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SPILLWAY DESIGN FLOOD

BY

* DR. V. C. KULANDAISWAMY, B.E., M.TECH., PH.D.

I. Synopsis—

The importance of “hydrologic design” in hydraulic projects is stated. The economic and social considerations involved, besides technical problems, in the determination of maximum flood for which the surplussing works of a reservoir is to be designed, are mentioned. The development of various methods for determining maximum probable flood is traced. The different procedures, and their combinations now in use, are briefly described. The need for collecting basic hydrologic data, to enable the development of more reliable and rigorous methods of analysis, is stressed.

II. Introduction—

There are three aspects in the design of a hydraulic structure:

- (a) hydrologic design,
- (b) hydraulic design and
- (c) structural design.

Stated briefly, the hydrologic design deals with the determination of the rate or quantity of water to be handled. It may be the maximum discharge, or minimum stream flow, or dependable yield and its time distribution from a catchment, depending on the nature of the project. The hydrologic design is important, not only for the safety of the structure, but also from the point of view of accurate estimates of the benefits from the project. However, this branch of engineering did not develop as much as it should have, and till about 1,900, or even later, highly empirical methods were used for solving the hydrologic aspect of an engineering problem. One of the most important hydrologic events which affects many branches of engineering besides those dealing with control of water, is “maximum flood run off”. Design of, culverts,

bridges, drainage works and surplussing works in reservoirs needs a reliable basis for estimating the run off for which the design is to be made. The scope of this paper is restricted to the *discussion of methods in use* for determining the maximum discharge for which the surplus outlets of a reservoir must be designed.

For absolute safety of the structure, the design must provide for the maximum probable flood. But this might increase the cost of the structure and render the project economically unsound. If the design is made for a flood discharge less than the maximum, there is a risk involved. Supposing that provision is made for a 1,000 year flood, a discharge equal to or exceeding this value might occur even the following year or during the life of the structure. A calculated risk is therefore implied, and to what extent the structure and thereby the life and property downstream can be exposed to such a risk is a matter that involves economic, social and even political considerations. However, to form the basis for a decision, the probable maximum flood and various flood magnitudes with their recurrence intervals must be determined. The problem of design flood estimation is peculiarly difficult and complicated one. In estimating run off from a catchment, discrimination must be made between small and large catchments. In what follows, the various methods or calculating maximum discharge will be discussed and at the end, their application with respect to areas will be stated.

III. Methods of prediction—

The method to be adopted depends on what data are available for the given catchment. The field practice ranges from use of highly empirical formulas to hydrometeorological studies and

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design storm application to unit hydrographs. In general, the methods used are as follows:—

1. Flood formulas.
2. (a) Envelope curves and (b) Increasing the observed maximum by a certain percentage.
3. Frequency analysis and use of unit hydrograph when necessary records are available either for the basin under question, or for the hydrologically similar region in which the catchment lies. The analysis may be with respect to one or more of the following:—
 - (a) Flood peaks.
 - (b) Flood volumes.
 - (c) Rainfall.
4. Design storm and use of unit hydrograph. For an economic design, consistent with safety, it is necessary to have the hydrograph of design flood rather than merely the value of flood peak. Methods 1 and 2 are used to predict the peak values. In method 3, the hydrograph of design flood can also be computed if sufficient data are available for deriving a unit hydrograph for the basin. In method 4, the hydrograph of design flood is computed.

III. 1 Flood formulas—

Flood formulas are about the earliest to be used for calculating the peak discharge from a catchment. Many are purely empirical and some are based on theoretical considerations with liberal approximations, assumptions and oversimplifications. Depending upon the basis adopted, they could be classified in various ways. They may perhaps be considered under the following heads:—

1. Maximum flood formulas:
 - (a) Formulas that take area of basin only for consideration.
 - (b) Formulas that take into account one or more of the basin and rainfall characteristics besides area.
2. Flood frequency formulas—
 - (a) Formulas that have recurrence interval implied.

(b) Formulas having recurrence interval explicitly used.

Most of the formulas are of the form $Q = CA^x$ where Q is the maximum discharge in cusecs, A the area of the catchment in square miles, x an exponent which varies from 0.3 to 0.8, and C a co-efficient varying from about 200 to 10,000. The formulas developed by Dickens, Ryves, Myer, Fanning, Possenti, and many others come under this category. The formulas developed by Craig, Lillie, Rhind, Chamier, Iszkowski and others use one or more of the basin, or basin and rainfall, characteristics. Under this category will fall the rational formula $Q = CIA$ which is popular even now for calculating peak run off from small catchments. It was in 1913 that Weston E. Fuller¹ published the first comprehensive study of statistical methods applied to floods and gave his formula introducing the concept of a magnitude, frequency relationship following a line of reasoning that was introduced into studies of river flow about the same time by Allen Hazen. Formulas developed by Kuichling, Pettis and others have frequency implied and those of Fuller, Lane Creager and others have frequency explicitly stated. Further studies have developed a different approach to the problem of flood prediction, by methods involving an analysis of rainfall records and corresponding hydrographs. The formulas given by Kinnison and Colby² deserve special mention. All these formulas were derived mostly for local use, but many of them were later adopted for the whole country of origin or sometimes even outside. The flood discharge of river Godavari in India with a catchment area of 115,570 square miles at the site under consideration was calculated³ using 24 of the well known flood formulas. The discharge ranged from 0.74 million to 3.46 million cusecs, the variation being of the order of 350%. Even in the region for which it is derived, the use of flood formula—if at all necessary should be resorted to with great caution.

“As time passes and more and more data are available, the record flood from any size drainage area continues to increase”. So the value of constants fixed with the meagre data available at the time of its derivation will no longer be correct. For instance, in Ryves' formula for South India, $Q = CA^{2/3}$, he fixed the value of C as ranging from 450 to 675. Observed floods have shown the value to be as high as 2360⁴. If a formula could be derived taking into consideration all the variables that influence discharge it may perhaps be the most ideal solution. It is said that there are at least 23 factors⁵ affecting magnitude of floods and it is impossible to account for all of them in an analytical approach. However, B.D. Richards⁷ claims that a set of formulas derived by him take into account all the major factors and their application to various catchments give satisfactory results. Flood formulas cannot be depended upon for flood estimation. They may be used, if at all, for purposes of comparison. Chow⁸ gives a list of 66 formulas that fall under

one or the other of the categories discussed in the foregoing.

III. 2. a. Envelope Curves—

These may be prepared either for peak discharge or for flood volumes. All the maximum recorded values for the catchments in the region may be plotted against the area on log-log paper. The extent of the region is fixed by conditions of hydrological homogeneity. The values used should represent similar type floods (rain floods or snowmelt floods). When only a part of the area happened to contribute to a maximum flood recorded in one catchment, only that area must be used in the plot. Data thus plotted will usually indicate a curved line envelopment which may be approximated to a straight line for small ranges. Discharges from small areas due to thunderstorms may suggest the need for considering two curves—one for small areas and another for large areas. In drawing the envelope curve, the points through which the enveloping line passes are called control points (Fig. 1). The

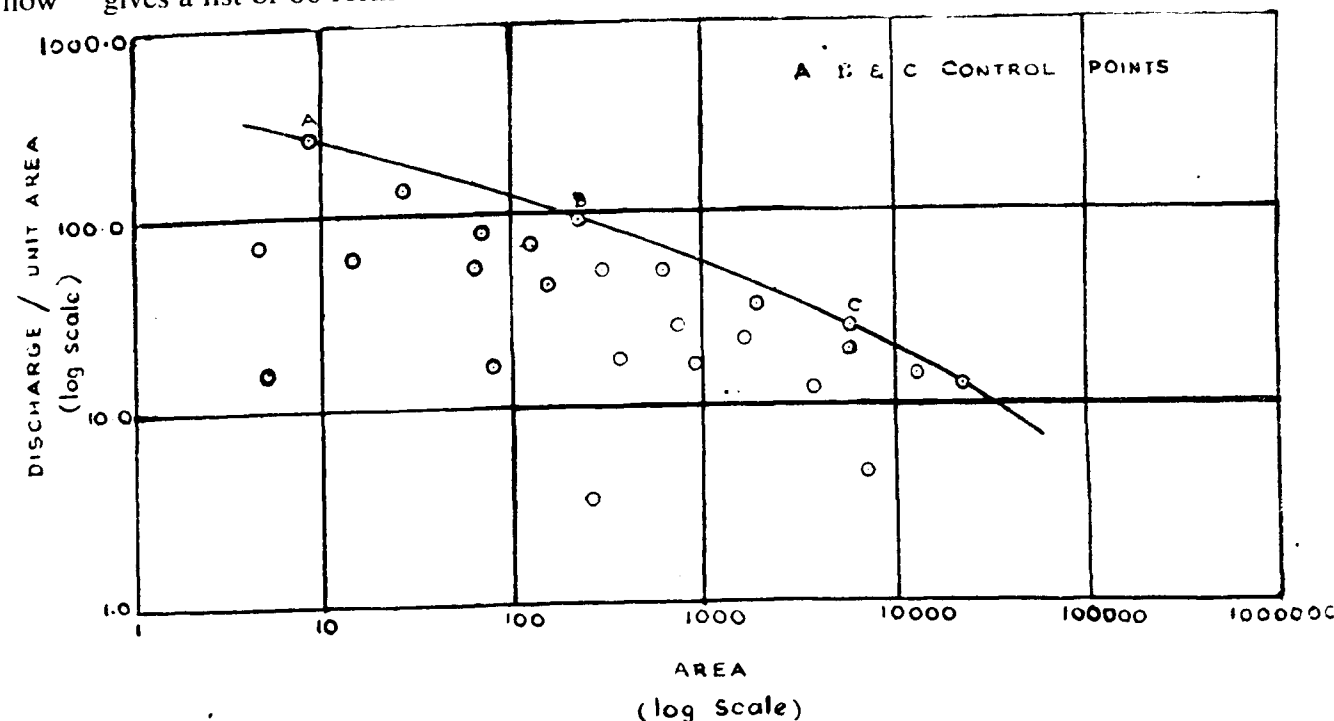


FIG. 1.

¹ Creager, Justin and Hinds, "Engineering for Dams" Vol. 1, 1944, P. 125.

² Fuller, Weston, E., "Flood Flows," Trans. A.S.C.E., Vol. LXXVII, 1914.

³ Kinnison and Colby, "Flood Formulas Based on Drainage Basin characteristics", Trans. A. S. C. E. Vol. 110, 1945.

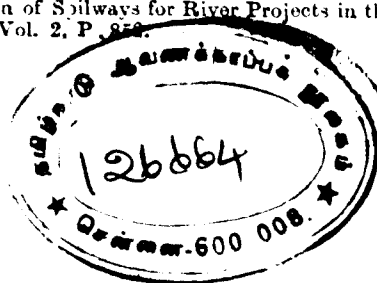
⁴ Rao K. L., "Design of Spillways for River Projects in the State of Mysras", International Congress on large Dams, Fourth Congress, 1951, Vol. 2, P. 855.

⁵ Creager, Justin and Hinds, "Engineering for Dams" Vol. 1, P. 140.

⁶ Creager, Justin and Hinds, "Engineering for Dams", Vol. I, P. 139, 1944.

⁷ Richards, B.D., "Flood Estimation and Control" London, 1955, P. 5.

⁸ Chow Ven Te, "Hydrologic Determination of Waterway Areas for the Design of Drainage Structure in Small Drainage Basins" University of Illinois, Engineering Experiment Station, Bulletin No. 462, 1962.



nature of the catchment and the history of the floods which form the control points need careful scrutiny to ensure that no abnormal condition like the breach of a dam is represented by these floods. Having drawn the envelope curve, the maximum value for the catchment under question can be read by entering the plot with the area of the catchment. When long records are not available for the basin concerned, the envelope curve helps us to gain in space what we do not have in time. However, the results from envelope curves should not be relied upon wholly as a means of estimating maximum probable flood. Design flood values, if they are to be maximum probable flood, should be normally higher than those obtained from envelope curves.

III. 2.6. Percentage addition to observed flood.

For many catchments, either flood records are not available or they cover only a brief period.

One of the methods adopted is to find out by local inquiries or observation of records of maximum flood levels reached near bridges, buildings, etc., the maximum flood that has so far occurred and the number of years during which this flood is the maximum. This is increased by a certain percentage to get the design flood. If the observed flood is maximum over a longer period, the percentage increase may be less. If it is over a short period, the increase may have to be more. As a sort of a rough guidance for the designer in such situations, graph connecting the years of record and the percentage by which the design flood adopted is more than the observed flood is plotted for various projects executed in India⁹ (Fig. 2). For the catchment under consideration, the plot may be entered with the number of years during which the known flood

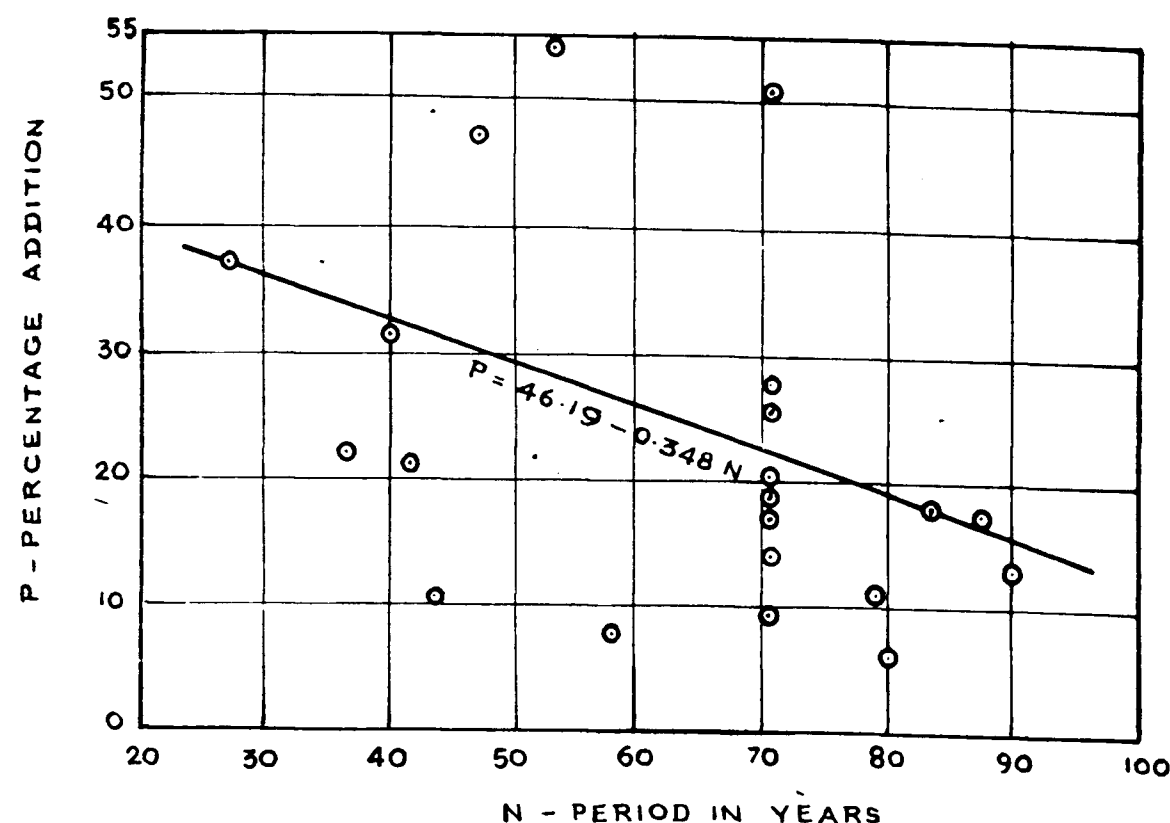


FIG. 2.

