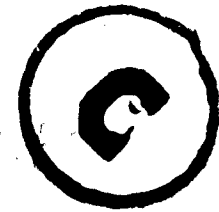


The New Institution for

1. 9

1. 9

No



JL
D3 m 211, NS2
N65-9.3.
126662

THE NEW IRRIGATION ERA

Published by the Irrigation Branch of the Madras Public Works Department, Madras-5, South India.

(For Departmental Circulation only)

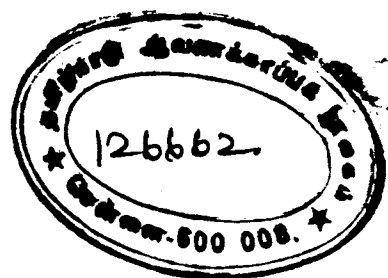
“ Treat a friend as a person who may some day become your enemy ;
An enemy as a person who may some day become your friend ”

—GEORGE BERNARD SHAW.

Vol. IX	MARCH 1965	No. 3
---------	------------	-------

	PAGE
DOES THE ENGINEERING PROFESSION FIT ONE TO CONDUCT PUBLIC AFFAIRS ?—	
By Sri Louis Armand, Non-Member, French Academy, Paris	3
DESIGN OF ANICUTS AND COMPONENTS—	
By Sri J. Walter, B.A., B.E., M. Irrn. Engg. (Colorado), M.I.E. (Ind.), Superintending Engineer (Designs), P.W.D., Madras-5	5
PUMPING SCHEMES IN VENNAR SUB-DELTA—	
By Sri P. T. D. Santhosham, B. E., Executive Engineer, P.W.D., Vennar Division, Thanjavur; and Sri T. G. Gopalakrishnan, B.Sc., B.E., Assistant Engineer, P.W.D., Drainages, Nagapattinam	20
FLAP VALVE FOR UPLIFT PRESSURE RELIEF IN CANALS—	
By Sri S. Shanmugham, B.E., M.Sc., Executive Engineer, P.W.D., Irrigation Research Division, Poondi	30
OVERFLOW BREAST WALL—	
By Sri J. Walter, B.A., B.E., M. Irrn. Engg. (Colorado), M.I.E. (Ind). Superintending Engineer (Designs), P.W.D., Madras-5	36
NEWS ROUND UP—	41

The Editor or Madras Public Works Department accepts no responsibility for the views expressed by the authors.



DOES THE ENGINEERING PROFESSION FIT ONE TO CONDUCT PUBLIC AFFAIRS?

BY

* LOUIS ARMAND.

It is desirable that the engineer should play his part in public affairs and he is highly qualified to do so. Why?

Firstly, because he is a realist and accepts the fact that everything cannot be perfect. The scientist stands by rigid principles and that is as it should be. Most of those who formulate mathematical laws think they are final, that like present day Prometheuses they have snatched them from God, just as the jurists write their laws and, fearfully enough, used to carve them on marble thinking they would never change. Now the engineer must agree to deal only with imperfect material. Surely a person used to assembling imperfect materials is well qualified to assemble men to form a society. That is an exceedingly smooth transition that should make the engineer an architect in society as well as in technology.

These things were in my mind during a recent visit to industry. Everyone marvels at those materials called uranium, zirconium and germanium a few traces of which make a transistor. Imagine my surprise to hear a research engineer saying, 'The germanium we are working on ought to be a crystal, an absolutely perfect single crystal!' I had thought that it was almost perfect. And he went on, 'There are still too many breaks in this crystal, millions per sq. cm. but without them we should not be able to weld it with indium'. That is rather like human society. We come together with our weaknesses and our inadequacies as well as our strong points.

The engineer proceeds by successive stages. According to old French tradition, one assumes

such titles as 'engineer of arts and crafts', 'engineer of arts and manufacturers'. You can see that being so greatly imbued with culture, our forefathers knew what they were talking about. Let us continue to think that we serve not a science but an art and that brings us closer to the art of politics. For I am very much afraid of people who would reduce the profession of engineering to a question of geometry. I pity them because they are bound to encounter setbacks throughout their careers. Nothing is geometrical. In the 19th century, in a surge of rationalism that cost us very dear, they believed that the Creator was a geometer. Since Le Verrier had found the planet Neptune by calculation, they thought they possessed the key to it all. But it was not so. Great things are not geometrical. You have to simply to turn it all into geometry. The planets do not describe circles around the sun—and this disappointed centuries of scientists—and not cones, because at the same time the sun is itself moving and it is horrible curve that does not even fit on to a cylinder and its approximate plan in our terms is an ellipse. We make abstractions the Creator never made. The world is not geometrical, it is statistical. To-day all our science is statistical.

Let us accept the World as it is. For an engineer to succeed he must not be a victim of that geometrical spirit. But do you realize who is in danger of being a victim to it? The jurists. Technocracy has been denounced. But the real technocrats have never been engineers. Some bear the title of engineer but are among those who side tracked and are only engineers in name. You can only be an engineer when you accomplish something, for then you are definitely freed of the excess of geometry and

* Non-member, French Academy, Paris.

can help to prevent the jurists aiming, from an inferiority complex, to be more exact.

What is more, jurists live on the past ; whereas an engineer lives in the future and that is one of the reasons why he should make a good citizen and take his place in the State. One has to be clear about whether one looks to the future or to the past. And between ourselves, the Creator has made us in such a way that we draw far more of a distinction between front and back than between right and left . . . If you think of it, that is the crux of the matter. Looking ahead, above all else, is the art of governing. The ancients did not believe in evolution. The notion of progress is recent. What progress can there be if there is no possibility of developing technology along lines most favourable to mankind?

Young people are going to live through the great age when the technicians become actually aware of their responsibilities. Hitherto, technical methods were limited. Every fruit was left to ripen. One part was negative and that was war; one part was positive and that was peace. But everything was left to grow. Technical possibilities have increased to such a degree to-day—exponentially that we shall soon be unable to develop all possible forms of progress. A choice will have to be made and what is favourable for culture only developed. That brings us to the core of the problem of the connection between the engineer and culture. A tremendous responsibility is involved there. Who can be a better judge in this case than the engineer who has had a technical training?

There we have two reasons why the engineer should make a good citizen if he wishes. A third reason is that engineer organizes. He assembles or joins atoms, molecules, electric wires, machines and men. Why should human society not avail itself of the engineer's natural bent for assembling? The doctrinaires would separate planners and non-planners. But one word covers the whole thing, whether in Moscow or in New York, there is need to 'organize'

factories, human societies, town halls, Governments, the U.N.O. There is need for organization everywhere, even in the Ecumenical Council.

Finally, engineers are able, or at any rate are obliged, to work as a team. Naturally they do quarrel amongst themselves, but the main thing is that their quarrels are not determined by political frontiers. They may want to run their neighbours down, they may be obstinate, they may be insincere, but in technical discussion and quite apart from national frontiers.

Since it is stimulating to quarrel, let us quarrel about technical problems and forget the frontiers. We shall do a good turn to those who are unfortunately, obliged to take them into account. However that may be, the engineer knows he cannot work on his own and in the team he makes no distinction between nationalities. This is excellent preparation for building the great teams of the future, whether they be political or technical.

There are many reasons therefore why the engineer should make a good citizen. Yet it is a fact that he rarely takes part in public affairs and those who are not engineers may well accuse him of rejecting the responsibility. One of the explanations is that the engineer is satisfied with his profession and feels no urge to find relief elsewhere. In relation to public life, he is an escapist . . .

To be good citizens, the main thing is to understand and to be understood. The European society of the future should be formed of engineers, on the one hand willing to be informed on civic problems and on the other hand, of men trained in the social sciences, made responsible for establishing and running human societies, but who would agree to acquire some notion of technology in order to respect it. The day our two sources of culture find a common language, each having agreed to learn something of the other's language, we should hope to see Europe setting the tone for the civilization of the future.

DESIGN OF ANICUTS AND COMPONENTS

BY

* SRI J. WALTER, B.A., B.E., M. IRRN. Eng. (Colorado), M.I.E. (Ind).

An anicut is a structure built across a stream to cause its diversion or storage for proper utility of the flowing water for irrigation purposes. Both hydraulic analysis and structural analysis are necessary for the proper design of anicuts and its components. Hydraulic analysis deals with the discharge, velocity, water slopes, water levels etc., and the general arrangement of the work, its main components, dimensions, etc., will be based on the result of this analysis. The structural design covers the calculations and design of all the structural parts and the preparation of the final drawings.

Design Aspects

Anicut Proper—

Anicut consists mainly of a body wall protected by an upstream and downstream apron. The structural design of body wall of various types is dealt with elaborately in the Irrigation Manual by ELLIS and hence is not included in this article.

Foundation—

Foundation for weirs can be classified into pervious one and impervious one. Hard rock alone can be treated as impermeable. Other foundations including clay and soft disintegrated rock are not wholly impervious and structures on them are usually designed as on permeable foundation to be conservative.

Uplift—

The structure on permeable foundation has to be designed for the uplift pressure and exit gradient. Water seeps through the pores of the structure itself. Uplift is the upward pressure exerted over the portion of the structure above the seepage path by the seeping water through the foundations or through the structure. This

causes a reduction in the effective unit weight of the structure. The law of flow of water through permeable soils was first enunciated by Darcy who, as a result of experiments found that the velocity of flow varied directly as the head and inversely as the length of the path of flow. Darcy's law is of the same form as the law for the movement of water in capillaries enunciated by Poiseville.

Hydraulic Gradient Theory—

The Hydraulic Gradient Theory for Weir Design was first originated between Sri John Otteley and Thomas Higham and was developed as a result of experiments by Col. Clibborn. According to this, the true measure of security of a weir in a permeable bed is to provide sufficient length of passage of seeping water.

Bligh's Creep Theory—

According to this theory, the creep is more important and destructive to undermine a weir than the percolation through the soil below it. Bligh assumed the percolation water to creep along the contact of the base profile of the weir with the sub-soil losing head on route, proportional to the length of its travel. This loss of head per unit length as creep co-efficient and assigned a safe value of C for different classes of soil. Bligh has recommended that full uplift be allowed for the masonry below low water level by taking the effective specific gravity of masonry as (P—I), and for the portion above the low water level by taking the effective specific gravity as p per C ft.

If the foundation is very pervious and the crest of the weir is much above the maximum tail water level, this allowance will be insufficient and partial floatation for the full profile, and full floatation for the portion of the profile below tail

water has to be allowed. In addition the weir must be protected against failure by undermining due to percolation and scour. This can be effected by the provision of solid aprons both upstream and downstream of the body wall. The reader may refer to Ellis Irrigation Manual for the design for the length of solid and loose aprons by using empirical relationship.

Electrical analogy method—

Pavlosky approached the problem of water through sub-soils of hydraulic structures from the analogy of flow of electricity through a conductor. He achieved success in solving a number of problems but as the laboratory results could not be shown to agree with the field results, this method remained more or less of academic interest. But now recently, the development in this branch of engineering has led to a final solution of the problem of design of weirs on permeable foundations.

Lane's Weighted Creep Theory—

This theory is an improvement of Bligh's Theory corrected for vertical cut-offs and sloping faces. Lane adopted a weight of three for vertical creep and one for horizontal creep in determining the length of seepage. It was an empirical one and it lacked the background for a rational basis for design.

Exit Gradient Theory—

Dr. Terzaghi stated and proved by laboratory experiments that failures occurred by undermining, if the hydraulic gradient at exit was in excess of what he called the "floatation gradient". This was the same as the critical gradient enunciated by Khosla.

Khosla's Theory—

According to this theory, the flow of water through the sub-soil is in stream lines and follows Darcy's Law. The structure has to be designed both for the uplift pressure and safe exit gradient.

Dr. A. N. Khosla has demonstrated that simple weir profiles are susceptible of complete mathematical treatment. But usually the weir section

happens to be a combination of the elementary forms and mathematical treatment in these cases is quite involved. To overcome this difficulty, Khosla proposed the method of independent variables which goes under his name.

In this method, the complex weir is split up into its elementary standard forms (for detailed information the reader may refer "The Designs of Weirs on Permeable foundations"—C.B.I.P. Publication No. 12) the entire length of floor with any one of the pile lines making up one such form. Each elementary form thus split up is treated as independent of the others. The pressure at the Key points are then read off from the curve in Plate VII-6 of the publication. These key points are junction points of the floor and the pile line of that particular elementary form, the bottom point of that pile line and the bottom corner in the case of depressed floor. These pressure values obtained are then corrected for (i) mutual interference of piles; (ii) floor thickness; and (iii) the slope of the floor.

Mutual Interference of piles—

The mutual interference of piles is local. It mainly extends to a distance equal to the depth of pile beyond which it gradually falls off and is negligible at twice that distance. In most cases where the spacing is close and the depth of piles great, the residual effect may be considerable. The following equation gives the influence for all cases:—

$$C = 19 \sqrt{\frac{D}{b_1}} \times \frac{D+d}{b}$$

Where C = correction to be applied as percentage head;

b_1 = distance between two piles;

D = depth of pile whose influence has to be determined on neighbouring piles of depth 'd';

d = depth of pile on which the effect of pile of depth 'D' is to be determined; and

b = total flow length.

The correction 'C' is additive for points in the rear or back water and subtractive for points in the forward direction of flow.

Sloping Floor—

The theoretical investigation on sloping floors by Malhotra is plotted in plate XI of the publication. A suitable percentage correction has to be applied for the sloping glacis, the correction being plus for the downward slope and minus for the upward slope following the direction of flow.

Exit Gradient—

Exit gradient is the hydraulic gradient at the exit point $\frac{dh}{ds}$ where h residual head at the point on the stream lines, and S distance measured along the stream line from a reference point in it. To safeguard against undermining the exit gradient must not be allowed to exceed a certain limit. It is normal practice to give a factor of safety of 4 to 7 depending on the materials used. Factors of safety recommended by Sri A. N. Khosla are as follows:—

Fine sand	..	..	..	6 to 7
Coarse sand	..	..	..	5 to 6
Shingle	..	..	..	4 to 5

Plate VII-5 in the publication gives the exit gradient.

Since a flat foundation without piles or legs will not be able to withstand the upward force due to the exit gradient even for very small heads of flow, a downstream pile or a masonry leg must be provided at the exit end.

Energy Dissipation—

The fall of water over a weir can be negotiated as a single vertical drop or a series of such drops or by a sloping glacis. In the flow after such a drop is allowed over the river bed, there is bound to be bed scour which may eventually undermine the works. Hence it is essential that the excess energy due to fall should be destroyed before the flow reaches the raw river bed. In the case of a vertical drop, no special arrangement is necessary except by making the apron thick enough to withstand the force of the falling water. For this purpose, Bligh recommends a minimum thickness of 3 to 4 feet water cushion, sloping glacis, etc., may be used in

other cases for dissipation of the residual energy. But in some cases where tail water build up cannot be relied upon, it is usual to adopt baffles, basing, baffle blocks, etc., which help to destroy the energy by impact and hydraulic jump.

For dams, a thumb rule had been evolved which gives the depth of the impact type of stilling basin to be provided in rear of spillways.

$$D = \frac{H}{15} + \frac{h}{5}$$

Where D = depth of stilling basin;

H = height of fall up to basin floor level from the crest of spillway; and

h = head of over flow.

The curve $\frac{D}{D_1}$ Vs Kinematic flow factor was found to be a straight line where D—depth of incoming jet and Kinematic flow factor is square of Froude number, i.e. $\frac{V}{\sqrt{gD_1}}$. It was found that up to a value of F^2 equal to 530, it is a straight line and even after that it is a straight line of different slope. It is not known how far this is applicable for lower Froude numbers (less than 5).

Some of the typical energy dissipating-cum-scour preventing arrangements that have been adopted in the various hydraulic structures executed in this State (based on the recommendation by I.R.S. Poondi) have been collected. It is seen that various types of arrangements like sunk type and baffle type of stilling basins, glacis, friction blocks or a combination of these have been adopted. Now there is no definite design economical design of energy dissipating and scour preventing arrangements in anicut. The present practice is to select a certain type of arrangement according to the whims and fancies of the designer and to confirm it in the model studies.

