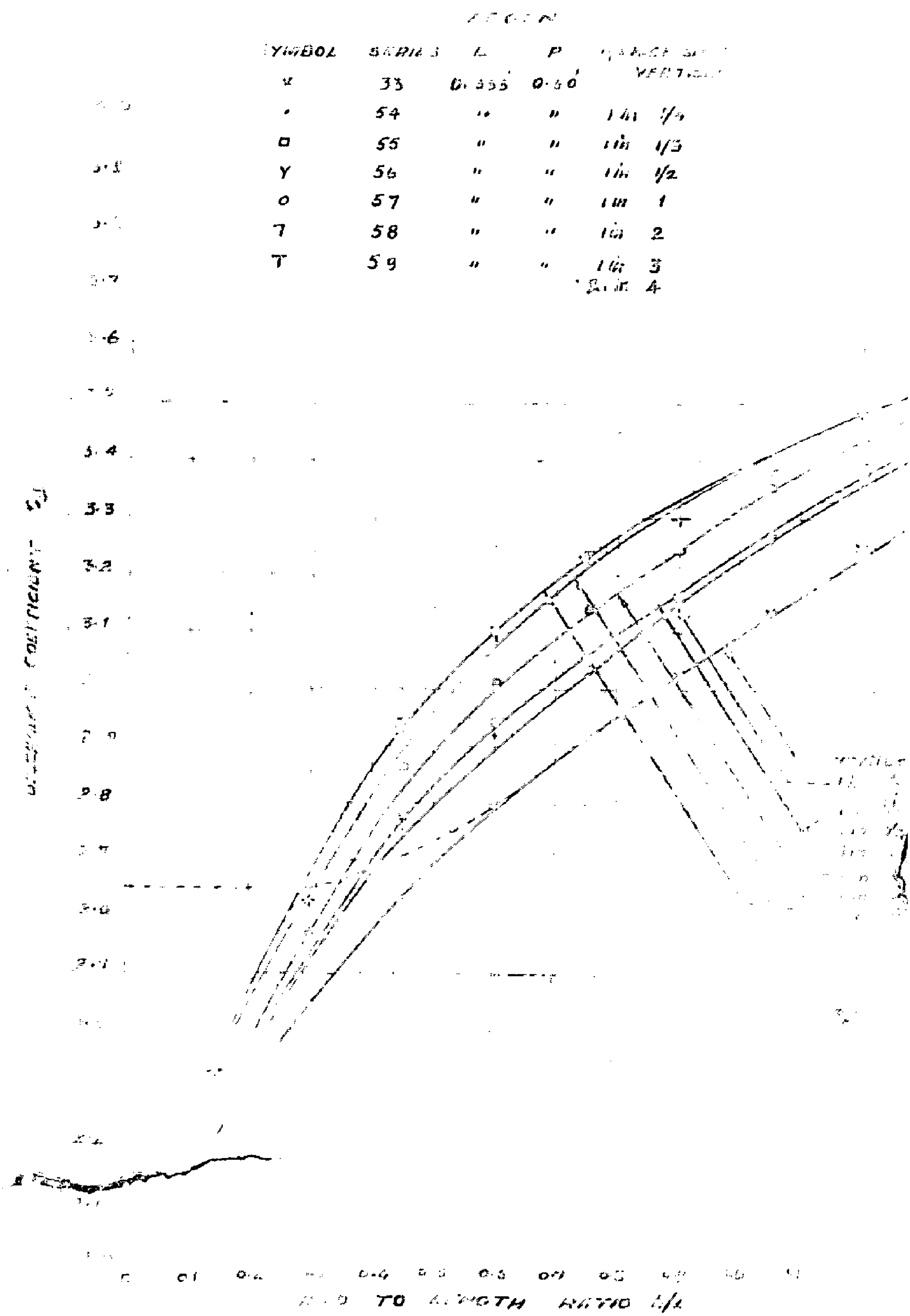
[illegible] CURVES [illegible] FOR VERTICAL DROP FALLS WITH [illegible]
[illegible] CORNER [illegible]

for the design of vertical drop falls, when the extra cost involved in the flatness of slope has to be weighed against the advantage gained in the hydraulic characteristics of the structure.

In figures 24 and 25 are shown the curves of the variation of "slope factor" C_{sl}/C representing the increase in the discharge coefficient due to sloped upstream face above that with vertical upstream face, with the h/L ratio. The values of C_{sl}/C are computed from the concerned curves in figures 22 and 23. As is evident from these curves, the slope factor initially increases with h/L upto a maximum value and then decreases for still higher h/L ratios. Thus, there is a maximum value for each slope corresponding to an optimum value of h/L. Again, the optimum value of h/L corresponding to the maximum slope factor value has a decreasing trend with increasing flatness of slopes. Further, for higher values of h/L, the values of the slope factor for flatter slopes tend to be equal.

5.8 <u>Effect of Boundary Roughness on the Discharge Coefficient</u>.-

In model testing, the simulation of boundary resistance is one of the most difficult tasks. For a



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In figures 24 and 25 are shown the curves of the variation of "slope factor" C_{sl}/C representing the increase in the discharge coefficient due to sloped upstream face above that with vertical upstream face, with the h/L ratio. The values of C_{sl}/C are computed from the concerned curves in figures 22 and 23. As is evident from these curves, the slope factor initially increases with h/L upto a maximum value and then decreases for still higher h/L ratios. Thus, there is a maximum value for each slope corresponding to an optimum value of h/L. Again, the optimum value of h/L corresponding to the maximum slope factor value has a decreasing trend with increasing flatness of slopes. Further, for higher values of h/L, the values of the slope factor for flatter slopes tend to be equal.

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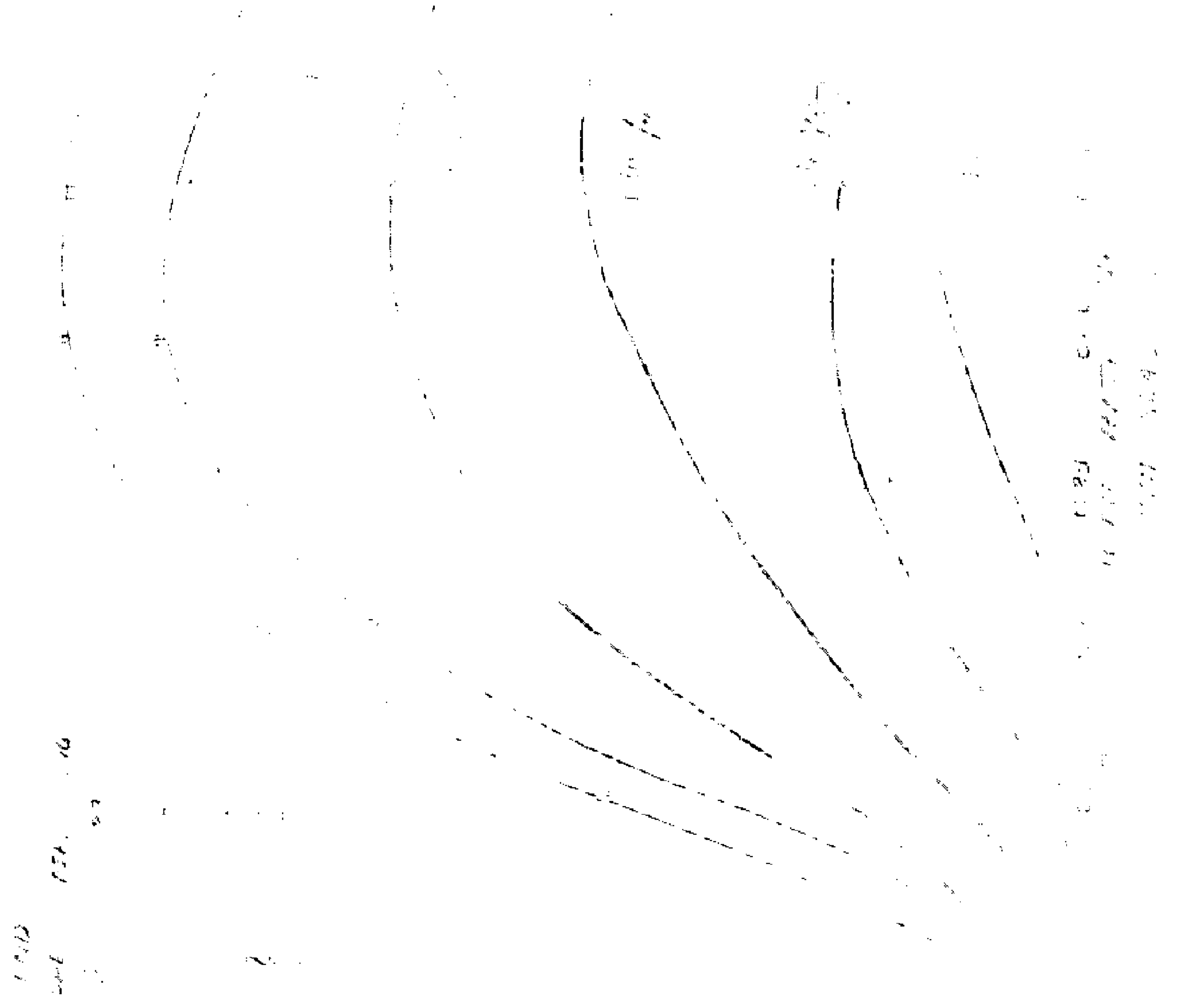


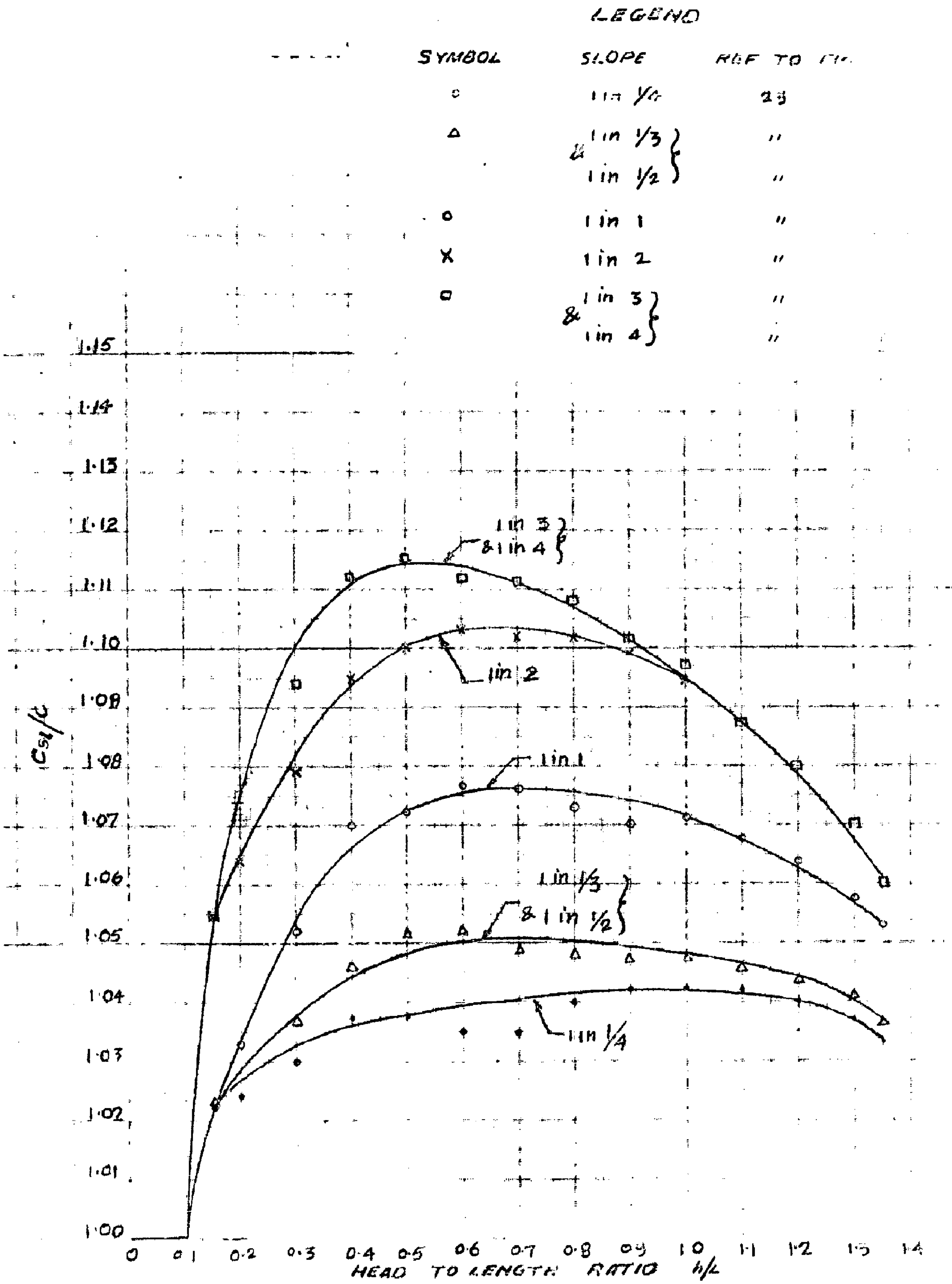


FIG :- 25 VARIATION OF SLOPE FACTOR C_{SL}/C WITH HEAD TO LENGTH RATIO h/L (UP TO 1.35) given structure having a given boundary roughness, the energy losses due to boundary resistance are a function of a characteristic Reynolds number and the relative roughness. As the scale is increased, the relative roughness increases and the boundary shear force becomes disproportionately large if the boundary surfaces are of the same material for the various scales.

The relative roughness is generally described by relating the mean height of the boundary projections above the nominal surface level to some significant length. In the analysis made in article 3.6, the ratio h/K is used, where K is the mean roughness height. It is evident, therefore, that the relative effect of boundary resistance is the same in the model and prototype, only if the relative roughness h/k and the Reynolds number R are the same in both scales. However, it is impracticable to produce equal values of R or h/k. Requirements of similitude are compromised by either ignoring the difference in relative roughness or by extension to the scale of the prototype by some method based on experience.

No tests were made in this investigation, to determine the effect of relative roughness; nor is definitive data based on previous studies available. However, except when the heads h are very small, the discharge coefficients obtained on the basis of model tests are considered to apply to much larger structures also with a very small margin of error. In general, the discharge coefficientshould be expected to increase slightly as the scale of the model increases.

5.9 <u>Effect of Reynolds number on the discharge Coefficient</u>.-

The significance of Reynolds number R is that it describes the relative importance of the viscous forces involved. The "scale effect", due to which there will be a variation in the discharge coefficient with the scale ratio, is the result of the action of the forces due to the viscosity of the fluid. The larger the value of the Reynolds number R, the less important the influence of viscosity upon the flow pattern is.

It is seen from equation (23), that for a given fluid, the Reynolds number is proportional to $h^{3/2}$, or the discharge per unit width of the crest. It is obvious, therefore, that the Reynolds number of model and prototype can be equal only at the same scale. Thus, it is generally impracticable to satisfy the Reynolds law. However, the Reynolds numbers obtaining in the laboratory tests are generally high enough to include the range in which the viscous effects are negligible.

Figure 14 shows the plot of the results of tests made on vertical drop falls having sharp upstream corner, with crest lengths of 1.25 ft., 1.00 ft. 0.333 ft. and crest heights of 1.00 ft., 0.75 ft. and 0.50 ft., thus producing different scale ratios. The scatter of the experimental points indicates the scale effects, which is apparently considerable in the range of h/L = 0.50 as seen in the figure.

To get a better appreciation of the effect of the Reynolds number on the discharge coefficient, the data from Bazin's experiments, covering a wider range of Reynolds numbers, are plotted in figure 14(a). These results pertain to tests on broad-crested weirs with sharp upstream corner and crest lengths of 0.33 ft., 0.66 ft., 1.315 ft., 2.62 ft., and 6.56 ft. The scale of the crest height P (which is equal to 2.46 ft. in these tests) is not considered, as it has been shown to have little effect on the discharge coefficient.

It is pertinent to note here, that the scatter of the points is very slight in the range of h/L above about 0.50; but considerable for h/L values upto 0.50. This is due to the fact that in this range of h/L upto 0.50, the heads on the crests and consequently the Reynolds numbers are small enough to cause appreciable scale effects. The slight variation of the values of C above h/L = 0.50, may perhaps be ascribed to some differences in energy required to maintain the eddy motion in the separation zone near the upstream sharp corner, as the scale ratio is changed, together with the very much reduced effect of the high Reynolds numbers in this range of h/L above 0.50.

5.10 Effect of submergence on the discharge Coefficient.-

After modularity limit is reached, the discharge coefficient of vertical drop falls is affected by the submergence of the crest of the fall. The non-modular flow is thus governed both by the upstream head h and the downstream head Z of the water above the crest (see Fig. 3). The dependence of the discharge coefficient on these two variables is evidenced by the fact that the coefficient is shown to be a function of dimensionless parameter Z/h in the discharge equation 24 in article 3.6.

With a view to determine the modularity limit, and to study the variation of discharge coefficient C_s in the non-modular range of discharge, tests were made on models having a crest length of 1.0 ft., with both sharp and well rounded upstream corner. Two different crest heights of 1.0 ft. and 0.75 ft. were used. These tests were carried out with four heads - 0.20 ft., 0.30 ft., 0.40 ft. and 0.50 ft. In each series of these experiments, the head was maintained constant and the downstream water level was varied to obtain various degrees of submergence, by the simultaneous control of both the valve on the supply pipe and the tail gate in the experimental flume.

The results of these experiments are presented in tables 49 to 60 in Appendix II.

<u>Forms of Flow</u>.- The following flow forms were observed while increasing the submergence of crest by raising the tail water level.

(a) Diving jet;- For small degrees of submergence, jet of water falling over the crest retained its wetted underneath form, though hidden by its immersion in the downstream channel (see figs. 3 and 27).

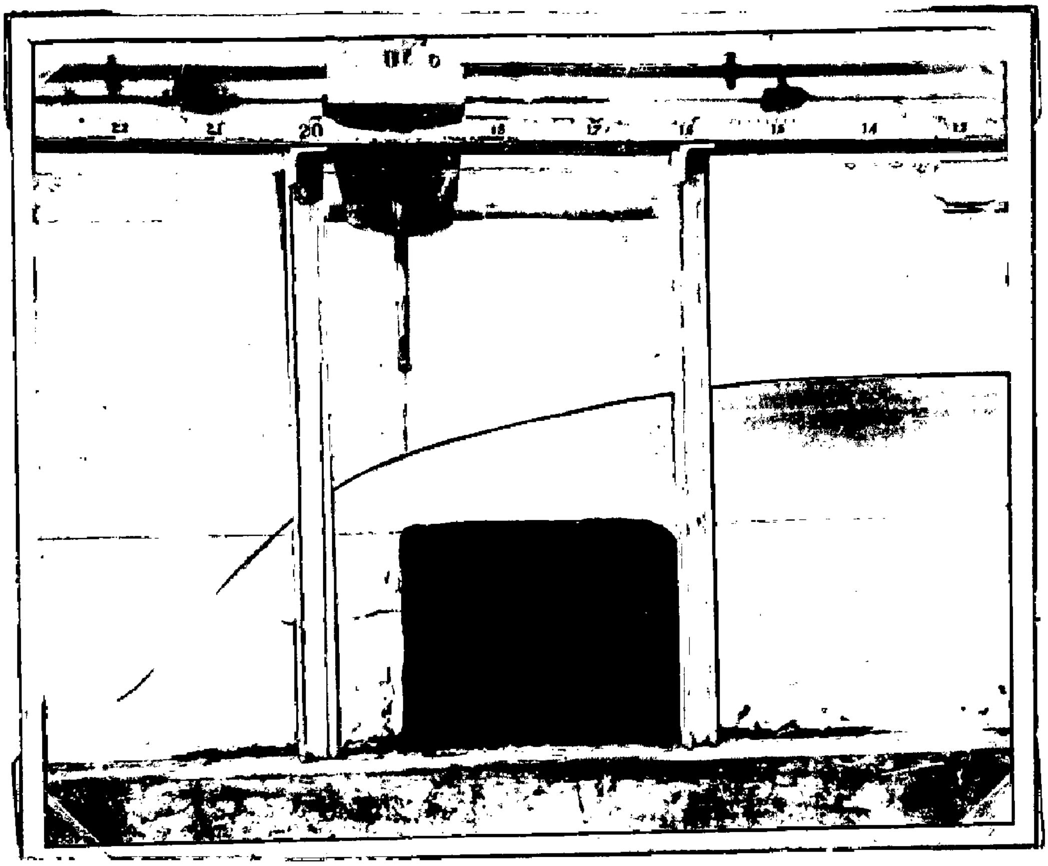


Fig.26. Free flow over crest 1.00 ft. long, 0.75ft. high and upstream crest corner rounded to 2" radius, under a head of 0.20 ft.