⁹ "Symposium on Spillway Capacity of Dams", Central Board of Irrigation, India, Publication No. 65.

is maximum and the percentage increase may be read. The observed maximum may be increased by this percentage to get the design flood. This is typical of the many empirical methods resorted to under helpless situations, and has not much theoretical justification.

III. 3. Frequency Analysis—

It is self evident that the only way to forecast the future is by a study of the past and it therefore follows that a statistical examination of past flood records taken in conjunction with an appraisal of the meteorological and other factors pertaining to the river in question might form a satisfactory means of estimating future flows. Flood flow studies using the statistical approach are based on the assumption that combinations of the numerous factors which produce flood flows are a matter of pure chance and therefore are subject to analysis according to the mathematical theory of probabilities. For predicting maximum probable floods, annual series are being used. It is held by some that peak annual flows are not a true sample of the universe of occurrences of peak flows during the period of record. The use of monthly peak flows is advocated¹⁰. The frequency curve is well defined in the central portion, but is likely to be poorly defined at the upper end. This condition often makes difficult the extrapolation of the curve with dependable accuracy. For design purposes, very often the flood to be predicted is one of rare occurrence. "It is believed by some that such outstanding floods follow some law of their own that comes into operation at very long intervals"¹¹. If the records cover only normal floods, they cannot help to predict the maximum floods of long recurrence intervals accurately. There is again the practical limit to which simple frequency curve can be extrapolated with reasonable accuracy. In its review of flood frequency methods, the sub-committee on floods concludes that "present (1951) methods of determining flood probabilities are suitable for use in connection with coffer dams, highways, and other minor structures, but

believes that existing methods which use solely extrapolations of frequency plotting of existing flood records are to be condemned for the determination of the probable frequency of floods for which major structures should be designed the failure of which would result in a calamity"¹². However, with these limitations in mind frequency analysis could be used as a valuable tool in flood studies to predict peak discharge or maximum volume, or peak rainfall rate for a given duration.

a. Peak Discharges—

When records of peak flow from the catchment under question are available, the maximum annual momentary peaks are chosen, arranged in the descending order of magnitude and the recurrence interval is calculated for each event using plotting position formula 1 or 2, given below:—

$$T = \frac{N+1}{M} \quad \dots (1)$$

$$T = \frac{2N}{2M-1} \quad \dots (2)$$

Where T is the recurrence interval, M the order number and N, number of years of record. Formula 1 may be used for extreme value plotting and 2 may be used for log normal plotting. The values of T obtained are plotted on the corresponding probability paper. If the plotted points are not too scattered and the line drawn through them is straight, a moderate extrapolation can be made by eye. When the line is curved, a straight line plot can be obtained by adding or subtracting a constant from the peak values¹³. The resulting plot can be extrapolated and the peak discharge of required recurrence interval can be found out. When extrapolation is necessary, control curves fixing confidence limits may also be drawn. The peak discharge so derived must be made use of with a "spillway design unit hydrograph" to arrive at the "design hydrograph of inflow". The term spillway design unit hydrograph will be explained later. The required reservoir elevation can be decided by routing the inflow through the reservoir. When records

¹⁰ Whisler and Smith, "The estimation of Frequencies of Rare Floods", Proc. A.S.C.E., Vol. 83, HYD, 2, April 1957, P. 1200.

¹¹ "Review of the Flood Frequency Methods", final report of the sub-committee of the Joint Division Committee on Floods., Trans. A.S.C.E., 1953, P. 1223.

¹² "Review of Flood Frequency Methods" Trans. A.S.C.E., 1953, P. 1220.

¹³ Goodrich, R.D. "Straight Line Plotting of Skew Frequency Data", Trans. A.S.C.E., Vol. 91, 1927.

available cover only daily discharge, the peak discharge can be derived from daily discharge. However, Foster's¹⁴ equation which is used for converting daily flow into momentary peak is not considered accurate.

b. Peak Volumes—

When the storage behind the dam, or above the spillway crest, is sufficient to materially modify the peak discharge, the shape of the inflow hydrograph is less important than volume of the flood, and the frequency with which various volumes of flood run off are equalled or exceeded becomes more useful. Flood hydrographs over the period of record must be analyzed and peak volumes for various durations, say 12 hours, one day 2 days, etc., must be tabulated. The minimum and the maximum duration and intervals to be adopted will depend on the size and run off characteristics of the catchment. For each duration, a frequency curve can be drawn and the data can be cross plotted to give a volume duration frequency relation (Fig. 3.). It is very

unlikely that volumes of equal frequency for all duration will occur in the same flood, but such an assumption is on the safe side. The volume duration curve for a certain chosen recurrence interval can be used as a mass curve of inflow with satisfactory results.

c. Frequency analysis for ungauged areas

Computed flood frequency relations from gauged areas may be used for estimating flood frequencies at ungauged points in the same stream or on streams in adjacent areas provided they are hydrologically homogeneous. From the volume-duration-frequency curves for each gauged area (Fig. 3) a family of curves connecting volume discharge per sq. mile versus area, with recurrence interval as parameter, can be obtained.

By entering the plot with the area of the ungauged basin, volume discharges for various recurrence intervals can be read, and using these values a graph connecting volume discharge

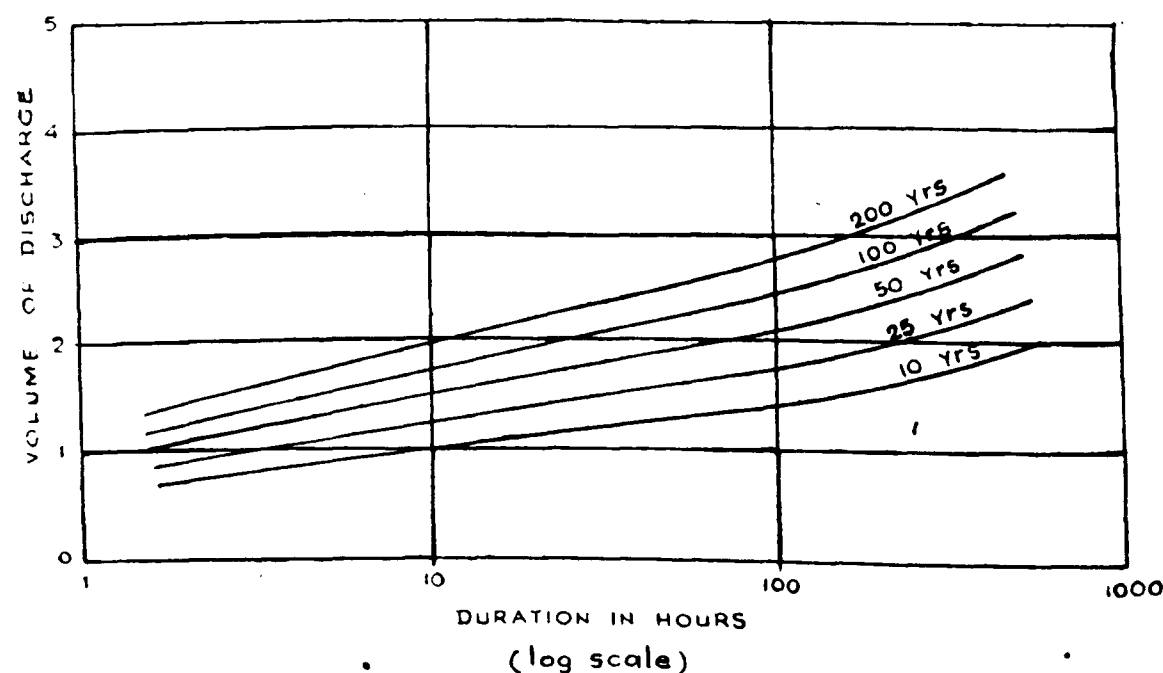


FIG. 3.

¹⁴ Foster, H. Alden, "Theoretical Frequency Curves and their Application to Engineering Problems", Trans. A.S.C.E. Vol. 87, 1924.

verses recurrence interval can be plotted for the ungauged area (Fig. 4). Similarly the peak discharge data can also be extended for ungauged areas. The U.S.G.S. had developed a two step procedure for areal interpolation of peak-flow frequencies¹⁵.

d. Peak rainfall—

If only rainfall records are available, then, the maximum average depth over the basin for various durations is assembled for the period of record. Frequency analysis for each duration is made and a family of curves connecting depth and duration with frequency as parameter is drawn (Fig. 5). For a specified recurrence interval, the depth for various durations can be read from Fig. 5 and a maximum depth duration curve can be plotted. From the maximum depth duration curve the rainfall increments can be read for a chosen unit duration. They may be arranged in a critical sequence. Adopting the initial loss and infiltration index method, and

assuming reasonably minimum values for both, the hyetograph of rainfall excess can be derived. Applying this hyetograph to the "spillway design unit hydrograph" derived for the basin, the inflow hydrograph can be obtained. Following the method already described for flood records, the rainfall relations from gauged areas can also be extended to ungauged areas. It may be mentioned that the recurrence interval of the rainfall may be less than the recurrence interval of the flood for which the design is intended to be made because the maximum flood requires the combination of critical rainfall and favourable antecedent conditions.

III. 4. Design Storm Method—

The terms frequently used in the discussion of this method are (1) maximum probable precipitation (M.P.P.) (2) standard project storm (S.P.S.) and (3) design storm. We know that the first denotes all time maximum. It denotes a value that will never be exceeded. The standard project storm gives the maximum rainfall

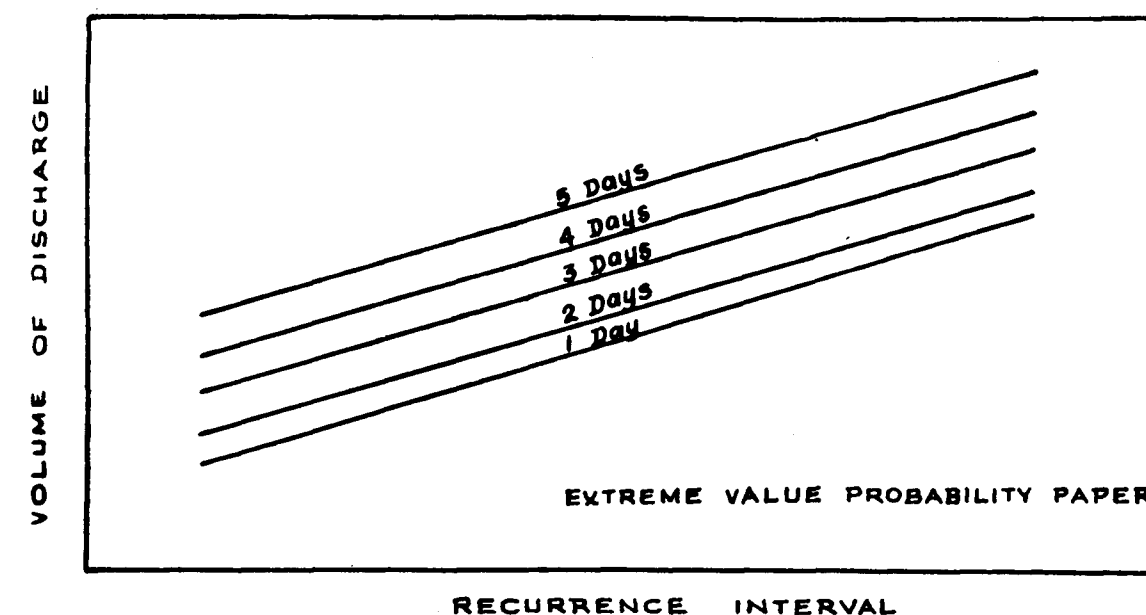


FIG. 4.

¹⁵ U.S. Geol. Survey, Cir. 204, 1952.

that is probable of occurrence under the most critical combination of conditions, considered reasonably probable. Rare combinations are excluded. The design storm is the one adopted for purposes of design. It may be equal to M.P.P., may lie between M.P.P. and S.P.S. values or may be lower than S.P.S. depending on the importance of the project and the degree of

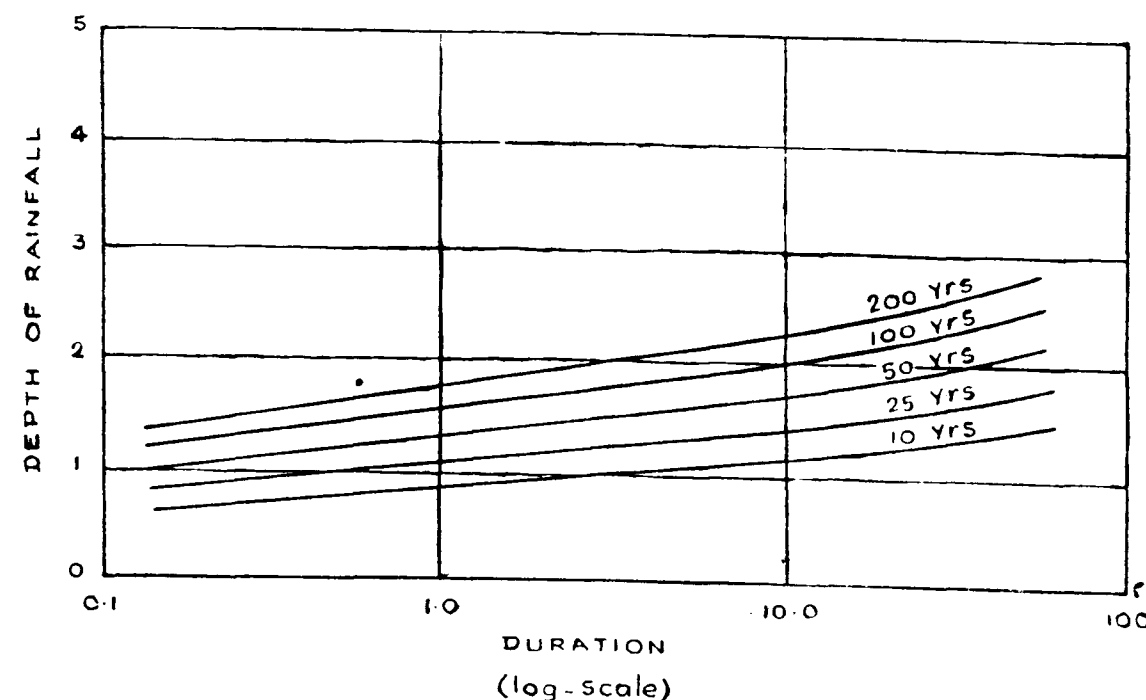


FIG. 5

risk taken. A rational determination of critical design storm criteria for a particular basin requires a comprehensive study of major storms of record in the region and an evaluation of the effects of local conditions on rainfall and run off rates. Very briefly stated, the studies may be on the following lines:—

a. All major storms of record in the region where the basin lies may be analysed.

b. On the basis of air mass properties and synoptic situations that prevailed during the storms of record, the increase in rainfall considered probable under more critical conditions may be estimated.

c. Since steps 'a' and 'b' are for the region as a whole, any modification necessary for using the storm in the catchment under consideration may be made.

d. In the light of studies from 'a' to 'c' select the storm that may be adopted for design.

e. In the areas where snow melt might contribute to runoff, the combination of maximum snow melt and critical rainfall that is reasonably probable must be determined.

f. Once the storm is determined, the hydrograph of excess rainfall can be arrived at with a critical sequence of rainfall increments. This may be used with the spillway design unit hydrograph to get the design flood hydrograph. Although special comprehensive studies are fully justified, it will be frequently necessary to prepare design storm estimates on the basis of data immediately available. The three methods for quantitative design storm estimates in general use are ¹⁶.