It is seen that the relationship $D = \frac{H}{15} + \frac{h}{5}$ holds good for the anicuts also. However, further detailed studies are required for standardising economical design of energy dissipating and scour preventing arrangements for various heights of anicuts and different heads of flow of water over the anicut. Similarly studies are

required for the energy dissipating arrangements for the flow through under sluices.

Scour—

The scour is the progressive erosion and removal of material of the river bed by the flowing water. For any discharge per foot run, Kennedy's silt formula gives

$$V_0 = 0.84 d^{0.64} \text{ for Punjab conditions.}$$

$$q = V_0 d = 0.84 d^{1.64}$$

$$\text{or } d = 1.11 q^{0.61} \text{ where } q = \text{discharge per foot run.}$$

The critical depth is the depth 'd' to which scour will take place in normal flow.

Lacey gives scour depth 'R' as—

$$R = 0.7305 \left[\frac{Q}{3.8 f} \right]^{\frac{1}{3}}$$

Where Q = maximum flood discharge; and
 f = silt factor.

This is modified by multiplying by a suitable factor to take into account the disturbances due to bends, etc.

Retrogression —

Retrogression is progressive lowering of the river bed level downstream of an anicut or weir and is different from scour. It is mainly due to the ponding action of weirs. When the water enters the reservoir formed upstream of the works, its velocity is checked and its silt load is deposited. By the time the flow reaches the weir, it is comparatively clear and its silt content is far less than it is capable of carrying. When this flow has passed the weir, it tends to pick up silt to replace that which has been deposited above, lowering the bed for some miles below the work. Retrogression will thus cease once the upstream pond is filled with silt and thereafter the downstream river bed level is gradually restored.

This phenomenon has got an important bearing on the design of downstream pile line and downstream apron in the case of weirs. With the lowering of the river bed, the tail water level goes down and the hydraulic jump tends

to form below the apron and entails serious erosion there. Hence in the cases where the apron is the only energy dissipator, the apron level should be lower than the original bed level. But in cases where some other devices also are utilized it may be sufficient if the loose apron is depressed. The weir and its aprons upstream and downstream get subjected to a greater head at a later date due to retrogression. Hence provision for retrogression must be made even in the first instance.

Scouring Sluices—

The main appurtenant work in an anicut is scouring sluice. The primary functions of a scouring sluice are (1) to keep open a deep channel in front of the head sluice; and (2) to keep the storage above anicut from silting up. It is usual to locate the scouring sluices in the flanks and within the zone of the head sluice taking off from the anicut. If channels take off from both the flanks, two sets of scouring sluices will be necessary.

The capacity of the scouring sluices is fixed on the following consideration:—

(a) The scouring sluices should be capable of passing the low flood discharge.

(b) At high floods, the scouring sluices should be capable of passing about 10 to 15 % of maximum flood discharge.

(c) The sill should be kept at a low level as can conveniently be arranged and generally at about the deep bed level of the river.

(d) Studies on the data collected from existing anicuts indicate that the linear waterway provided for scouring sluices should be more or less equal to the linear waterway provided for head sluice.

Further the scouring sluices should be kept as wide as possible within the limits of economic feasibility to effect quick and proper disposal of silt. It is usual practice to provide a breast wall in the front from M.W.L. elevation to the top of sluice shutters.

Design Criteria—

The conditions under which the scouring sluices operate are different from

those of weirs. Water flows with high velocity due to high head and as such the apron will have to be longer.

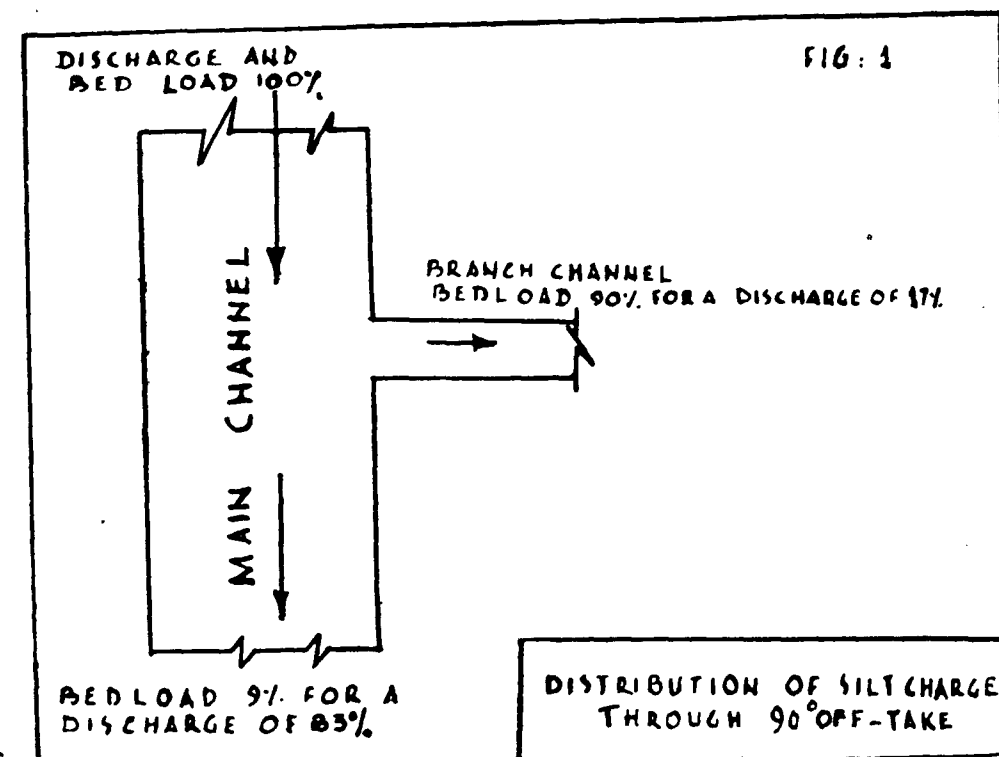
The formula for the aprons of scouring sluices has been furnished in Ellis Engineering Manual.

Off Takes:—Water is taken to the irrigation fields by channels taking off from the storage formed above the anicut. A well designed channel is one which draws only its share of the silt load, which maintains its regime and which calls for the least maintenance. To achieve all these proper location and orientation of the off-takes is necessary.

increases, the bed load drawn into the branch increases considerably (*vide* figure 1).

No conclusive data on the effect of obliquity of off-takes is available. But experience confirmed by model experiments shows that a right angular orientation does not help materially in the exclusion of silt and an obtuse angled off-take draws comparatively less silt.

It is best to site the off-take on the concave side for the simple reason that the high velocity top layers which carry comparatively less silt charge, flow outwards before they are deflected towards the bed and the slow moving bottom layers are

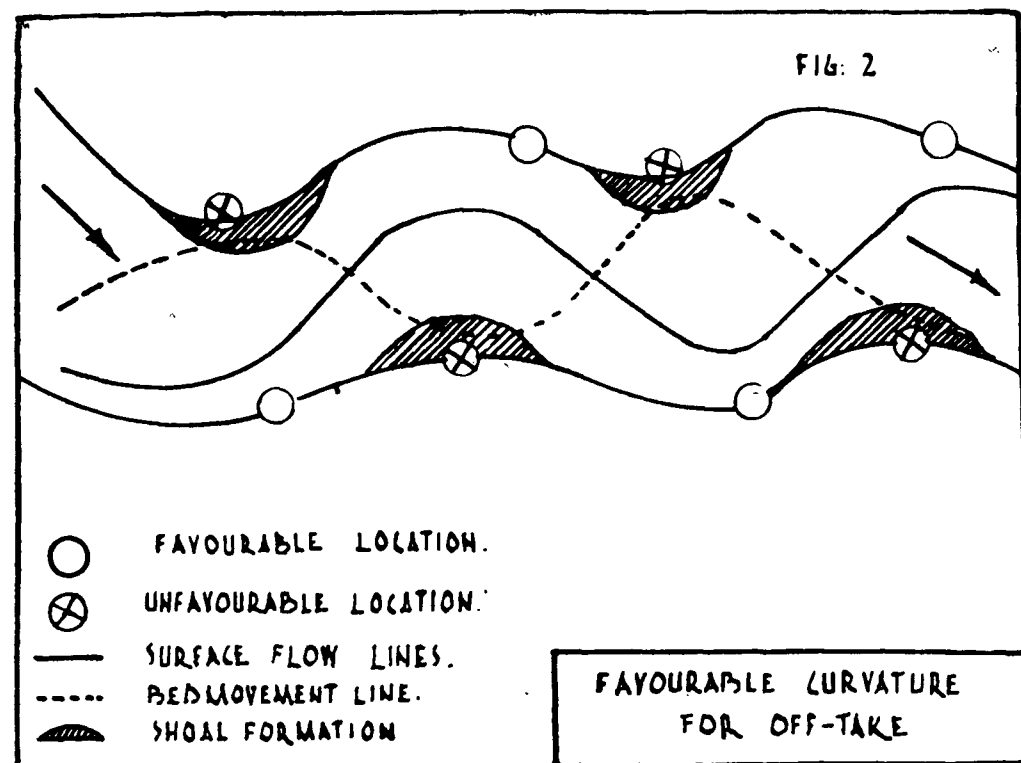


Location.—The off-take may be from a straight channel or from a curve. When it is from a straight channel and small compared to the parent stream, it is enough to raise the sill of the off-take 2' or 3' above the bed of the stream to obviate a major portion of the silt entering the channel. It is shown by experiments that the branch draws about 90 per cent of the bed load when it draws only 17 per cent of the total discharge. As the discharge in the branch

deflected across the bed of the stream towards the inner convex side. The effect of curvature is so dominant that off-takes sited at the outer concave side draw little silt while those sited on the inner side draw excessive silt (*vide* figure 2). When channels take off from both the banks, it is best to site the channels with the high discharge on the concave side and lower discharge on the convex side.

Hence position and orientation of off-take is by far the most important consideration. The whole success of design, so far as exclusion is considered, depends on this factor. The harm done due to faulty location can rarely be off-set by other methods of exclusion. The results of model studies conducted in Poondi Research station reveal that the optimum linear water-way for the scouring sluices in anicuts will be about one to one and a half times the water-way of the head sluices and that it is best to locate the scouring sluice perpendicular to the anicut

inasmuch as it reduces the canal depth where the material deposits and increases the slope. Dredging the canal in such cases becomes a task which is difficult, uneconomical and recurring. Hence it is best to design the headworks in such a way as to prevent coarse excess material enter into a canal and the canal should take only the normal charge both in quantity and grade. Care is to be taken in the design of headworks in the case of new projects and in the case of existing structures, training works and artificial devices are necessary.



and head sluice parallel to the anicut from the point of view of silt exclusion from canal (*vide* figure 3.)

Design criteria:—A detailed design for Tolu-dur Regulator is given in Ellis Irrigation.

Exclusion of silt from off-take:—An ideal irrigation channel is generally sufficient to pass the designed discharge, and normal charges which the canal can carry downstream with a certain amount of latitude. Excess entry of coarse sand into a canal reduces the discharging capacity

Only an outline of the preventive measures, generally associated with the silt exclusion are given below:—

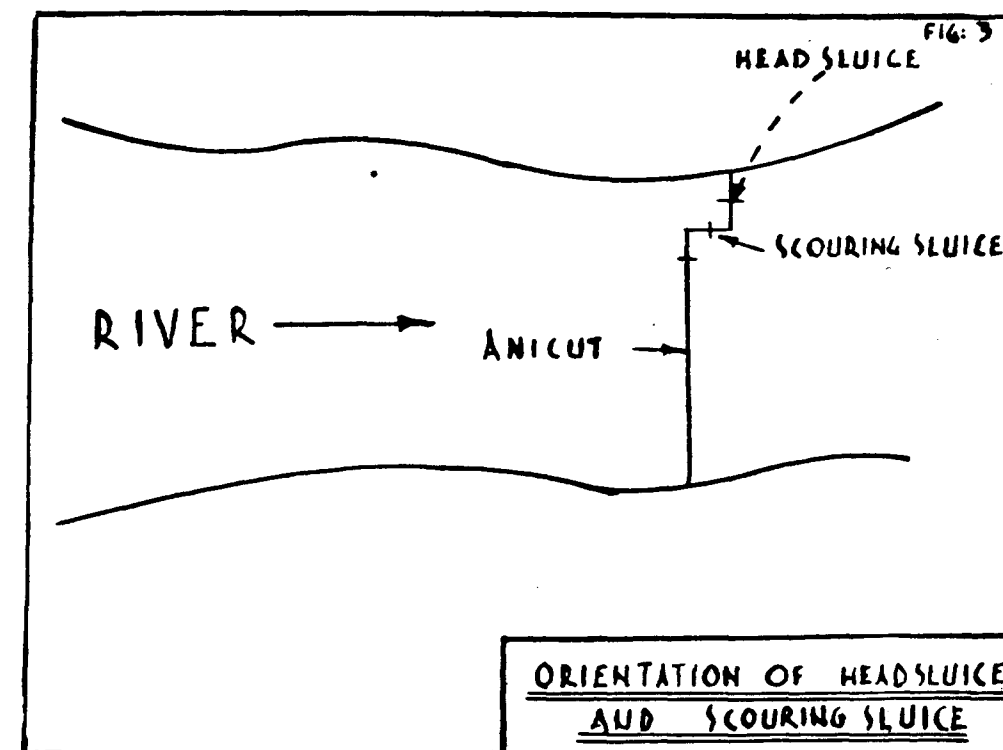
1. Fixing the crest level of the off-take at a higher level than the sill of the scouring sluice.
2. Giving suitable curvature to the flow.
3. Construction of divide wall upstream and at right angles to the anicut segregating the scouring sluice from anicut proper.
(A divide wall of 2/3 of head sluice length is generally adequate.)
4. Construction of silt excluders at the head.

Alignment of a weir has some modifying influence on the upstream curvature. This shall be given due consideration because a curvature which may seem satisfactory in pre-weir period may not prove to be so due to neutralizing of curvature on account of ponding and straightening of river after the construction of a weir.