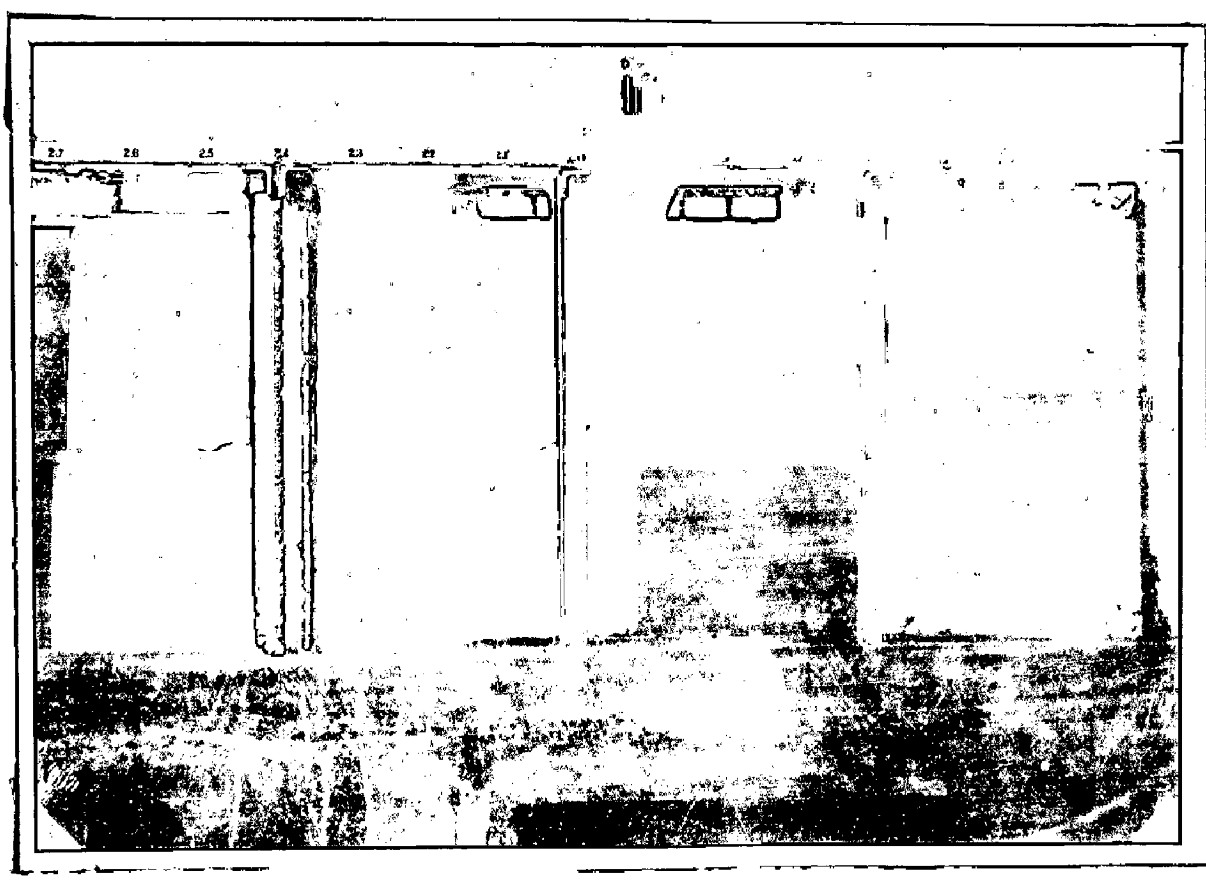


Fig.27. Submerged flow over crest 1.00 ft. long, 0.75ft. high and upstream crest corner rounded to 2" radius, under a head of 0.50 ft. with diving jet; Z/h=0.15.

(b) Undulating jet.- When the level of water was raised further, at some degree of submergence, which was not consistent for all the runs of experiments, the jet of falling water was found to spring abruptly to the surface with an undulating form. (See Figs. 3 and 28).

(c) With still further increase in the downstream water level, the undulations of the jet surface were found to be evened out with a more or less smooth surface, on crests with rounded upstream corner.

In the case of crests with sharp upstream corner, it was found that a standing wave forms near about the downstream end of the crest. This may be explained by the fact that the flow is supercritical on the crest, while that in the downstream channel is sub-critical.

The modularity limit was found to be reached, generally, at this stage of submerged flow.

(d) As the downstream water level was raised further, the surface of flow flattened out to a very even profile on the crests with rounded upstream corner.

On crests with sharp upstream corner, it was found that the standing wave formed earlier moved upstream on the crest, like an open channel surge, with the

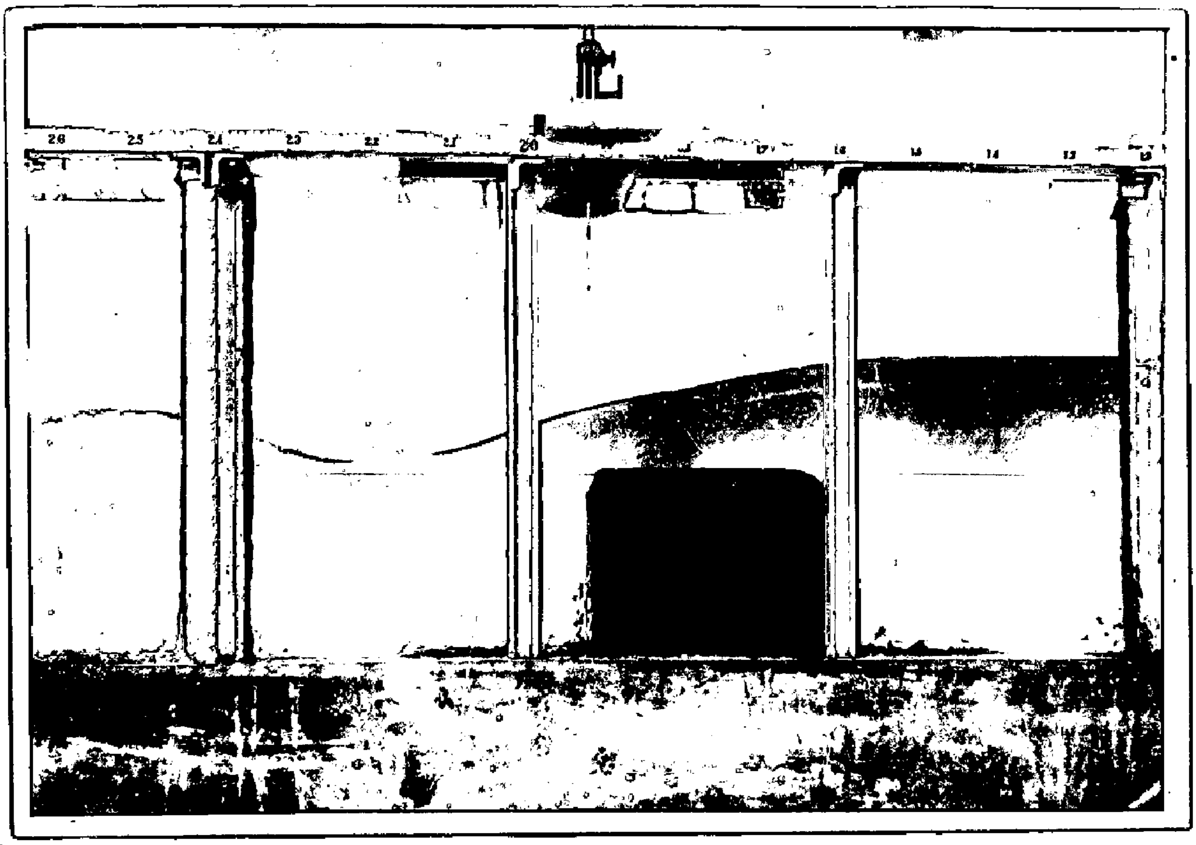


Fig.28. Submerged flow over crest 1.00 ft.long, 0.75 ft. high and upstream crest corner rounded to 2" radius, under a head of 0.50 ft., with undulating jet; Z/h= 0.45.

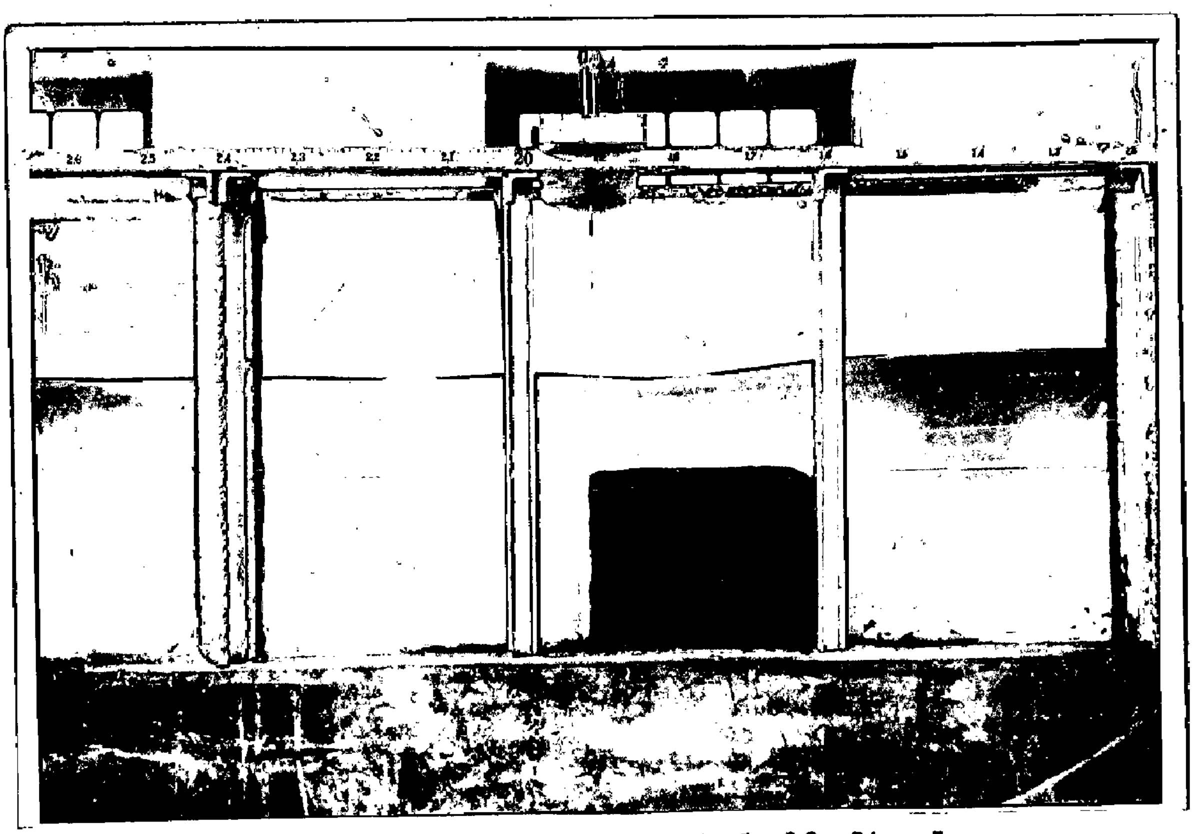
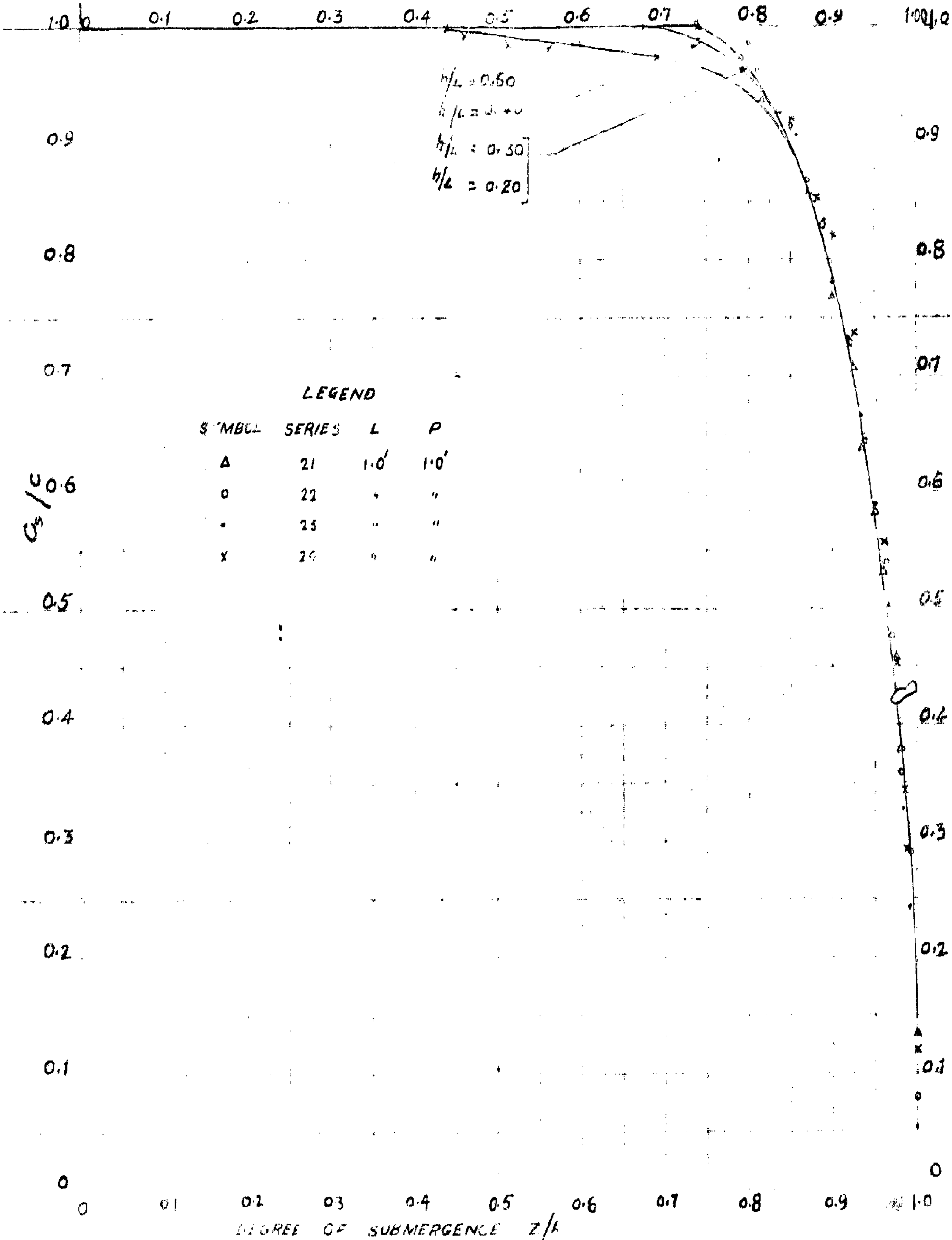


Fig.29. Submerged flow over crest 1.00 ft. long, 0.75 ft. high and upstream crest corner, rounded to 2" radius, under a head of 0.50 ft., with nearly smooth surface; z/h=0.92.



VARIATION OF C_s/C WITH DEGREE OF SUBMERGENCE Z/h
FOR A SUBMERGED VERTICAL DROP FALL WITH SHARP U/S CORNER

increase in the degree of submergence.

(e) When the downstream head was increased still, with a submergence degree of about 0.92 to 0.95, the profile of flow over crests was similar to the profile of flow over a raised floor in a flume, with a local drop in water level over the crest and a regain of flow depth downstream of the crest. (See Fig. 29).

This type of change of form was gradual in the case of crests with rounded upstream corner, while on crests with sharp upstream corner a relatively abrupt change of form was noticed, when the surge moving on the crest suddenly flattened out. This flow form evidently indicates that the flow over the crest is subcritical, and that is similar to the flow in a mild sloped channel with a local rise in its bed.

The flow in this range is perhaps better governed by the laws of flow in the channel in which the fall is existing.

Discharge Characteristics.-

Figures 30, 31 and 32 show the plot of C_s/C against the degree of submergence Z/h. Here, C_s represents the discharge coefficient in the submerged discharge conditions, and C represents the discharge coefficient

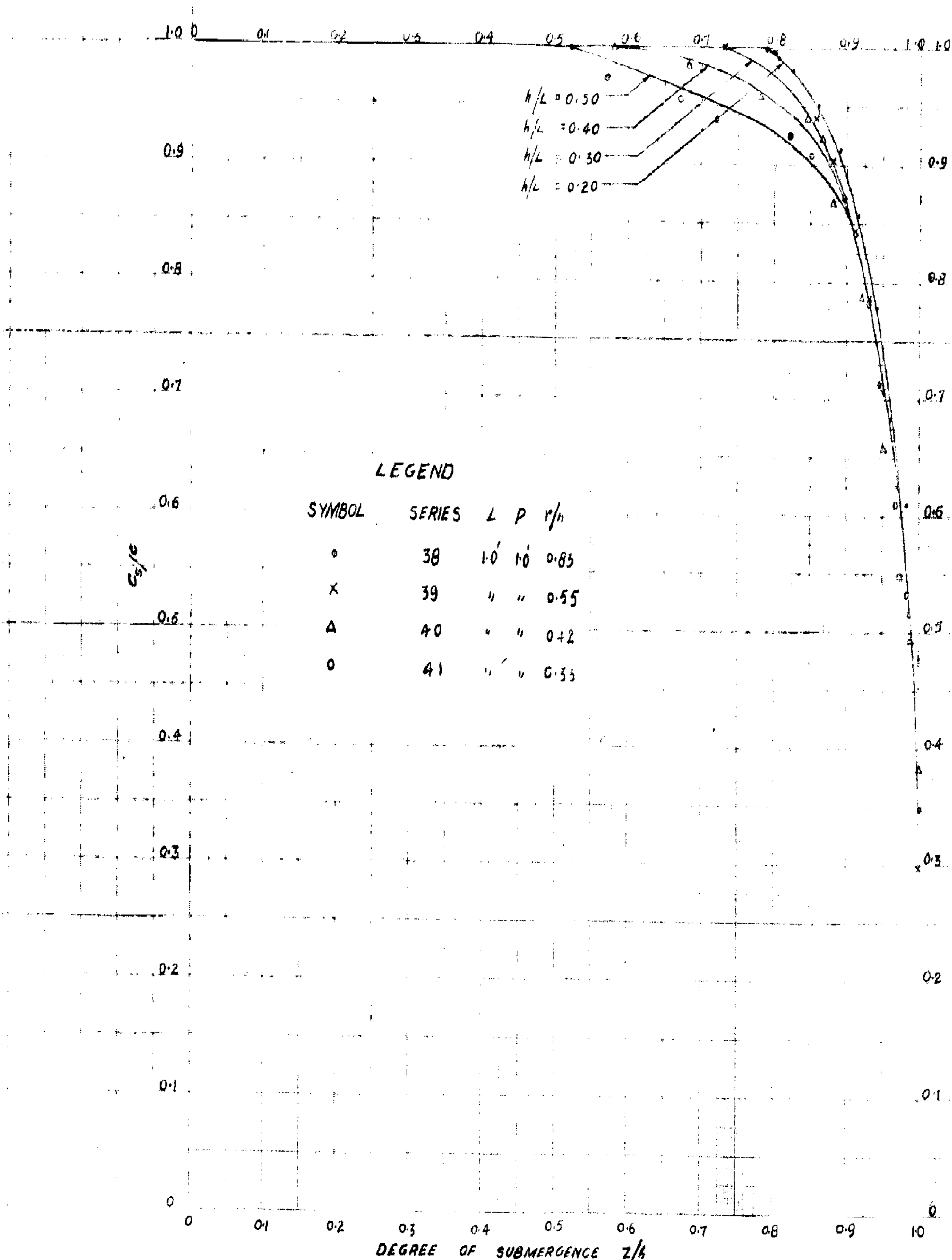


FIG: 31 VARIATION OF C_s/C WITH DEGREE OF SUBMERGENCE Z/h FOR A SUBMERGED VERTICAL DROP FALL WITH ROUNDED U/S CORNER in the free discharge conditions, both these values being for the same crest and the same head.

It can be seen from these plots, that the modularity limit is not constant for all the crests or for all the heads. It varies within a very wide range-from a degree of submergence of 0.48 (in Fig. 30) to 0.83 (in Fig. 32). Further, the modularity limit is reached generally, at a higher degree of submergence for crests with rounded upstream corner than for crests with sharp upstream corner, for a given head. This is a matter of great significance in the hydraulic design of vertical drop falls, as with a rounded upstream crest corner, the free discharge formula can be applied upto relatively high degrees of submergence.

The modularity limit varies with the head also, decreasing in value with increasing heads. However, all the curves for different heads merge into one at a degree of submergence of about 0.85 in the case of crests with sharp upstream corner and 0.95 in the case of crests with well rounded upstream corner (r/h above 0.33).

An important point to be observed is that all the curves meet the 1.0 degree of submergence line at some

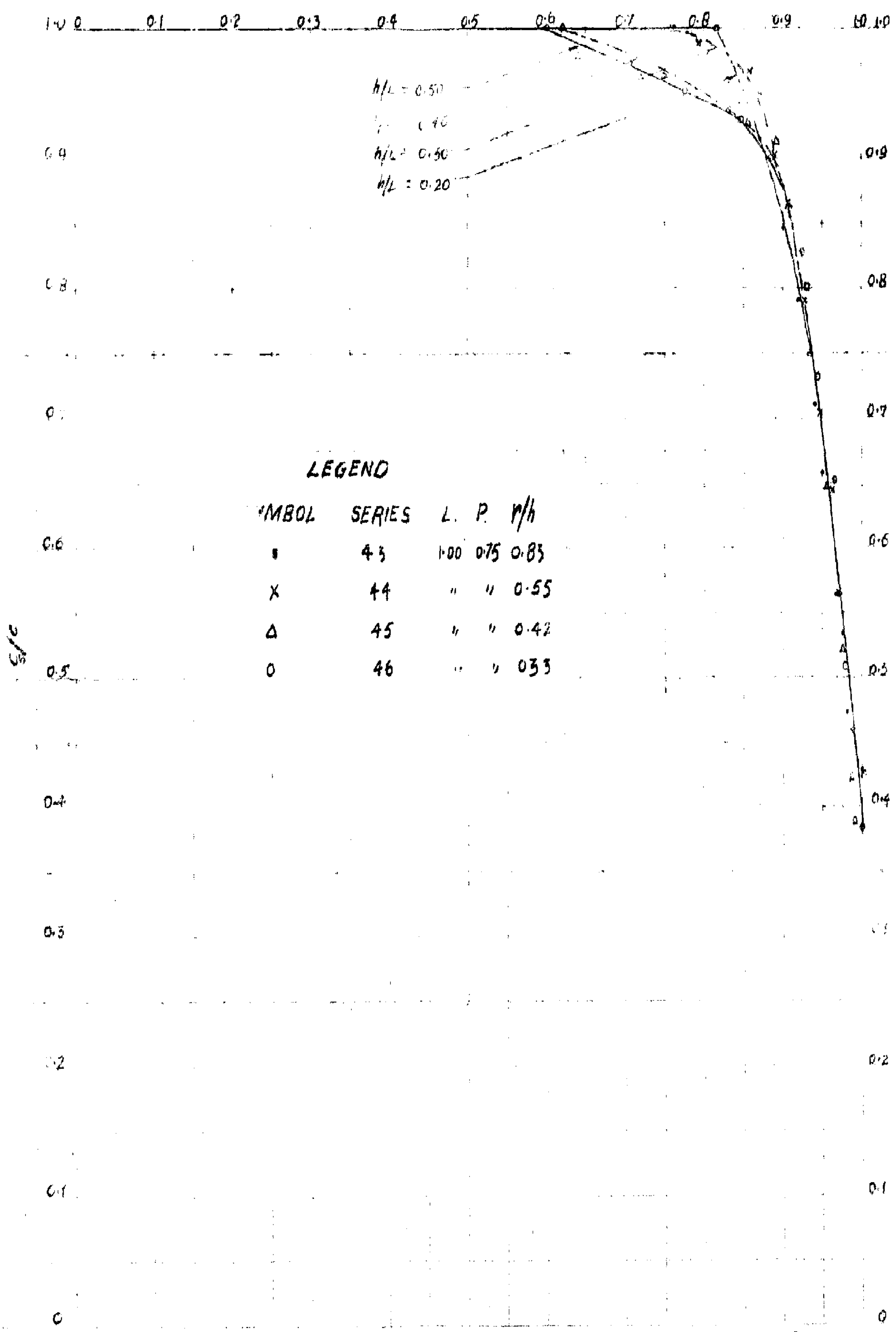


FIG - 32. VARIATION OF C_s/C WITH DEGREE OF SUBMERGENCE Z/h FOR A SUBMERGED VERTICAL DROP FALL WITH ROUNDED U/S CORNER finite value of C_s/C - about 0.1 in the case of crests with sharp upstream corner and about 0.35 in the case of crests with rounded upstream corner. This fact implies a certain minimum discharge, even for cent per cent submergence, flowing in the channel. This can be understood, considering the fact, that at high degrees of submergence (possibly above 0.95), the flow over the crest of the fall is like the flow over a local rise in the bed of a mild-sloped channel.