¹⁶ Creager, Justin and Hinds, "Engineering for Dams", Vol. 1, P. 181.

- (1) Envelope curve method.
- (2) Storm transposition method.
- (3) Modified storms transposition method.

The choice of the method depends on the size of the catchment and the data available.

Spillway design unit hydrograph—

It has been found that peak ordinates of unit hydrographs derived from major storms have peak values consistently higher than those derived from minor floods. If the unit hydrograph derived for the basin is based on average storms only, its peak value must be increased by a certain percentage to get the "spillway design unit hydrograph". The magnitude of increase depends on the size of the storms used for deriving the unit graph, and the size of the storm adopted for design. If for the basin, no flood hydrographs are available for deriving unit hydrograph, the derivation of synthetic unit hydrograph may be resorted to.

The details of flood calculations will also depend on the size of the catchment. Taking for instance the unit hydrograph, it may be applied up to about 2,000 sq. miles. It is said to have been successfully used up to 10,000 sq. miles. At any rate, it has an upper limit. For large catchments with many tributaries unit hydrograph must be applied for each tributary

and the flood at the point of interest must be determined by flood routing. The following methods may be adopted for calculating discharges from various sizes of catchments:—

1— 100 sq. miles.	Rational formula, unit hydrograph, flood frequencies, envelope curve.
100—2,000 sq. miles.	Unit hydrograph, flood frequencies, envelope curve.
Above 2,000 sq. miles.	Flood routing, flood frequencies, envelope curve.

IV. Conclusion—

The selection of an appropriate spillway design flood is an essential part of the engineering studies for a reservoir project. The many methods of investigation developed are valuable in themselves, but yet, do not offer a complete solution. The final determination will have to be a matter of judgment of an engineer fully conversant with the underlying principles of the phenomenon of floods. It is a subject that does not lend itself to controlled laboratory experiments. We have to depend on historical data; systematic and planned collection of extensive hydrologic data are a pre-requisite both for developing and using more reliable methods of flood estimation.

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OBLIQUE ANICUTS-DISCHARGE CHARACTERISTICS.

BY

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Synopsis—

The discharge characteristics of oblique anicut has been studied in a scale model. The variation of co-efficient with obliquity, under pertinent non-dimensional parameters arrived at by dimensional analysis, was studied. A final design chart is given. An illustrative example has been worked out.

1. Introduction—

Anicut is a low dam or weir used primarily for diversion of flow for irrigation purposes. These structures are generally located normal to the stream. But at times an anicut oblique to the flow direction is decided on for economy indicated by favourable foundation details. It is known that which oblique anicut, eventhough the total discharge may increase, the co-efficient of discharge decreases. This has certain disadvantages, there will be parallel flow along the upstream face which may damage the upstream talus and thus call for remedial measures such as cross groynes, the hydraulic jump in rear will be oblique thus complicating the energy dissipation arrangements. In spite of these, oblique anicuts are used at times. As there was no standard definite information regarding co-efficient to be adopted for these a scheme of basic study was drawn up. This report gives a description of the experiments conducted. Definite conclusions and design procedure are also indicated.

2. Dimensional analysis—

The geometry of the flow pattern is described by the length of the model anicut 'L', the height of anicut crest above bottom of the channel 'P', head of overflow 'H', referred to the level of the crest and measured in the uniform flow section upstream of the anicut, and the profile of the anicut 'P₁'. The fluid properties involved are

the specific weight 'r' and mass density 'p' the dynamic viscosity 'm' and surface tension 'o' being not considered as of minor importance. Only the independent flow characteristics is involved and it can be represented by the discharge 'Q'. 'Q' is conveniently selected as the dependent variable. Thus a complete statement of discharge function will be, as $Q = f(L, P, H, r, p, P_1)$.

By dimensional analysis the non-dimensional parameters are obtained as $Q/\sqrt{g} L H^{3/2} H/L, H/P, P_1$. Usually the co-efficient of discharge is made to include $\sqrt{g}$ so that

$$Q/LH^{3/2} = C = f(H/L, H/P, Pr)$$

$$\therefore Q = CLH^{3/2}$$

This is the simplest form of discharge equation. The relative influence of the non-dimensional factors on the co-efficient of discharge 'C' must be evaluated by experiment.

Significance of the parameters—

The H/L ratio is a measure of the shape of the discharging liquid stream in the plane of the weir. The influence of this ratio is believed to be negligible over the full practical range of other variables. The fact that most of the published results of research on full width weirs ignore the H/L ratio, indicates either that it was not recognised as an independent ratio or that its influence was not evident from the experimental data.

H/p ratio:—H/P ratio is a measure of the depth contraction characteristic of the weirs. Because the velocities in the channel upstream from the weir are proportional to the head, the Hp ratio, for the case of anicut with side contractions fully suppressed, is also a measure of the relative magnitude of the velocity in the approach

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channel. For this reason, its influence has been represented by a velocity head term. But in the present case, it is proposed to segregate the velocity head term by having the discharge equation as

$$Q = C.L. \left((H+ha)^{3/2} - ha^{3/2} \right)$$

$$\text{Where } ha = \frac{Va^2}{2g}$$

This ratio H/P has been described as Froude number for the flow in the approach channel. But Froude number is not a significant ratio. Thus it has to be concluded that H/P is just a geometric parameter.

Pf—Profile of anicut: It is a general practice that the profile of anicut is designed for design head (H_o) of overflow. The co-efficient is known to vary with head of overflow. To take into account the effect of the profile, it is proposed to introduce another non-dimensional parameter H/H_o .

Thus the experiments are proposed to be directed to study the variation of co-efficient 'C' with the non-dimensional parameters ' H/P ' and ' H/H_o '.

3. Experimental set up—

The experimental set up consisted of a long flume 4.344 ft. (1.324 m) wide across which, anicuts of different height with different obliquity can be constructed with end contractions fully suppressed. Anicuts of heights 0.1, 0.2, 0.3, 0.4, 0.5 and 0.6 ft. (3.048 cm., 6.096 cm., 9.144 cm., 12.192 cm., 15.24 cm., 18.288 cm.) with profile designed for head of overflow of 0.3 ft. were constructed. Creager's profile was chosen on account of its universality. The oblique anicuts were always constructed with profile at right angles to the line of anicut.

4. Experiments and observations—

Experiments were conducted as follows. A constant flow was maintained, when condition settled the heads over weir and V-notch were observed. Flow characteristics were also observed. The discharge was then slightly

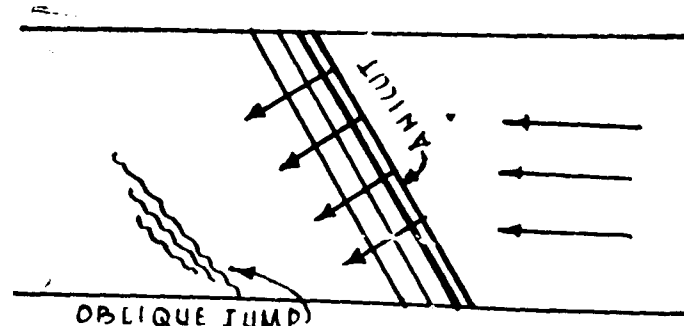
increased. The heads over weir and V-notch were noted down after constancy was obtained. This was carried on up to a head of overflow slightly in excess of the design head of 0.3 feet. Standard tables were used for computing the discharge over the V-notch.

The co-efficients of discharge taking into account the velocity of approach was computed using the formula,

$$Q = C.L. \left((H+ha)^{3/2} - ha^{3/2} \right)$$

where 'L' is length of anicut (normal or oblique as the case may be), 'ha' is head due to velocity of approach, which was computed based on the width of the flume. The charts of this co-efficient discharge versus H/H_o are shown in Figs. 1 and 2. It can be seen that for normal anicut and one with obliquity, 10 degrees three separate curves are obtained for $H_o/P = 3$, $H_o/P = 1.5$, and $H_o/P = 1.5$ for obliquity of 20 degrees two curves are seen for $H_o/P = 3$ and the other $H_o/P = 3$. For greater obliquity, only one curve was obtained for all values of H_o/P . The percentage reduction in co-efficient is worked out for each value of H/H_o independently, i.e., for one given H/H_o the reduction in percentage with obliquity is computed over the co-efficient for that value of H/H_o for normal obliquity. These are shown plotted in figures 3, 4 and 5. A perusal of these figures indicates that percentage reduction in co-efficient increases with H_o/P and H/H_o for a given obliquity.

This was observed to be due to the following flow characteristics. For low heads of overflow and high anicut heights, the flow over the anicut took place normal to it and not parallel to the flume.



But as the head of overflow increased or the height of anicut height decreased or in other words with increase in factor H/H_o or H/P , the flow straightened and tend to become parallel to the flume, this tendency showing an increase with increase in obliquity. Since the profiles were constructed normal to the alignment, this straightened flow was oblique to the profile with a consequent reduction in co-efficient of discharge. But it is of significant note that for H/H_o above 0.9 up to 1.2, the variation in co-efficient and percentage reduction in co-efficient is not significant.

It would be obvious that data regarding heads other than design head, would not be of much value in design. Hence reduction in co-efficient of discharge with obliquity for $H/H_o = 1.0$ is given in figure 6 for H_o/P value of 3, 1.5 and less than 1.5.

5. Conclusion—

As a result of studies, it is confirmed that co-efficient of discharge will be reduced in the

case of oblique anicut and the reduction in co-efficient increasing with obliquity and the parameter H_o/P and H/H_o .

Eventhough there is reduction in co-efficient indicating reduction in discharge per foot length of anicut, there is a net increase in discharge due to the increase in length of anicut which reduces the head of overflow for passing a given flood discharge. This will in consequence, reduce the cost of flood banks and other bank protection works upstream.

By utilising the design chart in Figure 6 the co-efficient for any given angle of obliquity head of overflow, anicut height, and the design co-efficient for a normal profile can be determined. An illustrative example is worked out in the appendix.

It is to be remembered that the charts are applicable only in the case of free overfall conditions.

APPENDIX.

An anicut has been constructed across a river of 100 ft. (30.48 m) width with an oblique angle of 25°. The maximum head of overflow on the anicut will be 3 ft. (0.9144 m) and anicut height also is 3 ft. (0.9144 m). The Co-efficient of discharge C in the formula $Q = C.L.H^{3/2}$ for the normal anicut chosen = 3.5 (1.933) (i.e., for the designed maximum head of 3 ft.). Find out the coefficient of discharge, C_v for the anicut with the above obliquity and the maximum discharge that will pass through both over the oblique anicut and in the case of normal anicut for the same conditions:—

$$P = 3 \text{ ft. (0.9144 m) i } H_o = 3 \text{ ft. (0.9144 m)}$$

$$L = 100 \text{ ft. (30.48 m) } C = 3.5 (1.933)$$

$$\theta = 25^\circ$$

$$H_o/P = 1 : \frac{H}{H_o} = 1 \text{ for this problem assuming that}$$

the profile for the design head will be adopted.

(1) To find out C_v for normal anicut for head of overflow of 3 feet:—

$$Q = 3.5 \times 100 \times 3^{3/2} = C_v \times 100 \left[(3+ha)^{3/2} - (ha)^{3/2} \right]$$

$$Q = 1819 \text{ cusecs (51.5 cu. mes)}$$

$$V_a = 1819/100 \times (3+3) = 1819/100 \times 6 = 3.0317 \text{ ft./sec. (0.9241 M/sec.)}$$

$$h_a = 3.0317^2 / 64.4 = 0.1428 \text{ ft. (0.04352 m)}$$

$$\text{Now } 1819 = C_v \times 100 \left[(3+0.1428)^{3/2} - (0.1428)^{3/2} \right]$$

$$\text{Hence } C_v = 3.296 (1.820)$$

(2) C_v for oblique anicut:—

$$\text{Length of oblique anicut, } L = 100 / \cos 25^\circ = 110.4 \text{ ft. (33.64 m.)}$$

As H_o/P is < 1.5 (i.e. = 1)

Figure 5 should be used.

From the charts, the percentage reduction in C_v for 25° obliquity and $H/H_o = 1$ is equal to 1.6

$$\text{Hence } C_v \text{ for oblique anicut} = 3.296 - 3.296 \times 1.6/100 = 3.2433 (1.791) \therefore (1)$$

for purpose of determining approach velocity, let us assume that the ratio C/C_v remains the same both for normal and oblique anicut. Now value of C for oblique anicut = $3.2433 \times 3.500/3.296 = 3.443 (1.901)$.