If the natural curvature of a channel is insufficient or not well defined, it can be accentuated by constructing guide banks, upstream of the off-take. Sharp curvatures should be avoided as they are not only useless, but definitely harmful. The

3. Providing tunnel excluders in the pocket.

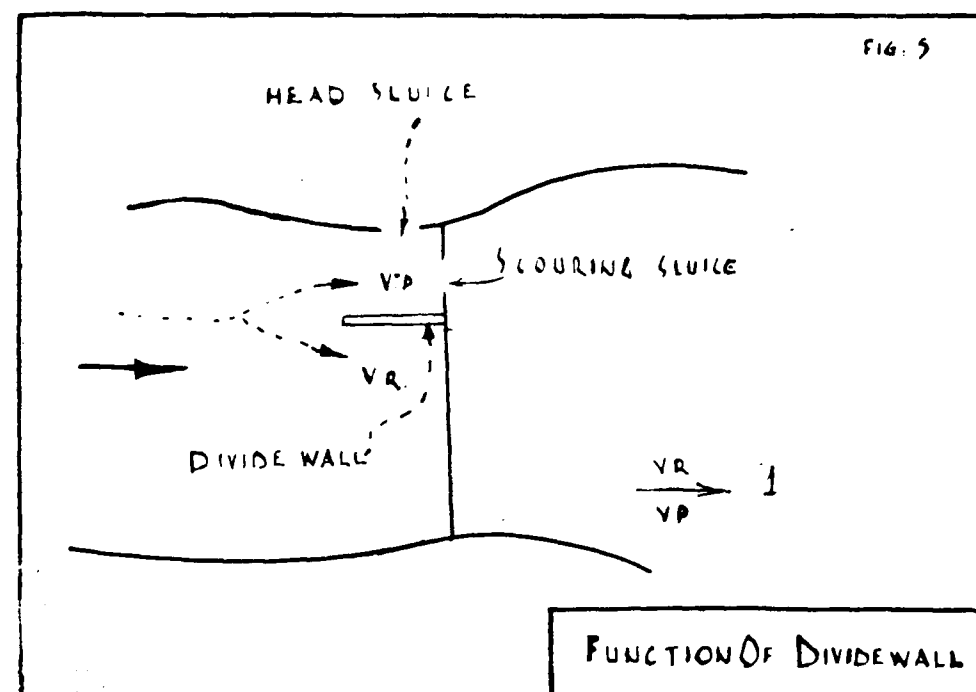
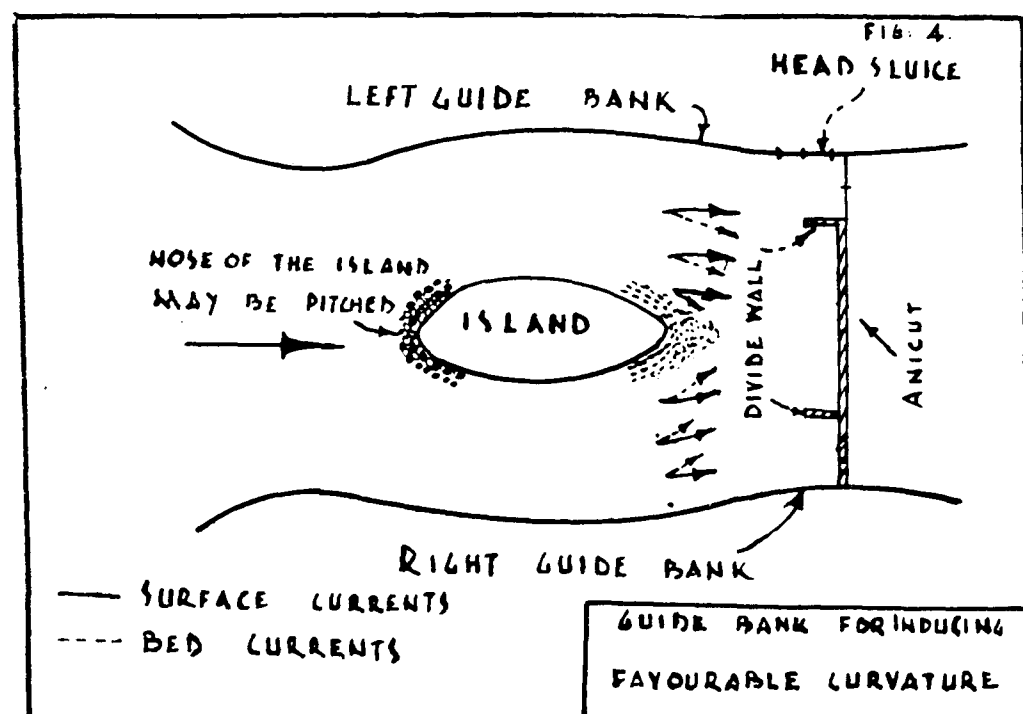
The nose of the divide wall shall be given a flat slope on the pocket side and a steep slope on the river side (*vide* figure 5). A pocket slightly converging towards the undersluices is to be preferred to a straight pocket as this helps scouring operation, but the convergence should not exceed 1 in 10. The effect of splay sharper than 1 in 10, on exclusion, is adverse and with increase in convergence, the area of slack flow is increased resulting in a heavy deposit of material in the pocket.



problem becomes more difficult when canal takes off from both banks upstream of the weir, as is generally the case. Conditions can be partly or totally improved by one of the following methods:—

1. If the river is sufficiently wide, a central island may be constructed, with concave flank embankments to impose favourable curvature on both flanks (*vide* figure 4).
2. Providing a curved vane upstream of an off-take to divert bed material to the mid-stream.

Regulation of Flow.—There are two types of regulation. In the first type called the still pond regulation, the scouring sluices remain closed while the head sluices are discharging. The quantity required for sending in the full supply discharge in the canal is drawn from the pocket. In doing so most of the silt gets deposited at the pond (pocket itself) to be subsequently flushed by operating the scouring sluices. The only limitation of this method in the necessity of the closure of the canal every now and then in order to flush the pocket. The second type is



called semi-open flow regulation where the scouring sluice shutters are partially opened when the head sluice is drawing its full supply. This method of regulation is not so good as the still pond regulation since the velocity of flow in the pocket is more than in the still pond regulation. During high floods when the discharge over the anicut is more, the semi-open flow regulation can be adopted with advantage.

Certain amount of water has to be wasted through the scouring sluices (at least 30 per cent of the total discharge) and the pocket has to be maintained without allowing the silt to be deposited.

Guide banks.—These are the artificial embankments placed in the direction of flow of a river, for properly guiding the flow past an anicut. They are constructed both upstream and downstream of the structure on one or both the flanks. Properly designed guide banks, apart from guiding the flow of water, protect the approach embankments from attack.

There are no hard and fast rules regarding the design of guide banks. Different types of guide banks that are usually provided are (1) parallel; (2) converging (away from the weir towards upstream); (3) diverging; (4) bottle neck; (5) concave; and (6) concavo-convex. Out of these only the parallel and diverging types are used. Between these two, diverging bank is found to be better for it helps to protect a much longer length of the marginal bunds.

Bank connections

The component structures which are required to securely connect the anicut with the banks are called the bank connections. These are designed to secure a proper bond between the anicut and the high banks on either side of it. It should also prevent dangerous percolation round the flanks of the work.

The two popular types of bank connections are:—

1. Abutments and Wings.
2. Core walls and wrap arounds.

The abutments and wings though more expensive are more satisfactory and gives better protection to the work and also guides the water smoothly in and out of the anicut. This type is economical in the long run and is also easily maintained.

The core wall type is cheaper and easier to construct. When the height of the earth to be retained is small, this type is also as effective as the wings and abutments type.

Design of Bank connections—

(a) *Wings and abutments type.*—The abutments, wings and returns are designed to withstand the earth pressure due to the banks applying the usual Rankine's or Coulomb's theory.

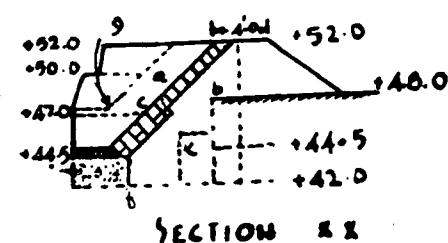
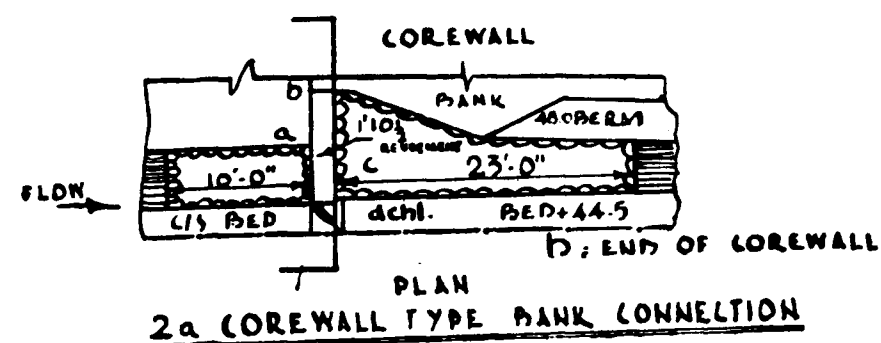
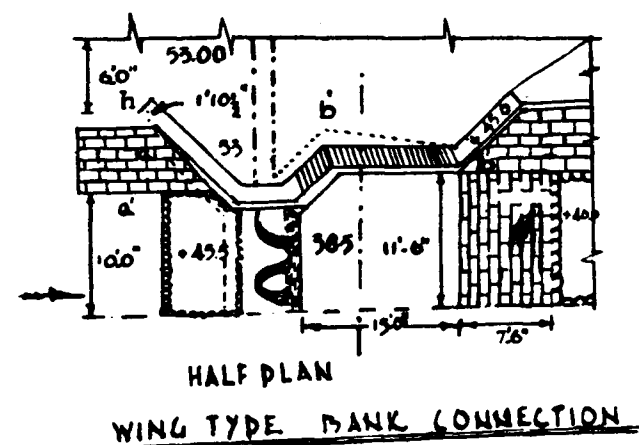
One more check that is required is for safe length of percolation round and under the wings.

This can be done similar to that for a canal drop (*vide* pages 298, 299 of Ellis Irrigation, Manual) which is reproduced below. The drop is 6 feet and taking the value of C as 5 the percolation length should not be less than 30 feet (*vide* figure 6).

One percolation line is that under the foundation of the splayed returns from 'a' to 'b' on plan a distance of 27 feet. In addition, the vertical lengths of earth through which the percolation must pass under the stepped up wing gives about 8 feet more. The shortest line of percolation might be at a' b' b under the wing or ahh around the end of it. The length of all the three lines are about 35 feet and hence the design is safe.

(b) *Design of core walls type of bank connection.*—As regards the percolation gradient through the soil past the core wall a design for core wall at a drop is reproduced from p. 301, Ellis Irrigation Manual.

Round the end of the wall length abc plus the thickness of the end of the wall is the length of the line of percolation and is just over 12 feet giving a gradient of 1 in 4.



2b COREWALL TYPE BANK CONNECTION

Under the foundation of the wall, the line of percolation runs from 'g' on the upper side to 'd' on the lower side. This distance scaled from the section is 8' plus the width of the foundation ($4\frac{1}{2}'$) and is also just over 12' giving a gradient 1 to 4.

Generally in the case of weirs wing type of bank connection is the most satisfactory as giving best protection to the work and forming best guide to the water on the approach and run off and containing the earth slopes in the most

effective way. They are also cheaper to maintain.

Conclusion

This note is not to be considered in any way exhaustive. Only a few useful information regarding the anicut, components, silt exclusion, etc., are given to serve as a guide for field engineers. Detailed information on any of the subjects discussed can be had from the references appended at the end and a typical calculation for the Design of Anicut is enclosed as an appendix.

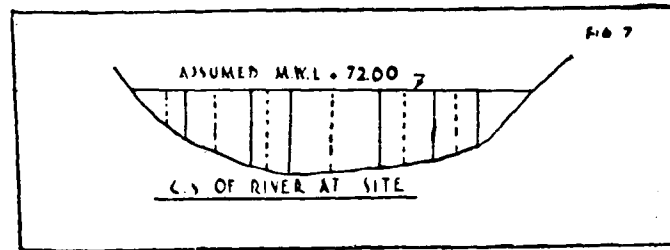
References.

1. Handbook of Applied Hydraulics by C. V. Davies.
2. Engineering for Dams, Volume 1-Creager, Justin and Hinds.
3. Steady flow in open channels—C. J. Posey and S. M. Woodward.
4. Design of Weirs on permeable foundations by A. N. Khosla, M. Tayler and N. K. Bose C.B.I.P. Publication No. 12.
5. Irrigation and Hydraulic design, Volumes I and II by Serge Leliavsky.
6. Manual on River Behaviour control and training C.B.I.P. Publication, 1960.
7. Report of the Fourth Meeting of the International Association of Hydraulic Research, Bombay, India, 1951.
8. Irrigation Research Station Report No. 15/62 on Basic studies on optimum linear waterway for scouring sluices.
9. College of Engineering Manual, Irrigation by Ellis.

APPENDIX.

TYPICAL CALCULATION FOR THE DESIGN OF ANICUT.

Calculation of Flood discharge and M. F. L.



Flood discharge values are calculated as per Ryve's formula, from the actual cross section of the river at site and from recorded gauge readings at any masonry bridge nearest to the site of the anicut. The highest of the above values is taken as the design flood. Discharge calculated from Ryve's formula—180,000 cusecs. Assumed M. F. L. is plus 72.00.

The area of cross section of the river is calculated to be 13,173.8 sq. ft. at M. F. L.

Wetted Perimeter = 1,444 ft.

$$R = \frac{13173.8}{1444} = 9.122$$

$$V = \frac{1.486}{n} (R)^{\frac{2}{3}} (S)^{\frac{1}{2}}$$

A value of 0.03 is taken for n . The slope of the river 'S' was determined from 5 cross sections taken upstream of the site $S = 1/837.5$.

Hence flood discharge = $8.96 \times 13173.8 = 118,000$ cusecs.

The design flood is taken as 180,000 cusecs.

Calculation of M. F. L.:

For the design discharge of 180,000 cusecs. M.F.L. has to be determined. $\left(\frac{Q^2}{R}\right)$ is assumed approximately constant.

$$\text{i.e. } \frac{118,000}{(9.122)^{\frac{2}{3}}} = \frac{180,000}{R^{\frac{2}{3}}}$$

$$R = 11.75$$

Assumed M. F. L. for 118,000 cusecs = 72.00

Average depth of flow = $72.00 - 61.5 = 10.5$ ft.

$$\text{Average depth for 180,000 cusecs} = \frac{10.5}{9.122} \times 11.75 = 13.53 \text{ ft.}$$

M. F. L. for 180,000 cusecs = 61.5 plus $13.53 = 75.03$ say 75.00.

$$\text{Velocity is proportional to } R^{\frac{2}{3}}$$

$$\frac{8.96}{(9.122)^{\frac{2}{3}}} = \frac{V}{(11.75)^{\frac{2}{3}}}$$

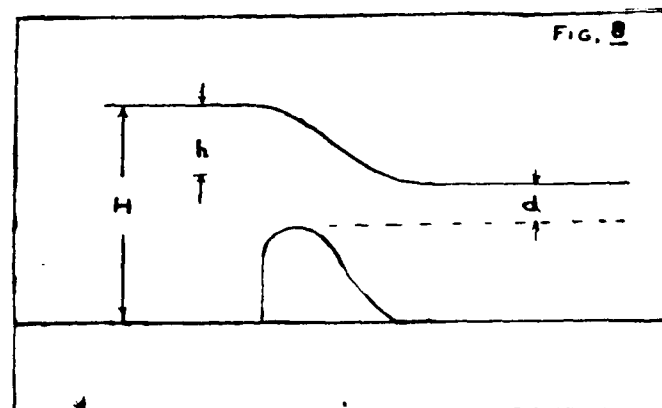
$$V = 10.6 \text{ ft./sec.}$$

$$\text{Head due to velocity of approach} = h_a = \frac{v^2}{2g} = 1.745$$

2. Length of Anicut:

The length of the anicut is generally fixed based on available waterway at site. After determining the afflux, the length has to be modified so as to pass the maximum flood discharge with an afflux not exceeding 3 feet. In the case under consideration a length of 1,300 ft. is provided.

The crest level of the anicut has to be fixed based on consideration of ayacut commandability, permissible afflux (3 ft.) submergence, etc. The crest of the anicut is fixed as 65.00 in this case.



3. Afflux and front M. F. L.:

The afflux can be calculated from the formula

$$Q = L \left[3.1 \left\{ \left(h + h_a \right)^{\frac{3}{2}} - h_a^{\frac{3}{2}} \right\} + 0.8d + 2g(h + h_a)^{\frac{1}{2}} \right]$$

where Q = discharge in cusecs

L = length of anicut

h = afflux

d = depth of water above crest

h_a = head due to velocity of approach

Putting $X = (h + h_a)^{\frac{1}{2}}$ and solving the cubical equation in X , the value of h can be determined.

$$h = 1.5 \text{ feet (say)}$$

$$\text{Front M. F. L.} = 75.0 + 1.5 = 76.50$$

The front M. F. L. can also be calculated by using the formula $Q = C L H^{\frac{3}{2}}$. Where C is the submerged coefficient (value to be taken considering the percentage reduction of coefficient due to submergence as indicated in Chief Engineer (Irrigation)'s circular Memo. No. 8074/59A-2, dated 24th September 1960) and H is the head of overflow above the crest of the anicut. The afflux must be restricted to 3 feet. If the calculated afflux is more than 3 feet, the length of the anicut will have to be increased suitably.

4. Design of weir proper:

The weir may be designed as a solid body wall with protective apron or as a raft type.