The curves of C_s/C present a variation in their general configuration as well as their individual values, with different crest heights of vertical drop falls. This is, perhaps, due to the momentum unbalance in the flow over the crest, on account of a change in the height of crest. More experiments are needed to investigate this aspect of the problem.

The general pattern of the variation of C_s/C with Z/h is, however, similar in all the cases tested.

6. CONCLUSIONS

6. CONCLUSIONS

A review of literature has shown that there exists, today, no rational method of fixing the discharge coefficient of vertical drop falls, whose general form is that of a broad-crested weir, over the full range of their function. The flow characteristics of the vertical drop falls have been analysed theoretically, which led to the conclusion that they are not subject to complete mathematical analysis. Therefore, a comprehensive solution for the discharge characteristics based on dimensional analysis and experiments has been presented. This solution covers both the free and submerged discharge over the falls.

The discharge over the vertical drop falls is influenced by the physical characteristics of the crest of fall, and the channel in which the fall exists. For this reason, the discharge coefficient is expressed as a function of dimensionless parameters characteristic of the significant factors influencing the flow.

The study of the flow pattern over crests has led to a rational classification of broad-crested weirs.

The general trend of the variation of the discharge coefficient in relation to significant variables are:

1. The discharge coefficient varies appreciably with the h/L ratio above about 0.35 for crests with sharp upstream corner and 0.25 for crests with rounded upstream corner. For h/L below the above limits, the value of the coefficient is nearly constant.

2. The crest height does not affect the discharge coefficient.

3. The shape of the discharging stream represented by h/B ratio does not affect the discharge coefficient.

4. Reynolds number of flow over crest is significant in its lower range, while it is negligible in its higher range.

5. The effect of crest boundary roughness is not yet fully known and further investigation is necessary to evaluate it.

6. The discharge coefficient increases with the rounding of the upstream corner upto a certain degree of rounding (r/h = 0.33), beyond which any further rounding will not result in any increased value of the coefficient.

7. The discharge coefficient increases with increasing flatness of slope of upstream face, in the range tested (from vertical to 1 in 4).

8. Submergence of crest of the vertical drop fall has no effect on the discharge coefficient upto a limiting degree of submergence (modularity limit). Thereafter, the discharge coefficient falls in value appreciably, the rate of fall increasing with increasing degree of submergence.

The curves representing this variation differ with the head, shape of upstream crest corner and the height of crest. Additional experiments are needed to investigate this aspect fully.

BIBLIOGRAPHY

BIBLIOGRAPHY

1. G.W.Rafter : "On the flow of water over dams" Trans. A. S. C. E., 1900 pp. 220 to 398.

2. Robert, E. Horton : "Weir experiments, coefficients and formulas" Water Supply and Irrigation Paper No. 200, Dept. of Interior, United States Geological Survey, 1907.

3. Richard. R. Lyman : "Measurement of the flow of streams by approved forms of weirs with new formulas and diagrams". Trans. A.S.C.E. Vol. 77, 1914.

4. Parshall, R.L. : "The improved Venturi Flume" Trans. A.S.C.E. Vol. 89, 1926.

5. J.G.Woodburn with appendix by A.R.Webb : "Tests on broad-crested weirs" with discussion Trans. A.S.C.E., 1932.

6. Hunter Rouse : "Discharge Characteristics of the Free Overfall". Civil Engineering, April, 1936 pp. 257 to 260.

7. "Annual Report of the Punjab Irrigation Research Institute, Amritsar", 1938.

8. Hunter Rouse : "Fluid Mechanics for Hydraulic Engineers", Engineering Societies Monographs, McGraw-Hill Book Company, 1938.

(9) H. A. Doeringsfield and C.L. Barker: "Pressure-momentum theory applied to the broad-crested weir" with discussion, Trans.A.S.C.E.,Vol.106, 1941, pp. 934 to 969.

10. "Annual Report of the Punjab Irrigation Research Institute, Amritsar", 1943.

11. "Boulder Canyon Projects", Final Reports Part VI, Bulletin 3, U.S. Bureau of Reclamation, Denever, U.S.A.

12. "Instructions for designing a Sarda Type fall with metering properties", U.P. P.W.D.(Irrigation Branch) Technical Paper No. 6, 1945.

13. "Note on the design and behaviour of Sarda Type (vertical drop) fall as developed by the Irrigation Department of the United Provinces". U.P., P.W.D. (Irrigation Branch) Technical Paper No. 7, 1945.

14. J. Allen: "Scale models in hydraulic engineering" Longmans, Green & Co., 1947.

15. J.I.Coil Pillai: "Kistna Anicut at Vijayawada" - Determination of Coefficient of discharge". Annual Report of the Irrigation Research Institute, Poondi (Madras) 1949.

16. A.T.Ippen : "Control Sections". Article 9, Chapter VIII in "Engineering Hydraulics" ed. Hunter Rouse, John Wiley & Sons., 1950, pp. 525-528.

17. J.V.Rao : "Anicut across the Godavari at Dowlaishwaram" - Study of discharge characteristics". Annual Report of the Irrigation Research Institute, Poondi (Madras), 1951.

18. Charles Grant Edson: "Hydraulic drop as a function of velocity distribution" Civil Engineering, December, 1954.

19. Charles Jaeger : "Engineering Fluid Mechanics" - Blackie & Son Limited, 1956.

20. S.V.Chitale: "Model experiments for the estimation of discharge coefficients of the proposed Hanumannagar Barrage on Kosi river". Irrigation and Power Journal (India) Vol. XIV, No. 2, April, 1957.

21. P.K.Kandaswamy: "Discharge Characteristics of Kodiveri anicut" Annual Report of the Irrigation Research Institute, Poondi(Madras), 1958.

22. Smith, C.D. : "Open channel water measurement with the broad-crested weir". Annual Bulletin of the International Commission on Irrigation &

Drainage, 1958.

23. Richard. A. Smith: "Calibration of the submerged broad-crested weir" Journal of the Hydraulics Division. Vol. 85, No. HY.3, Proceedings. A.S.C.E., March, 1959.

24. P.K.Kandaswamy, N.Rajaratnam and T.R.Anand : "Discussion on Calibration of a submerged broad-crested weir (by Richard A. Smith)" Journal of the Hydraulics Division, HY 9, Proceedings A.S.C.E., September, 1959.

25. Carl. E. Kindsvater and Rolland. W. Carter: "Tranquil flow through open channel constrictions" Trans. A.S.C.E. Vol. 120, 1950, pp. 976-977.

APPENDIX I

NOTATION

NOTATION

The following symbols have been adopted for use in this thesis, and are presented here alphabetically for convenience of reference.

a = area of cross-section of flow.

B = width of crest (normal to the flow direction)

C = discharge coefficient in the formula $Q = CBh^{3/2}$ for vertical drop falls with sharp U/S corner and vertical U/S face.

C_1 = discharge coefficient in the formula $Q = C_1BH^{3/2}$ for vertical drop falls with sharp U/S corner and vertical U/S face.

C_r = discharge coefficient in the formula $Q = C_rBh^{3/2}$ for vertical drop falls with rounded U/S corner and vertical U/S face.

C_{s1} = discharge coefficient in the formula $Q = C_{s1}Bh^{3/2}$ for vertical drop falls with sharp U/S corner and sloped U/S face.

c_v = coefficient of velocity.

d = depth of crest below a considered point in the flow.

E = specific energy (or head) of flow.

F = Froude number $\frac{v}{\sqrt{gh}}$

g = acceleration of gravity.

h	=	upstream piezometric head of water, above the crest of vertical drop fall measured at a section of normal flow in the channel upstream.
H	=	total head $(h+\frac{v_o^2}{2g})$ upstream of crest.
K	=	ratio of depth of flow on crest to the piezometric head upstream.
k	=	mean height of boundary roughness projections, above the nominal level of boundary.
L	=	length of crest parallel to the direction of flow .
P	=	height of crest above the bed of upstr am channel.
Q	=	total discharge
q	=	discharge per unit width of crest.
R	=	Reynolds number $\frac{v\,h\,\rho}{\mu}$.
r	=	radius of rounding of U/S crest corner
v	=	velocity of flow over the crest.
v_o	=	velocity of flow in approach channel
w	=	specific weight.
x	=	distance along the crest with reference to the upstream crest corner.
y	=	depth of flow or elevation of water surface over the crest.
y_c	=	critical depth of flow.

z = downstream piezometric head of water, above the crest of vertical drop fall, measured at a section of normal flow in the tail channel (downstream).

δ = boundary displacement thickness

ρ = mass density

μ = dynamic viscosity.

ν = kinematic viscosity

APPENDIX II

TABLES

Table 1

Series No. 1 Crest length L = 1.25 ft. ; Crest height P = 1.0 ft. ; Both faces vertical ; Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.04	0.0220	1.994	0.0194	0.3880
2.	0.10	0.08	0.0713	2.290	0.0394	0.3940
3.	0.15	0.12	0.1350	2.358	0.0571	0.3806
4.	0.20	0.16	0.2150	2.441	0.0789	0.3946
5.	0.25	0.20	0.3075	2.497	0.1027	0.4109
6.	0.30	0.24	0.4100	2.531	0.1164	0.3881
7.	0.35	0.28	0.5237	2.566	0.1394	0.3983
8.	0.40	0.32	0.6462	2.592	0.1608	0.4020
9.	0.45	0.36	0.7713	2.594	0.1779	0.3954
10.	0.50	0.40	0.9037	2.594	0.2018	0.4035

Table 2

Series No. 2 — Crest length L = 1.25 ft. ; Crest height P = 0.75 ft. ; Both faces vertical Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.04	0.0220	1.994	0.0194	0.3880
2.	0.10	0.08	0.0713	2.290	0.0407	0.4070
3.	0.15	0.12	0.1350	2.358	0.0591	0.3941
4.	0.20	0.16	0.2150	2.441	0.0787	0.3936
5.	0.25	0.20	0.3075	2.497	0.1034	0.4136
6.	0.30	0.24	0.4100	2.531	0.1197	0.3990
7.	0.35	0.28	0.5237	2.566	0.1462	0.4178
8.	0.40	0.32	0.6462	2.592	0.1657	0.3956
9.	0.45	0.36	0.7713	2.594	0.1865	0.4050
10.	0.50	0.40	0.9037	2.594		

‡ Nappe depressed

Table 3

Series No. 3 Crest length L = 1.25 ft. ; Both faces vertical
Crest height P = 0.50 ft. ; Sharp **U/S** corner

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.04	0.0225	2.039	0.0197	0.3941
2.	0.10	0.08	0.0725	2.328	0.0394	0.3940
3.	0.15	0.12	0.1363	2.381	0.0591	0.3941
4.	0.20	0.16	0.2175	2.469	0.0820	0.4101
5.	0.25	0.20	0.3143	2.552	0.1056	0.4224
6.	0.30	0.24	0.4200	2.592	0.1292	0.4307
‡7.	0.35	0.28	0.5413	2.653		
8.	0.40	0.32	0.6812	2.732		
9.	0.45	0.36	0.8238	2.770		
10.	0.50	0.40	0.9787	2.808		

‡ Nappe depressed.

Table 4

Series No. 19 Crest length L = 1.0 ft. ; Both faces vertical
Crest height P = 1.0 ft. ; Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.05	0.0243	2.202	0.0197	0.3940
2.	0.10	0.10	0.0762	2.447	0.0400	0.4000
3.	0.15	0.15	0.1425	2.489	0.0633	0.4220
4.	0.20	0.20	0.2200	2.498	0.0836	0.4175
5.	0.25	0.25	0.3100	2.517	0.1032	0.4128
6.	0.30	0.30	0.4143	2.558	0.1230	0.4100
7.	0.35	0.35	0.5343	2.617	0.1444	0.4126
8.	0.40	0.40	0.6650	2.668	0.1662	0.4155
9.	0.45	0.45	0.8030	2.701	0.1882	0.4182
10.	0.50	0.50	0.9437	2.708	0.2118	0.4236

Table 5

Series No. 20 Crest length L = 1.0 ft. ; Both faces vertical
Crest height P = 1.0 ft. ; Sharp U/S corner

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C	Flow depth at D/S end of crest in ft. y_e	y_e/h
[a] 1.	0.20	0.20	0.2200	2.498	0.0820	0.4100
[b] 2.	0.30	0.30	0.4143	2.558	0.1200	0.4000
[c] 3.	0.40	0.40	0.6650	2.668	0.1592	0.3980
[d] 4.	0.40	0.40	0.6650	2.668	0.1542	0.3855
[e] 5.	0.50	0.50	0.9437	2.708	0.2084	0.4168
[f] 6.	0.50	0.50	0.9437	2.708	0.2034	0.4068

a Nappe depressed

b Nappe depressed

c Nappe depressed

d Nappe wetted underneath

e Nappe depressed

f Nappe wetted underneath

Table 6

Series No. 25 Crest length L = 1.0 ft. ; Both faces vertical
Crest height P = 0.75 ft.; Sharp U/S corner

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.05	0.0243	2.202	0.0197	0.3940
2.	0.10	0.10	0.0762	2.447	0.0400	0.4000
3.	0.15	0.15	0.1425	2.489	0.0633	0.4220
4.	0.20	0.20	0.2200	2.498	0.0838	0.4190
5.	0.25	0.25	0.3100	2.517	0.1036	0.4144
6.	0.30	0.30	0.4143	2.558	0.1232	0.4107
7.	0.35	0.35	0.5350	2.622	0.1445	0.4126
8.	0.40	0.40	0.6700	2.687	0.1667	0.4167
‡9.	0.45	0.45	0.8125	2.733		
10.	0.50	0.50	0.9575	2.748		

‡ Nappe depressed

Table 7

Series No. 26 Crest length L = 1.0 ft. ; Both faces vertical
Crest height P = 0.50 ft. ; Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.05	0.0243	2.202	0.0194	0.3880
2.	0.10	0.10	0.0762	2.447	0.0404	0.4040
3.	0.15	0.15	0.1425	2.489	0.0621	0.4140
4.	0.20	0.20	0.2200	2.498	0.0853	0.4265
5.	0.25	0.25	0.3106	2.522	0.1050	0.4200
6.	0.30	0.30	0.4188	2.585	0.1230	0.4100
‡7.	0.35	0.35	0.5356	2.625		
8.	0.40	0.40	0.6750	2.708		
9.	0.45	0.45	0.8237	2.770		
10.	0.50	0.50	0.9925	2.848		

‡ Nappe depressed

Table 8

Series No. 51 Crest length L = 0.333 ft. ; Crest height P = 1.0 ft. ; Both faces vertical Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.15	0.0244	2.211	0.0184	0.3680
2.	0.10	0.30	0.0316	2.447	0.0381	0.3810
3.	0.15	0.45	0.1490	2.602	0.0583	0.3886
4.	0.20	0.60	0.2388	2.711	0.0800	0.4000
5.	0.25	0.75	0.3472	2.819	0.1094	0.4376
6.	0.30	0.90	0.4738	2.925	0.1413	0.4710
7.	0.35	1.05	0.6190	3.033	0.1783	0.5094
8.	0.40	1.20	0.7850	3.149	0.2160	0.5400
9.	0.45	1.35	0.9762	3.283	0.2570	0.5711

Table 9

Series No. 52 Crest length $L = 0.333$ ft. ; Both faces vertical
Crest height $P = 0.75$ ft. ; Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.15	0.0244	2.211	0.0184	0.3680
2.	0.10	0.30	0.0762	2.447	0.0384	0.3840
3.	0.15	0.45	0.1490	2.602	0.0590	0.3933
4.	0.20	0.60	0.2450	2.782	0.0840	0.4200
5.	0.25	0.75	0.3560	2.890	0.1114	0.4456
6.	0.30	0.90	0.4894	3.021	0.1500	0.5000
‡7.	0.35	1.05	0.6368	3.120		
8.	0.40	1.20	0.8100	3.249		
9.	0.45	1.35	0.9862	3.316		

‡ Nappe depressed

Table 10

Series No. 53 — Crest length L = 0.333 ft. ; Crest height P = 0.50 ft. ; Both faces vertical Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.15	0.0244	2.211	0.0184	0.3680
2.	0.10	0.30	0.0762	2.447	0.0384	0.3840
3.	0.15	0.45	0.1508	2.634	0.0595	0.3966
4.	0.20	0.60	0.2460	2.793	0.0845	0.4225
‡5.	0.25	0.75	0.3582	2.908		
6.	0.30	0.90	0.4905	3.028		
7.	0.35	1.05	0.6398	3.136		
8.	0.40	1.20	0.8100	3.249		
9.	0.45	1.35	0.9862	3.316		

‡ Nappe depressed

<u>Table 11</u>

Series No. 27 — Crest length L = 1.0 ft. ; Crest height P = 0.50 ft.; U/S face slope 1 in ¼ ; Sharp U/s corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_{s1}	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.05	0.0243	2.202	0.0164	0.3280
2.	0.10	0.10	0.0768	2.447	0.0404	0.4040
3.	0.15	0.15	0.1425	2.489	0.0608	0.4053
4.	0.20	0.20	0.2200	2.498	0.0860	0.4300
5.	0.25	0.25	0.3134	2.545	0.1055	0.4220
6.	0.30	0.30	0.4250	2.623	0.1240	0.4133
‡7.	0.35	0.35	0.5431	2.662		
8.	0.40	0.40	0.6862	2.752		
9.	0.45	0.45	0.8342	2.805		
10.	0.50	0.50	1.0037	2.880		

‡ Nappe depressed

Table 12

Series No. 28 Crest length L = 1.0 ft. ; U/S face slope 1 in 1/3 ;
Crest height P = 0.50 ft.; Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_{s1}	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.05	0.0243	2.203	0.0164	0.3280
2.	0.10	0.10	0.0762	2.447	0.0404	0.4040
3.	0.15	0.15	0.1431	2.500	0.0617	0.4113
4.	0.20	0.20	0.2262	2.563	0.0860	0.4300
5.	0.25	0.25	0.3158	2.564	0.1065	0.4260
6.	0.30	0.30	0.4312	2.662	0.1240	0.4133
‡7.	0.35	0.35	0.5550	2.719		
8.	0.40	0.40	0.7075	2.838		
9.	0.45	0.45	0.8438	2.837		
10.	0.50	0.50	1.0062	2.887		

‡ Nappe depressed

Table 13

Series No. 29 Crest length L = 1.0 ft. ; Crest height P = 0.50 ft.; U/S face slope 1 in 0.4 ; Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_{s1}	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.05	0.0243	2.202	0.0164	0.3280
2.	0.10	0.10	0.0762	2.447	0.0404	0.4040
3.	0.15	0.15	0.1450	2.533	0.0617	0.4113
4.	0.20	0.20	0.2300	2.611	0.0866	0.4330
5.	0.25	0.25	0.3250	2.638	0.1082	0.4328
6.	0.30	0.30	0.4362	2.693	0.1247	0.4157
‡7.	0.35	0.35	0.5688	2.787		
8.	0.40	0.40	0.7150	2.868		
9.	0.45	0.45	0.8575	2.884		
10.	0.50	0.50	1.0142	2.910		

‡ Nappe depressed

<u>Table 14</u>

Series No. 30 Crest length L = 1.0 ft. ; Crest height P = 0.50 ft.; U/S face slope 1 in ½ ; Sharp U/S corner .

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_{s1}	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.05	0.0243	2.202	0.0164	0.3280
2.	0.10	0.10	0.0762	2.447	0.0416	0.4160
3.	0.15	0.15	0.1475	2.575	0.0623	0.4153
4.	0.20	0.20	0.2301	2.612	0.0868	0.4340
5.	0.25	0.25	0.3250	2.638	0.1082	0.4328
6.	0.30	0.30	0.4400	2.716	0.1285	0.4283
‡7.	0.35	0.35	0.5744	2.815		
8.	0.40	0.40	0.7175	2.878		
9.	0.45	0.45	0.8625	2.900		
10.	0.50	0.50	1.0158	2.915		

‡ Nappe depressed

Table 15

Series No. 31 Crest length L = 1.0 ft. ; U/S face slope 1 in 1 ;
Crest height P = 0.50 ft.; Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_{s1}	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.05	0.0243	2.202	0.0164	0.3280
2.	0.10	0.10	0.0762	2.447	0.0407	0.4070
3.	0.15	0.15	0.1475	2.575	0.0626	0.4173
4.	0.20	0.20	0.2338	2.655	0.0875	0.4375
5.	0.25	0.25	0.3330	2.704	0.1108	0.4432
6.	0.30	0.30	0.4512	2.786	0.1345	0.4483
‡7.	0.35	0.35	0.5900	2.892		
8.	0.40	0.40	0.7450	2.988		
9.	0.45	0.45	0.8875	2.984		
10.	0.50	0.50	1.0512	3.016		

‡ Nappe depressed

Table 16

Series No. 32 Crest length L = 1.0 ft. ; Crest height P = 0.50 ft.; U/S face slope 1 in 2 ; Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_{s1}	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.05	0.0243	2.202	0.0171	0.3420
2.	0.10	0.10	0.0762	2.447	0.0426	0.4260
3.	0.15	0.15	0.1506	2.629	0.0662	0.4413
4.	0.20	0.20	0.2438	2.768	0.0918	0.4590
5.	0.25	0.25	0.3438	2.791	0.1147	0.4588
6.	0.30	0.30	0.4650	2.871	0.1377	0.4590
‡7.	0.35	0.35	0.6075	2.977		
8.	0.40	0.40	0.7575	3.039		
9.	0.45	0.45	0.9062	3.047		
10.	0.50	0.50	1.0775	3.090		

‡ Nappe depressed.