$$Q = 3.443 \times 110.4 \times 3^{3/2} = 1975 \text{ cusecs (55.93 cumecs).}$$

$$V_a = 1975/100 \times (3+3) = 3.292 \text{ ft./Sec. (1.003 m/Sec.)}$$

$$h_a = 3.292^2/64.4 = 0.1683 \text{ ft (0.05130 m)}$$

3. Maximum discharge that will pass through oblique anicut at 3' head—

$$\text{Correct discharge } Q = C_v L \times \left[\frac{(H \times h_a)^{3/2}}{(3 + 1.683)^{3/2}} - \frac{(h_a)^{3/2}}{(0.1683)^{3/2}} \right]$$

$$= 3.2433 \times 110.4 \left[\frac{(3 \times 0.1683)^{3/2}}{(3 + 1.683)^{3/2}} - \frac{(0.1683)^{3/2}}{(0.1683)^{3/2}} \right]$$

$$= 3.2433 \times 110.4 \times 5.570$$

$$= 1994 \text{ cusecs. —(2) (56.46 cu mecs)}$$

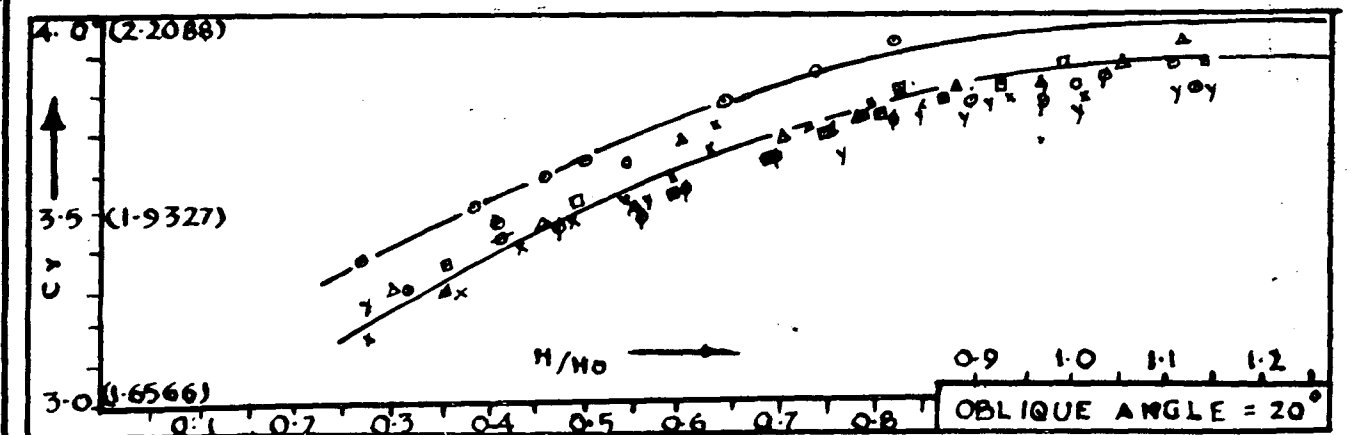
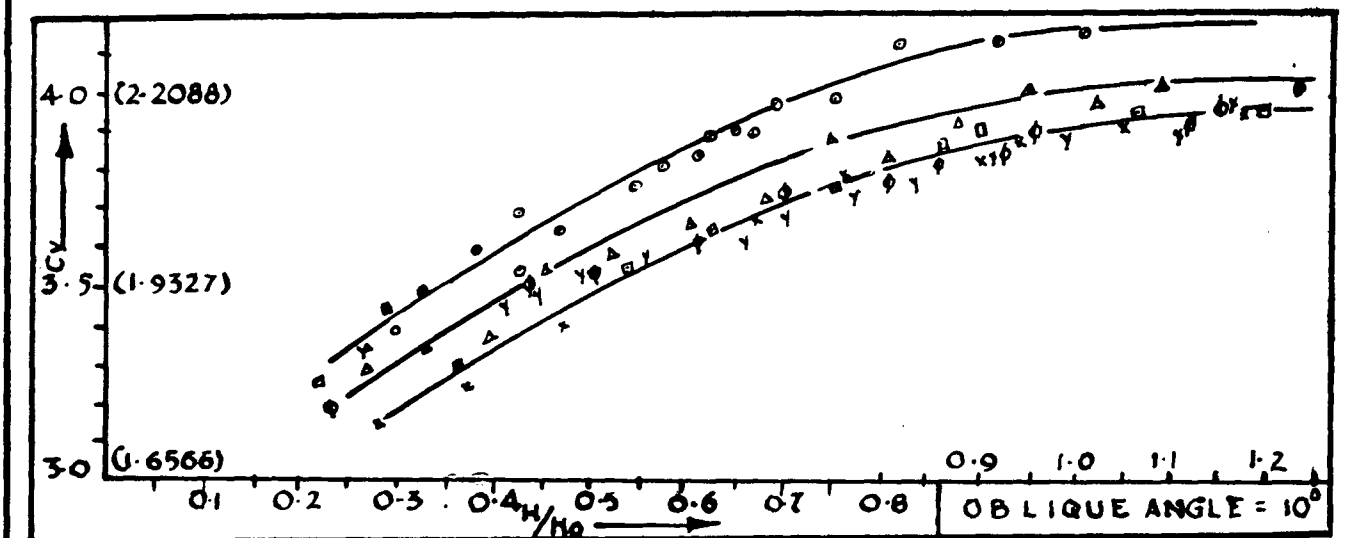
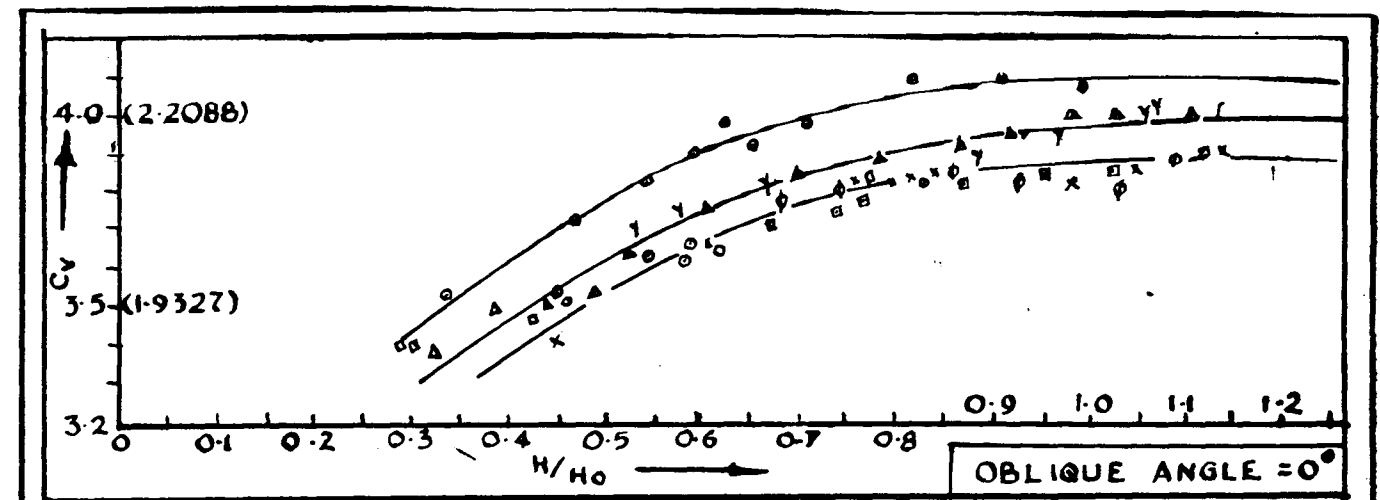
Check by applying this value of 'Q' for velocity of approach V_a the correct discharge obtained is equal to $3.2433 \times 110.4 \times 5.575 = 1995 \text{ cusecs (56.49 cu mecs)}$.

Maximum discharge that will pass through a normal anicut of the same height under the same head = 1819.0 cusecs (51.5 cusecs) from (1) ... (3)

Note.—(1) Instead of C for normal anicut, C_v itself is given, then C_v for oblique anicut can be straightaway computed and the maximum can be arrived at after 2 or 3 trials.

(2) In similar ways, the angle of obliquity required for an anicut of particular height and discharge co-efficient to pass a given flood discharge at any head of over flow can be determined by making use of the charts given.

(3) All measurements given are in British Units. The figures given within the bracket both in charts and in the calculations indicate the corresponding values in the metric units.



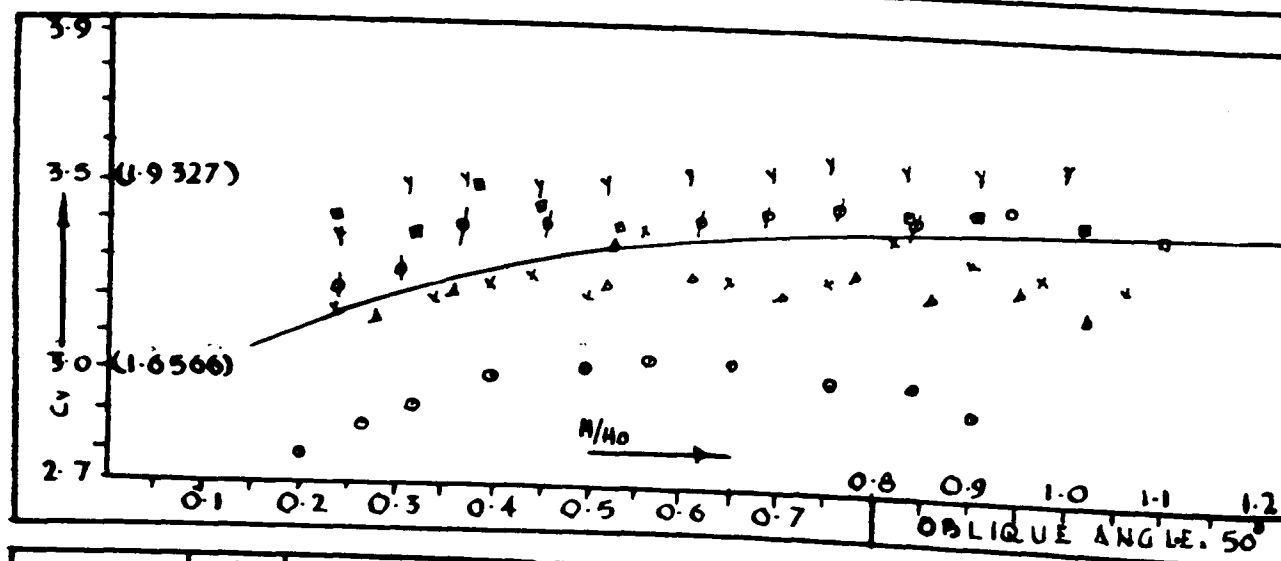
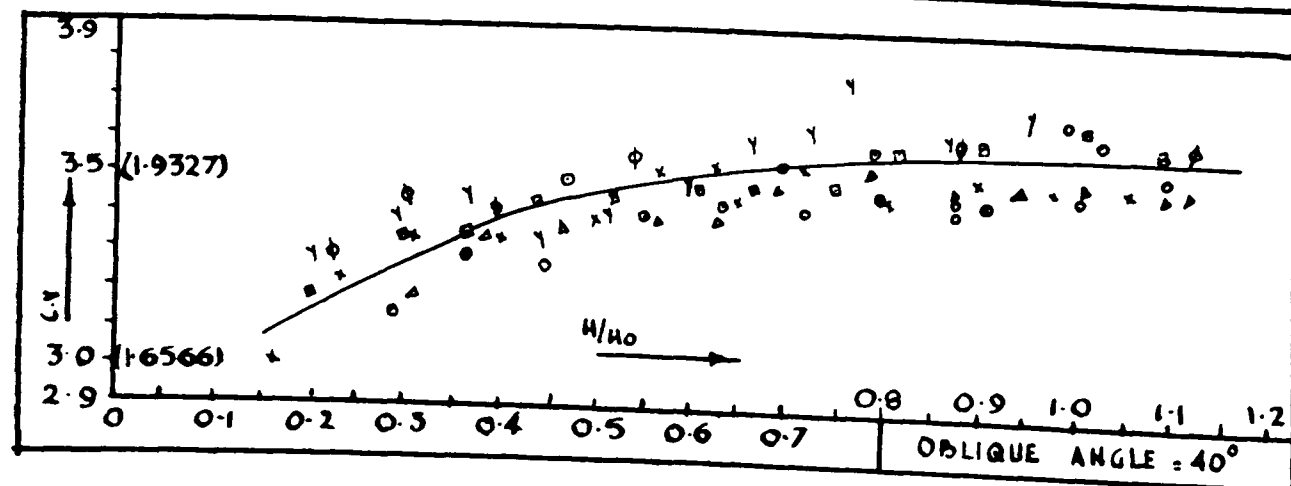
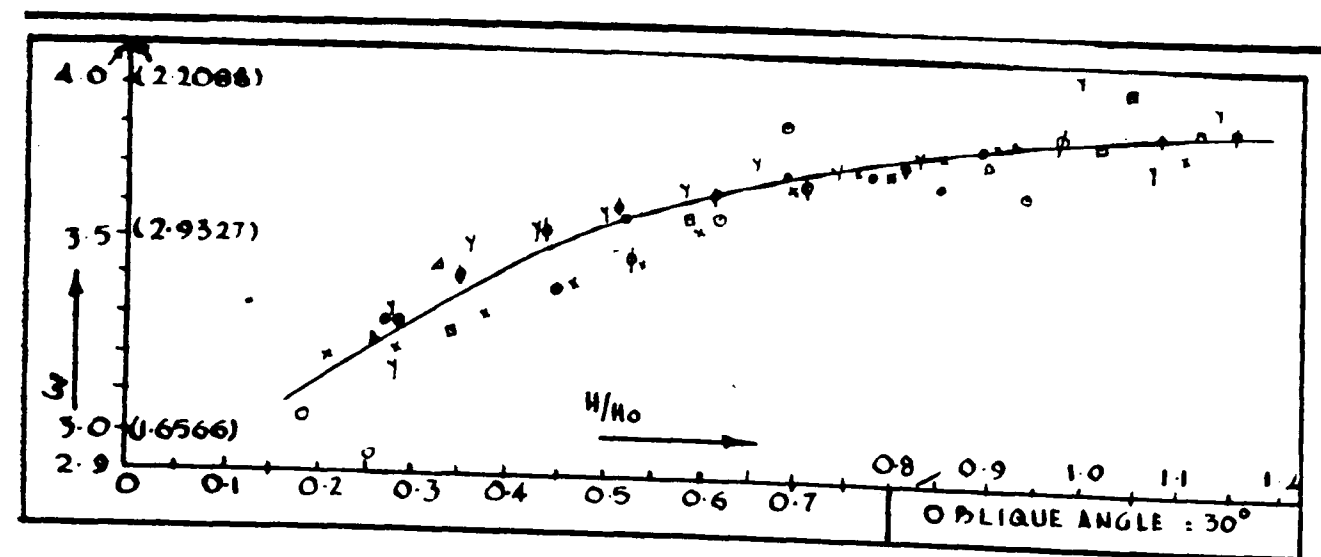
LEGEND:-

DESIGNATION	H_0/p	p
—○—○—○	3.00	0.1' (0.03048m)
—△—△—△	1.50	0.2' (0.06096m)
—x—x—x	1.00	0.3' (0.09144m)
—□—□—□	0.75	0.4' (0.12192m)
—◇—◇—◇	0.60	0.5' (0.15240m)
—γ—γ—γ	0.50	0.6' (0.18288m)