4.1. Floor elevation:

The present practice is to have the floor at about the deepest bed level of the river. But provision for retrogression must be made in fixing the level of the apron in relation to the riverbed (vide page 8 in the article). Alternatively provision has to be made in the energy dissipation arrangements taking into account lowering of tail water due to retrogression. As a result of retrogression in bed levels, while the low water levels have been found to drop from 4 to 7 feet, the maximum flood levels have

not been known to have dropped by more than 1 to 1.5 feet.

The floor level for the case under study is fixed as 60.00 (i.e.) below bed level.

4.2. Section of weir:

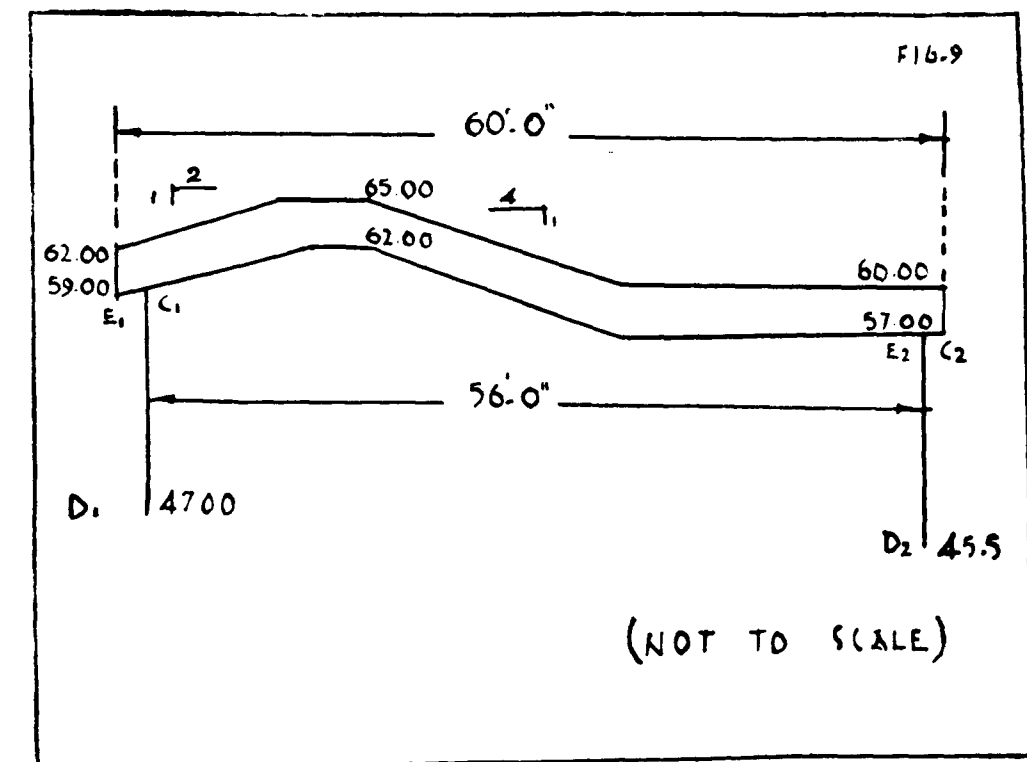
Proper section of the weir for the body wall may be selected based on Chief Engineer (Irrigation)'s circular Memo. No. 8074/59 A-1, dated 20th January 1960 and No. 8074/59-A-2, dated 24th September 1960. The stability of the body wall of the weir has to be checked for the following conditions of maximum stress (1) with head water at the crest of the weir and tail water at maximum flood level or at the level H_1 above the base of the weir whichever is lower if the weir is founded on impervious soil and (2) tail water either at crest level or at its maximum flood level whichever is lower for weirs on pervious soils.

4.3. Design of cut off:

Scour depth for any water way other than the minimum stable water way is given by $R = 0.9 \left(\frac{q}{f} \right)^{\frac{1}{3}}$ where q is the discharge per foot run and f is the silt factor.

Value of 'f' for coarse sand is 1.5

$R = 25.17$ feet. (say calculated value). The depths are to be reckoned from the respective flood levels. Hence the upstream cut off will reach a depth of $76.50 - 25.17 = 51.33$ and the down stream one to $75.00 - 25.17 = 49.83$. To allow for retrogression the cut offs are taken to E.L. 47.00 and 45.50 respectively.



(NOT TO SCALE)

4. Width and thickness of floor :

4.4.1. The head across the weir will be the maximum afflux or difference in level between crest and downstream bed. In this case the head is 5 ft. (say). According to Bligh a value of 12 is taken for c the width of the floor should be $5 \times 12 = 60$ ft. This length has to be checked for safe exit gradient. Assume a thickness of 3 ft.

4.4.2. Uplift pressure :

The section in figure 9 is proposed.

(Vide C. B. I. Publication No. 12)

length of floor = 60'

$d = 62.0 - 47.0 = 15.0$ ft.

$$\alpha = \frac{b}{d} = \frac{60}{15} = 4.0$$

$$\phi D_1 = 71 \text{ per cent}$$

$$\phi C_1 = 57 \text{ per cent}$$

$$\phi D_1 - \phi C_1 = 14 \text{ per cent}$$

$$\phi C_1, \text{ correction for depth} = \frac{62.00 - 59.00 \times 14}{15} = 2.80$$

Correction for mutual interference of pile :

Vide page 6 of the article.

Interference of down stream pile ;

$$d = 59.0 - 47.0 = 12.0$$

$$D = 59.0 - 45.5 = 13.5$$

$$C = + 19 \sqrt{\frac{13.5}{56}} \times \frac{12 + 13.5}{60} = 3.97 \text{ per cent}$$

$$\therefore \phi C_1 \text{ corrected} = 57 + 2.80 + 3.97 = 63.77 \text{ per cent.}$$

Correction for sloping Floor ;

Correction for a slope of 2 to 1 is 6.5 per cent.

Upstream length of slope = 6'. Spacing of piles = 56 feet.

$$\text{Correction at ends of slope} = \frac{6}{56} \times 6.5 = 0.70 \text{ per cent}$$

$$\text{Corrected potential at } C_1 = 63.77 - 0.7 = 63.07 \text{ per cent.}$$

Similarly the potentials at various points will have to be determined.

$$\text{Uplift pressure at } C_1 = \frac{63.07 \times 5}{100} = 3.1535 \text{ ft. head of water}$$

Full uplift is allowed and hence the specific gravity

$$\text{of concrete under water becomes } \frac{144 - 62.5}{62.5} = 1.3$$

$$\therefore \text{Floor thickness required at } C_1 = \frac{3.1535}{1.3} = 2.43 \text{ ft.}$$

Hence assumed thickness of 3 ft. is safe. Similarly floor thickness has to be determined at all salient points.

4.4.3. Exit Gradient (Vide page 7 of the article)

$$\frac{b}{d} = \frac{60}{14.5} = 4.138$$

$$\tau = \frac{1 + \sqrt{1 + (4.138)^2}}{2} = 2.6285$$

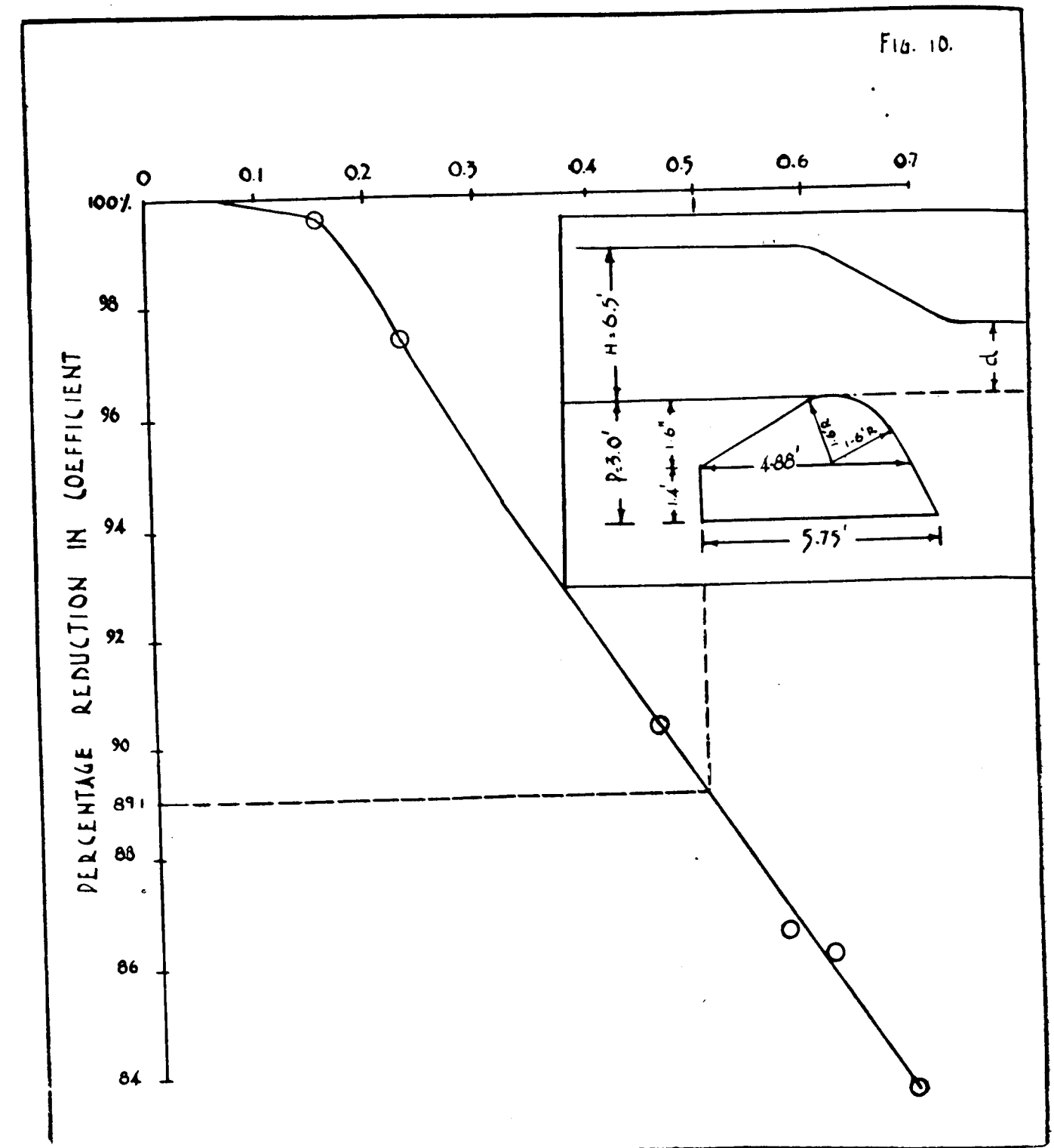
$$G_E = \frac{H}{d} \times \frac{1}{\pi \sqrt{\tau}} = \frac{5}{14.5} \times \frac{1}{\pi \sqrt{2.6285}} = 1 \text{ in } 14.75$$

Critical value of G_E for coarse sand is 1. Allowing a factor of safety of 6, the safe value is 1 in 6. Since the value obtained is less, it is safe. Otherwise the length of floor, depth of downstream cut off will have to be increased and tried.

Similarly the impervious floor of head sluice has to be designed for uplift and exit gradient. The loose apron will be designed using the Bligh's formula.

The Woods are Lovely, Dark and Deep,
But I have Promises, To Keep,
And Miles to go Before I Sleep
And Miles to go Before I Sleep.

RICHARD FROST.



PUMPING SCHEMES IN VENNAR SUB-DELTA

BY

* SRI P. T. DEVYSINGH SANTHOSHAM, B.E.

AND

** SRI T. G. GOPALAKRISHNAN, B.SC., B.E.

The rivers, Channels and drains in the Vennar Sub-delta can be divided into two viz., (a) East sea board rivers, (b) South sea board rivers. All the East sea board rivers except Kaduvayar, Odambogiyar and Vettar in fall into Vedaranyam Canal. This is a navigation canal running in a southern direction from Nagapattinam Port to Vedaranyam Salt pans. The South sea board rivers in their turn infall into a large lagoon called Mulliapallam creek and Sethalaikodu creek. So most of the rivers in Vennar Sub-delta have got only an indirect access to sea.

Most of the rivers in this Sub-delta function as purely irrigation sources in their head reaches where they take off from their parent rivers. In the middle reaches, as well as towards the tail end in some cases, they perform the dual function of irrigation source and drainage carrier. Some of the rivers become purely drainage carriers in their tailend reaches. The rivers, by being combined irrigation sources and drainage carrier, effect utmost economy in the usage of water. The drainages have not been allowed to go as waste, instead being utilised to irrigate lower down lands, thereby achieving higher efficiencies. Tail end regulators to head up and feed the ayacut before the rivers infall either into Vedaranyam Canal or into the lagoon, are the order of this system. Thus most of the lands inside the delta, have been brought under plough.

But there are still some lands left undeveloped inside the delta. These lands are situated in the high ridges between the rivers and drains of this Sub-delta. Also another higher portion is left

between Vedaranyam Canal and the sea. These portions remain undeveloped because they could not be commanded by any of the rivers or the drains.

Only Manavari cultivation of paddy and other dry crops like millets, and casurina and coconut thopes are undertaken in these tracks. Vegetable growing is also undertaken in some of these parts. But all these cultivations are dependent on Monsoon rains and so susceptible to the vagaries. Thus the people living in these parts are subjected to constant hardship and sufferings.

In this sub-delta, a saturation point has almost been reached in cultivable lands, which could normally be commanded by the usual age old gravitation flow. Owing to the increase in the population, the ryots of this area were in search for lands to be brought under plough and the source to command them. Naturally their attentions were diverted to the un-commandable high ridges, mentioned supra, and some of the enterprising ryots thought of making use of these tracts together with the drainage flows available nearby. They ventured out for small scale pumping schemes.

Three to five horse-power diesel pumpsets were used to pumpout water from the adjacent drains to irrigate about 5 acres of lands, here and there in these high ridges. Thus these areas were slowly being brought under cultivation. Since these lands are situated in high ridges, they are having good drainage facilities, which is a rarity in the tail end areas. This has resulted

in reaping bumper harvests which attracted more attention towards these high ridge tracts. Public opinion gathered momentum and they have started agitating for the Government sponsored major pumping schemes for these localities.

Accordingly number of pumping schemes were taken up for investigation. Seven numbers of such schemes have been executed and they are now serving their ayacuts. Two more

schemes are under execution. Still about a dozen schemes are under contemplation and they are under various stages of investigation and examination. The ayacut proposed to be brought under irrigation by the implementation of the above mentioned seven numbers of pumping schemes is 7,100 acres at a total estimated cost of Rs. 11.54 lakhs. The following statement furnishes the list of schemes already executed.

STATEMENT I.--DETAILS OF PUMPING SCHEMES.

Serial number and name of scheme.	Location.	Cost in Rs.	Ayacut in acres.		Proposed duty.	Details of pumps.			Source.	Head.
			Proposed.	Developed.		No.	H.P.	Hours of working.		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1 Mulliar Pumping Scheme	Thagattur village, T. T. Poondy taluk.	2.17 lakhs	1,539	1,214	40	4	35 each	18 to 20	Mulliar Rajan Adappar.	8
2 Deevambalpatnam	Deevambalpatnam, T. T. Poondy taluk.	0.42 ..	350	359	40	1	35 ..	Do.	..	15
3 Aymoor Pumping Scheme.	Aymoor village, T. T. Poondy taluk.	1.56 ..	717	675	80	3	30 ..	Do.	Mallanar	15
4 Ayakkarambalam	Ayakkarambalam village.	1.30 ..	750	581	80	1+2	20 h.p. and 15 h.p.	Do.	Mulliar	10
5 Valavanar Pumping Scheme.	Valavanar village, T. T. Poondy taluk.	2.58 ..	1,970	1,105	60	6+1	85 each and 1 No. 30 h.p.	Do.	Valavanar	14
6 Theenadar Pumping Scheme.	Theenadar village, T. T. Poondy taluk.	1.75 ..	1,040	650	80	3	20 h.p. each	Do.	Mulliar	12
7 Panani Mullar Pumping Scheme.	Panani village, T. T. Poondy taluk.	1.76 ..	740	729	80	3	30 h.p. each.	14	Adappar	12

As mentioned above, the surplus drainage flows are utilised for these pumping schemes. The flows were assessed wherever it was feasible. From a study of the above, it was inferred that it can be safely assumed, that about 10 per cent of the irrigation water is realised as drainage flow in the drainages. This assumption or thumb rule was adopted in framing up proposals where drainage flows could not be assessed. This has never proved to be wrong. In fact it is found that more supply, than what it could be assessed by adopting the 10 per cent thumb

rule is available. Hence whether the same can be revised to 20 per cent is under active consideration. Thus the availability of drainage is ascertained at the outset. Then the suitability of the drainage water for irrigation is examined.