Table 17

Series No. 33 Crest length L = 1.0 ft. ; U/S face slope 1 in 3 ;
Crest height P = 0.50 ft.; Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_{sl}	Flow depth at D/S endo of crest in ft. y_e	y_e / h
1.	0.05	0.05	0.0243	2.202	0.0194	0.3880
2.	0.10	0.10	0.0762	2.447	0.0426	0.4260
3.	0.15	0.15	0.1537	2.685	0.0662	0.4413
4.	0.20	0.20	0.2488	2.825	0.0925	0.4625
5.	0.25	0.25	0.3475	2.821	0.1153	0.4612
6.	0.30	0.30	0.4662	2.878	0.1377	0.4590
‡7.	0.35	0.35	0.6112	2.994		
8.	0.40	0.40	0.7662	3.073		
9.	0.45	0.45	0.9200	3.094		
10.	0.50	0.50	1.0875	3.121		

‡ Nappe depressed

Table 18

Series No. 34 Crest length L = 1.0 ft. ; Crest height P = 0.50 ft. ; U/S face slope 1 in 4 ; Sharp corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_{s1}	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.05	0.0243	2.202	0.0194	0.3880
2.	0.10	0.10	0.0762	2.447	0.0426	0.4260
3.	0.15	0.15	0.1537	2.685	0.0662	0.4413
4.	0.20	0.20	0.2488	2.825	0.0925	0.4625
5.	0.25	0.25	0.3475	2.821	0.1153	0.4612
6.	0.30	0.30	0.4700	2.901	0.1377	0.4590
‡7.	0.35	0.35	0.6125	3.002		
8.	0.40	0.40	0.7662	3.073		
9.	0.45	0.45	0.9200	3.094		
10.	0.50	0.50	1.0875	3.121		

‡ Nappe depressed

Table 19

Series No. 54 Crest length L = 0.333 ft. ; U/S face slope 1 in $\frac{1}{4}$;
Crest height P = 0.50 ft. ; Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_{s1}	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.15	0.0244	2.211	0.0171	0.3420
2.	0.10	0.30	0.0785	2.520	0.0377	0.3777
3.	0.15	0.45	0.1577	2.755	0.0610	0.4066
4.	0.20	0.60	0.2572	2.920	0.0868	0.4340
‡5.	0.25	0.75	0.3723	3.023		
6.	0.30	0.90	0.5102	3.150		
7.	0.35	1.05	0.6653	3.260		
8.	0.40	1.20	0.8420	3.378		
9.	0.45	1.35	1.0250	3.447		

‡ Nappe depressed

Table 20

Series No. 55 — Crest length L = 0.333 ft. ; Crest height P = 0.50 ft. ; U/S face slope 1 in 1/3 ; Sharp U/S corner

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_{s1}	Flow depth at D/S end of crest in ft.	y_e/h
1.	0.05	0.15	0.0250	2.266	0.0171	0.3420
2.	0.10	0.30	0.0787	2.527	0.0377	0.3770
3.	0.15	0.45	0.1592	2.780	0.0606	0.4040
4.	0.20	0.60	0.2593	2.945	0.0868	0.4340
‡5.	0.25	0.75	0.3756	3.049		
6.	0.30	0.90	0.5128	3.165		
7.	0.35	1.05	0.6653	3.260		
8.	0.40	1.20	0.8426	3.380		
9.	0.45	1.35	1.0300	3.465		

‡ Nappe depressed

Table 21

Series No. 56 Crest length $L = 0.333$ ft. ; U/S face slope 1 in $\frac{1}{2}$;
Crest height $P = 0.50$ ft. ; Sharp U/S corner .

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_{s1}	Flow depth at D/S end of crest in ft.	y_e / h
1.	0.05	0.15	0.0250	2.266	0.0171	0.3420
2.	0.10	0.30	0.0787	2.527	0.0377	0.3770
3.	0.15	0.45	0.1594	2.785	0.0610	0.4066
4.	0.20	0.60	0.2598	2.950	0.0872	0.4360
‡5.	0.25	0.75	0.3766	3.058		
6.	0.30	0.90	0.5136	3.170		
7.	0.35	1.05	0.6688	3.270		
8.	0.40	1.20	0.3672	3.393		
9.	0.45	1.35	1.031	3.470		

‡ Nappe depressed

Table 22

Series No. 57 Crest length : $L = 0.333$ ft. ; U/S face slope 1 in 1;
Crest height $P = 0.50$ ft. ; Sharp U/S corner .

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_{s1}	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.15	0.0250	2.266	0.0171	0.3420
2.	0.10	0.30	0.0800	2.570	0.0387	0.3870
3.	0.15	0.45	0.1640	2.864	0.0639	0.4260
4.	0.20	0.60	0.2655	3.013	0.0918	0.4590
‡5.	0.25	0.75	0.3868	3.140		
6.	0.30	0.90	0.5252	3.242		
7.	0.35	1.05	0.6852	3.358		
8.	0.40	1.20	0.8582	3.442		
9.	0.45	1.35	1.0450	3.515		

‡ Nappe depressed

Table 23

Series No. 58 Crest length L = 0.333 ft. ; U/S face slope 1 in 2 ;
Crest height P = 0.50 ft. ; Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_{s1}	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.15	0.0257	2.328	0.0171	0.3420
2.	0.10	0.30	0.0818	2.628	0.0400	0.4000
3.	0.15	0.45	0.1680	2.934	0.0655	0.4366
4.	0.20	0.60	0.2727	3.095	0.0935	0.4675
‡5.	0.25	0.75	0.3982	3.233		
6.	0.30	0.90	0.5346	3.300		
7.	0.35	1.05	0.6910	3.386		
8.	0.40	1.20	0.8700	3.490		
9.	0.45	1.35	1.0550	3.551		

‡ Nappe depressed

Table 24

Series No. 59 Crest length L = 0.333 ft. ; U/S face slope 1 in 3 ;
Crest height P = 0.50 ft. ; Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_{s1}	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.15	0.0257	2.328	0.0174	0.3480
2.	0.10	0.30	0.0825	2.650	0.0403	0.4030
3.	0.15	0.45	0.1687	2.947	0.0656	0.4373
4.	0.20	0.60	0.2738	3.108	0.9400	0.4700
‡5.	0.25	0.75	0.3990	3.240		
6.	0.30	0.90	0.5346	3.300		
7.	0.35	1.05	0.6910	3.386		
8.	0.40	1.20	0.8700	3.490		
9.	0.45	1.35	1.0550	3.551		

‡ Nappe depressed

Table 25

Series No. 60 Crest length $L = 0.333$ ft. ; U/S face slope 1 in 4 ;
Crest height $P = 0.50$ ft. ; Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_{sl}	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.15	0.0257	2.328	0.0174	0.3480
2.	0.10	0.30	0.0828	2.660	0.0403	0.4030
3.	0.15	0.45	0.1689	2.950	0.0656	0.4373
4.	0.20	0.60	0.2749	3.120	0.9400	0.4700
‡5.	0.25	0.75	0.4000	3.248		
6.	0.30	0.90	0.5346	3.300		
7.	0.35	1.05	0.6910	3.386		
8.	0.40	1.20	0.8700	3.490		
9.	0.45	1.35	1.0550	3.551		

‡ Nappe depressed

Table 26

Series No. 4 — Crest length L = 1.25 ft. ; Crest height P = 1.0 ft. ; Both faces vertical ; U/S corner rounded to $\frac{1}{2}$" radius.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.04	0.0275	2.492	0.0197	0.3936
2.	0.10	0.08	0.0843	2.707	0.0426	0.4260
3.	0.15	0.12	0.1575	2.751	0.0656	0.4450
4.	0.20	0.16	0.2494	2.831	0.0875	0.4375
5.	0.25	0.20	0.3512	2.851	0.1132	0.4529
6.	0.30	0.24	0.4637	2.863	0.1345	0.4483
7.	0.35	0.28	0.5900	2.892	0.1580	0.4514
8.	0.40	0.32	0.7230	2.900	0.1805	0.4512
9.	0.45	0.36	0.8637	2.905	0.2050	0.4556
10.	0.50	0.40	1.0150	2.913	0.2245	0.4489

Table 27

Series No. 7 Crest length L = 1.25 ft. ; Crest height P = 1.0 ft. ; Both faces vertical ; U/S corner rounded to 1" radius.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.04	0.0294	2.660	0.0203	0.4060
2.	0.10	0.08	0.0875	2.809	0.0442	0.4420
3.	0.15	0.12	0.1612	2.816	0.0665	0.4433
4.	0.20	0.16	0.2533	2.876	0.0886	0.4430
5.	0.25	0.20	0.3570	2.898	0.1150	0.4601
6.	0.30	0.24	0.4750	2.932	0.1388	0.4627
7.	0.35	0.28	0.6087	2.983	0.1623	0.4636
8.	0.40	0.32	0.7575	2.998	0.1875	0.4686
9.	0.45	0.36	0.8917	2.998	0.2130	0.4734
10.	0.50	0.40	1.0606	3.044	0.2360	0.4720

Table 28

Series No. 10 Crest length L = 1.25 ft. ; Both faces vertical ;
Crest height P = 1.0 ft. ; U/S corner rounded to 1½" radius.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.04	0.0294	2.660	0.0206	0.4120
2.	0.10	0.08	0.0875	2.809	0.0442	0.4420
3.	0.15	0.12	0.1637	2.860	0.0675	0.4450
4.	0.20	0.16	0.2562	2.908	0.0895	0.4485
5.	0.25	0.20	0.3600	2.922	0.1150	0.4601
6.	0.30	0.24	0.4812	2.970	0.1388	0.4627
7.	0.35	0.28	0.6118	2.997	0.1640	0.4685
8.	0.40	0.32	0.7525	3.018	0.1885	0.4712
9.	0.45	0.36	0.9075	3.052	0.2120	0.4711
10.	0.50	0.40	1.0800	3.099	0.2360	0.4720

Table 29

Series No. 13 Crest length L = 1.25 ft. ; Both faces vertical
Crest height P = 1.0 ft. ; U/S corner rounded to 2" radius.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.04	0.0294	2.660	0.0206	0.4120
2.	0.10	0.08	0.0875	2.809	0.0460	0.4600
3.	0.15	0.12	0.1637	2.860	0.0698	0.4653
4.	0.20	0.16	0.2562	2.908	0.0927	0.4635
5.	0.25	0.20	0.3600	2.922	0.1177	0.4708
6.	0.30	0.24	0.4812	2.970	0.1404	0.4680
7.	0.35	0.28	0.6087	2.983	0.1640	0.4686
8.	0.40	0.32	0.7500	3.009	0.1895	0.4737
9.	0.45	0.36	0.9075	3.052	0.2107	0.4682
10.	0.50	0.40	1.0800	3.099	0.2360	0.4720

Table 30

Series No. 16 Crest length L = 1.25 ft. ; Both faces vertical ;
Crest height P = 1.0 ft. ; U/S corner rounded to $2\frac{1}{2}$" radius.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.04	0.0294	2.660	0.0206	0.4120
2.	0.10	0.08	0.0875	2.809	0.0460	0.4600
3.	0.15	0.12	0.1637	2.860	0.0698	0.4653
4.	0.20	0.16	0.2562	2.908	0.0927	0.4635
5.	0.25	0.20	0.3600	2.922	0.1177	0.4708
6.	0.30	0.24	0.4812	2.970	0.1404	0.4680
7.	0.35	0.28	0.6087	2.983	0.1640	0.4686
8.	0.40	0.32	0.7500	3.009	0.1895	0.4737
9.	0.45	0.36	0.9075	3.052	0.2107	0.4682
10.	0.50	0.40	1.0800	3.099	0.2361	0.4722

Table 31

Series No. 5 Crest length L = 1.25 ft. ; Both faces vertical ;
Crest height P = 0.75 ft. ; U/S corner rounded to ½" radius.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.04	0.0275	2.492	0.0197	0.3936
2.	0.10	0.08	0.0843	2.707	0.0426	0.4260
3.	0.15	0.12	0.1575	2.751	0.0672	0.4450
4.	0.20	0.16	0.2494	2.931	0.0885	0.4425
5.	0.25	0.20	0.3512	2.851	0.1132	0.4529
6.	0.30	0.24	0.4637	2.863	0.1360	0.4533
7.	0.35	0.28	0.5900	2.892	0.1575	0.4450
8.	0.40	0.32	0.7255	2.910	0.1820	0.4550
‡9.	0.45	0.36	0.8675	2.917		
10.	0.50	0.40	1.0200	2.927		

‡ Nappe depressed

Table 32

Series No. 8 Crest length L = 1.25 ft. ; Both faces vertical
Crest height P = 0.75 ft. ; U/S corner rounded to 1" radius.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.04	0.0294	2.660	0.0203	0.4060
2.	0.10	0.08	0.0875	2.809	0.0442	0.4420
3.	0.15	0.12	0.1612	2.816	0.0675	0.4450
4.	0.20	0.16	0.2533	2.876	0.0886	0.4430
5.	0.25	0.20	0.3587	2.913	0.1150	0.4601
6.	0.30	0.24	0.4762	2.940	0.1388	0.4627
7.	0.35	0.28	0.6112	2.983	0.1623	0.4636
8.	0.40	0.32	0.7525	3.018	0.1875	0.4686
9.	0.45	0.36	0.9119	3.066	0.2130	0.4737
‡10.	0.50	0.40	1.0800	3.099		

‡ Nappe depressed

Table 33

Series No. 11 Crest length $L = 1.25$ ft. ; Both faces vertical ;
Crest height $P = 0.75$ ft. ; U/S corner rounded to $1\frac{1}{2}$" radius.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.04	0.0294	2.660	0.0206	0.4120
2.	0.10	0.08	0.0875	2.809	0.0442	0.4420
3.	0.15	0.12	0.1637	2.860	0.0675	0.4450
4.	0.20	0.16	0.2562	2.908	0.0895	0.4475
5.	0.25	0.20	0.3600	2.922	0.1155	0.4619
6.	0.30	0.24	0.4812	2.970	0.1390	0.4634
7.	0.35	0.28	0.6175	3.026	0.1655	0.4729
‡8.	0.40	0.32	0.7713	3.094		
9.	0.45	0.36	0.9212	3.098		
10.	0.50	0.40	-	-		

‡ Nappe depressed

Table 34

Series No. 14 — Crest length L = 1.25 ft. ; Crest height P = 0.75 ft. ; Both faces vertical; U/S corner rounded to 2" radius.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.04	0.0294	2.660	0.0210	0.4200
2.	0.10	0.08	0.0875	2.809	0.0460	0.4600
3.	0.15	0.12	0.1637	2.860	0.0698	0.4653
4.	0.20	0.16	0.2562	2.908	0.0927	0.4635
5.	0.25	0.20	0.3600	2.922	0.1177	0.4708
6.	0.30	0.24	0.4812	2.970	0.1404	0.4680
7.	0.35	0.28	0.6125	3.002	0.1650	0.4714
8.	0.40	0.32	0.7525	3.018	0.1930	0.4825
‡9.	0.45	0.36	0.9162	3.081		
10.	0.50	0.40	-	-		

‡ Nappe depressed

Table 35

Series No. 17 Crest length L = 1.25 ft. ; Both faces vertical ;
Crest height P = 0.75 ft. ; U/S corner rounded to $2\frac{1}{2}$" radius

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.04	0.0294	2.660	0.0210	0.4200
2.	0.10	0.08	0.0875	2.809	0.0460	0.4600
3.	0.15	0.12	0.1637	2.860	0.0698	0.4653
4.	0.20	0.16	0.2562	2.908	0.0927	0.4635
5.	0.25	0.20	0.3600	2.922	0.1177	0.4708
6.	0.30	0.24	0.4812	2.970	0.1404	0.4680
7.	0.35	0.28	0.6125	3.002	0.1650	0.4714
8.	0.40	0.32	0.7525	3.018	0.1930	0.4825
‡9.	0.45	0.36	0.9162	3.081		
10.	0.50	0.40	-	-		

‡ Nappe depressed

Table 36

Series No. 6 Crest length L = 1.25 ft. ; Both faces vertical;
Crest height P = 0.50 ft. ; U/S corner rounded to $\frac{1}{2}$" radius

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.04	0.0275	2.492	0.0210	0.4150
2.	0.10	0.08	0.0843	2.707	0.0426	0.4260
3.	0.15	0.12	0.1575	2.751	0.0662	0.4414
4.	0.20	0.16	0.2500	2.838	0.0852	0.4260
5.	0.25	0.20	0.3525	2.862	0.1115	0.4461
6.	0.30	0.24	0.4675	2.886	0.1351	0.4503

Table 37

Series No. 9 — Crest length L = 1.25 ft. ; Crest height P = 0.50 ft. ; Both faces vertical ; U/S corner rounded to 1" radius

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.04	0.0294	2.660	0.0206	0.4120
2.	0.10	0.08	0.0875	2.809	0.0442	0.4420
3.	0.15	0.12	0.1637	2.860	0.0675	0.4450
4.	0.20	0.16	0.2550	2.895	0.0895	0.4485
5.	0.25	0.20	0.3600	2.922	0.1150	0.4601
6.	0.30	0.24	0.4863	3.002	0.1395	0.4650
‡7.	0.35	0.28	0.6280	3.077		
8.	0.40	0.32	0.7775	3.119		
9.	0.45	0.36	0.9368	3.151		

‡ Nappe depressed

Table 38

Series No. 12 Crest length L = 1.25 ft. ; Crest height P = 0.50 ft. ; Both faces vertical U/S corner rounded to $1\frac{1}{2}$" radius

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.04	0.0294	2.660	0.0206	0.4120
2.	0.10	0.08	0.0875	2.809	0.0442	0.4420
3.	0.15	0.12	0.1637	2.860	0.0675	0.4450
4.	0.20	0.16	0.2550	2.895	0.0903	0.4515
5.	0.25	0.20	0.3600	2.922	0.1165	0.4660
6.	0.30	0.24	0.4863	3.002	0.1395	0.4650
‡7.	0.35	0.28	0.6300	3.087		
8.	0.40	0.32	0.7775	3.119		
9.	0.45	0.36	0.9275	3.119		
10.	0.50	0.40	-	-		

‡ Nappe depressed

Table 39

Series No. 15 Crest length L = 1.25 ft. ; Both faces vertical
Crest height P = 0.50 ft. ; U/S corner rounded to 2" radius.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.04	0.0294	2.660	0.0213	0.4260
2.	0.10	0.08	0.0875	2.809	0.0460	0.4600
3.	0.15	0.12	0.1637	2.860	0.0722	0.4814
4.	0.20	0.16	0.2562	2.908	0.0950	0.4750
5.	0.25	0.20	0.3600	2.922	0.1203	0.4812
6.	0.30	0.24	0.4812	2.970	0.1460	0.4867
‡7.	0.35	0.28	0.6137	3.007		
8.	0.40	0.32	0.7600	3.049		
9.	0.45	0.36	0.9212	3.098		
10.	0.50	0.40	-	-		

‡ Nappe depressed

Table 40

Series No. 18 Crest length L = 1.25 ft. ; Both faces vertical ;
Crest height P = 0.75 ft. ; U/S corner rounded to 2½" radius.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft.	y_e / h
1.	0.05	0.04	0.0294	2.660	0.0213	0.4260
2.	0.10	0.08	0.0875	2.809	0.0460	0.4600
3.	0.15	0.12	0.1637	2.860	0.0722	0.4813
4.	0.20	0.16	0.2562	2.908	0.0950	0.4750
5.	0.25	0.20	0.3600	2.922	0.1203	0.4812
6.	0.30	0.24	0.4812	2.970	0.1460	0.4867
‡7.	0.35	0.28	0.6137	3.007		
8.	0.40	0.32	0.7600	3.049		
9.	0.45	0.36	0.9212	3.098		
10.	0.50	0.40	-	-		

‡ Nappe depressed

Table 41

Series No. 35 — Crest length L = 1.0 ft. ; Both faces vertical
Crest height P = 1.0 ft. ; U/S corner rounded to 1" radius.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.05	0.0275	2.492	0.0204	0.4080
2.	0.10	0.10	0.0842	2.704	0.0439	0.4390
3.	0.15	0.15	0.1587	2.772	0.0678	0.4520
4.	0.20	0.20	0.2500	2.838	0.0915	0.4575
5.	0.25	0.25	0.3496	2.839	0.1137	0.4548
6.	0.30	0.30	0.4658	2.876	0.1370	0.4540
7.	0.35	0.35	0.6000	2.941	0.1613	0.4608
8.	0.40	0.40	0.7617	3.056	0.1834	0.4585
9.	0.45	0.45	0.9117	3.066	0.2075	0.4611
10.	0.50	0.50	1.0875	3.121	0.2340	0.4680

Table 42

Series No. 36 — Crest length L = 1.0 ft. ; Crest height P = 1.0 ft. ; Both faces vertical U/S corner rounded to 1½" radius.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.05	0.0275	2.492	0.0207	0.4140
2.	0.10	0.10	0.0842	2.704	0.0443	0.4430
3.	0.15	0.15	0.1600	2.795	0.0682	0.4547
4.	0.20	0.20	0.2519	2.860	0.0918	0.4590
5.	0.25	0.25	0.3575	2.902	0.1148	0.4592
6.	0.30	0.30	0.4688	2.894	0.1395	0.4650
7.	0.35	0.35	0.6125	3.002	0.1640	0.4686
8.	0.40	0.40	0.7806	3.131	0.1890	0.4725
9.	0.45	0.45	0.9413	3.166	0.2138	0.4751
10.	0.50	0.50	1.1050	3.170	0.2400	0.4800

Table 43

Series No. 37 — Crest length L = 1.0 ft. ; Both faces vertical
Crest height P = 1.0 ft. ; U/S corner rounded to 2" radius.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.05	0.0275	2.492	0.0207	0.4140
2.	0.10	0.10	0.0842	2.704	0.0446	0.4460
3.	0.15	0.15	0.1600	2.795	0.0685	0.4566
4.	0.20	0.20	0.2537	2.880	0.0918	0.4590
5.	0.25	0.25	0.3580	2.907	0.1154	0.4616
6.	0.30	0.30	0.4750	2.932	0.1432	0.4773
7.	0.35	0.35	0.6230	3.053	0.1647	0.4706
8.	0.40	0.40	0.7837	3.145	0.1915	0.4787
9.	0.45	0.45	0.9415	3.166	0.2199	0.4886
10.	0.50	0.50	1.1050	3.170	0.2465	0.4930

Table 44

Series No. 48 — Crest length $L = 1.0$ ft. ; Crest height $P = 1.0$ ft. ; Both faces vertical ; U/S corner rounded to $2\frac{1}{2}$" radius.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.05	0.0275	2.492	0.0207	0.4140
2.	0.10	0.10	0.0842	2.704	0.0446	0.4460
3.	0.15	0.15	0.1600	2.795	0.0685	0.4567
4.	0.20	0.20	0.2537	2.880	0.0918	0.4590
5.	0.25	0.25	0.3580	2.907	0.1154	0.4616
6.	0.30	0.30	0.4750	2.932	0.1432	0.4773
7.	0.35	0.35	0.6230	3.053	0.1647	0.4706
8.	0.40	0.40	0.7837	3.145	0.1915	0.4787
9.	0.45	0.45	0.9415	3.166	0.2199	0.4887
10.	0.50	0.50	1.1050	3.170	0.2465	0.4930

Table 45

Series No. 42 Crest length L = 1.0 ft. ; Crest height P = 0.75 ft.; Both faces vertical U/S Corner rounded to 2" radius.