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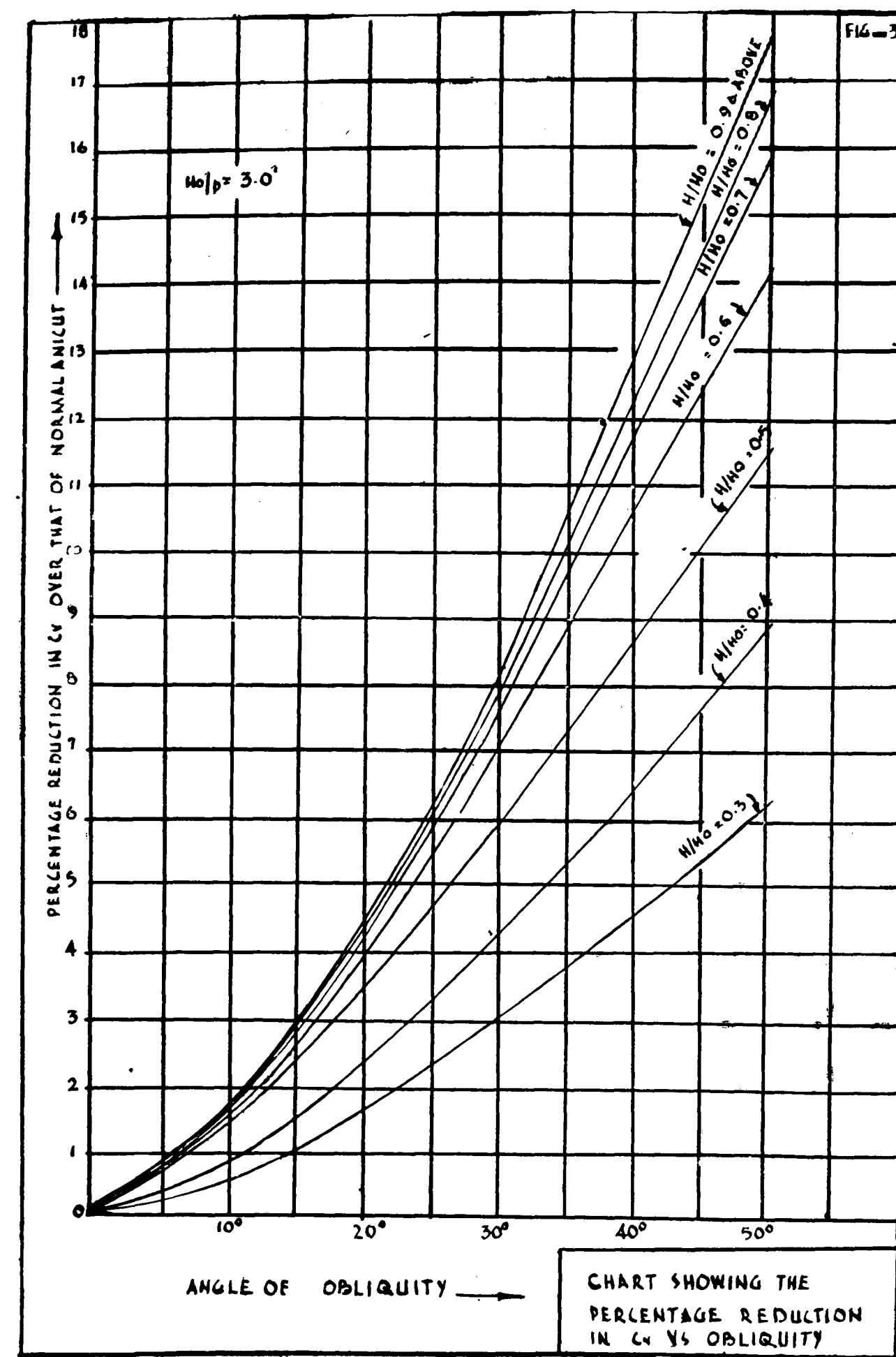
OBLIQUE ANICUT
DISCHARGE CHARACTERISTICS
CHARTS OF C_v , V_a , H/H_0

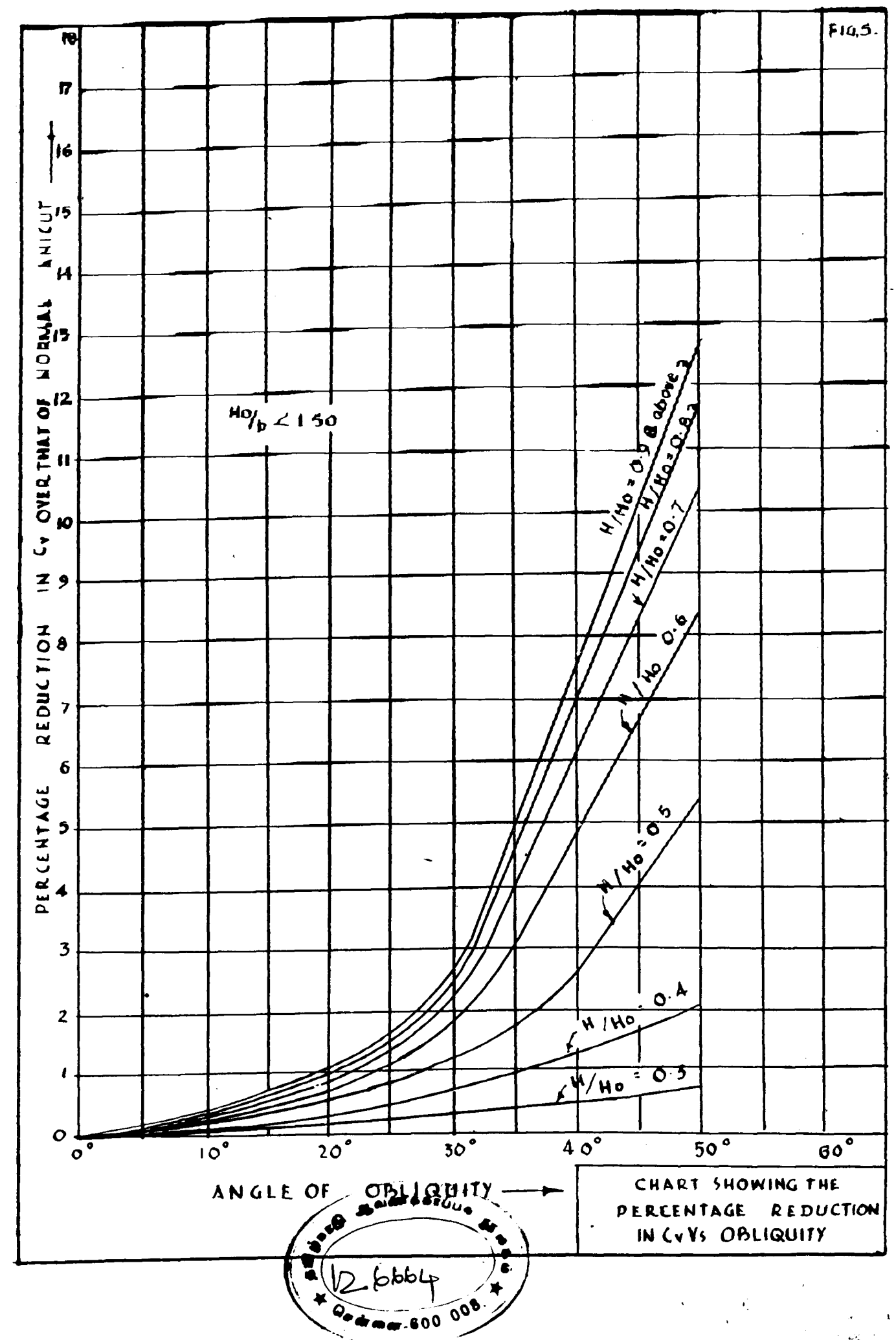
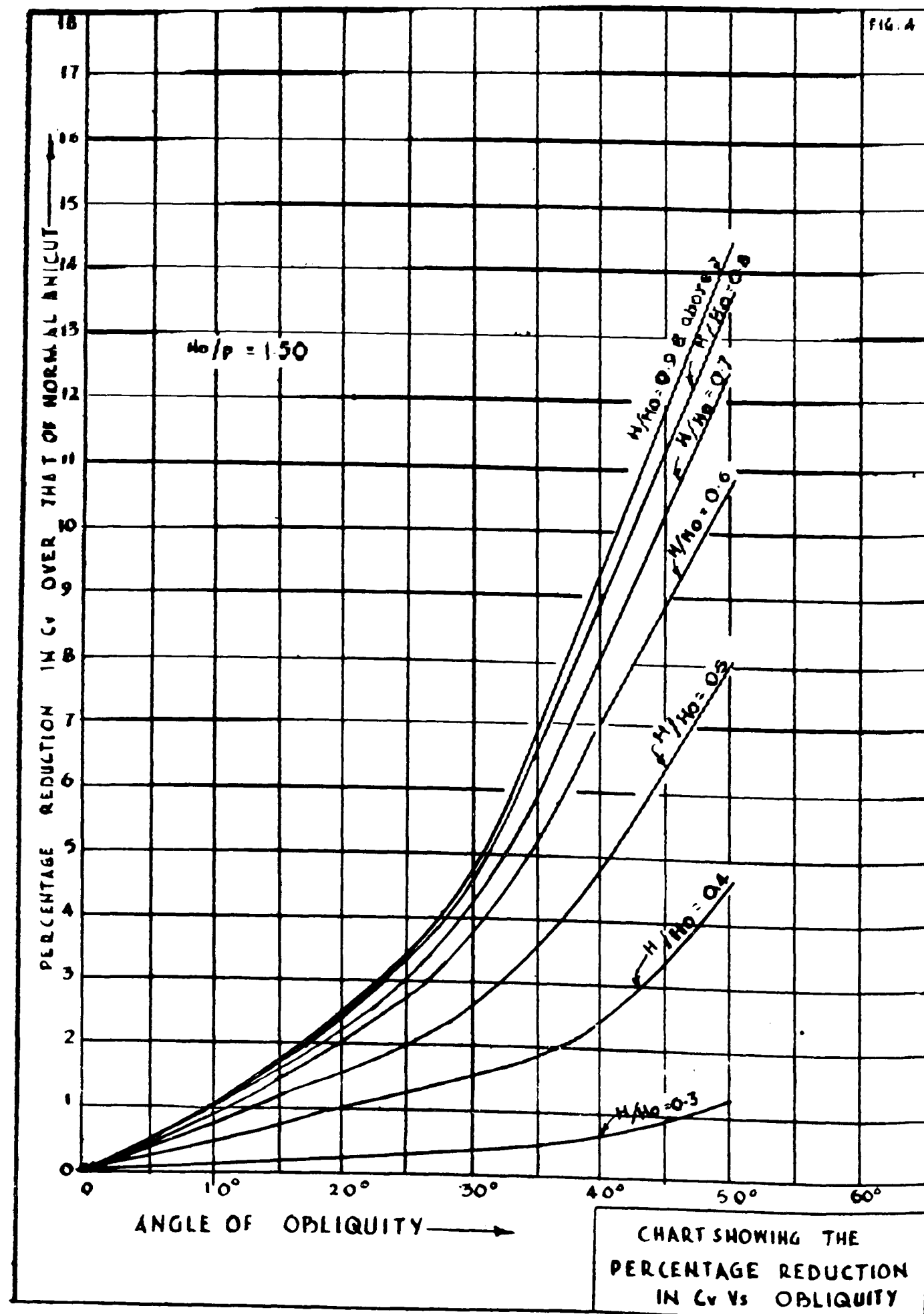
Sd: xxx	Sd: xxx	Sd: xxx.
J. E. R.	A. R. O.	E. E. R.
Trd: C. S. K.	Ckd: ...	FIG. 1

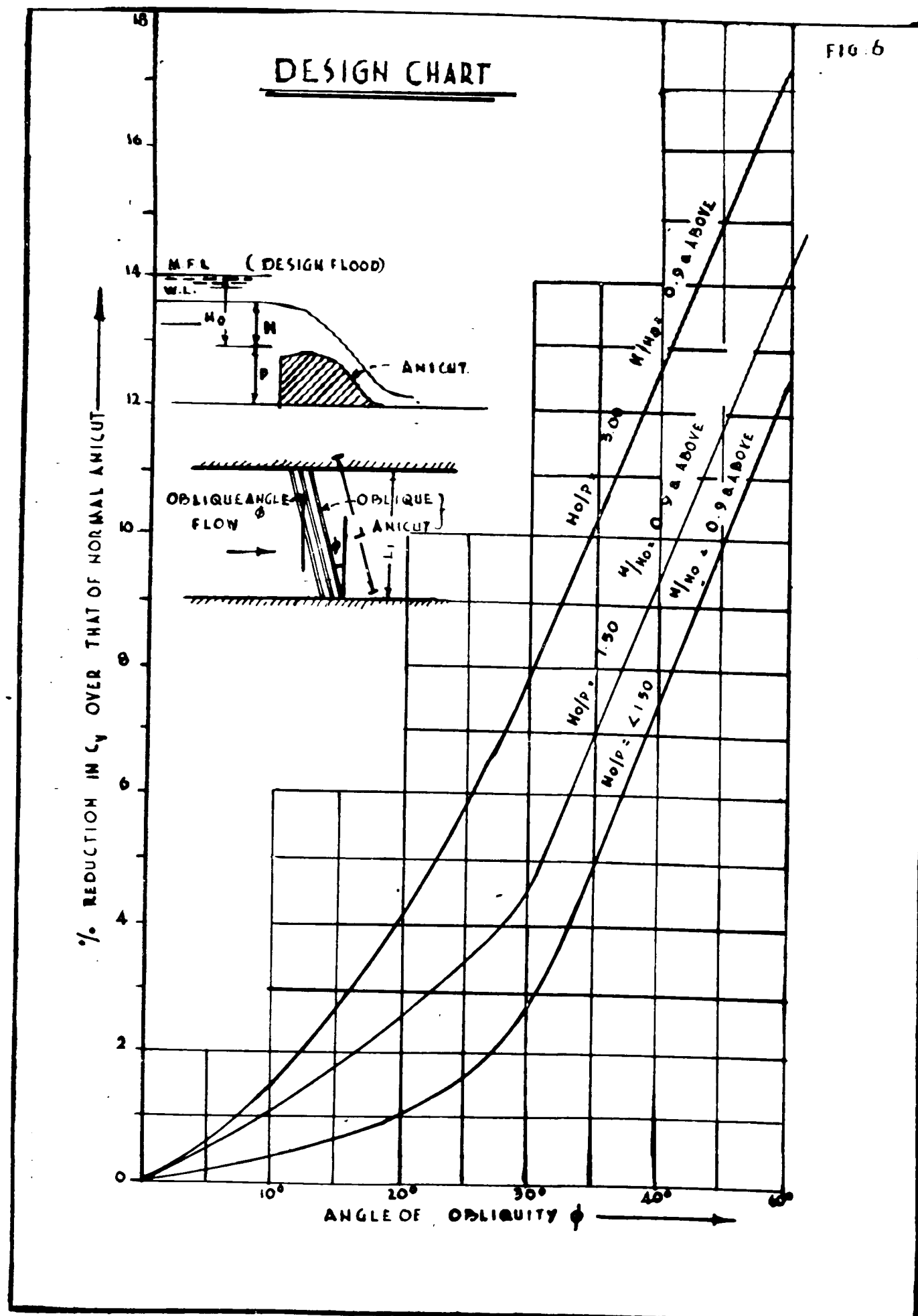


Designation	H_o/p	p
○-○-○	3.00	0.1' (0.03048m)
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PUBLIC WORKS DEPARTMENT, MADRAS. IRRIGATION RESEARCH STATION, POONCH		
OBLIQUE ANICUT DISCHARGE CHARACTERISTICS CHARTS OF C_v Vs H/H_o .		
Sd. xxx	Sd. xxx	Sd. xxx.
J. E. R.	A. R. O.	E. E. R.
Trd: C. S. K.	Chd. xxx	FIG. 2.







METRIC SYSTEM AND BRICKS

BY

* SRI T. K. VISWANATHAN, B.Sc., B.E.

Introduction—

The metric system has come to stay in this country and according to "The Standards of Weights and Measures Act, 1956" enacted by the Government of India the changeover to metric system will have to be completed within a period of 10 years (i.e., by the end of 1966) in all the spheres of activity.

Bricks—

With particular reference to 'bricks', there are in existence several sizes being manufactured in various parts of the country. A survey conducted in this direction revealed that there are as many as about 30 sizes of building bricks, leaving out minor variations in dimensions. This diversity not only creates confusion but makes it impossible to have a common basic size of bricks throughout. This would also deprive the chances of having reasonable prices for the bricks in the market. Having uniform and standard sizes as far as practicable for all commodities throughout the country, would facilitate rapid industrial growth among different States. In the International trade, the advantages of having uniform standards are evident and need no greater emphasis.

Sizes—

The Indian Standard Institution, which was entrusted with the work of suggesting standard sizes of bricks, has finalised the following two sizes:—

Nominal size.	Actual size.
(1) 20 cm. × 10 cm. × 10 cm.	19 cm. × 9 cm. × 9 cm.
(2) 20 cm. × 10 cm. × 5 cm.	19 cm. × 9 cm. × 4 cm.