Water samples have been collected from drainages at suitable places near and at site of pumping. These samples were collected at various depth, i.e., at the surface, at the centre and very near the bed. These samples were sent to

* Executive Engineer, Vennar Division, Thanjavur.
** Assistant Engineer, P.W.D., Drainages, Nagapattinam.

the Research Station at Aduthurai Farm and was ascertained. The following are some of the their suitability for irrigating the paddy crop results:—

STATEMENT No. 2.

Place of collection of water samples. (1)	P.H. Value. (2)	E.C. Value. (3) m.mhs. cm.	Remarks. (4)
Valavanar (Near Valavanar pumphouse) ..	7.1	0.35	Suitable for irrigation.
Mulliar tail end (Near Thennadar pump-house).	7.2	0.35	Do.
Overur Overkudy drain	7.2	0.20	Do.
Mulliar (Near Thagattur village)	7.1	0.20	Do.
Mulliar in Ayakkarambalam village limits.	7.4	0.20	Do.
Deevambalpatnam (South Rajan tail end reaches).	7.2	0.20	Do.

As a next step, the area to be brought under irrigation, among the high ridges, is to be demarcated. This is done with the help of natural boundaries, viz., the valleys and the drainage courses adjacent to them. After demarcating the area, the net area, excluding the village site, burial ground, etc., is ascertained. Then duties are fixed. Originally the schemes were designed for 40 duties. At present they are being designed for 70 and 80 duties. Though the designed duties are 40 to 80, the duty actually realised in them ranges between 100 to 130 and in some cases it is 140 also. The following table furnishes the duty proposed and the duty actually realised for some of the schemes executed already for the year 1963-1964.

STATEMENT No. 3.

Serial number and name of scheme. (1)	Ayacut developed. (2) ACRES.	Duty as proposed. (3)	Duty actually realised. (4)
1 Deevambalpatnam Scheme	346	40	135
2 Aymoor Scheme	670	80	109
3 Pamini Mullur Scheme	720	80	130
4 Valavanar Scheme	1,105	60	147
5 Thennadar Scheme	650	80	94

The pumps are designed as per the formula 9.089 AH/dt. which includes one-third the capacity of the standby also. Axial flow pumps and centrifugal pumps have so far been used in the schemes. The Axial flow pump is being driven by the vertically mounted motor with a long connecting shaft to the pump. The centrifugal pumps are either driven by direct coupled motors or by belt driven motors. In belt driven pumps, much of efficiency is lost in the belt drive. The performance of axial flow pumps appear to be better than the centrifugal ones. So far, mixed flow and submergible pumps have not been tried in any of the schemes. Submergible pump should exhibit better performance over the axial flow type, as the foot value, a source for drop in energy, is dispensed with. Further the suction head is practically eliminated. The only snag in these pumps is that they cannot be repaired locally.

The pumpsets are housed in pump houses of width 15 feet. A clear space of 10 feet is provided alround and between pumpsets. Sump

wells are provided in front of the pump houses. In case of axial flow pumps, wells are provided inside the pump house and these are connected to the sump provided in the front. Over these wells, the pumps are mounted and they draw supplies from them. In addition to the sump wells, collection ponds are provided for. The capacity will be usually 1 day's irrigation supplies and in some cases is 3 to 4 days. This provision safeguards the ayacut against any dearth of supply of intermittent flows in the drains. Gan-trays are provided inside the pump houses for moving the pumpsets inside the pump house, in case they have to be moved for repairs. An operator's room is provided for in the pump-house, where stores like spares, lubricant materials, etc., could be kept and the operator could stay while working the pumpsets.

either minimum or nil. This involved taking the walls of the pump house below ground level and they were provided wide to resist the earth and water pressures. Further the floor was made thick enough to resist the uplift pressure and the arch action of the floor is also taken into consideration. All these amounted to extra expenditure. The depressed floor also caused rise in temperature and increase in humidity. This made the working condition worse. Hence to reduce the expenditure and to improve the working condition it was examined whether the pumpsets could be erected at the ground level. This will amount to introduction of a suction head and the effect of such introduction over the over-all efficiency of the pumpset was investigated. Opinions were called for from some of the leading pump manufacturers and experts in the field. The extracts are given hereunder for reference:—

The pumps were placed originally, over depressed floors, to reduce the suction head to

STATEMENT No. 4.

Source of information. (1)	Extract. (2)
1 Voltas Limited, Madras	It is not necessary that the pump efficiency should increase only when the suction head increases.
2 The Madanagopal Copper Works, Sirkali.	Large output pumps whose total heads may not exceed 40 feet will almost shed about 15 per cent of their operative efficiency when the suction level is raised from 0 to 8 feet.
3 Sri K. T. Ganapathy, B.E. (Hons.) Executive Engineer, Research.	When the suction lift has varied from 2.6 feet to 11.3 feet the variation in efficiency had been small compared to variation when it has increased to 19.3 feet. This appears to confirm the statement by an Indian author which runs as follows: "Centrifugal pumps are designed for a total suction lift of 15 feet of liquid to be handled. So long as the suction lift is kept below this designed value, the capacity and the efficiency are unaffected. When once this limit is exceeded the efficiency drops abruptly.

Considering the above, it appears that efficiency still not be much impaired if a suction head, to a maximum limit of 8 feet excluding frictional losses, is introduced. This is confirmed by the observation that for a 11.3 feet of suction lift the variation in efficiency will be small (vide item No. 3 of the extracts shown above). Allowing a loss of head of 3 feet for friction, loss in pipe specials, valves, etc., the net suction head will be 8 feet. Thus in schemes under contemplation now, suction heads up to a maximum limit of 8 feet are being introduced.

Delivery cisterns are provided for the pump house wherein the pump empties the supplies. Various dissipation arrangements are provided in rear of the delivery cistern. Bell mouth entry into the leading channel is one such arrangement. Providing baffles is another such arrangement. Arrangements like inverted cell is provided in one of the pumping station. But the

best dissipation arrangement, so far tried is 'Honey comb' arrangement. Honey combs of size 4½ inches by 4½ inches in 1 foot 6 inches thick wall are provided for in the rear of the delivery cistern. This arrangement dissipates the energy to a maximum extent. Turbulence free flow is observed in the leading channel, in this type. Sketches of the above mentioned arrangements are furnished elsewhere.

The discharges passing through the channel are measurable with the help of drop weirs provided at the rear of dissipation arrangement. Log books for the pumpsets in which the hours of pumping energy consumed and quantity of water pumped are recorded daily. This furnishes the initial record for making duty observations. The pump driver maintains this log book and it is periodically checked. Extracts of the log book for every fortnight are being communicated to all officers.

The leading channel is provided without any berm in it. This divides into distributaries or branch channels. Sluices for every block of 50 acres are provided for at present and hence the delay in land acquisition and the consequent delay in the excavation of field bothie are avoided. Incidentally such a provision of sluices helps in reducing the disputes over the distribution of supply among the ryots. Irrigation conservancy subordinates and lascars are also employed. They look into the economical utili-

sation of irrigation supplies. They throttle up the sluices and push down supplies to the needy reach. They also attend to the maintenance of the channel.

The pumping was originally proposed to be started by middle of August by which time enough drainage flow is expected to set, in the drainage areas. But the date of starting the pumping operations are fixed based on the availability of supply. It is advanced either to the first week of August or to the last week of July, sometimes.

The entire capital cost of the scheme are borne by the Government. Due to Grow More Food value, the schemes are taken up under special Minor Irrigation Programme. However, the running expenses are to be borne by the ayacutdars to whose land supplies are effected. The running expenses include energy, other charges like lubrication, replacements, etc., and depreciation taking the life of pumpset to be 15,000 hours. A water cess of Rs. 5 is also levied. At initial stages when development is less, the running expenses are higher than that when the development is more. The ryots willingly shoulder these running expenses as these schemes brought prosperity to their home direct.

The following statement furnishes the working expenses arrived at various pumping schemes:—

STATEMENT No. 5.

Serial number and name of scheme.	Lift head.	Ayacut proposed.	Ayacut developed.	Year of development.	Rate per acre.	Rate per acre per foot lift.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
	FEET.	ACRES.	ACRES.		RS.	RS.
1 Mulliar	8	1,539	1,127.31	1961-62	13.43	1.68
2 Deevambalpatnam	15	350	337.58	1962-63	12.75	0.75
3 Aymoor	15	717	675	1962-63	13.64	0.91
4 Ayakkarambalam	10	750	615	1962-63	13.84	1.38
5 Pamini Mullar	12	740	682	1962-63	10.96	0.91
6 Valavanar	14	1,970	1,103	1963-64	25.17	1.80

The mean running expenses per acre per foot lift is about Rs. 1.25. For 3 schemes, which have developed nearly to the full extent, the running expenses are lower. However the rate of Rs. 1.25 can be taken as mean running expenses per foot lift per acre.

Prior to introduction of the scheme, the average yield per acre, when it was manavari is reported to be about 10 to 12 kalams. The

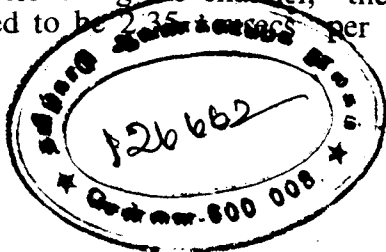
average yield of the lands after the introduction of the scheme has increased to 24 kalams. Thus there is an increased production of 12 to 14 kalams of paddy per acre. Seeing the increased production and the benefits accrued thereby, more and more people are coming forward clamouring for pumping schemes, wherever it is feasible. The following are few of the schemes that are under various stages of execution and examination:—

STATEMENT No. 6.

Serial number and scheme.	Location.	Source.	Cost in lakhs of rupees.	Ayacut in acres.	Stage.
(1)	(2)	(3)	(4)	(5)	(6)
1 Velanganni	Velanganni village, Nagapattinam taluk.	Maravanar	6.50	1,264	Under execution.
2 Villundamavadi	Tiruthuraipoondi village, Nagapattinam taluk.	Cauvery Kondanar.	4.42	1,611	Do.
3 Kallimodu	Kallimodu village, Tiruthuraipoondi taluk.	Pokku Voikkal	4.00	1,317	Under investigation.
4 Vilangadu	Vilangadu village, Tiruthuraipoondi taluk.	Ovarar Grain.	1.70	427	Do.
5 Thondiakkadu	Thondiakkadu village, Tiruthuraipoondi taluk.	Marakka Korayar	4.46	1,336	Under examination.
6 Oradiambalam	Oradiambalam village, Tiruthuraipoondi taluk.	Land drainages and Nallar when required.	7.00	640	Under investigation.
7 Kilathangiar	Thillavilagam, Tiruthuraipoondi taluk.	Kilathangiar	2.80	550	Under examination.
8 Manangondanar	Panchandikulam Kila- sethi and Adanur of Tiruthuraipoondi taluk.	Manangondanar	1.87	450	Do.
9 Kokkalady	Kokkalady village, Tiruthuraipoondi taluk.	Adappar	1.40	336	Do.
10 Vettaikaraniruppu	Vettaikaraniruppu vil- lage, Nagapattinam taluk.	Aladi channel or Harichandra Nadhi.	2.5	600	Do.
11 Valavanar Extension Scheme.	Voimedu village, Tiru- thuraipoondi taluk.	Valavanar	1.0	300	Do.
12 Aymoor Extension Scheme..	Aymoor village	New Mallianar	1.75	450	Do.
13 Pinnathur Tholi Scheme ..	Tholi village, Tiru- thuraipoondi taluk.	Tholi drain and Koraiyar whenever necessary.	3.0	300	Do.

In conclusion there are a few observations to be made in light of the experience gained from the schemes executed and functioning. At present a portion of precious irrigation supply is lost by percolation. The leading channels from pumping stations mostly run in embankments in the upper reaches. Hence the losses due to see-

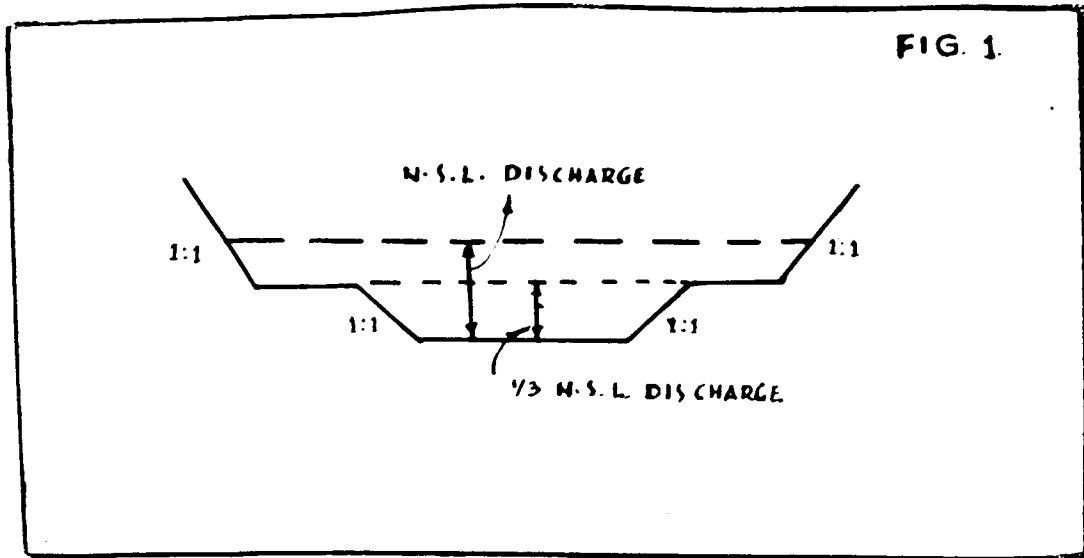
page are large. In Lower Bhavani Project canals, the transmission losses, which include losses due to evaporation, percolation, etc., were observed. It was found to be 6.5 cusecs per million square feet of wetted area for earthen channel. After lining the channel, the losses were observed to be 2.35 cusecs per million



square feet of wetted area, i.e., losses due to percolation have been eliminated. Thus percolation losses amount to two-thirds of total losses. Hence for the existing pumping schemes also lining is being undertaken.