S.No.	U/s head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.05	0.0275	2.492	0.0207	0.4140
2.	0.10	0.10	0.0842	2.704	0.0446	0.4460
3.	0.15	0.15	0.1600	2.795	0.0685	0.4567
4.	0.20	0.20	0.2537	2.880	0.0918	0.4590
5.	0.25	0.25	0.3580	2.907	0.1154	0.4616
6.	0.30	0.30	0.4750	2.932	0.1432	0.4773
7.	0.35	0.35	0.6230	3.053	0.1647	0.4706
8.	0.40	0.40	0.7837	3.145	0.1915	0.4787
9.	0.45	0.45	0.9415	3.166	0.2144	0.4764
‡10.	0.50	0.50	1.1050	3.170	—	—

‡ Nappe depressed

Table 47

Series No. 47 Crest length L = 1.0 ft. ; Both faces vertical
Crest height P = 0.5 ft. ; U/S corner rounded 2" radius.

S. No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.05	0.0275	2.492	0.0207	0.4140
2.	0.10	0.10	0.0842	2.704	0.0446	0.4460
3.	0.15	0.15	0.1600	2.795	0.0688	0.4587
4.	0.20	0.20	0.2537	2.880	0.0918	0.4590
5.	0.25	0.25	0.3580	2.907	0.1154	0.4616
6.	0.30	0.30	0.4750	2.932	0.1432	0.4773
‡ 7.	0.35	0.35	0.6230	3.053		
8.	0.40	0.40	0.7837	3.145		
9.	0.45	0.45	0.9415	3.166		
10.	0.50	0.50	1.1050	3.170		

‡ Nappe depressed

Table 46

Series No. 49 Crest length $L = 1.0$ ft. ; Both faces vertical
Crest height $P = 0.75$ ft.; U/S corner rounded to $2\frac{1}{2}$" radius.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.05	0.0275	2.492	0.0207	0.4140
2.	0.10	0.10	0.0842	2.704	0.0446	0.4460
3.	0.15	0.15	0.1600	2.795	0.0685	0.4567
4.	0.20	0.20	0.2537	2.880	0.0918	0.4590
5.	0.25	0.25	0.3580	2.907	0.1154	0.4616
6.	0.30	0.30	0.4750	2.932	0.1432	0.4773
7.	0.35	0.35	0.6230	3.053	0.1647	0.4706
8.	0.40	0.40	0.7837	3.145	0.1915	0.4787
9.	0.45	0.45	0.9415	3.166	0.2199	0.4887
‡10.	0.50	0.50	1.1050	3.170		

‡ Nappe depressed

Table 47

Series No. 47 — Crest length L = 1.0 ft. ; Both faces vertical
Crest height P = 0.5 ft. ; U/S corner rounded 2" radius.

S. No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e / h
1.	0.05	0.05	0.0275	2.492	0.0207	0.4140
2.	0.10	0.10	0.0842	2.704	0.0446	0.4460
3.	0.15	0.15	0.1600	2.795	0.0688	0.4587
4.	0.20	0.20	0.2537	2.880	0.0918	0.4590
5.	0.25	0.25	0.3580	2.907	0.1154	0.4616
6.	0.30	0.30	0.4750	2.932	0.1432	0.4773
‡ 7.	0.35	0.35	0.6230	3.053		
8.	0.40	0.40	0.7837	3.145		
9.	0.45	0.45	0.9415	3.166		
10.	0.50	0.50	1.1050	3.170		

‡ Nappe depressed

Table 48

Series No. 50 Crest length L = 1.0 ft. ; Crest height P = 0.50 ft.; Both faces vertical U/S corner rounded to $2\frac{1}{2}''$ radius.

S.No.	U/S head in ft. h	h/L	Discharge in c.f.s. Q	Discharge Coefficient C_r	Flow depth at D/S end of crest in ft. y_e	y_e/h
1.	0.05	0.05	0.0275	2.492	0.0207	0.4140
2.	0.10	0.10	0.0842	2.704	0.0446	0.4460
3.	0.15	0.15	0.1600	2.795	0.0688	0.4586
4.	0.20	0.20	0.2537	2.880	0.0918	0.4590
5.	0.25	0.25	0.3580	2.907	0.1154	0.4616
6.	0.30	0.30	0.4750	2.932	0.1432	0.4773
‡7.	0.35	0.35	0.6230	3.053		
8.	0.40	0.40	0.7837	3.145		
9.	0.45	0.45	0.9415	3.166		
10.	0.50	0.50	1.1050	3.170		

‡ Nappe depressed

Table 49

Series No. 21 — Crest length L = 1.0 ft. Crest height P = 1.0 ft. — Both faces vertical ; Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Submergence depth in ft. Z	Submergence ratio Z/h	Discharge in c.f.s. Q	Discharge Coefficient C_s	Modular discharge Coefficient C	C_s/ C
ab1.	0.20	0.20	0.148	0.740	0.2213	2.513	2.513	1.000
2.	0.20	0.20	0.158	0.790	0.2150	2.441	2.513	0.9714

Table 50

Series No. 22 — Crest length L = 1.0 ft. Crest height P = 1.0 ft. — Both faces vertical ; Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Submergence depth in ft. Z	Submergence ratio Z/h	Discharge in c.f.s. Q	Discharge Coefficient C_s	Modular discharge Coefficient C	C_s / C
a 1. b	0.30	0.30	0.222	0.740	0.4143	2.558	2.558	1.000
2.	0.30	0.30	0.240	0.800	0.4087	2.523	2.558	0.9863
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.4000	2.463	2.558	0.9627

Table 49

Series No. 21 Crest length L = 1.0 ft. Both faces vertical ;
Crest height P = 1.0 ft. Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Submergence depth in ft. Z	Submergence ratio Z/h	Discharge in c.f.s. Q	Discharge Coefficient C_s	Modular discharge Coefficient C	C_s/C
a b 1.	0.20	0.20	0.148	0.740	0.2213	2.513	2.513	1.000
2.	0.20	0.20	0.158	0.790	0.2150	2.441	2.513	0.9714
3.	0.20	0.20	0.163	0.815	0.2100	2.384	2.513	0.9486
4.	0.20	0.20	0.167	0.835	0.2050	2.328	2.513	0.9264
5.	0.20	0.20	0.174	0.870	0.1900	2.157	2.513	0.8582
6.	0.20	0.20	0.180	0.900	0.1700	1.930	2.513	0.7680
7.	0.20	0.20	0.185	0.925	0.1562	1.773	2.513	0.7057
8.	0.20	0.20	0.187	0.935	0.1432	1.626	2.513	0.6471
c 9.	0.20	0.20	0.190	0.950	0.1287	1.461	2.513	0.5815
b 10.	0.20	0.20	0.192	0.960	0.1175	1.334	2.513	0.5309
111	0.20	0.20	0.195	0.975	0.1012	1.149	2.513	0.4573
12.	0.20	0.20	0.196	0.980	0.0838	0.951	2.513	0.3785
13.	0.20	0.20	0.200	1.000	0.0293	0.333	2.513	0.1323

a. Modularity limit ; b. Surge at D/S end of crest ; c. Surge flattens out;

d. Smmoth nappe surface.

Table 50

Series No. 22 | Crest length L = 1.0 ft. | Crest height P = 1.0 ft. | Both faces vertical ; | Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Submergence depth in ft. Z	Submergence ratio Z/h	Discharge in c.f.s. Q	Discharge Coefficient C_s	Modular discharge Coefficient C	C_s / C
[a,b] 1.	0.30	0.30	0.222	0.740	0.4143	2.558	2.558	1.000
2.	0.30	0.30	0.240	0.800	0.4087	2.523	2.558	0.9863
3.	0.30	0.30	0.243	0.810	0.4000	2.463	2.558	0.9627
4.	0.30	0.30	0.255	0.850	0.3812	2.353	2.558	0.9200
5.	0.30	0.30	0.261	0.870	0.3600	2.222	2.558	0.8688
6.	0.30	0.30	0.266	0.887	0.3438	2.122	2.558	0.8294
7.	0.30	0.30	0.270	0.900	0.3237	1.998	2.558	0.7811
8.	0.30	0.30	0.276	0.920	0.3020	1.864	2.558	0.7286
[c] 9.	0.30	0.30	0.281	0.937	0.2667	1.646	2.558	0.6434
[d] 10.	0.30	0.30	0.289	0.963	0.2237	1.381	2.558	0.5399
11.	0.30	0.30	0.291	0.970	0.1975	1.219	2.558	0.4765
12.	0.30	0.30	0.294	0.980	0.1487	0.918	2.558	0.3588
13.	0.30	0.30	0.297	0.990	0.1206	0.744	2.558	0.2910
14.	0.30	0.30	0.300	1.000	0.0325	0.199	2.558	0.0778

a. Modularity limit ; b. Surge at D/S end of crest ; c. Surge flattens out

d. Smooth nappe surface.

Table 51

Series No. 23 Crest length L = 1.0 ft. Crest height P = 1.0 ft. Both faces vertical ; Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Submergence depth in ft. Z	Submergence ratio Z/h	Discharge in c.f.s. Q	Discharge Coefficient C_s	Modular discharge Coefficient C	C_s / C
[a]1.	0.40	0.40	0.270	0.675	0.6650	2.668	2.668	1.000
2.	0.40	0.40	0.295	0.737	0.6575	2.638	2.668	0.9888
[b]3.	0.40	0.40	0.317	0.792	0.6413	2.572	2.668	0.9638
4.	0.40	0.40	0.343	0.858	0.6034	2.420	2.668	0.9069
5.	0.40	0.40	0.355	0.890	0.5550	2.226	2.668	0.8345
6.	0.40	0.40	0.367	0.918	0.4863	1.951	2.668	0.7311
7.	0.40	0.40	0.373	0.933	0.4438	1.780	2.668	0.6671
[c]8.	0.40	0.40	0.380	0.950	0.3925	1.574	2.668	0.5899
[d]9.	0.40	0.40	0.386	0.965	0.3337	1.339	2.668	0.5019
10.	0.40	0.40	0.390	0.975	0.2837	1.139	2.668	0.4270
11.	0.40	0.40	0.393	0.982	0.2175	0.873	2.668	0.3271
12.	0.40	0.40	0.396	0.990	0.1617	0.749	2.668	0.2431
13.	0.40	0.40	0.400	1.000	0.0350	0.140	2.668	0.0526

a. Modularity limit ; b. Surge on D/S end of crest ; c. Surge flattens out.

d. Smooth nappe surface.

Table 52

Series No. 24 — Crest length L = 1.0 ft. Crest height P = 1.0 ft. — Both faces vertical ; Sharp U/S corner.

S.No.	U/S head in ft. h	h/L	Submergence depth in ft. Z	Submergence ratio Z/h	Discharge in c.f.s. Q	Discharge Coefficient C_s	Modular discharge Coefficient C	C_s / C
[a] 1.	0.50	0.50	0.220	0.440	0.9425	2.708	2.708	1.000
2.	0.50	0.50	0.230	0.460	0.9369	2.689	2.708	0.9928
3.	0.50	0.50	0.256	0.512	0.9300	2.669	2.708	0.9856
4.	0.50	0.50	0.345	0.690	0.9200	2.640	2.708	0.9747
[b] 5.	0.50	0.50	0.425	0.850	0.8675	2.490	2.708	0.9194
6.	0.50	0.50	0.440	0.880	0.8287	2.379	2.708	0.8784
7.	0.50	0.50	0.450	0.900	0.7750	2.224	2.708	0.8213
8.	0.50	0.50	0.462	0.924	0.6956	1.996	2.708	0.7370
[c] 9.	0.50	0.50	0.480	0.960	0.5262	1.510	2.708	0.5576
[d] 10.	0.50	0.50	0.487	0.974	0.4287	1.230	2.708	0.4541
11.	0.50	0.50	0.492	0.984	0.3250	0.933	2.708	0.3444
12.	0.50	0.50	0.494	0.988	0.2750	0.789	2.708	0.2915
13.	0.50	0.50	0.500	1.000	0.1137	0.326	2.708	0.1205

a. Modularity limit ; b. Surge at D/S end of crest ; c. Surge flattens out

d. Smooth nappe surface.

Table 53

Series No. 38 — Crest length L = 1.0 ft. Crest height P = 1.0 ft. — Both faces vertical. U/S corner rounded to 2" radius.

S.No.	U/S head in ft. h	h/L	Submergence depth in ft. Z	Submergence ratio Z/h	Discharge in c.f.s. Q	Discharge Coefficient C_s	Modular discharge Coefficient C	C_s / C
‡1.	0.20	0.20	0.158	0.790	0.2537	2.880	2.880	1.000
2.	0.20	0.20	0.161	0.805	0.2520	2.861	2.880	0.9936
3.	0.20	0.20	0.165	0.825	0.2487	2.823	2.880	0.9802
4.	0.20	0.20	0.172	0.860	0.2413	2.739	2.880	0.9510
5.	0.20	0.20	0.178	0.890	0.2319	2.632	2.880	0.9139
6.	0.20	0.20	0.183	0.915	0.2175	2.469	2.880	0.8572
7.	0.20	0.20	0.188	0.940	0.1975	2.243	2.880	0.7787
8.	0.20	0.20	0.192	0.960	0.1737	1.972	2.880	0.6847
9.	0.20	0.20	0.1965	0.982	0.1550	1.760	2.880	0.6110
10.	0.20	0.20	0.200	1.000	0.1213	1.377	2.880	0.4780

‡ Modularity limit

Table 54

Series No. 39 : Crest length L = 1.0 ft.; Crest height P = 1.0 ft. Both faces vertical ; U/S. corner rounded to 2" radius.

S.No.	U/S. head in ft. h	h/L	Submergence depth in ft. Z	Submergence ratio Z/h	Discharge in c.f.s. Q	Discharge Coefficient C_s	Modular discharge Coefficient C	C_s / C
‡1.	0.30	0.30	0.219	0.730	0.4750	2.932	2.932	1.000
2.	0.30	0.30	0.240	0.800	0.4737	2.924	2.932	0.0072
3.	0.30	0.30	0.257	0.857	0.4470	2.759	2.932	0.9408
4.	0.30	0.30	0.264	0.880	0.4287	2.646	2.932	0.9024
5.	0.30	0.30	0.273	0.910	0.4000	2.469	2.932	0.8420
6.	0.30	0.30	0.279	0.930	0.3737	2.307	2.932	0.7866
7.	0.30	0.30	0.285	0.950	0.3363	2.076	2.932	0.7081
8.	0.30	0.30	0.290	0.967	0.2969	1.832	2.932	0.6248
9.	0.30	0.30	0.296	0.987	0.2444	1.509	2.932	0.5146
10.	0.30	0.30	0.300	1.000	0.1413	0.8720	2.932	0.2974

‡ Modularity limit

Table 55

Series No. 40 — Crest length L = 1.0 ft. Crest height P = 1.0 ft. — Both faces vertical U/S corner rounded to 2" radius.

S.No.	U/S head in ft. h	h/L	Submergence depth in ft. Z	Submergence ratio Z/h	Discharge in c.f.s. Q	Discharge Coefficient C_s	Modular discharge Coefficient C	C_s / C
‡1.	0.40	0.40	0.232	0.580	0.7837	3.145	3.145	1.000
2.	0.40	0.40	0.272	0.680	0.7713	3.094	3.145	0.9840
3.	0.40	0.40	0.312	0.780	0.7500	3.009	3.145	0.9568
4.	0.40	0.40	0.338	0.845	0.7370	2.956	3.145	0.9399
5.	0.40	0.40	0.346	0.865	0.7225	2.898	3.145	0.9215
6.	0.40	0.40	0.352	0.880	0.6812	2.732	3.145	0.8688
7.	0.40	0.40	0.368	0.920	0.6175	2.477	3.145	0.7875
8.	0.40	0.40	0.380	0.950	0.5150	2.065	3.145	0.6567
9.	0.40	0.40	0.389	0.972	0.4300	1.725	3.145	0.5486
10.	0.40	0.40	0.395	0.987	0.3875	1.554	3.145	0.4941

‡ Modularity limit

Table 56

Series No. 41 Crest length L = 1.0 ft. Crest height P = 1.0 ft. Both faces vertical ; U/S corner rounded to 2" radius.

S.No.	U/S head in ft. h	h/L	Submergence depth in ft. Z	Submergence ratio Z/h	Discharge in c.f.s. Q	Discharge Coefficient C_s	Modular discharge Coefficient C	C_s / C
1.	0.50	0.50	0.260	0.520	1.1050	3.170	3.170	1.000
2	0.50	0.50	0.285	0.570	1.0750	3.085	3.170	0.9731

Table 57

Series No. 43 Crest length L = 1.0 ft. Crest height P = 0.75 ft. Both faces vertical ; U/S corner rounded to 2" radius.

S.No.	U/S. head in ft. h	h/L	Submergence depth in ft. Z	Submergence ratio Z/h	Discharge in c.f.s. Q	Discharge Coefficient C_s	Modular discharge Coefficient C	C_s / C
‡1.	0.20	0.20	0.162	0.810	0.2537	2.880	2.880	1.000
2.	0.20	0.20	0.169	0.845	0.2392	2.716	2.880	0.9430
3.	0.20	0.20	0.180	0.900	0.2150	2.441	2.880	0.8476
4.	0.20	0.20	0.184	0.920	0.2008	2.279	2.880	0.7912
5.	0.20	0.20	0.188	0.940	0.1800	2.044	2.880	0.7096
6.	0.20	0.20	0.190	0.950	0.1667	1.892	2.880	0.6569
7.	0.20	0.20	0.194	0.970	0.1425	1.617	2.880	0.5614
8.	0.20	0.20	0.195	0.975	0.1356	1.539	2.880	0.5344
9.	0.20	0.20	0.196	0.980	0.1200	1.362	2.880	0.4730
10.	0.20	0.20	0.200	1.000	0.0700	1.230	2.880	0.4271

‡ Modularity limit

Table 56

Series No. 41 Crest length L = 1.0 ft. Crest height P = 1.0 ft. Both faces vertical ; U/S corner rounded to 2" radius.

S.No.	U/S head in ft. h	h/L	Submergence depth in ft. Z	Submergence ratio Z/h	Discharge in c.f.s. Q	Discharge Coefficient C_s	Modular discharge Coefficient C	C_s / C
[a]1.	0.50	0.50	0.260	0.520	1.1050	3.170	3.170	1.000
2.	0.50	0.50	0.285	0.570	1.0750	3.085	3.170	0.9731
3.	0.50	0.50	0.335	0.670	1.0550	3.028	3.170	0.9550
4.	0.50	0.50	0.360	0.720	1.0350	2.970	3.170	0.9369
5.	0.50	0.50	0.410	0.820	1.0225	2.934	3.170	0.9255
6.	0.50	0.50	0.425	0.850	1.0037	2.880	3.170	0.9084
7.	0.50	0.50	0.447	0.894	0.9602	2.763	3.170	0.8716
8.	0.50	0.50	0.445	0.910	0.9294	2.668	3.170	0.8416
9.	0.50	0.50	0.465	0.930	0.8625	2.476	3.170	0.7811
10.	0.50	0.50	0.472	0.944	0.7875	2.260	3.170	0.7129
11.	0.50	0.50	0.483	0.966	0.6725	1.931	3.170	0.6091
12.	0.50	0.50	0.491	0.982	0.5900	1.693	3.170	0.5340
13.	0.50	0.50	0.500	1.000	0.3850	1.105	3.170	0.3485

a. Modularity limit

Table 57

Series No. 43

Crest length L = 1.0 ft.
Crest height P = 0.75 ft.

Both faces vertical ;
U/S corner rounded to 2" radius.

S.No.	U/S head in ft. h	h/L	Submergence depth in ft. Z	Submergence ratio Z/h	Discharge in c.f.s. Q	Discharge Coefficient C_s	Modular discharge Coefficient C	C_s / C
‡1.	0.20	0.20	0.162	0.810	0.2537	2.880	2.880	1.000
2.	0.20	0.20	0.169	0.845	0.2392	2.716	2.880	0.9430
3.	0.20	0.20	0.180	0.900	0.2150	2.441	2.880	0.8476
4.	0.20	0.20	0.184	0.920	0.2008	2.279	2.880	0.7912
5.	0.20	0.20	0.188	0.940	0.1800	2.044	2.880	0.7096
6.	0.20	0.20	0.190	0.950	0.1667	1.892	2.880	0.6569
7.	0.20	0.20	0.194	0.970	0.1425	1.617	2.880	0.5614
8.	0.20	0.20	0.195	0.975	0.1356	1.539	2.880	0.5344
9.	0.20	0.20	0.196	0.980	0.1200	1.362	2.880	0.4730
10.	0.20	0.20	0.200	1.000	0.0700	1.230	2.880	0.4271

‡ Modularity limit

Table 58

Series No. 44 — Crest length L = 1.0 ft. Crest height P = 0.75 ft. — Both faces vertical ; U/S corner rounded to 2" radius.