The difference of 1 cm. between the nominal and actual size is to allow for the thickness of the mortar joint, which is 1 cm. Thus with the joints in place the bricks will fit into modular sizes which are always in multiples of 10 cm. (vide Sketch I).

The bricks of 9 cm. height shall be moulded with a frog 1 cm or 2 cm deep on one of the flat sides (vide Sketch II).

The Indian Standard Institution has also classified the bricks as Class I, Class II and Class III, according to the quality requirements, rather than sizes. In Madras State, however, the present classification of bricks are based on sizes and mode of manufacture. Thus the first class bricks or stock bricks are table moulded and of size 9" × 4½" × 2½"; the second class bricks are of the same size but ground-moulded. The country bricks are ground-moulded and of size 8½" × 4½" × 2".

There was another practical difficulty in the manufacture of 9 cm deep bricks by the ground-moulding process. Due to the greater thickness of these metric bricks (viz. 9 cm or 3½ inches) as against the present thickness of ground-moulded bricks (viz., 2 inches), the bricks, even during the moulding stage, do not retain the shape. They get distorted and bulge when the mould is removed. The burning is also not uniform due to the greater thickness. The absence of the frog, which is not possible to be had with the present system of ground-moulding process of brick manufacture, is also a reason for the non-uniform burning of these bricks.

The Metric Cell, Public Works Department, who were seized with this problem, have worked out suitable lesser thickness to achieve better shaped bricks and at the same time to conform to the modular co-ordination, according to which all the dimensions should be in multiples of the module. A thickness of 5.7 cm is suggested in this connection. Three courses of these bricks with 3 mortar joints of 1 cm each will give 20.1 cm or 20 cm, which will be a multiple of the basic module (10 cm).

*Executive Engineer, Metric Cell, P.W.D., Madras.

Regarding the table moulded bricks, we might not have any difficulty as the manufacture will be under controlled condition and the frog will help uniform burning of the bricks.

The Metric Cell had also arranged the manufacture of some samples of these metric sized bricks of class II-A and got them tested in order to study the various properties like absorption,

compressive strength, tolerance in dimensions, etc.

Classification—

Taking into consideration the conditions and method of manufacture that are obtained in this State, we have suggested having the classification of bricks based on method of manufacture and the sizes, rather than the physical appearance alone. Please refer the table below:—

(1)	Class I bricks.	Class II-A bricks.	Class II-B bricks.
1 General requirements	Shall have a uniform colour; thoroughly burnt but not over-burnt. Shall have plane rectangular faces with parallel sides of sharp straight right angled edges. Shall have a fine, compact and uniform texture.	Shall have a fairly uniform colour; may be slightly over-burnt. May be slightly distorted and have slightly round edges. Same as for class I	Same as class II-A. May be slightly distorted and have round edges. Same as for class I.
1 (a) Sizes	19 × 9 × 9 cm. 19 × 9 × 4 cm.	19 × 9 × 5.7 cm. 19 × 9 × 4 cm.	19 × 9 × 5.7 cm. 19 × 9 × 4 cm.
2 Water absorption for 24 hours of immersion in cold water.	Not more than 20 per cent by weight.	Not more than 22 per cent by weight.	Not more than 22 per cent by weight.
3 Efflorescence	Slight	Slight	Slight.
4 Moulding	Table-moulded	Ground-moulded	Ground moulded.
5 Burning	Kiln-fired	Kiln-fired	Clamp-fired.
6 Minimum compressive strength.	70 kg./cm. ²	35 kg./cm. ²	25 kg./cm. ²

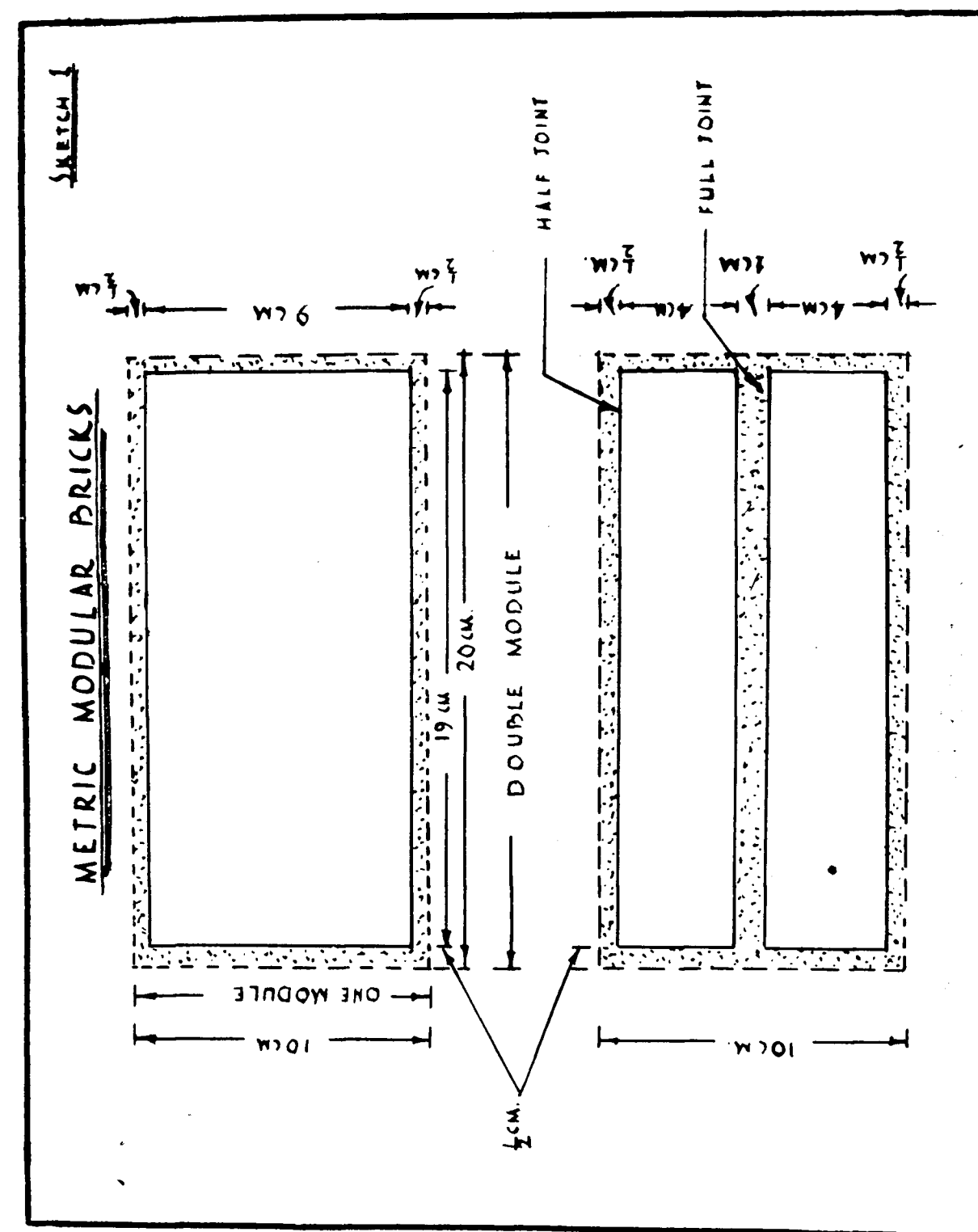
Advantages—

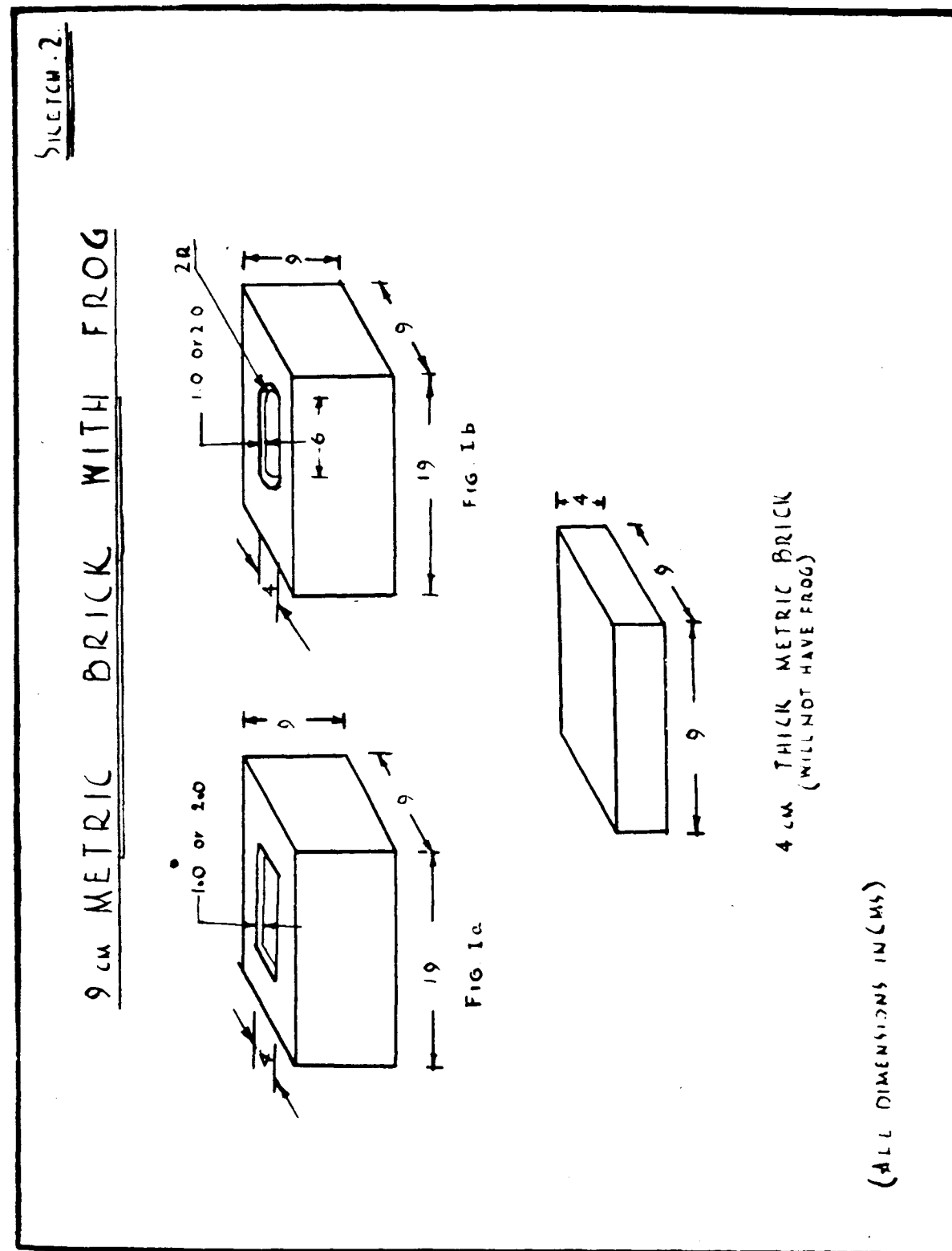
Though the metric bricks might appear superficially larger than the present table-moulded bricks due to their greater thickness, the metric bricks are 11 percent lesser in volume. The weight of a metric brick is 2.5 kg. as against the weight of 2.8 kg. of the present stock brick. Thus it may be seen that modular bricks can be handled with more ease; and the width of 9 cm. (3½") as against the width of 4½ inches of the present brick is an added advantage in easier handling of the bricks by the masons.

Since the length of the metric brick (7½ inches) is lesser than that of the present brick (9 inches), economy can be achieved in the use of the former. It is reported that the Central Public Works Department have constructed, in collaboration with the N.B.O., a two storeyed

model block using metric sized bricks and they have reported that there was no discomfort due to the reduced thickness of the walls. For the same area of masonry, there will be a saving of about 14.5 per cent in the number of bricks and about 3.8 per cent in mortar, if metric modular bricks are used instead of the present bricks. Due to the greater thickness of the metric bricks, the number of courses will also be reduced, resulting in saving of mason's time.

The traditionally established bonds like English bond, Flemish bond, etc., can be adopted for these metric sized bricks. Research carried out in the Central Building Research Institute have established that the drying time of modular bricks is less than that of the standard size of F.P.S. bricks irrespective of the clay content of soil used and the drying conditions.



**Conclusion—**

In summing up the above facts, it is seen that the cost of construction can be considerably reduced by using metric modular bricks.

When the design of a building is based on modular co-ordination, by which the dimensions of the building components are multiples of the basic module (i.e. 10 cm. which is also the module for the metric brick), the advantages are manifold.

(a) The architectural designs will be simplified and drafting time reduced.

(b) The preparation of plans and working drawings will be simplified due to the modular system.

(c) When the sizing of the components based on a "module", the construction and production costs are reduced, as the variety of building components are eliminated or at least brought to a minimum.

STUDIES ON WATER CUSHION FOR CANAL DROPS

BY

* S. SHANMUGAM, B.E., M.Sc.

SYNOPSIS

This report deals with the new technique of energy dissipation for notch type drops as an alternative to water cushion which is normally adopted for canal drops. Experiments conducted for various length and depths of the water cushion have been explained. The idea of splitting the nappe into a number of horizontal thin sheets was tried and found to be successful. This new type design consisting of divide slabs and honey comb was finalised in the place of water cushion. Detailed experimental procedure is given with necessary tabulations. Typical calculation using the new design is shown in Appendix II. The performance, and cost comparison between new design and Ellis design with water cushion have been worked out and furnished. As a result of the study, a new type design which is economical and better in performance is recommended as an energy dissipating device for notch type canal drop up to 10 feet fall and 1,000 cusecs carrying capacity.

1. Introduction—

Generally a water cushion is provided to serve as an energy dissipating and scour preventing device at a fall or drop. There are a number of empirical formulae and thumb rules available for designing the depth and length of cistern required for a canal drop. For a particular height of drop the water cushion is designed using all these formulae and thumb rules. The values obtained are furnished in Table 1. From the table it is seen that for the same drop the depth of cistern varies from 0.75 feet to 4.5 feet and the length varies from 13 feet to 42.5 feet.

There is such a large variation. Hence the present study has been taken up to standardise the design of water cushion for canal drops. Since most of the canal drops in Madras State are notch type, notch type drop has been taken for the studies.

2.1.1. Previous Work—

The notations used in this report are explained in Appendix I.

The empirical formulae developed by various authors may be divided into 3 groups depending upon the theory on which they are built up. The formulae developed by Ellis which is used in Madras State, formula formed by Etcheverry and the formula used in Pacific South-West region are based on the assumption that the length of water cushion must be sufficient to contain horizontal projection of the overall fall $2\sqrt{hd}$ with some additional margin.

$$L = 2\sqrt{hd} + d; \quad X = \frac{1}{2}d\sqrt{h-d} \quad \text{Ellis (1)}$$

$$L = 3\sqrt{hd}; \quad X = L/6 \quad \text{Etcheverry (2)}$$

$$L = 2.5\sqrt{hd}; \quad X = \frac{h}{8} \quad \text{Pacific South-West region (3)}$$

2.1.2. There are certain empirical formulae taking energy into consideration.

$$L = \frac{4E}{3\sqrt{h+d}} + E \quad \text{(Bombay P.W.D. Handbook (4))}$$

$$X = E + E^{\frac{1}{2}}h^{\frac{1}{2}} - d$$

$$L = 5(X + d) \quad \text{(Used in Bihar and Orissa (4))}$$

$$X = 2.25 E^{\frac{1}{2}} X h^{\frac{1}{2}} - d$$

$$L = 4 E_f$$

$$X = \frac{E_f}{2} \quad \text{(Montagu's formula (4))}$$

Taking hydraulic jump as the main criterion Elevator-sky recommends the length and depth of water cushion as:

$$L = 4.5(d_1 - d)$$

$$X = 0.2d$$

with friction blocks placed at a distance of 0.9 d.