The leading channels are designed in one section. Provision for berms are not being made. Hence the difference between the low supplies and N.S.L. is large. Such variation and fluctuation in water levels disturbs the setting

up of the channel and prevents it from attaining the regime. To avert this the channel can be designed to carry one-third N.S.L. supply in the section below the berm and the N.S.L. supply in the section including the berm. By designing so, the variation in depth of flow under the two conditions may not be large. This smaller variation in depth of flow will help the channel to attain its regime earlier than in channel section without the berm. The section proposed will be as shown below:—



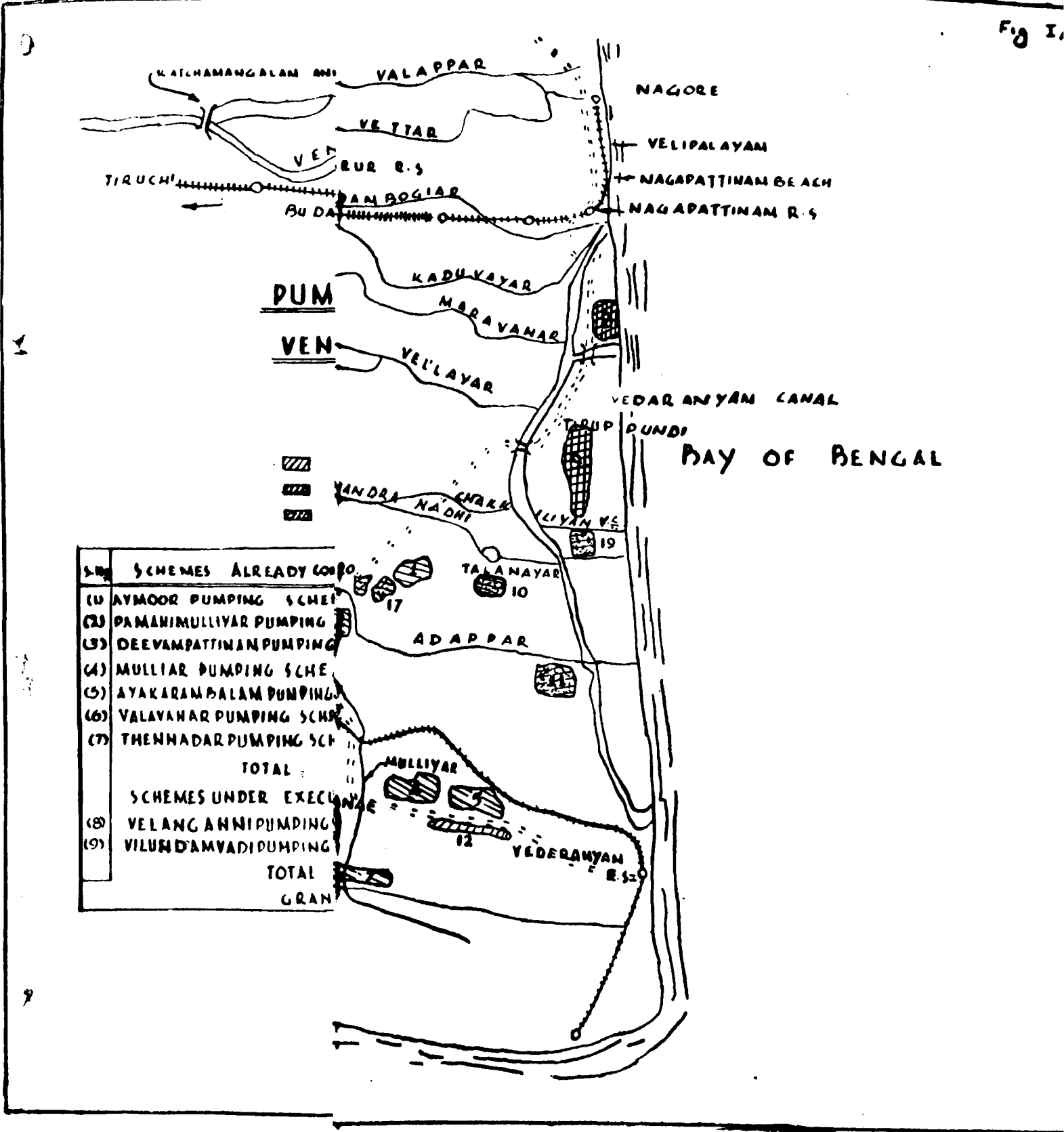
The following statement furnishes the average supplies for the season 1963-64 in various maximum supplies and average minimum schemes:—

STATEMENT No. 7.

Serial number and scheme.					Maximum supply.	Minimum supply.
(1)					(2)	(3)
					CUSECS.	CUSECS.
1	Deevambalpatnam	..	..	..	5	2
2	Thennadar	..	..	..	6	1.50
3	Valavanar ..	..	..	..	15	5
4	Pamini Mullur	..	..	..	6	2.50
5	Aymoor	..	..	..	4.5	2.00

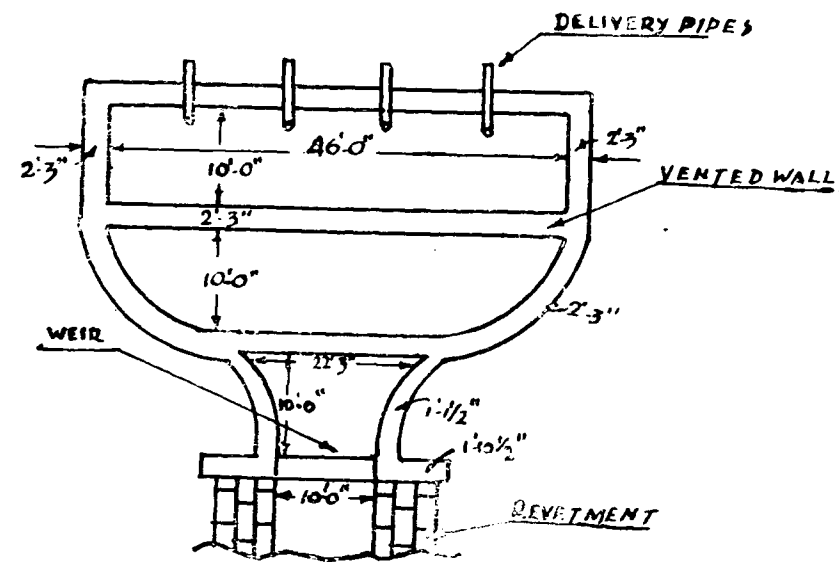
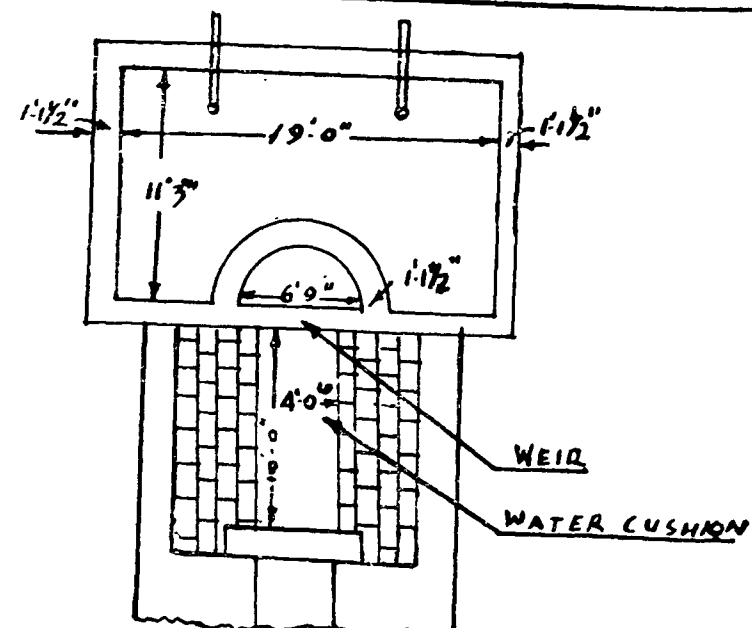
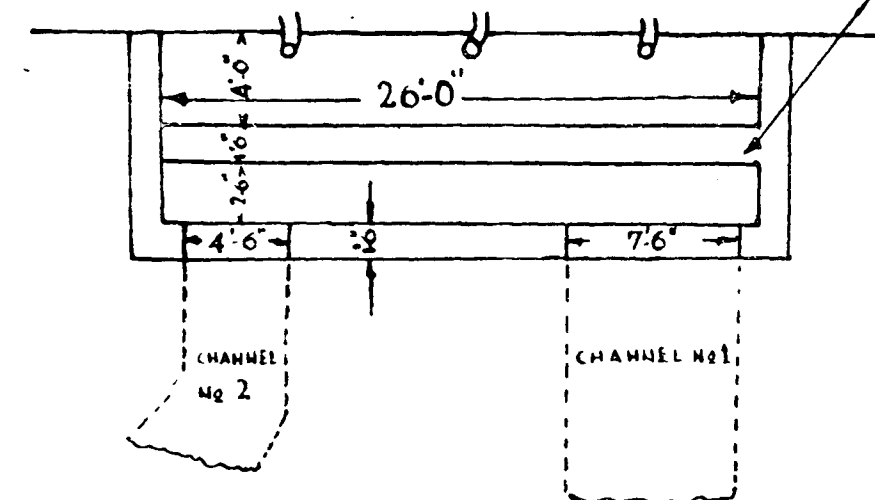
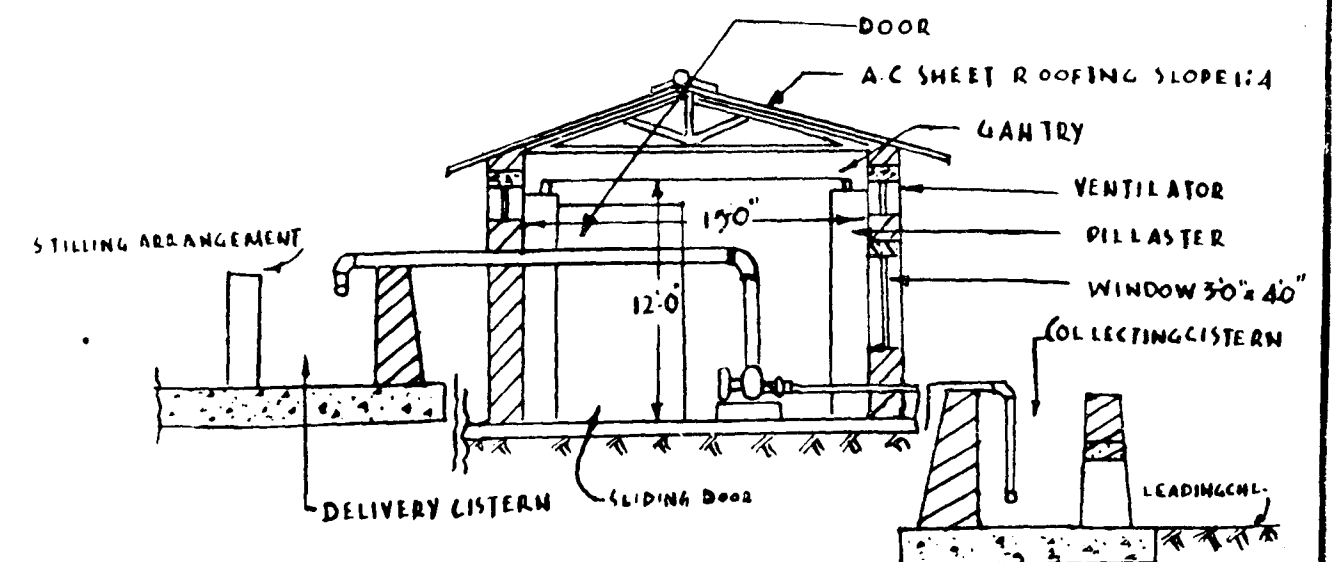
A study of the above shows that the maximum and minimum supplies are roughly in proportion 1:3. Hence the N.S.L. section can be designed for one-third of maximum discharge

and the section with berm can be designed for maximum discharge. This will prevent large variation in depths of flow between the low supply and N.S.L. supply conditions.



STILLING ARRANGEMENTS

Fig 2

MULLIAR PUMP HOUSESCALE 1 IN 200AYAKARAMBALAM
PUMP HOUSESCALE 1 IN 100HONEY COMBWORK (HONEY LOMB SIZE 4 1/2 x 4 1/2)THENNADAR PUMP HOUSESCALE 1 IN 100PUMP HOUSE
SCALE 1 IN 100

FLAP VALVE FOR UPLIFT PRESSURE RELIEF IN CANALS

BY

* SRI S. SHUNMUGAM, B.E., M.SC.

Introduction

Generally in lined canals the depth of lining is fixed arbitrarily. Unless the soil in the rear and underside of the lining is well drained the lining is likely to be subject to sizable uplift pressure built up by the seepage flow from canals or infiltration from precipitation. In such cases the lining is likely to be blown off. Hence it is desirable to have some device incorporated in the lining to maintain the uplift pressure within a certain limit. One such device, namely, "Flap valve" designed by the U.S.B.R. was being recommended to the field for adoption.

The flap valve is nothing but a device to turn the opening for pressure relief to a vertical direction so that a freely moving gate can act as a non-return valve. A protective hood in masonry is incorporated in the rear of the valve box to protect it from abrasive materials. By this arrangement the inertia due to weight of gate is also reduced considerably. Experiments were conducted in the Irrigation Research Station, Poondi, for studying its performance.

Model set-up

The flap valve, the details of which are shown in figure (1) was fabricated in mild steel the box being in 1/8 inch thick plate and the gate proper being in 1/4 inch thick plate. The strata of materials like sand, gravel, etc., were also prepared in a flume 21 feet long and 3 feet wide and 6 inches G.I. pipe with open joints and a right angled bend as shown in the figure was laid in position. Right angled bend was enclosed in a vertically placed R.C. flume pipe 2 feet diameter which was filled with Poondi red soil which is practically impervious. Over the soil

layers a 6 inches masonry floor was laid to preclude any possibility of blowing off. The inlet end of the 6 inches masonry floor was laid to preclude any possibility of blowing off. The inlet end of 6 inches pipe was closed with a perforated plate to simulate an open joint. An inlet tray of size 10 feet by 3 feet was constructed wherein desired uplift head up to a maximum of 3 feet above the centre line of the G.I. pipe could be maintained. Water from this tray was allowed to seep through a double layer dry brick wall. The downstream end of the flume was closed with an impervious wall 9 inches thick. Three piezometer tapings, two on the upstream and one on the downstream of the valve were fixed as shown in figure (1).

Experiments

The level in the tray was gradually raised from an initial level of that of the floor, the raising being 3 inches at a time. For each constant level in the tray the pressures at the piezometer points were observed. By extending the hydraulic gradient from the head water through piezometers (1) and (2) up to the valve point the pressure at the valve point could be read.

From the figure (1) it will be clear that the flap valve will open due to the momentum of seepage flow through the pipe and the opening and consequently the pressure at the point will depend on the weight of the valve and the position of the hinge. To study this aspect provision was made to add two more 1/4 inch thick plates to the gate and experiments were done for two hinge positions.