S.No.	U/S head in ft. h	h/L	Submergence depth in ft. Z	Submergence ratio Z/h	Discharge in c.f.s. Q	Discharge Coefficient C_s	Modular discharge Coefficient C	C_s / C
‡1.	0.30	0.30	0.228	0.760	0.4750	2.932	2.932	1.000
2.	0.30	0.30	0.238	0.793	0.4694	2.898	2.932	0.9883
3.	0.30	0.30	0.257	0.857	0.4600	2.840	2.932	0.9685
4.	0.30	0.30	0.261	0.870	0.4512	2.786	2.932	0.9499
5.	0.30	0.30	0.267	0.890	0.4283	2.643	2.932	0.9014
6.	0.30	0.30	0.272	0.907	0.4112	2.538	2.932	0.8656
7.	0.30	0.30	0.278	0.927	0.3756	2.318	2.932	0.7905
8.	0.30	0.30	0.284	0.947	0.3344	2.064	2.932	0.7039
9.	0.30	0.30	0.289	0.963	0.3062	1.890	2.932	0.6446
10.	0.30	0.30	0.291	0.970	0.2688	1.659	2.932	0.5657
11.	0.30	0.30	0.296	0.987	0.2175	1.343	2.932	0.4581
12.	0.30	0.30	0.300	1.000	0.1450	1.246	2.932	0.4249

‡ Modularity limit

APPENDIX III

Surface profiles of flow over crests

Table 59

Series No. 45. Crest length L = 1.0 ft. Crest height P = 0.75 ft. Both faces vertical ; U/S corner rounded to 2" radius.

S.No.	U/S head in ft. h	h/L	Submergence depth in ft. Z	Submergence ratio Z/h	Discharge in c.f.s. Q	Discharge Coefficient C_s	Modular discharge Coefficient C	C_s / C
a 1.	0.40	0.40	0.240	0.620	0.7837	3.145	3.145	1.000
2.	0.40	0.40	0.256	0.640	0.7675	3.079	3.145	0.9790
3.	0.40	0.40	0.300	0.750	0.7550	3.028	3.145	0.9627
4.	0.40	0.40	0.324	0.810	0.7450	2.988	3.145	0.9501
5.	0.40	0.40	0.342	0.855	0.7262	2.913	3.145	0.9262
6.	0.40	0.40	0.356	0.890	0.7158	2.872	3.145	0.9133
7.	0.40	0.40	0.362	0.905	0.6750	2.708	3.145	0.8612
8.	0.40	0.40	0.374	0.935	0.5875	2.357	3.145	0.7496
9.	0.40	0.40	0.382	0.955	0.5075	2.035	3.145	0.6471
10.	0.40	0.40	0.390	0.975	0.4081	1.637	3.145	0.5206
11.	0.40	0.40	0.394	0.985	0.3300	1.323	3.145	0.4207
12.	0.40	0.40	0.400	1.000	0.1737	1.2141	3.145	0.3860

a. Modularity limit.

Table 60

Series No. 46 — Crest length $L = 1.0$ ft. Crest height $P = 0.75$ ft. — Both faces vertical ; U/S corner rounded to 2" radius.

S.No.	U/S head in ft. h	h/L	Submergence depth in ft. Z	Submergence ratio S/h	Discharge in c.f.s. Q	Discharge Coefficient C_s	Modular discharge Coefficient C	C_s / C
[a] 1.	0.50	0.50	0.300	0.600	1.1050	3.170	3.170	1.000
2.	0.50	0.50	0.360	0.720	1.0612	3.045	3.170	0.9605
3.	0.50	0.50	0.388	0.776	1.0512	3.016	3.170	0.9515
4.	0.50	0.50	0.415	0.830	1.0363	2.974	3.170	0.9383
5.	0.50	0.50	0.423	0.846	1.0275	2.949	3.170	0.9301
6.	0.50	0.50	0.442	0.884	0.9938	2.852	3.170	0.8995
7.	0.50	0.50	0.453	0.906	0.9575	2.748	3.170	0.8670
8.	0.50	0.50	0.462	0.924	0.9137	2.622	3.170	0.8269
9.	0.50	0.50	0.465	0.930	0.8842	2.538	3.170	0.8005
10.	0.50	0.50	0.472	0.944	0.8075	2.318	3.170	0.7311
11.	0.50	0.50	0.483	0.966	0.7183	2.062	3.170	0.6504
12.	0.50	0.50	0.490	0.980	0.5617	1.612	3.170	0.5086
13.	0.50	0.50	0.495	0.990	0.4287	1.230	3.170	0.3880
14.	0.50	0.50	0.500	1.000	0.2525	1.219	3.170	0.3845

a. Modularity limit

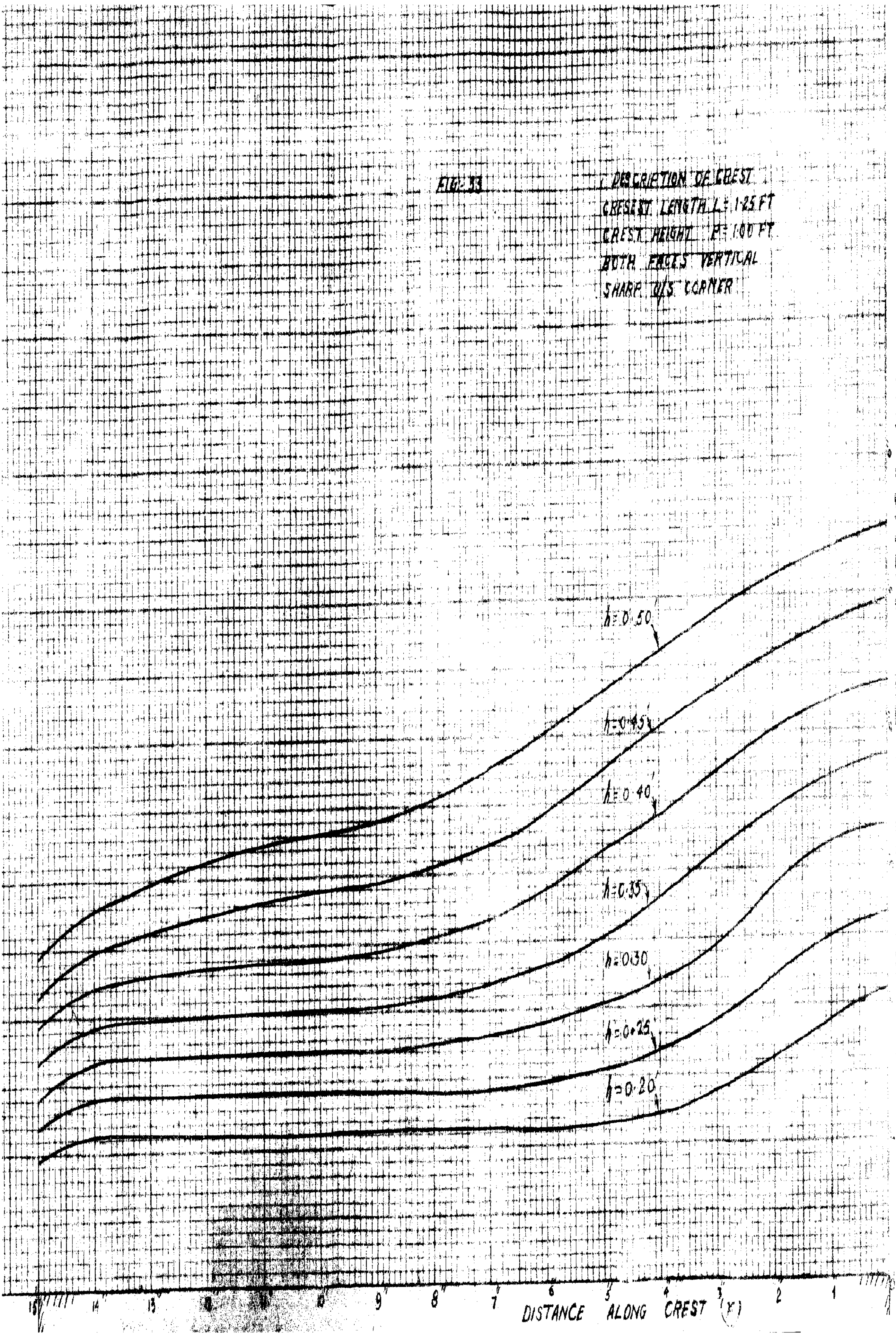


FIG. 33
DESCRIPTION OF CREST
CRESEST LENGTH L = 1.25 FT
CREST HEIGHT P = 1.00 FT
BOTH FACES VERTICAL
SHARP U/S CORNER
h = 0.50
h = 0.45
h = 0.40
h = 0.35
h = 0.30
h = 0.25
h = 0.20
15
14
13
10
9
8
7
6
5
4
3
2
1
DISTANCE ALONG CREST (X)
SURFACE (Y)

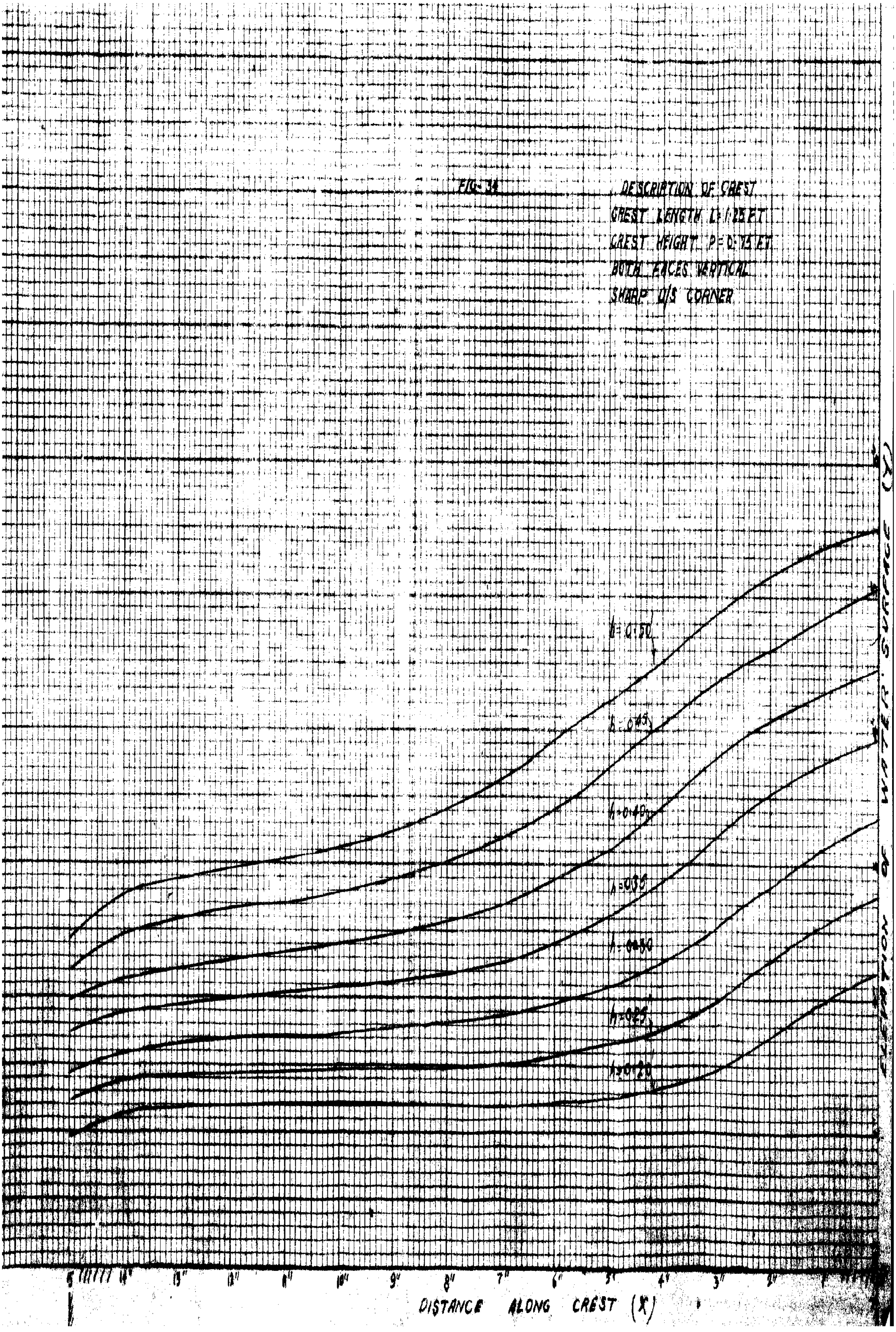
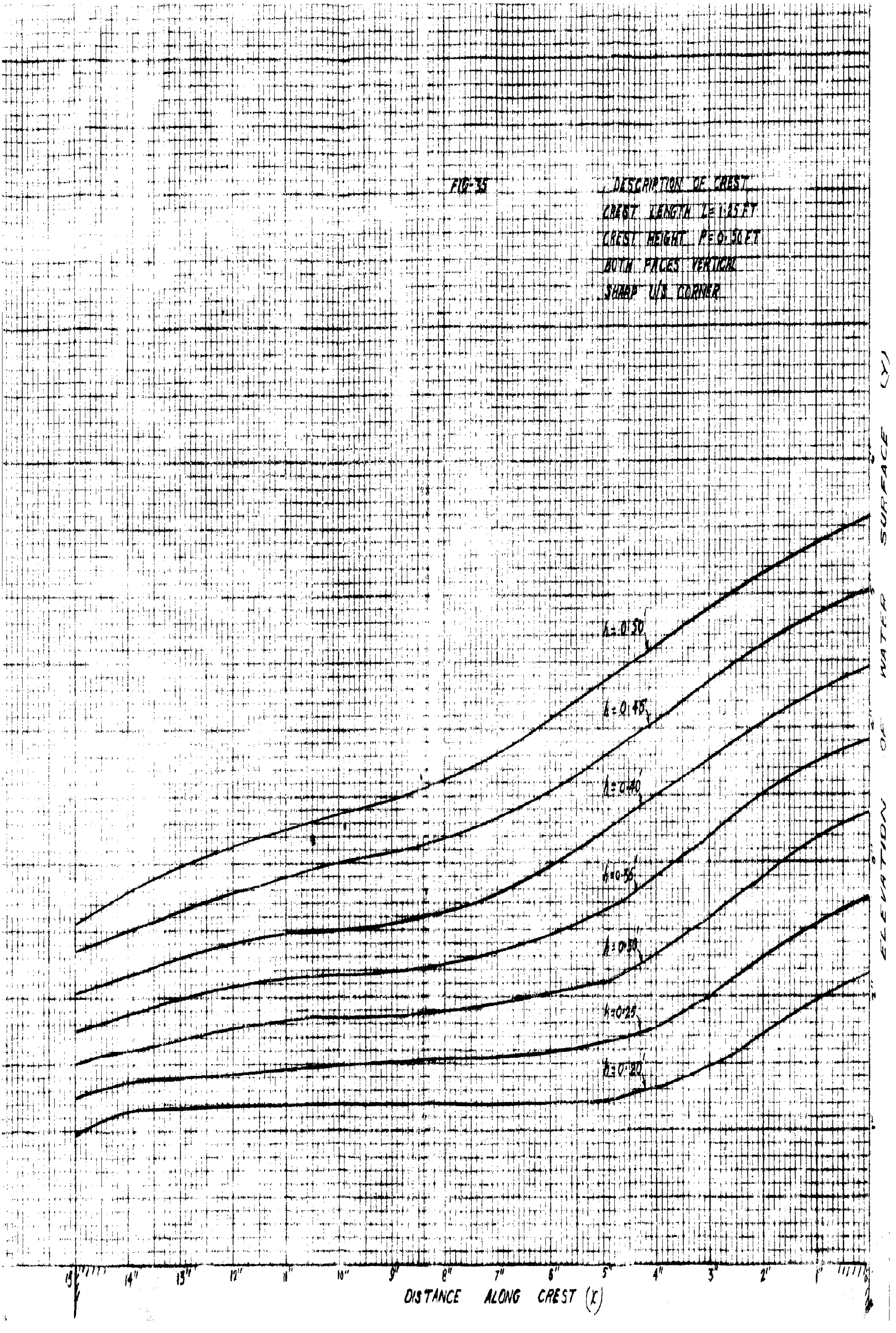
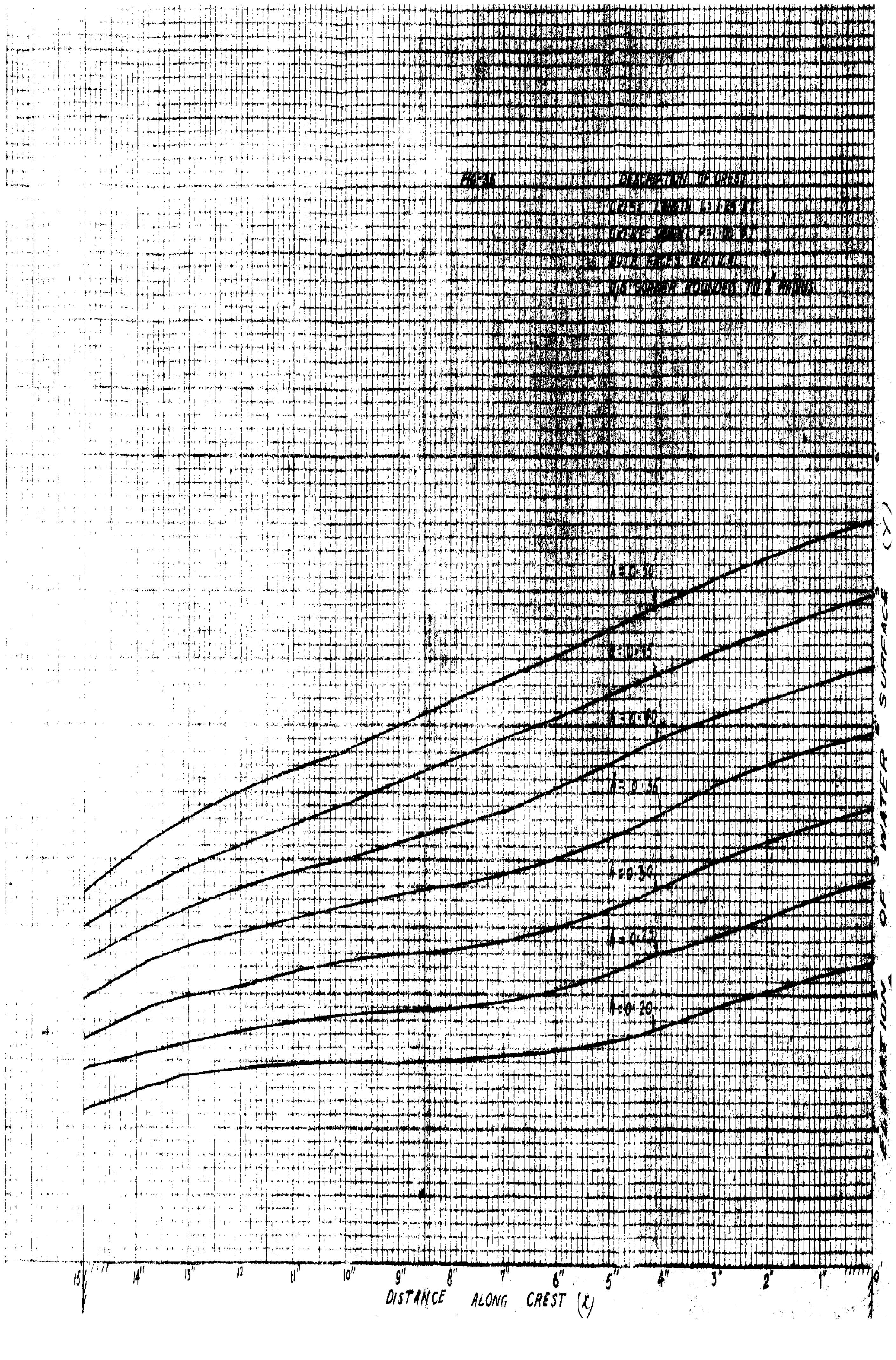


FIG-34
DESCRIPTION OF CREST
CREST LENGTH L=1·25 FT
CREST HEIGHT P=0·75 FT
BOTH FACES VERTICAL
SHARP U/S CORNER
h=0·50
h=0·45
h=0·40
h=0·35
h=0·30
h=0·25
h=0·20
14"
13"
12"
11"
10"
9"
8"
7"
6"
5"
4"
3"
2"
DISTANCE ALONG CREST (X)
ELEVATION OF WATER SURFACE (Y)


DESCRIPTION OF CREST
CREST LENGTH L=1.25 FT
CREST HEIGHT P=0.50 FT
BOTH FACES VERTICAL
SHARP U/S CORNER
h=0.50'
h=0.45'
h=0.40'
h=0.35'
h=0.30'
h=0.25'
h=0.20'
15"
14"
13"
12"
11"
10"
9"
8"
7"
6"
5"
4"
3"
2"
1"
0
DISTANCE ALONG CREST (X)
ELEVATION OF WATER SURFACE (Y)
FIG-35


h = 0.50
h = 0.45
h = 0.40
h = 0.35
h = 0.30
h = 0.25
h = 0.20
15
14"
13"
12
11"
10"
9"
8"
7"
6"
5"
4"
3"
2"
1"
0"
DISTANCE ALONG CREST (X)
6"
ELEVATION OF WATER SURFACE (Y)