2.2.1. Apart from the empirical formulae some experimental work also has been done for

energy dissipation downstream of a drop. In the year 1953 experiments were conducted in this Research Station (7) to suggest an economical type design for drop of 6 feet (canal capacity 250 cusecs). Various devices such as hanging baffles, dentated sill, R.C. rib along revetment were tried and finally one row of friction blocks was recommended instead of water cushion.

2.2.2. Detailed experiments were carried out at Poona Research Station (8) on canal drops and five type designs were recommended. For the vertical type drop structure they suggest

$$L = 5\sqrt{dh}$$

$$X = \frac{1}{6}(dh)^{\frac{2}{3}} \quad \text{with 2 rows of friction blocks.}$$

The same design has been recommended in draft manual on falls by Central Water and Power Commission (9).

2.2.3. B.T. Morris conducted experiments and derived the formula (3) as

$$L = \left(2.5 + 1.2 \frac{d}{h}\right)\sqrt{dh} \quad \text{and} \quad X = 0.1L \text{ or } 0.3\sqrt{dh}$$

or $\frac{d}{2}$. Since excessive scour was observed at

the outlet of a number of straight drop spillway located at Florido which were designed according to B.T. Morris, model studies were conducted and the following was recommended by Charles A. Donnally and F. Blaisedell (10):

$$L = L_n + 2.55d$$

$$\text{and} \\ X = 0.4d$$

with one row of friction blocks at a distance of $L_n 0.8d$ from upstream face.

3. Present Programme of work.—

The studies were taken up to give safe, economical and minimum length and depth of water cushion to be provided for notch drops. It was proposed to study the performance of drops for various conditions:—

1. Varying the length of cushion.
2. Varying the depth of cushion.
3. Varying the height of drop.
4. Varying the discharge.

4. Model set up.—

A canal carrying 200 cusecs and having 6 feet drop was designed as per Ellis design with three trapezoidal notches and represented in a hydraulic model to a scale 1 in 8. About 120 feet upstream length of canal and 150 feet downstream of notch were represented. Necessary stilling arrangements were made and water levels were recorded with precision pointer gauges. The discharge was measured using a calibrated V notch. Glass plates were fixed on the side wall to observe the hydraulic jump and positive bed roller. Sieved sand was used (mobile bed) downstream of the apron to study the scour pattern.

5. Experimental work.—

5.1. Various length and depth of water cushion.—First the experiments were done for the lengths $2d$ and $3d$, varying the depth as $0.2d$, $0.3d$, $0.4d$, $0.5d$ and $0.6d$, keeping the water level at full supply depth “ d ”, i.e., 4.5 feet. Surface profile velocity observations and scour readings were taken for each case. Similarly the experiments were done for half supply depth. The length of impervious apron was reduced by 12 feet and experiments were repeated.

5.2. Splitting the nappe.—

Meanwhile, as suggested by Ventechow (11), the effect of separating the jet into a number of long thin sheets thereby dissipating the energy without resort to hydraulic jump was studied in the model. G.I. pipes of 1 inch diameter and 1 foot long were placed in between the notches one over the other in the direction of flow. Observations were taken for 5.5 feet drop and upstream water depth 4.5 feet with up cistern of 9 feet by 1 foot and 9 feet by 0.5 foot considerable reduction in bed scour was observed. Eventhough this was found to be effective, study on this was dropped due to the difficulty of construction and the narrow openings which caused obstruction to the free passage of debris. Since most of the canals in Madras State are not usually carrying debris and logs, further studies to find a suitable means for separating the flow into thin sheets were done bearing in mind that it must be

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of simple type and easy for construction. To split the nappe into number of thin sheets two methods were found suitable and they were tried in the model.

5.2.1. Gratings—

Gratings of 1 inch square holes, 1.5 feet long in model were prepared in G-1. sheet. By providing gratings on the upstream side in between the notches the falling jet was separated into long thin sheets. For 5.5 feet drop with a stilling basin of 9 feet by 0.5 foot gratings were provided and the model was run for half an hour. Surface readings, velocity and scour observations were taken. About 35 per cent reduction in maximum scour was observed. The main drawback in this was the difficulty and higher cost of construction. Hence some other means was tried.

5.2.2. Divide slabs—

By providing divide slabs of 3 inches thickness at 1 foot intervals on the notch openings the falling water was separated into number of parts. Surface readings, velocity and scour observations were taken for a drop of 5.5 feet with a plain apron of 12.66 feet length for the discharge of 200 cusecs with and without these divide slabs. About 30 per cent reduction in maximum scour was observed. Since it was more economical than the gratings and simple for construction, further experiments were done on the following design aspects of divide slabs.

5.3. Design aspects.—

5.3.1. Coefficient.—Due to the introduction of divide slabs reduction in discharge was observed. So discharge studies were done for the discharges 100, 200, 330 and 530 cusecs for upstream depths 3.375 feet, 4.5 feet, 5.625 feet and 6.75 feet with divide slabs at 1 foot interval. The co-efficient to be used for notch drops with divide slabs was determined.

5.3.2. Interval and angle of slabs—

Further studies were done varying the interval and angle of divide slabs. Divide slabs were placed at 2 feet c/c for a drop of 5 feet and no reduction in bed scour was observed. Then, instead of placing the divide slabs horizontal,

they were placed at an angle of 20 degrees at 1 foot interval and observations were taken. No visible reduction in bed scour was observed. But the discharge was greatly reduced. For the effective interaction of jets the bottom most two slabs and the floor of the notch were kept at a slope 1 in 6 upward and other slabs were kept horizontally. Experiments were done in this condition. Scour pattern was not changed in the model but about 15 per cent reduction in discharge was observed. Hence studies were finalised for divide slabs placed horizontally at 1 foot c/c.

5.3.3. Honey-comb—

A honey-comb structure placed on the downstream end of apron was tried and found to give best performance. Experiments were conducted by varying the height of honeycombs in the model for various drops ranging from 4 feet to 10 feet. For each case, the scour profiles and water surface profiles were observed.

5.3.4. Apron length—

Experiments were done to find the minimum apron length to be provided downstream the drop. The formula formed by Etcheverry:

Apron length = $L = C \sqrt{hd}$ was taken for guidance. Experiments were done using honeycomb and divide slab for various apron lengths varying the co-efficient "C" from 2.6 to 3.2. Experiments were repeated for drops 4 feet to 10 feet and discharges 100 cusecs to 530 cusecs. The scour profiles were observed for each case. In all the cases apron lengths were measured from the end of the sill and not from the toe of the drop wall.

5.4. Comparative study between Ellis Design and new Design—

Experiments were conducted for a drop of 5 feet 6 inches high. The performance of the water cushion as designed by Ellis method (Fig. 1) was compared with that for the new design. Scour profiles and detailed velocity observations were taken and furnished in fig. 6.

6. Discussion—

6.1. Splitting the nappe into a number of thin sheets was found to reduce the bed scour considerably. By using the divide slabs for the same

purpose, the scour was found to get reduced by 30 per cent. The normal coefficient recommended by Ellis for the design of notch is 3.745. Due to the introduction of the divide slabs, the co-efficient has to be reduced. By experiments the reduced co-efficient was found to be 3.55. Thus in this new design, the area of the vent way increases causing a decrease in the size of the notch piers. No appreciable reduction in bed scour was observed either by increasing the spacings between the slabs or by slightly inclining the slabs. Hence horizontal divide slabs at 1 foot interval are recommended.

Further to improve the performance, a honey comb structure placed at the end of the apron was tried. When both divide slabs and honey-comb structure were used, about 70 per cent reduction in bed scour was observed. The honey-comb structure helps in deflecting the high velocity filaments upwards and formation of positive bed roller in rear. Hence end scour also was reduced considerably. It was found by experiments that a solid baffle wall of height $0.03d_1$ to $0.5d_1$ must be constructed and only over that honey comb structure must be built. For drops up to 5 feet, the total height of this structure (solid portion and the honey comb) was found to be $0.3d_1$ and for drops in between 5 to 10 feet, $0.33d_1$ for best performance. From the experiments, the minimum length of apron for drops up to 5 feet is found to be $2.9 \sqrt{hd}$ and for drops between 5 feet to 10 feet $2.8 \sqrt{hd}$. This length is less than that provided by Ellis. The length of the apron has to be measured from the end of the sill and not from the toe of the drop wall.

For the finalised design (i.e., with divide slabs, honeycomb structure and reduced length of apron), the performance was observed for various canal discharges and various drop heights and the observations are furnished in Table 2. It is seen that for all the cases, i.e., for canal discharges up to 1,000 cusecs and for drop heights less than 10 feet, the performance is very good with negligible scour. The main advantage of this method is that no water cushion is necessary. Absence of water cushion causes savings in the volume of drop

wall masonry excavation and in addition causes enormous savings due to reduced thickness of floor slab in the case of porous soils. As the depth of water cushion increases, increased thickness of floor slab has to be provided for resisting uplift pressures in previous soils. It was found by experiments that up to a slope of 2|3:1, the water falls on the apron and not on the sloping face of the drop wall. Hence, where soil permits, mere lining up to a slope of 2|3:1 can be adopted omitting the solid masonry wall for the drop thus causing a huge economy. The difference in cost between a drop designed by Ellis and by the new design is worked out for a particular case and shown in Appendix 2. Thus it is seen that the new design is cheaper than the Ellis design by about Rs. 1,200 for 8 feet drop.

7. Recommended Procedure—

The recommended new design is worked out for 8 feet drop and furnished in Appendix 3. The divide slabs can be of pre-cast type (weld-mesh or chicken mesh plastered on both sides) and can be introduced during the construction of the notches. If the top slab is designed properly, this can be used as a small canal crossing for men, sheep, etc.

8. Limitations—

This new design is economical only where it is necessary to maintain the water level upstream of the drop by providing the notches. This can be adopted for drops up to 10 feet and discharges up to 1,000 cusecs. In previous types of soils, there is considerable reduction in floor thickness due to the avoidance of the water cushion. This method is likely to be blocked often, if the canal carries logs or boulders.

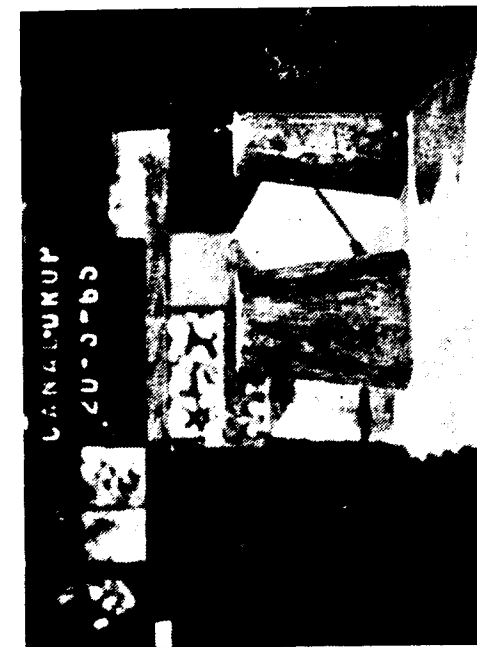
9. Conclusion—

A new type of energy dissipating arrangement for the notch drops with free fall in the place of the water cushions has been evolved and recommended. This design is more economical than the Ellis design usually adopted in Madras State with better performances. In addition, this serves as a canal crossing without any extra cost. Construction also is simple.

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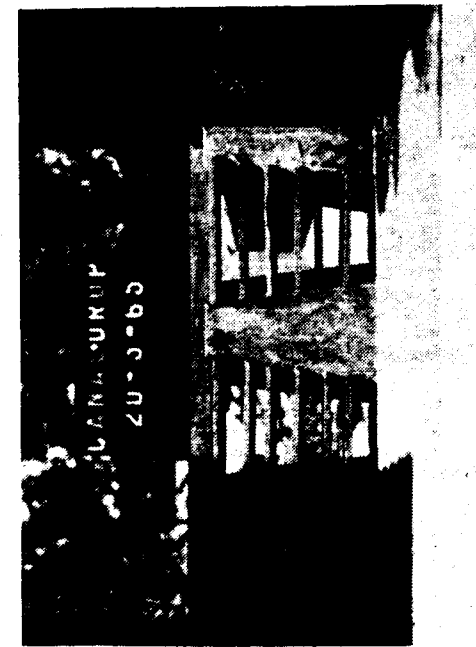
MODEL VIEW



ELLIS DESIGN—Trapezoidal Notch (Looking downstream).



NEW DESIGN—With Divide slab and honeycomb (Looking upstream).



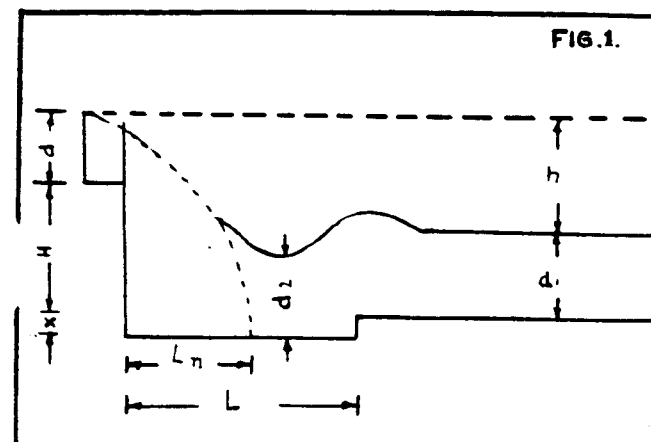
NEW DESIGN—Trapezoidal Notch with Divide slab.



PERFORMANCE OF NEW DESIGN.

APPENDIX 1.

NOTATIONS :



- L = Length of water cushion.
 X = Depth of water cushion.
 d = Upstream water depth.
 h = Height of fall (difference in water level between upstream and downstream).
 d_1 = Downstream water depth in the canal.
 H = Difference in level between upstream and down stream.
 d_2 = Depth at the fall before the formation of jump.
 E = Energy at crest.
 E_1 = Energy of flow downstream.
 L_n = Horizontal projection of falling jet.

APPENDIX 2.

COMPARISON OF COST BETWEEN WATER CUSHION (ELLIS) AND THE NEW DESIGN.

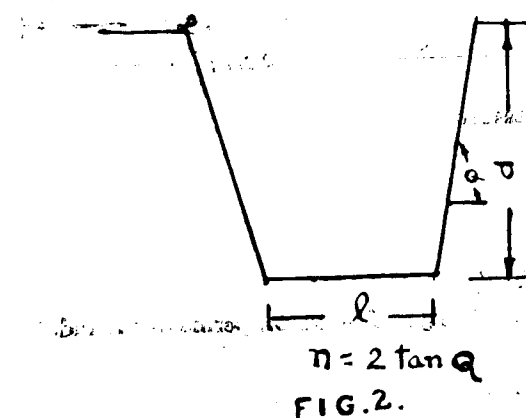
Height of drop—8'.