The pressures at the piezometers were reduced to unit head at tray and are shown plotted in

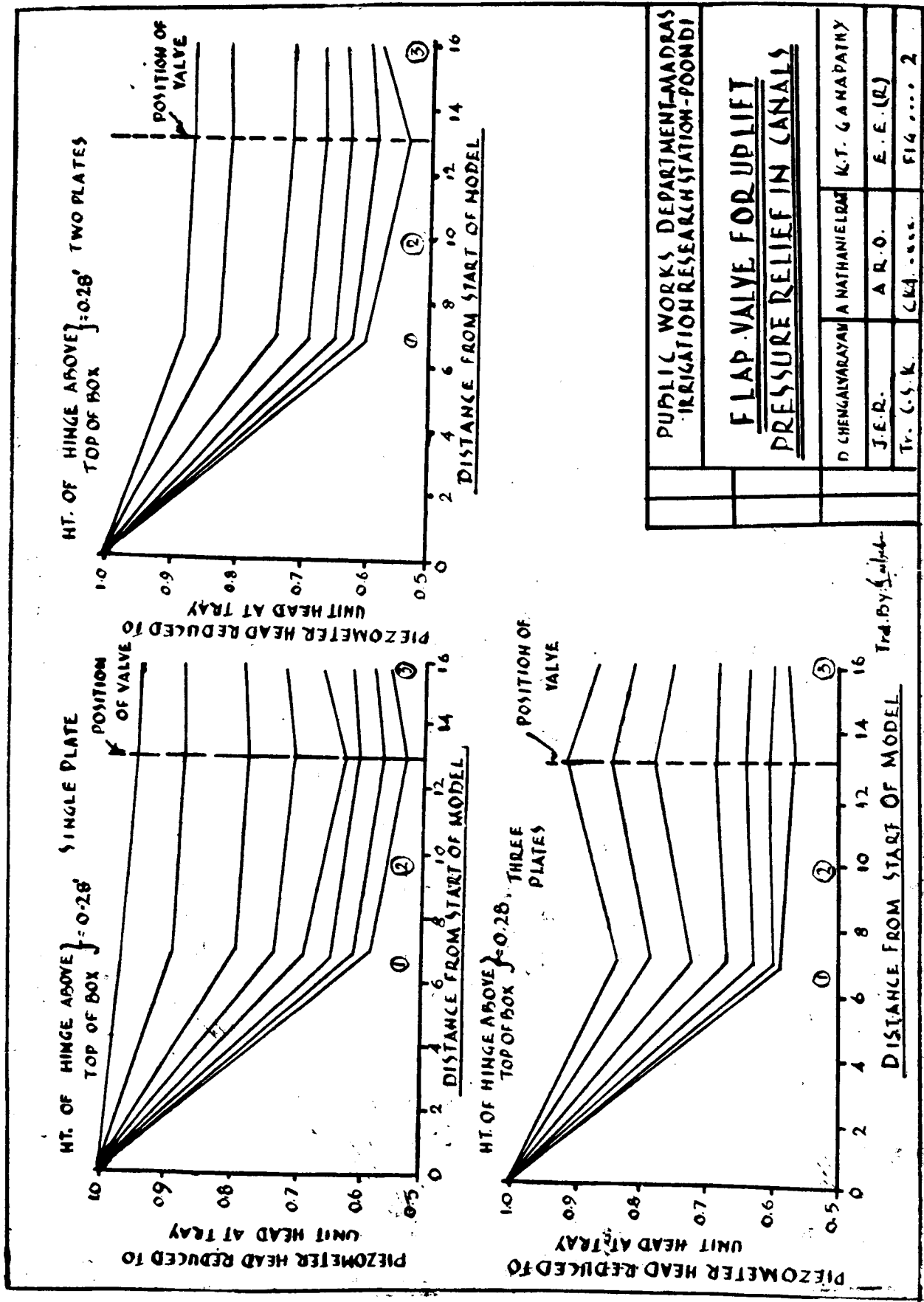
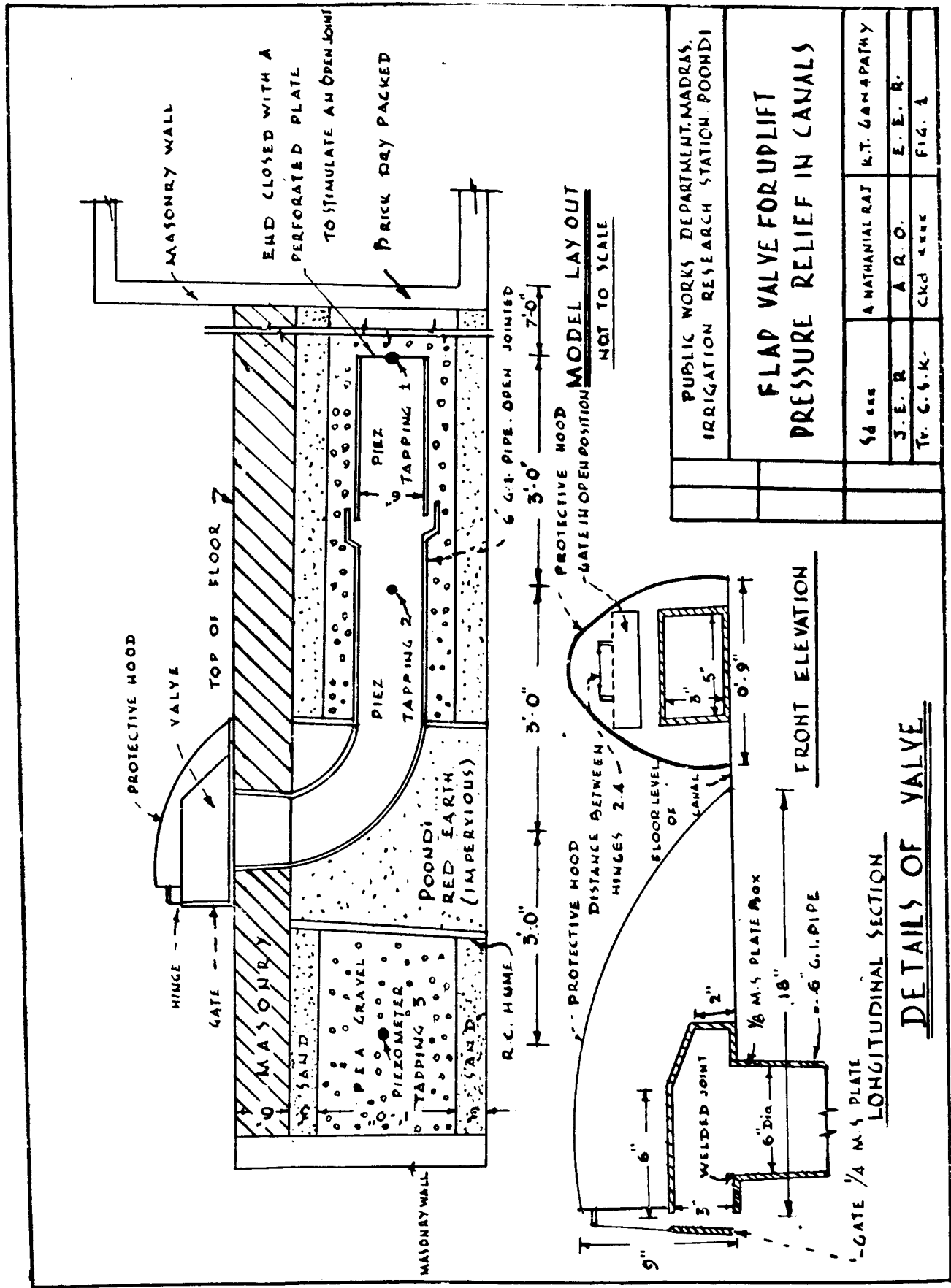
figures (2) and (3). From these figures the percentage pressure at valve could be read. Another plot of percentage pressure at valve vs. head of water at tray is shown in figure (4).

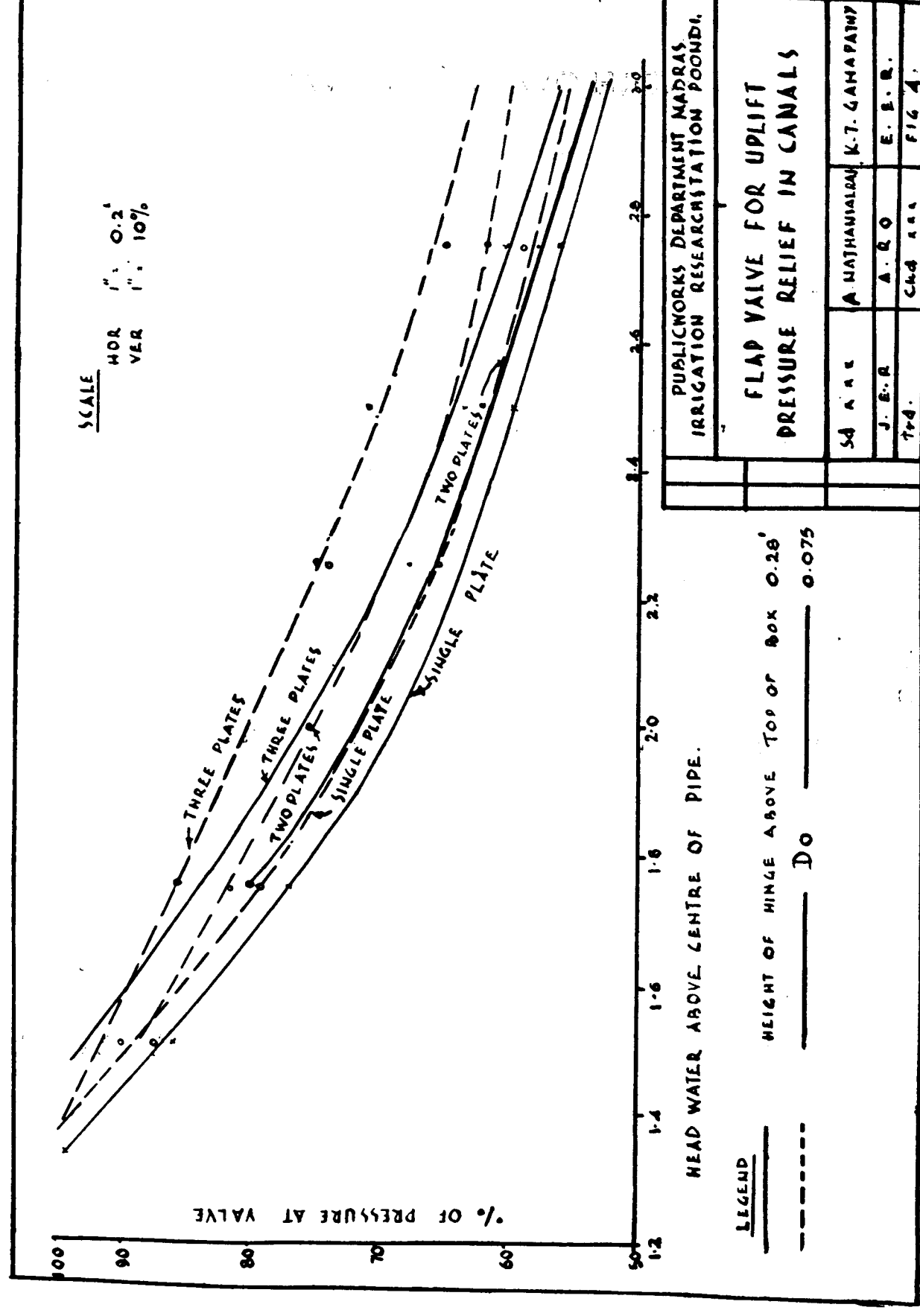
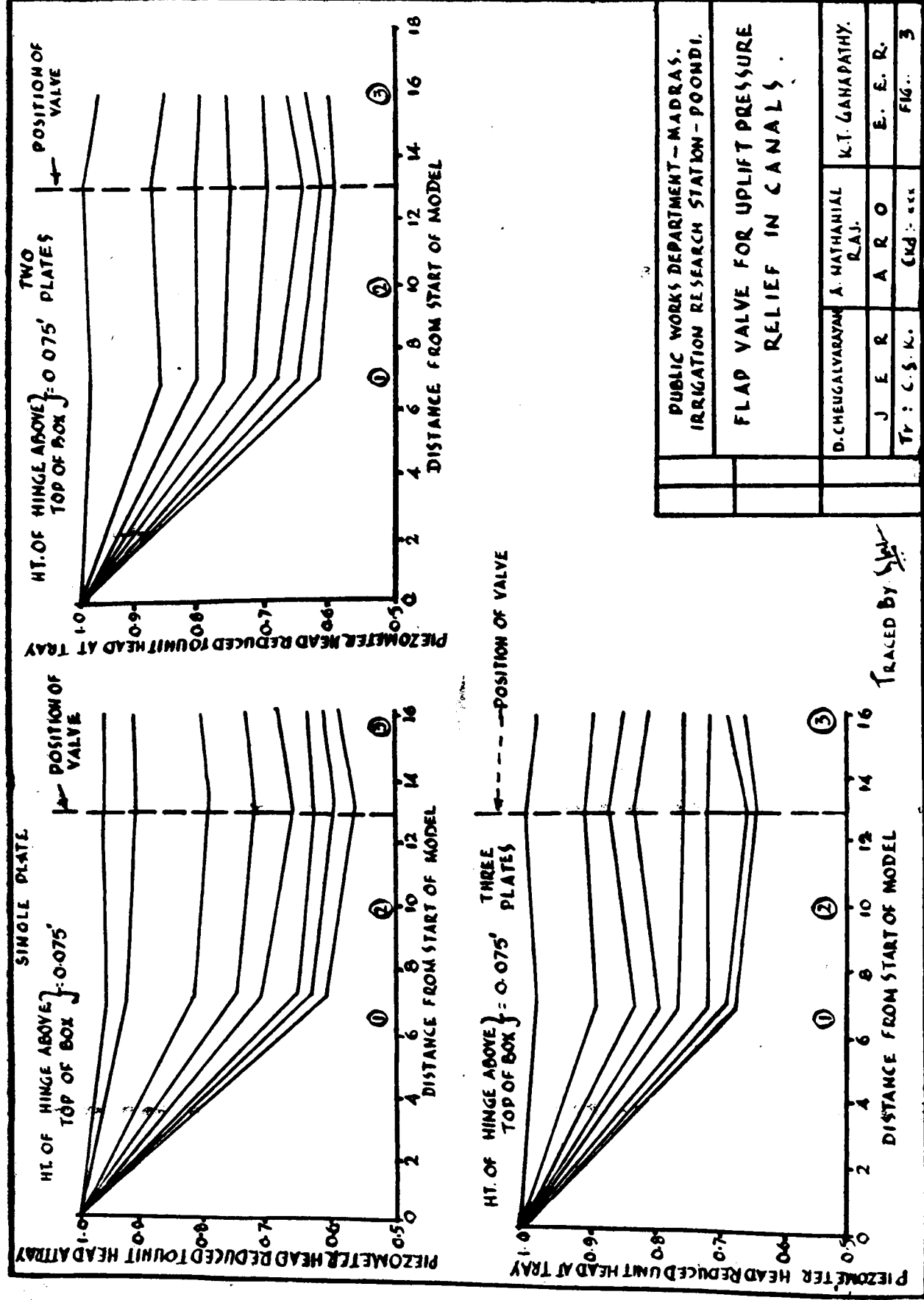
Conclusion

A perusal of figure (4) will indicate that the percentage pressure increases as the weight of

the gate increases. It also increases as the position of the hinge gets lowered. It will also be obvious that for an uplift head of as much as 2.5 feet the pressure beneath the floor is likely to be of the order of 60 per cent. It is recommended that the flap valve gate should be as light as possible and the hinge placed as high as practicable.

* Executive Engineer, Irrigation Research Division, Poondi.





OVERFLOW BREAST WALL

BY

* SRI J. WALTER, B.A., B.E., M. IRRN. Eng. (Colorado), M.I.E. (Ind.).

Normally, a breast wall is designed to retain water so that regulated supplies can be allowed through the gates below the breast wall. This paper deals about the design of an unique breast wall which allows not only regulated water through the controlled gates below, but also permits overflow over the breast wall. This type of overflow breast wall was designed to satisfy a set of special local conditions arising in the design of a small irrigation scheme, namely, Sathiar Odai Scheme.

The Sathiar Odai Scheme estimated to cost about Rs. 16 lakhs, aims at the maximum utilisation of occasional rains in the hilly catchment and the resulting flashy flows in the Odai. The scheme when executed is to benefit about 4,000 acres. The M.W.L. is kept at EL. 765.50. The various alternatives for the spillway for the above scheme were—

- (1) The conventional open type spillway with crest at E.L. 748.50;
- (2) The Siphon spillway; and
- (3) The breast wall spillway with crest at EL. 741.00.

The main consideration for designing a suitable spillway for Sathiar Odai dam were:

(1) Whatever inflow that is to be impounded in the reservoir comes only when the floods come, which are very flashy and of very short duration not exceeding a few hours. Therefore, the spillway gates have to be kept closed and the crest level of spillway has to be high to impound the flood waters.

(2) If and when the flood comes when the reservoir is full, the spillway gates have to be operated such that after impounding available water, the excess water is released quickly. If this flood comes in the night for a few hours when the operating personnel were not present

(being in an out of the place and are not available within short notice) two things can happen:—

(a) If the gates are closed, the flood waters will overflow the earth dam, or

(b) if the gates are kept open the flood waters will flow past the spillway and recede to crest level and there will be no impounding of the valuable waters above crest level.

Several proposals were posed by the field engineers.