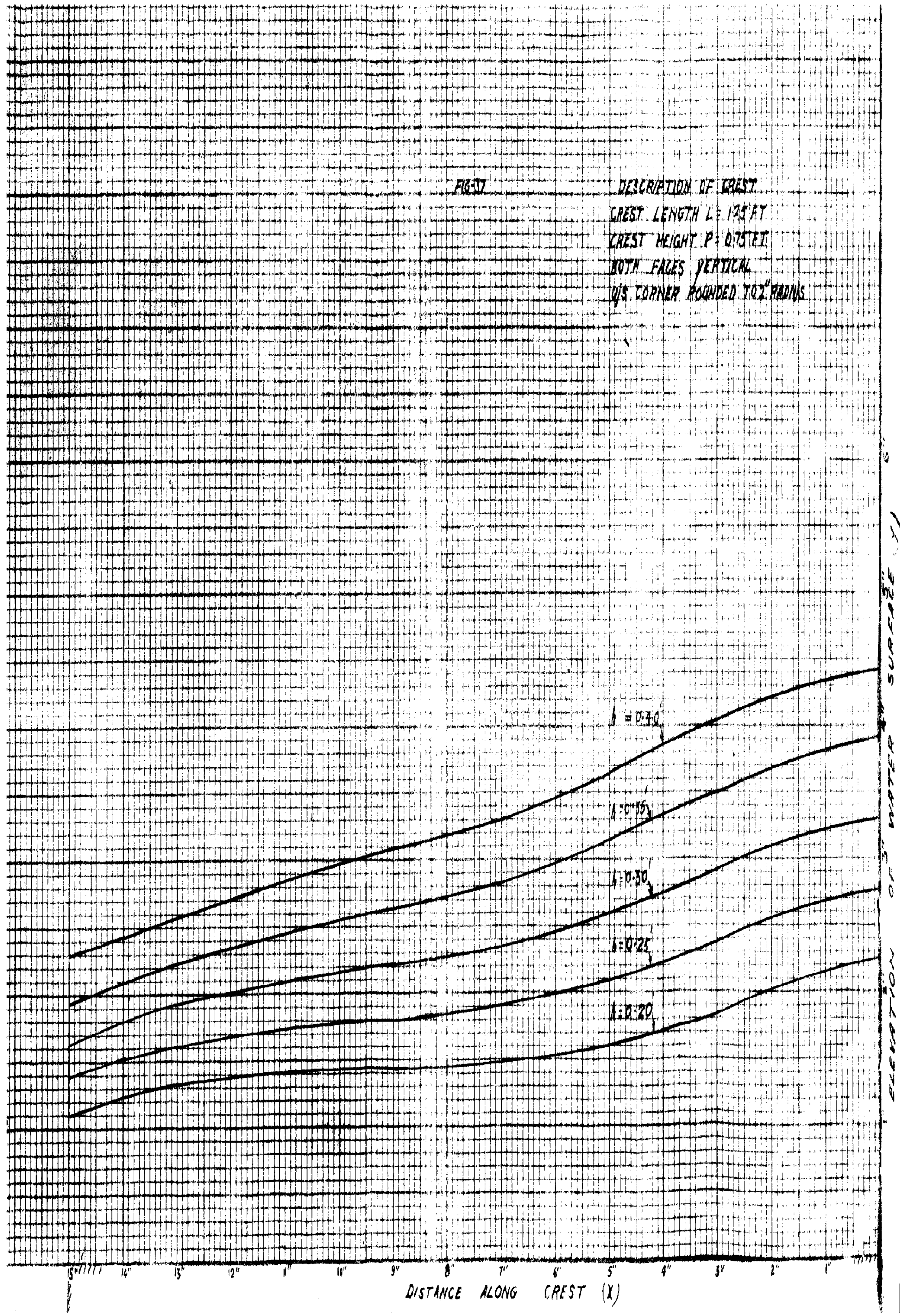


FIG-37
DESCRIPTION OF CREST
CREST LENGTH L = 1.25 FT
CREST HEIGHT P = 0.75 FT
BOTH FACES VERTICAL
U/S CORNER ROUNDED TO 2" RADIUS
h = 0.40
h = 0.35
h = 0.30
h = 0.25
h = 0.20
15" 14" 13" 12" 11" 10" 9" 8" 7" 6" 5" 4" 3" 2" 1"
DISTANCE ALONG CREST (X)
ELEVATION OF WATER SURFACE (Y)
3"
6"

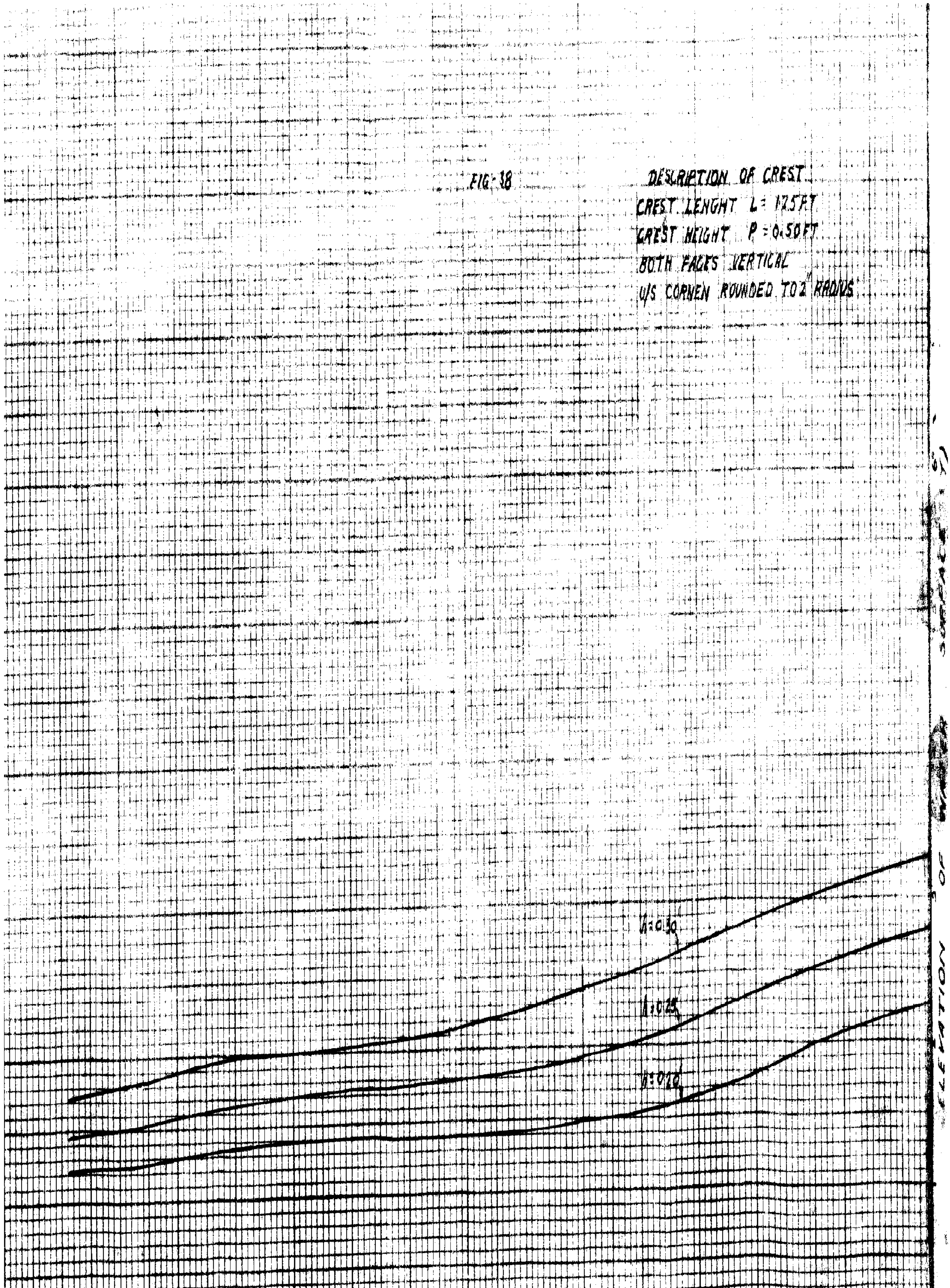
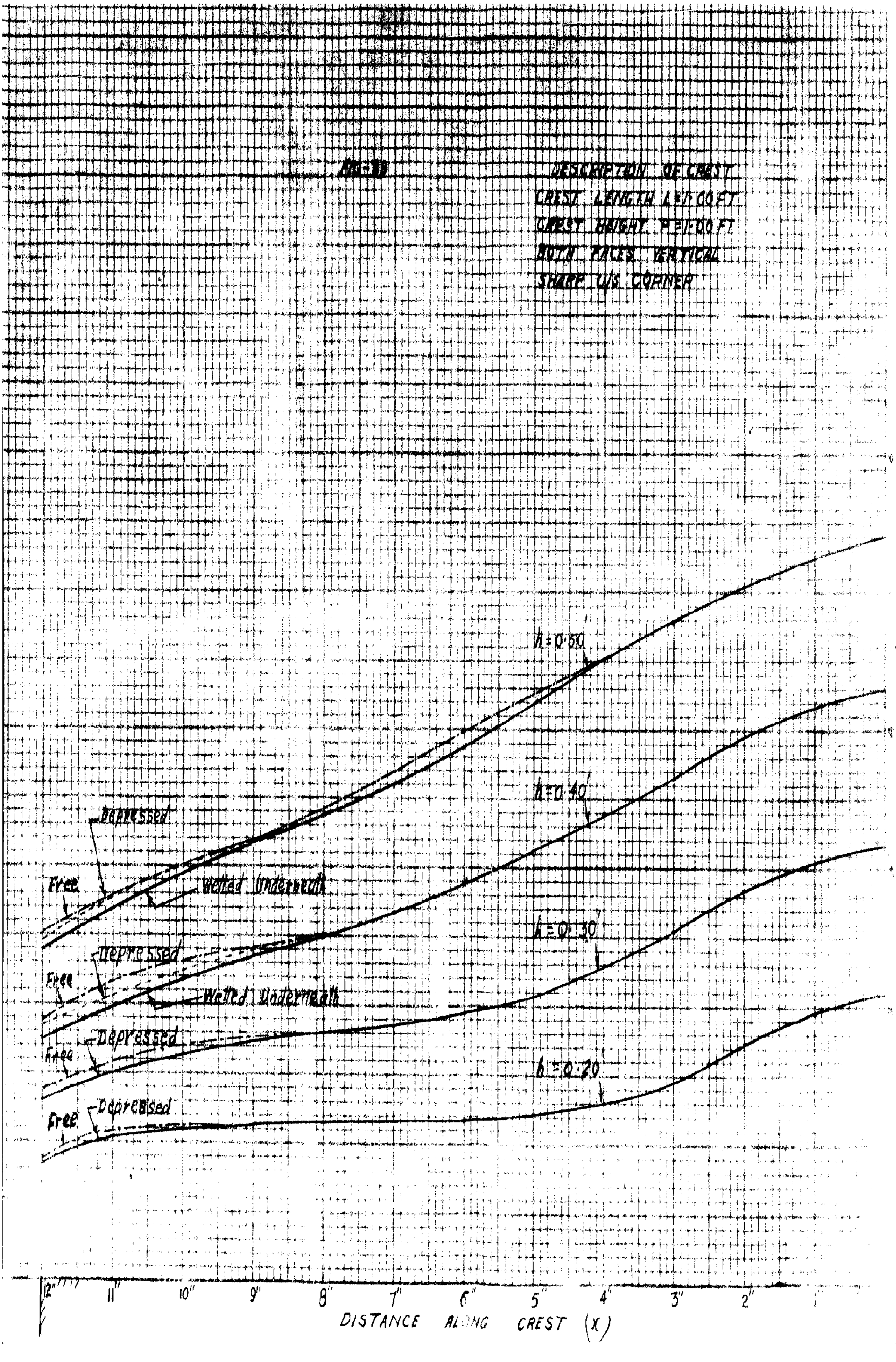

DESCRIPTION OF CREST
CREST LENGHT L = 17.5 FT
CREST HEIGHT P = 0.50 FT
BOTH FACES VERTICAL
U/S CORNEN ROUNDED TO 2" RADIUS
H=0.30
H=0.25
H=0.20
15" 14" 13" 12" 11" 10" 9" 8" 7" 6" 5" 4" 3" 2" 1"
DISTANCE ALONG CREST (X)
ELEVATION OF
SURFACE

FIG. 38


DESCRIPTION OF CREST
CREST LENGTH L=1.00 FT
CREST HEIGHT P=1.00 FT
BOTH FACES VERTICAL
SHARP U/S CORNER
h = 0.50
h = 0.40
h = 0.30
h = 0.20
Free
Depressed
Wetted Underneath
12"
11"
10"
9"
8"
7"
6"
5"
4"
3"
2"
DISTANCE ALONG CREST (X)

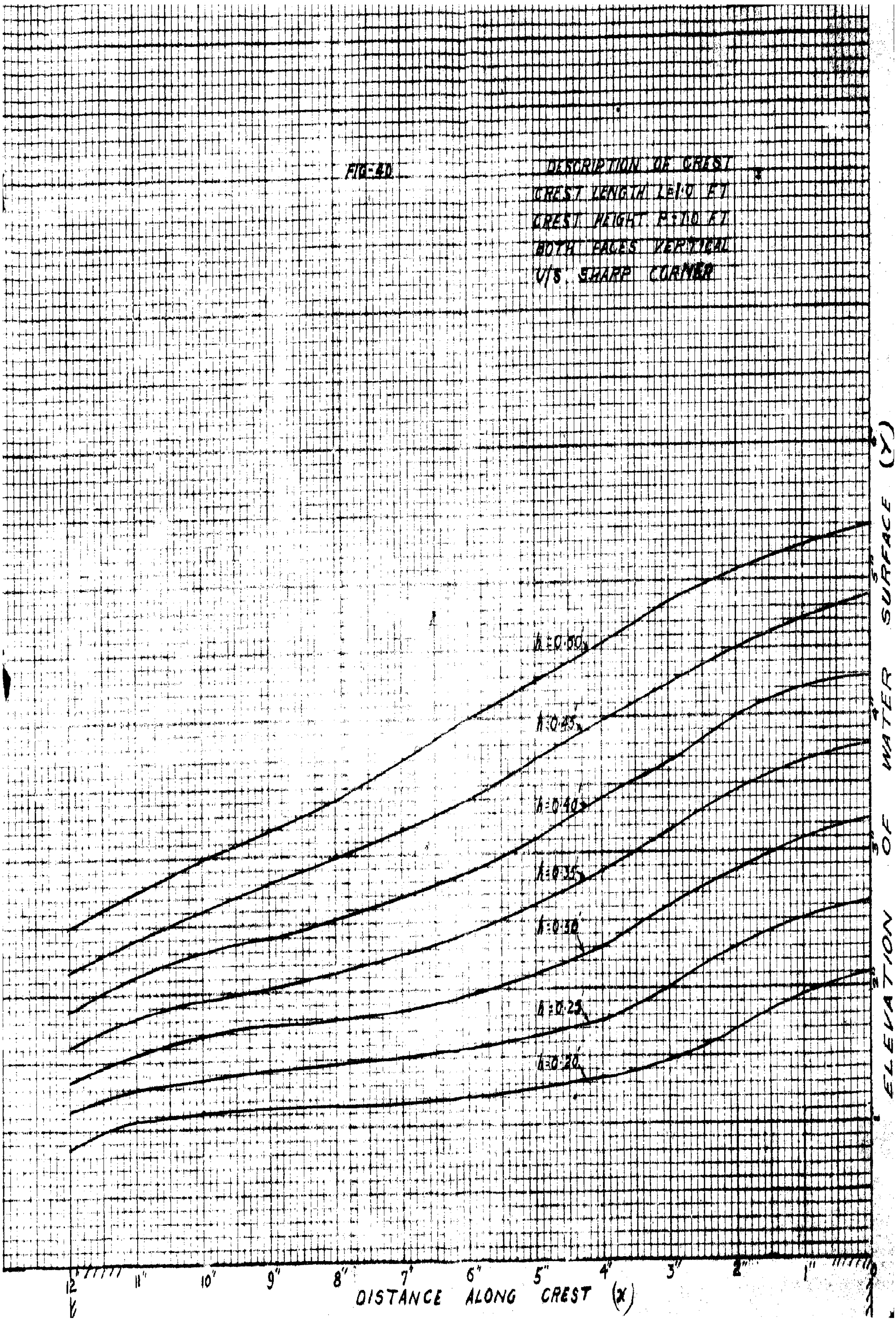


FIG-40
DESCRIPTION OF CREST
CREST LENGTH L=1.0 FT
CREST HEIGHT P=1.0 FT
BOTH FACES VERTICAL
U/S SHARP CORNER
h=0.50
h=0.45
h=0.40
h=0.35
h=0.30
h=0.25
h=0.20
12"
11"
10"
9"
8"
7"
6"
5"
4"
3"
2"
1"
0
DISTANCE ALONG CREST (x)
ELEVATION OF WATER SURFACE (Y)

FIG-41

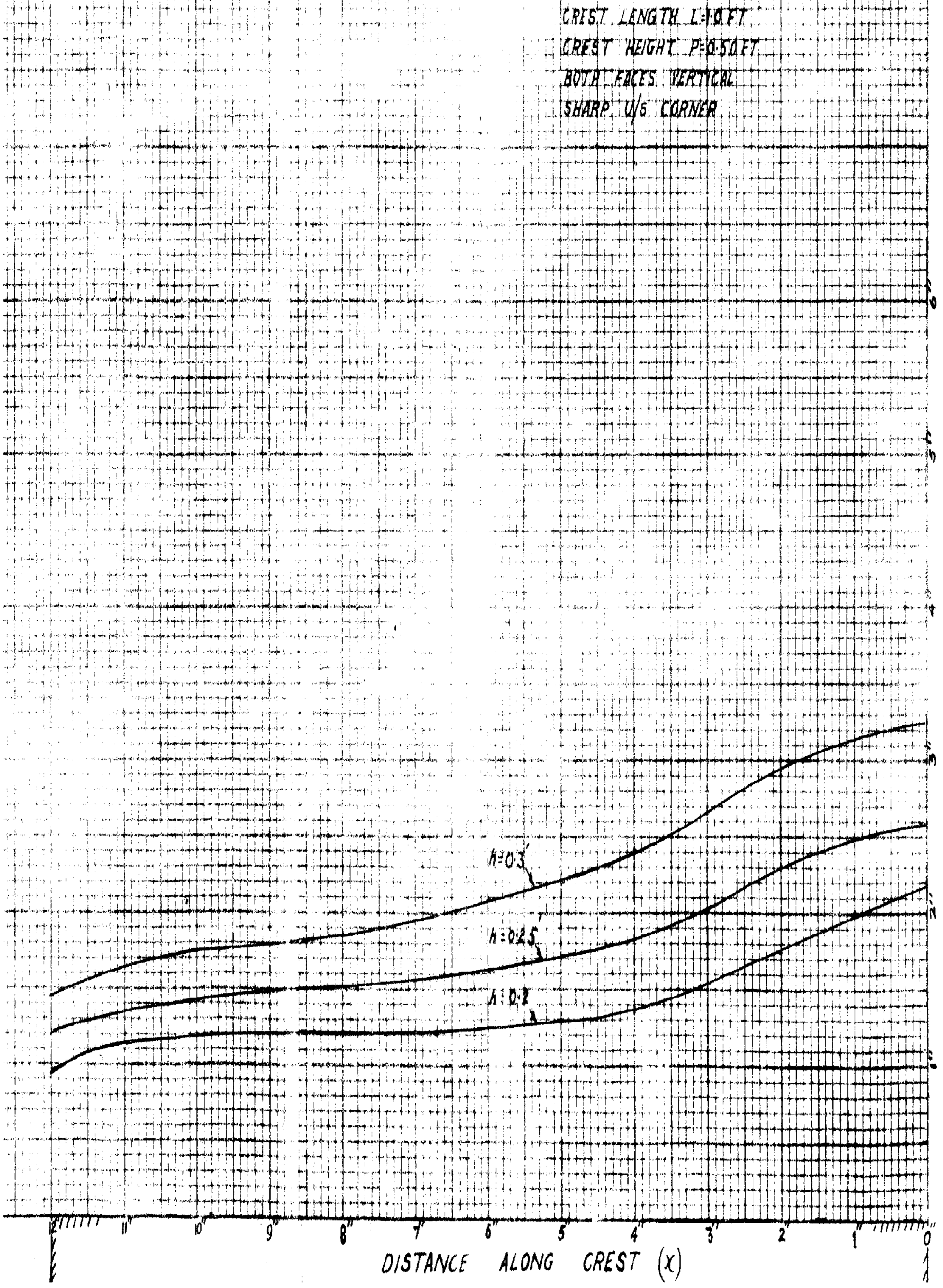
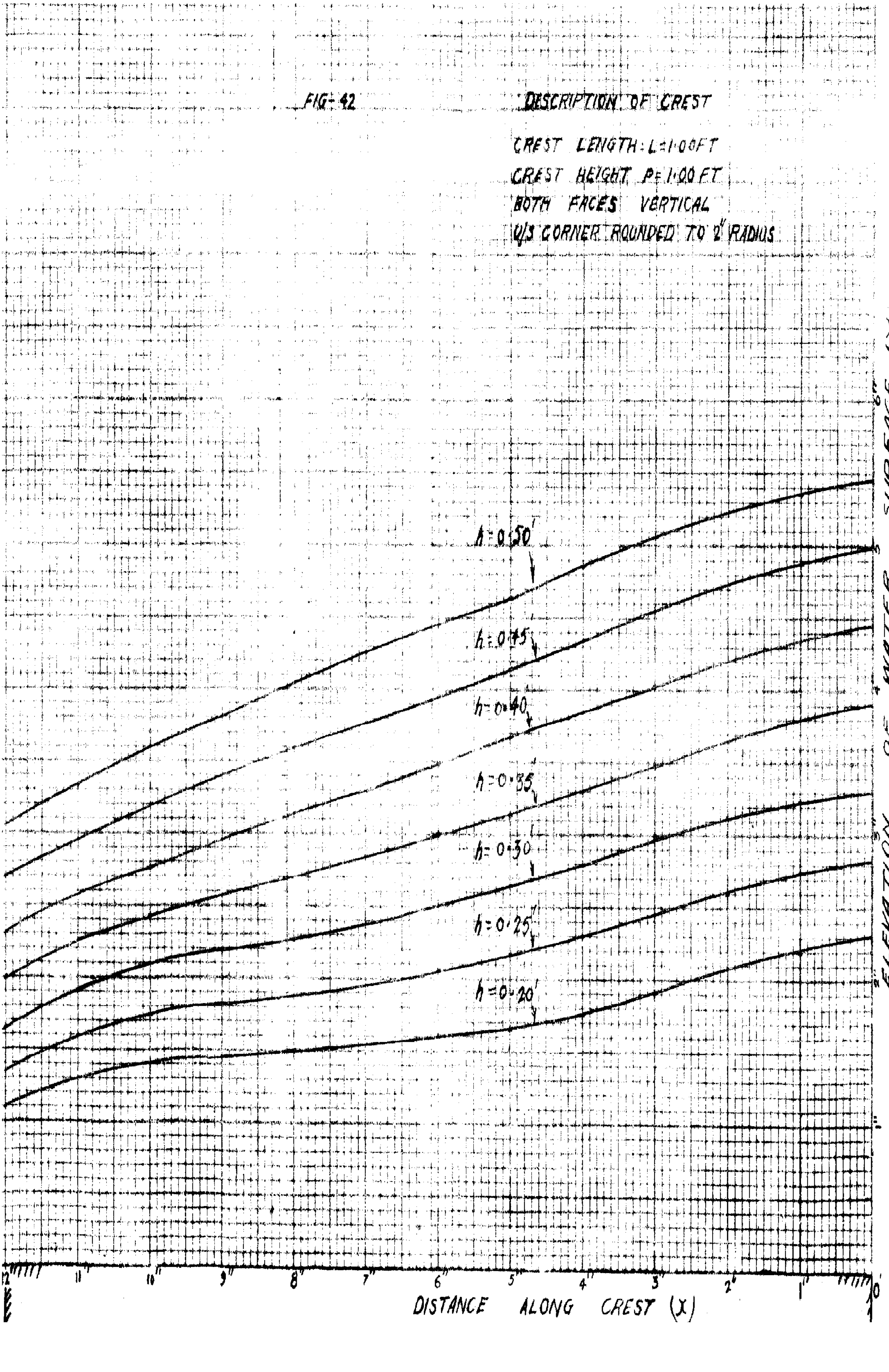