Flume width—25'.

	Excess cost in new design. RS.	Savings in new design. RS.
Divide slabs—		
In new design 3" thick slabs 18 numbers roughly 75 cft.	600.00	
Apron length—		
In water cushion 21.45'		
In new design 19.1'		
Savings 2.35' length.		

	Excess cost in new design. RS.	Savings in new design. RS.
Flume width 25'. Thickness of apron 4'. ∴ Savings $2.35 \times 25 \times 4 = 235$ c.ft. ..		235.00
Thickness of apron—		
In water cushion 4'		
In new design 3'		
Savings 1' thick concrete.		480.00
$= 1 \times 25 \times 19.1 = 480$ c.ft.		
Honey comb—		
2.25' height 25' long honey comb ..	150.00	
Drop walls and wing walls—		
In case of water cushion drop wall and wing walls are to be taken 2.75' more the savings is roughly		1,200.00
Earthwork cutting—		
In case of water cushion $21.75 \times 2.75 \times$ $25 = 1,500$ c.ft.		35.00
	750.00	1,950.00
Net savings		1,200.00

APPENDIX 3.



Calculations for the new design—

Height of drop —8'.

Upstream water depth 'd' = Downstream water depth $d_1 = 6.75'$.

Discharge = 530 cusecs.

Providing 3 notches,

$$\text{Discharge through one notch} = \frac{530}{3} = 176.6 \text{ cusecs.}$$

$$q = 3.55 (1 + 0.4 dn) d^{3/2}$$

$$176.6 = 3.55 (1 + 0.4 \times 6.75n) (6.75)^{3/2}$$

$$1 + 2.7n = 2.84 \dots (1)$$

For half supply depth, i.e., for
(2/3d) 4.5' depth

$$88.3 = 3.55 (1 + 0.4 \times 4.5n) (4.5)^{3/2}$$

$$1 + 1.8n = 2.6 \dots (2)$$

$$(1) - (2) = 0.9n = 0.24$$

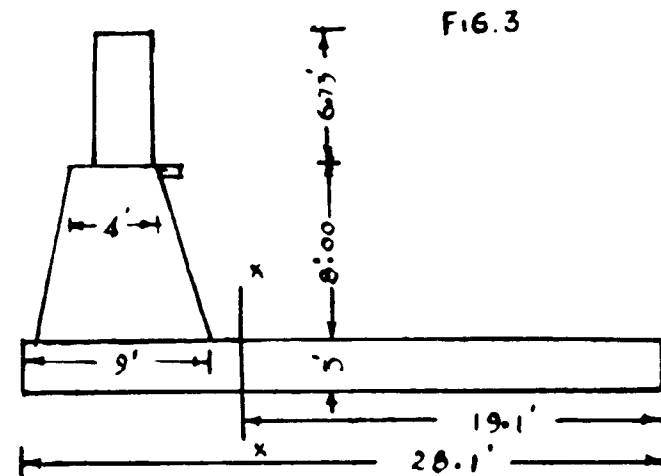
$$\therefore n = 1/3; \therefore \text{slope } 1/6$$

Then $l = 2'$

$$\text{Top width} = 2 + 6.75 \times 1/3 = 4.25'$$

Body wall. $\left\{ \begin{array}{l} \text{Top width} = \frac{3}{4} + \frac{d}{2} = 4' \\ \text{Bottom width} = \frac{h}{\sqrt{1}} + d_1 = 9' \end{array} \right.$

$$\text{Length of apron from the end of the sill} = 2.8 \sqrt{hd} = 20.6'$$



Thickness of apron.—For uplift pressure

Assuming 3' thick

Upstream end of apron:—

$$\frac{1}{\alpha} = \frac{11}{28.1} = 0.392$$

$$\phi_{D_1} = 100 - \phi_D = 100 - 23 = 77\%$$

(ϕ_D taken from plate VII-5 in Design of weirs on permeable foundation, publication No. 12)

Downstream end of apron:—

$$1/\alpha = 3/28.1 = 0.107$$

$$\phi_{D_1} = 14\%$$

$$\text{Uplift Pr. at } \times \times = 14 + 19.1 (77 - 14) = 14 + 42.8 = 56.8\%$$

$$\text{Absolute Pr.} = 56.8 \times 8/1.25 = 363.84$$

Assuming that 75% of it will act, thickness of floor required for net uplift Pr.

$$= 56.8 \times \frac{8}{1.25} \times \frac{75}{100} = 2.72'$$

Thickness of apron provided = 3'

Hence safe

$$\text{Height of honeycomb} = 0.33 d_1 = 0.33 \times 6.75 = 2.25'$$

TABLE 1.

Serial number.	Method.	Water cushion dimensions.	
		Length 'L'	Depth 'X'
(1)	(2)	(3)	(4)
1	Ellis formula	15.0'	1.5'
2	Formula formed by Etcheverry	15.6'	2.6'
3	Pacific South-West region S.C.S.	13.0'	0.75'
4	Punjab State	..	4.0'
5	Bombay P.W.D. Handbook	19.2'	4.5'
6	Bihar and Orissa—Glass formula	42.5'	4.5'
7	Montagu's formula	16.3'	2.0'
8	Professor Scholitch's formula	..	4.0'
9	Formula formed by B. T. Morris	17.7'	1.6'

Particulars—

Discharge = 200 cusecs.

Height of drop = 6'

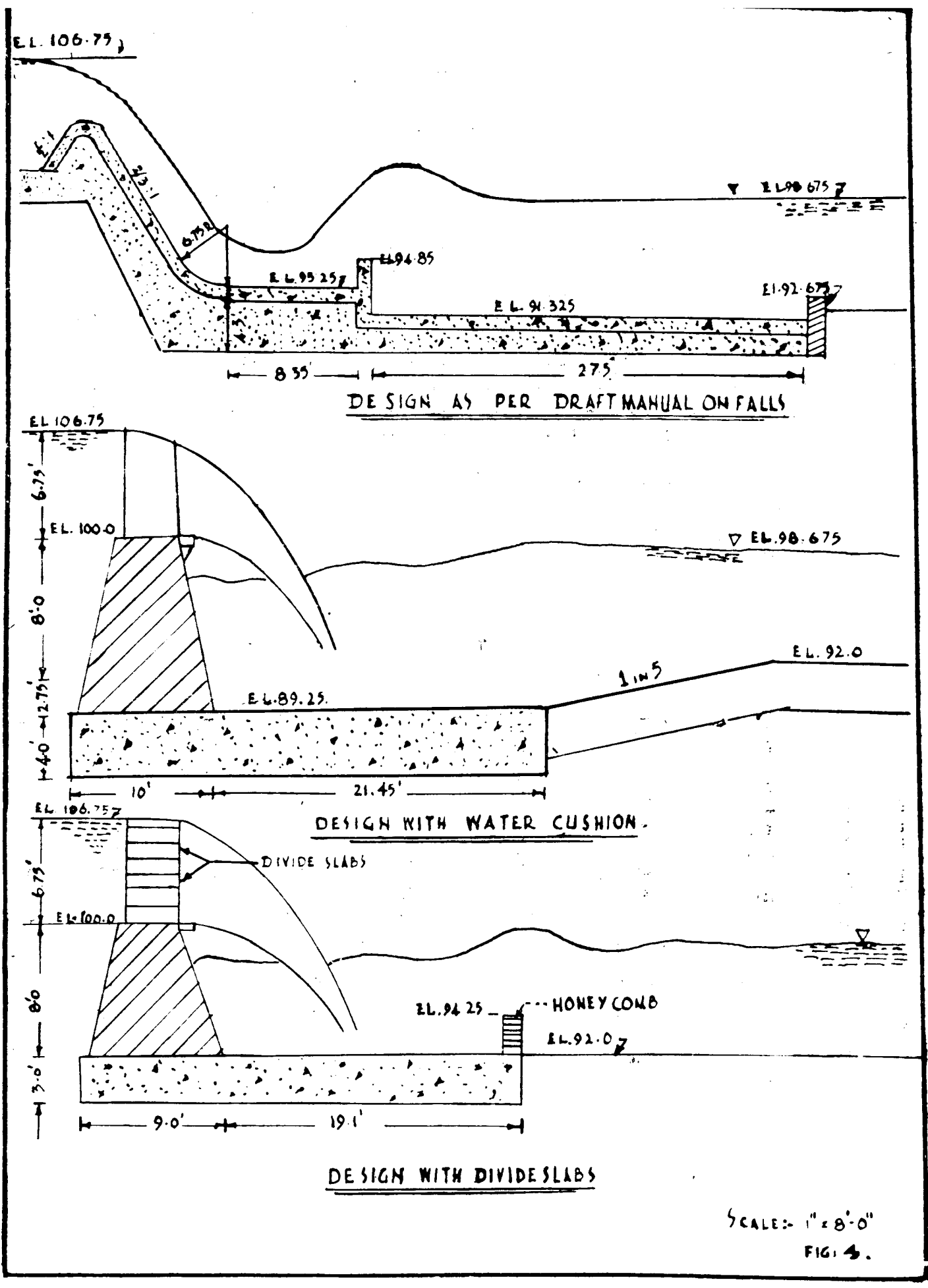
Upstream depth = 4.5'

Downstream depth = 4'

Slope = 1/4600

TABLE 2.
Observations for the finalised design.

Serial number.	Height of drop.	Upstream depth downstream depth.	Particulars.			Observations.			
			Apron length.	Height of honey comb.	Discharge.	End scour.	Maximum scour.	Distance for maximum scour.	Slope.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	4'	3.375'	$2.94\sqrt{hd}$ = 10.1	$0.33d_1$	100 cusecs	0.047'	0.45'	3'	0.133
2	5'	3.375'	$2.94\sqrt{hd}$ = 11.9	$0.33d_1$	100 do.	0.012'	0.28'	3'	0.094
3	6'	3.375'	$2.84\sqrt{hd}$ = 12.6	$0.33d_1$	100 do.	0.008'	0.4'	6.75'	0.06
4	6'	4.5'	$2.84\sqrt{hd}$ = 15.6	$0.33d_1$	200 do.	0.009'	1.65'	12'	0.136
5	8'	4.5'	$2.84\sqrt{hd}$ = 16.8	$0.33d_1$	200 do.	0.005'	0.462'	11'	0.042
6	8'	5.625'	$2.84\sqrt{hd}$ = 18.7	$0.33d_1$	300 do.	0.007'	0.7'	9'	0.078
7	10'	5.625'	$2.84\sqrt{hd}$ = 21.0	$0.33d_1$	300 do.	0.002'	0.45'	16'	0.027
8	8'	6.75'	$2.84\sqrt{hd}$ = 20.6	$0.33d_1$	530 do.	0.067'	1.265'	21'	0.057
9	10'	6.75'	$2.84\sqrt{hd}$ = 23.9	$0.33d_1$	530 do.	0.02'	0.65'	10.5'	0.062

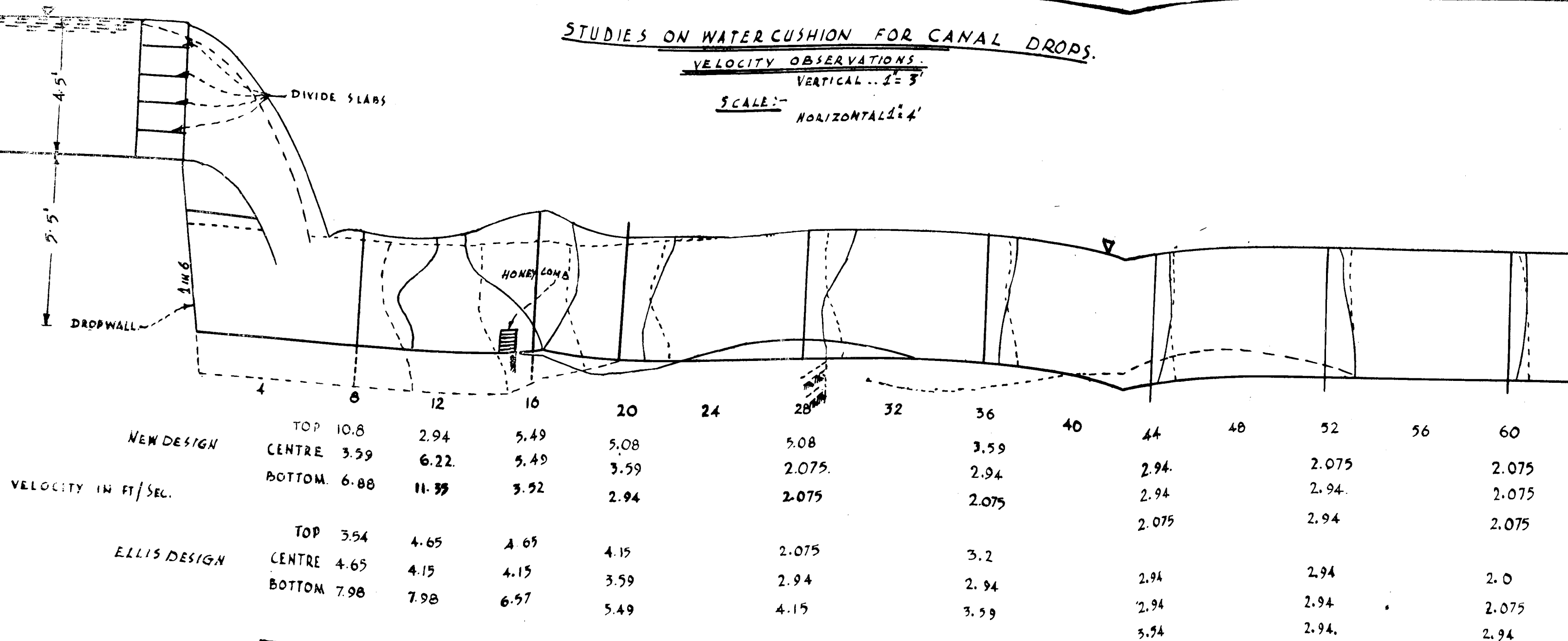


STUDIES ON WATER CUSHION FOR CANAL DROPS.

VELOCITY OBSERVATIONS.

VERTICAL .. 1" = 3'

SCALE:-
HORIZONTAL 1" = 4'



PARTICULARS	
UP STREAM DEPTH	4.5'
DOWN STREAM DEPTH	4.0'
DISCHARGE	125 CFS.
DROP	6'
WATER CUSHION	14.4 x 1.4

ENERGY DISSIPATION ARRANGEMENTS		OBSERVATIONS	DESIGN WITH	
IN NEW DESIGN	IN ELLIS DESIGN		DIVIDE SLAB	WATER CUSHION
DESIGN SLABS AT 1' 0" C/C	CISTERN LENGTH = $2\sqrt{hd+d_1}$	END SCOUR MAX. SCOUR DIS. FOR MAX. SCOUR.	0.195'	0.48'
HONEY COMB HL. $\frac{1}{3}d_1$	DEPTH = $\frac{1}{2}d_1\sqrt{h-d_1}$		0.535'	1.31'
APRON LENGTH $2.9\sqrt{hd}$			3.0'	5.0'

NEW
ELLIS

THE NEW IRRIGATION ERA.
Journal of the Irrigation Branch of the Madras P.W.D.

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