DESCRIPTION OF CREST
CREST LENGTH L=1.0 FT
CREST HEIGHT P=0.50 FT
BOTH FACES VERTICAL
SHARP U/S CORNER
h=0.3'
h=0.25'
h=0.2'
ELEVATION OF WATER SURFACE (Y)
DISTANCE ALONG CREST (X)

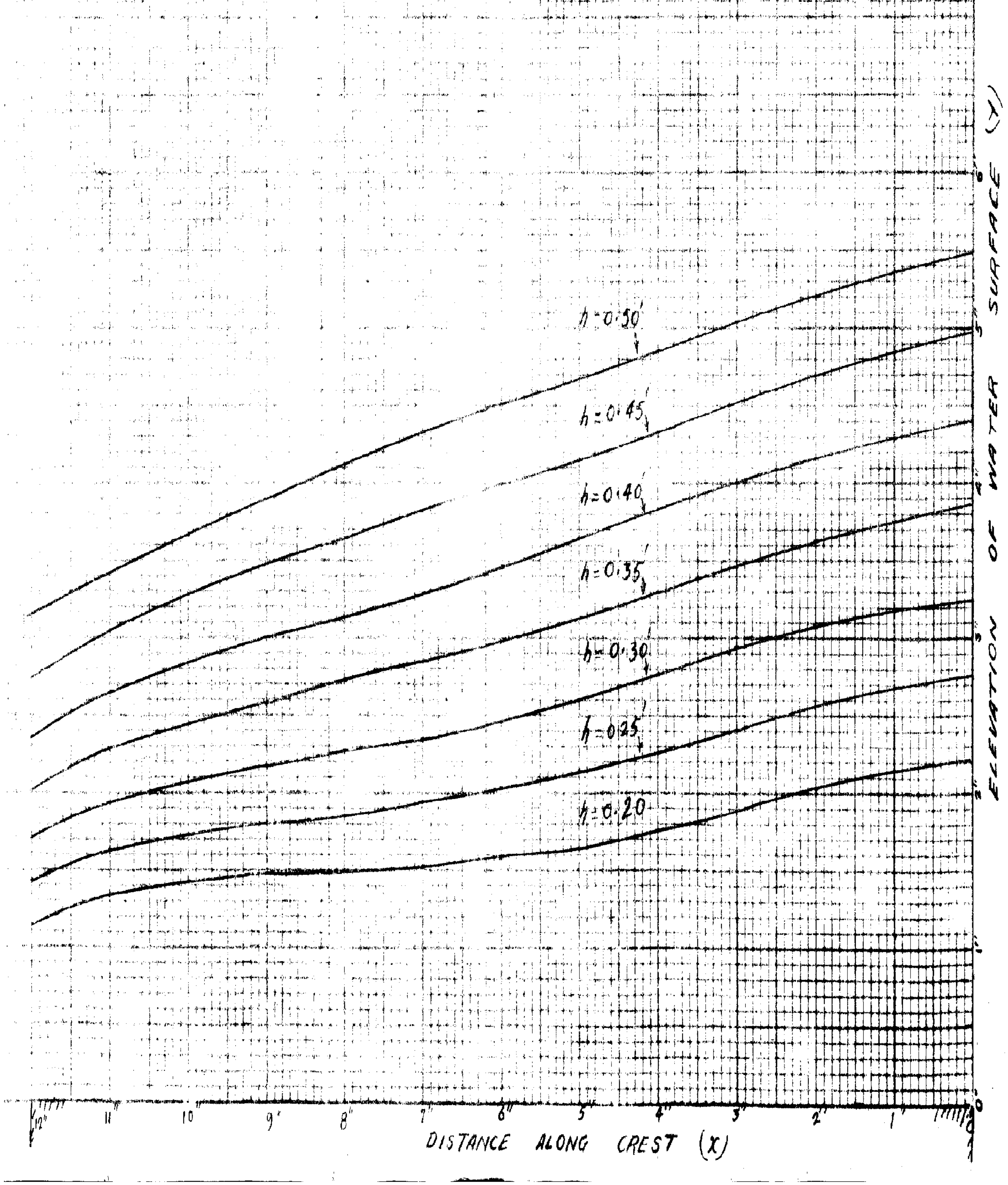
FIG- 42



DESCRIPTION OF CREST
CREST LENGTH: L=1·00FT
CREST HEIGHT P=1·00 FT
BOTH FACES VERTICAL
U/S CORNER ROUNDED TO 2" RADIUS
h=0·50'
h=0·45'
h=0·40'
h=0·35'
h=0·30'
h=0·25'
h=0·20'
ELEVATION OF WATER SURFACE (Y)
DISTANCE ALONG CREST (X)

FIG- 43

DESCRIPTION OF CREST
CREST LENGTH L = 1·00 FT
CREST HEIGHT P = 0·75 FT
BOTH FACES VERTICAL
U/S CORNER ROUNDED TO 1" RADIUS



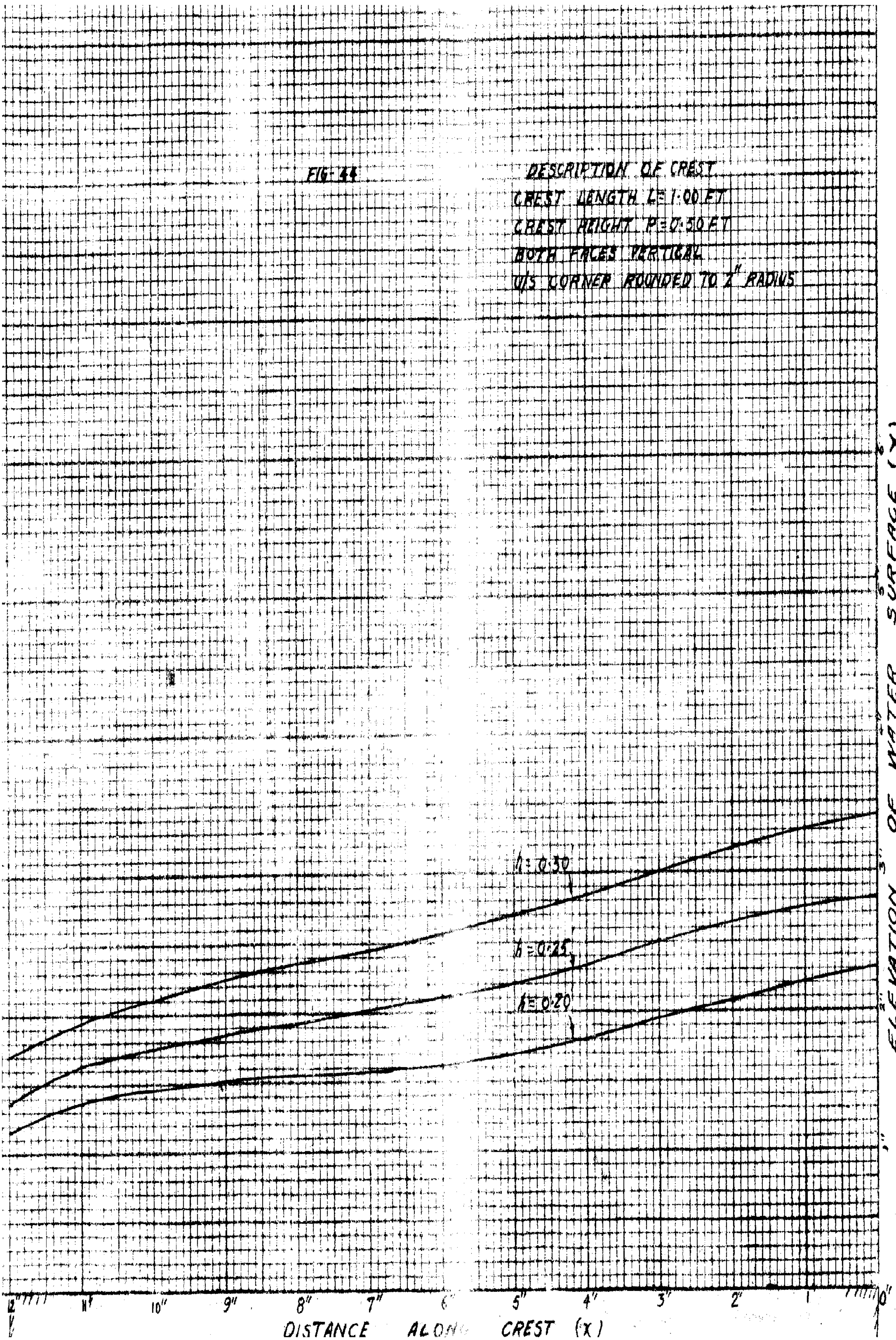


FIG. 44
DESCRIPTION OF CREST
CREST LENGTH L=1.00 FT
CREST HEIGHT P=0.50 FT
BOTH FACES VERTICAL
U/S CORNER ROUNDED TO 2" RADIUS
h=0.30
h=0.25
h=0.20
12" 11" 10" 9" 8" 7" 6" 5" 4" 3" 2" 1 0"
DISTANCE ALONG CREST (X)
1" 2" 3" 4" 5" 6"
ELEVATION OF WATER SURFACE (Y)

FIG- 46

DESCRIPTION OF CREST

CREST LENGTH L = 0.333 FT

CREST HEIGHT P = 0.75 FT

BOTH FACES VERTICAL

SHARP U/S CORNER

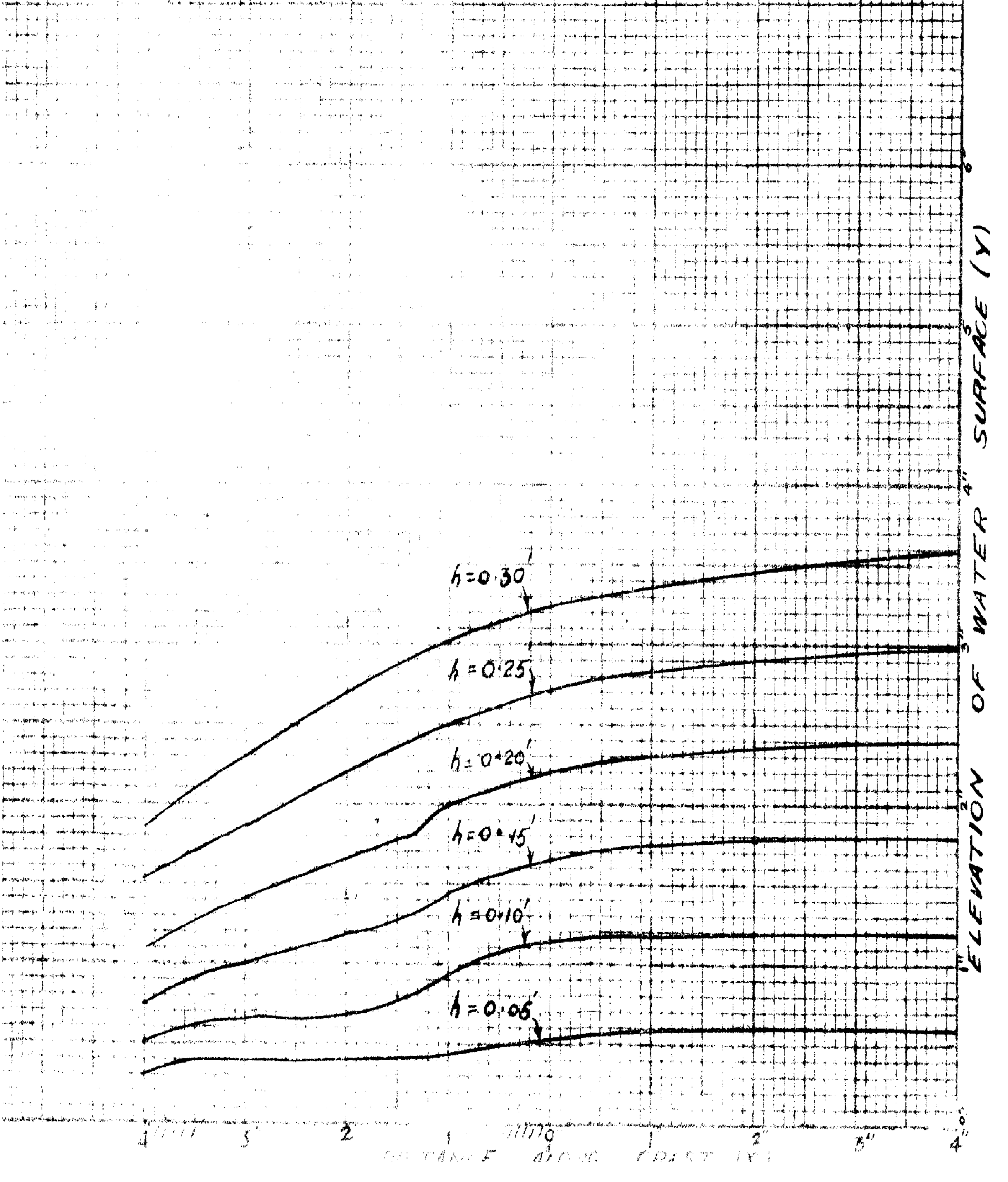
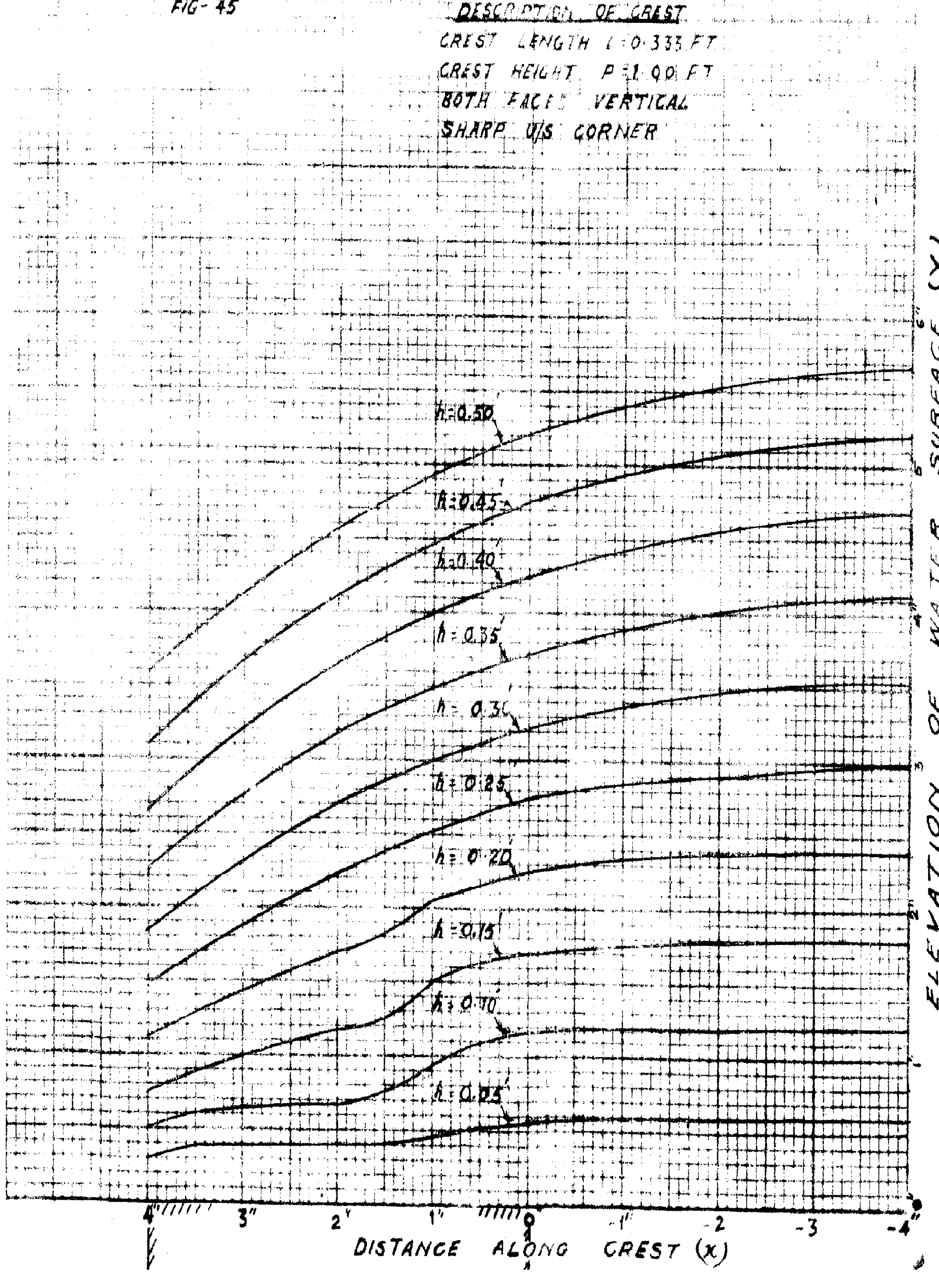


FIG- 45


DESCRIPTION OF CREST
CREST LENGTH L=0.333 FT
CREST HEIGHT P=1.00 FT
BOTH FACES VERTICAL
SHARP U/S CORNER
h=0.50
h=0.45
h=0.40
h=0.35
h=0.30
h=0.25
h=0.20
h=0.15
h=0.10
h=0.05
ELEVATION OF WATER SURFACE (Y)
4"
3"
2"
1"
0
-1"
-2
-3
-4"
DISTANCE ALONG CREST (x)

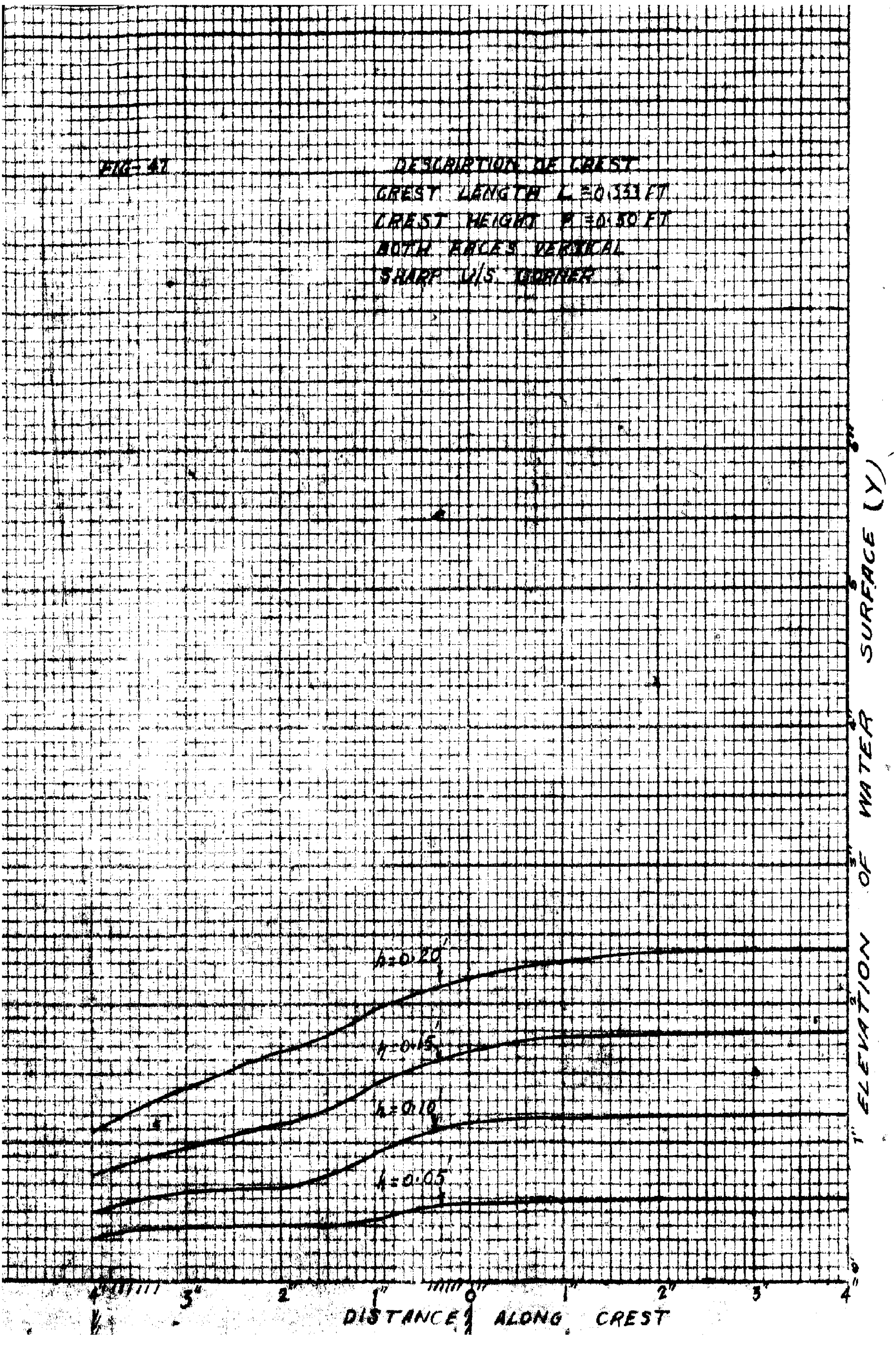


FIG- 47
DESCRIPTION OF CREST
CREST LENGTH L = 0.333 FT
CREST HEIGHT P = 0.50 FT
BOTH FACES VERTICAL
SHARP U/S CORNER
h = 0.20'
h = 0.15'
h = 0.10'
h = 0.05'
4"
3"
2"
1"
0"
1"
2"
3"
4"
DISTANCE ALONG CREST
0"
1"
2"
3"
4"
5"
6"
ELEVATION OF WATER SURFACE (Y)