A central spillway having two spans of 40 feet with 10.5 feet height shutters above the crest at EL. 748.50 was ruled out as the Central spillway in the river section is very high with the attendant prohibitive cost. The shifting of the spillway with crest at EL. 748.50 with 10½ feet height gates to the right flank, was also dropped again on cost consideration as the shutter area for this proposal works out to 840 square feet. The choice of the type of the spillway for a flashy stream like Sathiar Odai is governed by the major consideration that it may perhaps be not possible to forecast the sudden flood, alert the staff for timely action for the operation of the gates. Again, by keeping the crest as high as even at EL. 748.50, the effective storage is only 13.6 M.cft. against 45 M.cft. required and hence may not cater to the full designed demands of irrigation. This literally would mean lowering the F.R.L. to 748.50 in the interest of the safety of the dam against sudden draw down condition at the expense of the irrigation demands. Also the fact that the Sathiar Odai is not a perennial stream has got an important bearing that whatever flows prevail in Sathiar Odai which are only flashy, have to be harnessed for use for the irrigation demands. Hence summing

up, this flashy stream has to be harnessed to impound 45 M.Cft. at minimum cost by a safe structure and with minimum maintenance demands dispensing with the round the clock vigil, and ensuring that the structure is safe, even if by chance the maintenance staff fail to take timely precaution and action, the river being a flashy one.

Under the above conditions when once the F.R.L. is fixed the ideal spillway will be one to be a free overfall ungated structure. Such a spillway may not be possible in this case as site conditions do not accommodate the required length for free overflow spillway in either flanks economically.

As the next alternative, a siphon spillway is likely to be an ideal one for flashy streams like Sathiar Odai where there will be sudden on rush of floods within the shortest periods. This again has the disadvantage that it involves skilled form work for R.C.C. structures not so far used to be in the Madras P.W.D. and besides it may be rendered ineffective and endanger the structure, if by chance the primers got choked up due to floating debris, trees, etc., carried by flood waters. A novel idea was given shape for envisaging a high co-efficient weir for the sill and a gate with the breast wall type to gain the advantage of the extra head to dispose off the flood quickly and the gate and piers so designed that the water will top up the breast wall and even overflow, in case of emergency, if the maintenance gang by chance, do not turn up. The free board for the earth dam however is temporarily reduced during such abnormal overflows but it is designed that in no case flood waters can overtop the earth dam.

Taking all the above factors into consideration, the breast wall spillway was decided to be the best solution and the length of spillway also has been reduced to a minimum as maximum co-efficient of discharge is obtained in view of its stream line conditions, entry at the breast wall, etc. Considerable savings in the cost of shutter alone is anticipated as 7 feet shutter having a total shutter area of 399 square feet against the

original proposal of 840 square feet is suggested and the maintenance cost will also be the minimum.

The safety of the earth dam designed, has also to be ensured so that any sudden floods due to sudden rains may not endanger the safety of the earth dam, with the flood level rising considerably when the gates are closed, within the shortest period. The site being located in an out of the way place, the maintenance staff may not implement the necessary flood patrolling and other precautions in time. Some fool proof arrangements had to be evolved for surplusing only the excess flood water above the designed F.R.L. For this, as an additional safety with the available site conditions, a high co-efficient weir is provided on the right flank with its masonry solid crest at + 756.50 + 2.5 feet of temporary crest so that any sudden flood can be disposed off even with the gates closed, and floods overflowing the breast wall structure without encroaching too much of the earth dam free board.

The salient feature of the finalised proposals is that the top of the breast wall is kept at + 759 at M.W.L. itself and water is allowed to overflow freely when the flood level rises with the gates in closed position. This is a novel proposal in as much as generally the breast walls are designed only for static water pressure and not for overflowing conditions. Hence it may be interesting to study the design of the overflow breast wall for the following two conditions:—

- (1) When the gates are open—freely discharging.
- (2) When the gates are closed—assuming the water level has been raised to 1 foot 6 inches below T.B.L. + 767.00 and a head of 6.5 feet is functioning.

Condition 1.—In this case, the breast wall was designed for static pressure, the abnormal flood level 765.50 being 1 foot 6 inches below T.B.L. The section has been considered as strips and they are designed as beams for the corresponding water thrust. The load transmitted at any point

* Superintending Engineer (Designs), Madras-5.

will be $= (Wh)$. Where W = Weight of water, Cft , h is the height of the water above the point considered. In this particular case, the maximum height of water is 17.5 feet and a maximum thickness of 27 inches was adopted at the bottom. Main reinforcements were provided on the rear side in horizontal direction spacing between the piers, and distributors are also provided to cater for the bending stress horizontally. Nominal reinforcements were provided on the water faces for temperature, etc.

For vertical load.—The reinforcement for the breast wall has been calculated for the combined bending moment due to self weight and water load, assuming an average width for the beam. Since the section of the beam and the loadings are unsymmetrical, the above method of design assumption seems to be practical and suitable.

Condition II.—When the water is overflowing over the breast wall, when the gates are in closed position, the breast wall may be designed as in the case of condition I in addition to the following:—

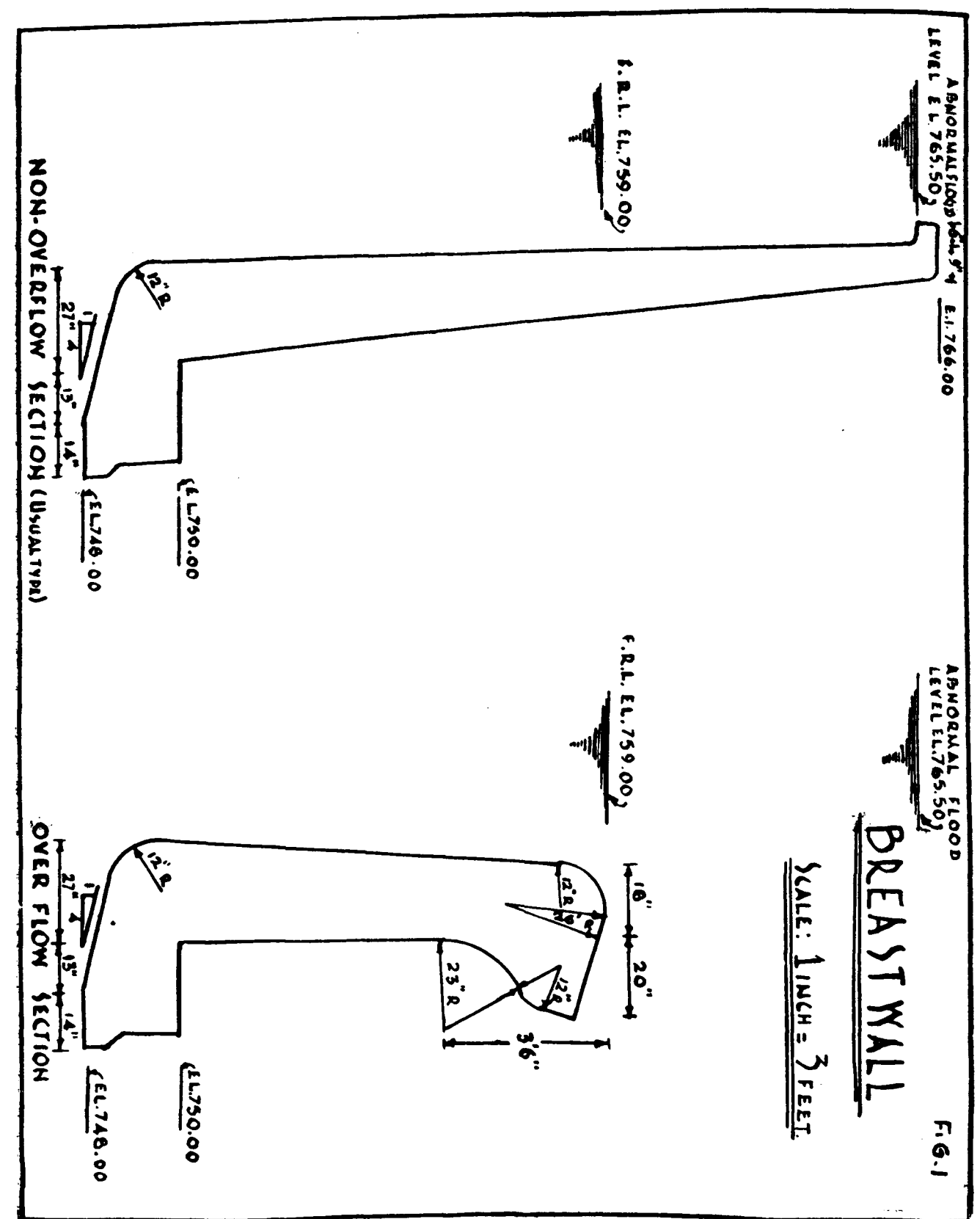
(1) the breast wall may be subjected to torsional effect on the section due to flow of water over the breast wall.

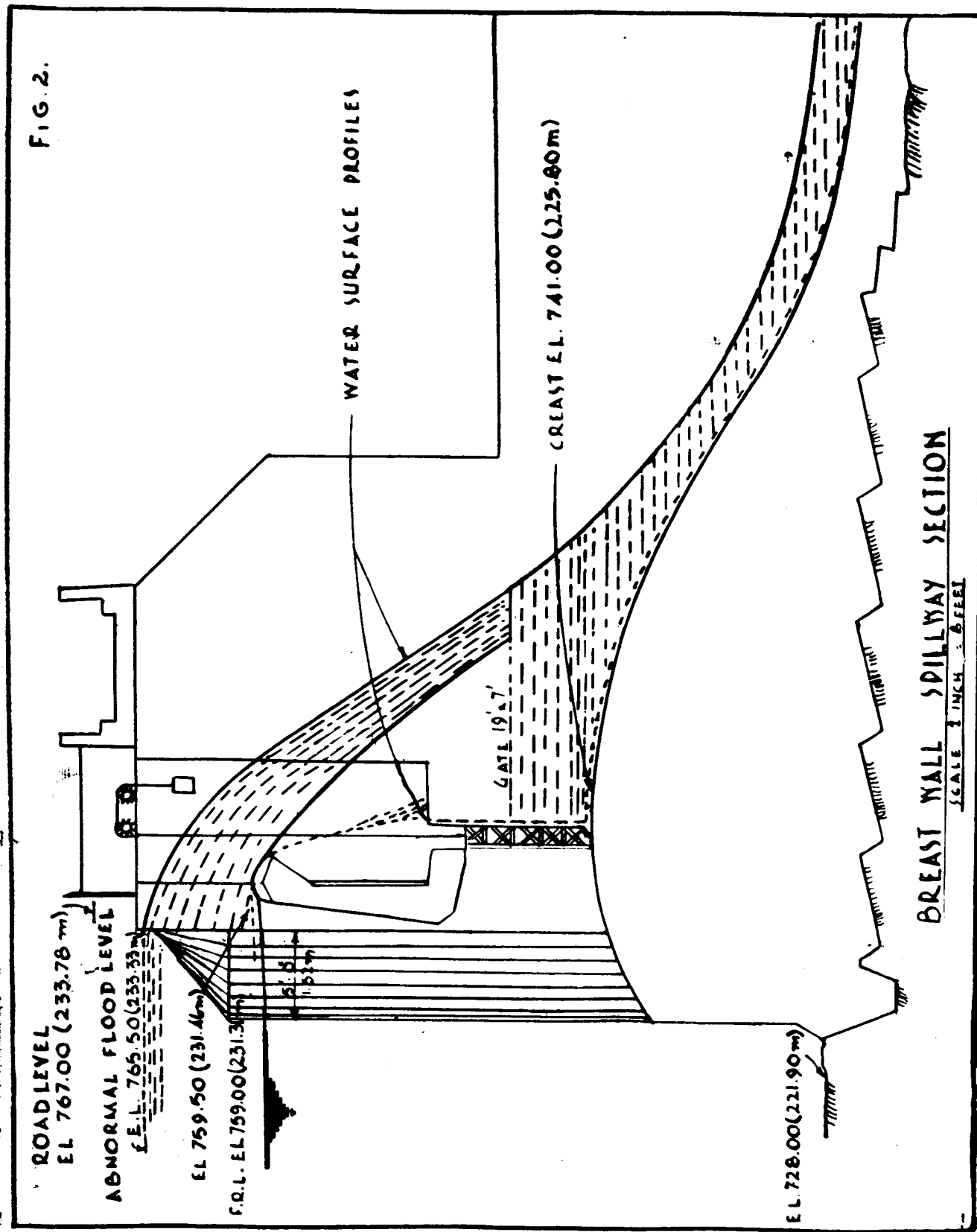
(2) the shape of breast wall at top to ensure the best flow performance over the breast wall.

As the torsional force is found to be an additional one different possible solutions were thought of. The possibility of the bottom and top portions to act independently is ruled out as the assumption is far from reality the whole structure being monolithic. Secondly torsional effect was considered taking the whole slab, and it was found that the net torque for the design generally is negligible and that it can be resisted by the reinforcements already provided for.

As regards the shape at top of breast wall and the hydraulic performance the same has been finalised based on hydraulic model experiments at the Irrigation Research Station, Poondi.

Studies conducted have revealed that the nappe passing over the breast wall falls clear of the gate without fouling it as shown in figure enclosed, even for small heads of flow, i.e., even for 1½ inch head over the breast wall. The water surface profiles for upstream water levels at EL. 765.50 and E.L. 759.50 are presented in the enclosed figures 1 and 2.

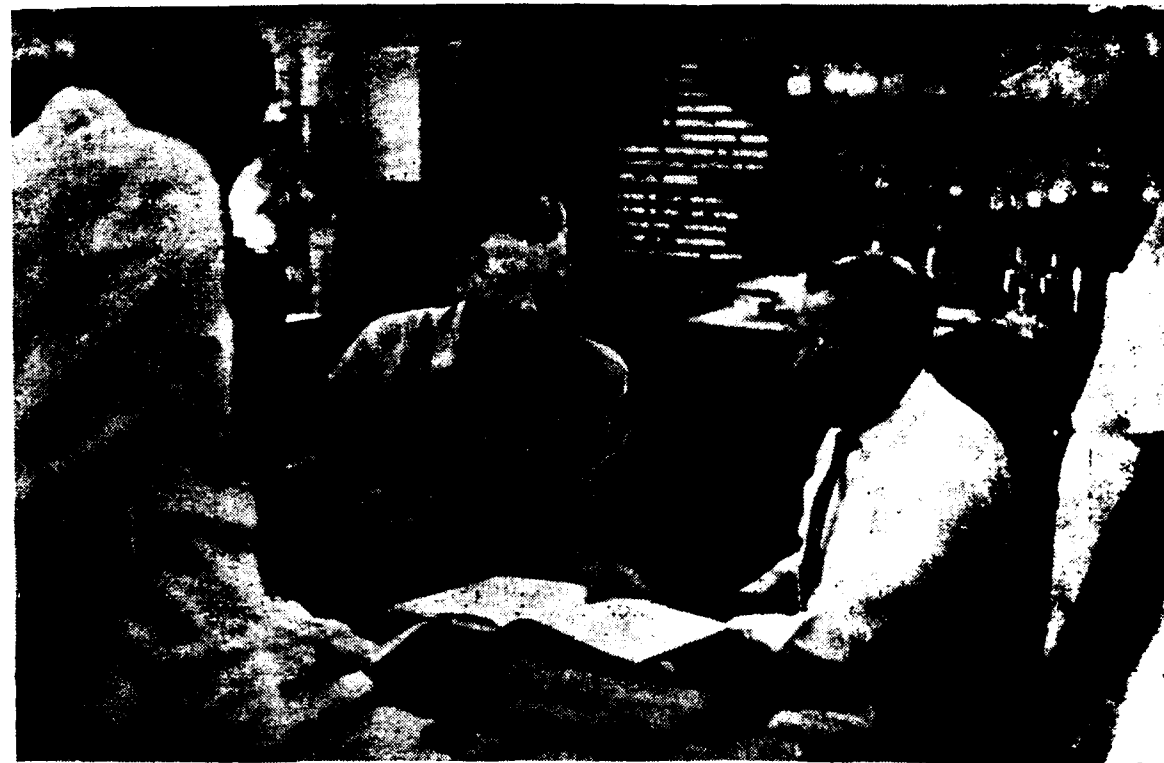




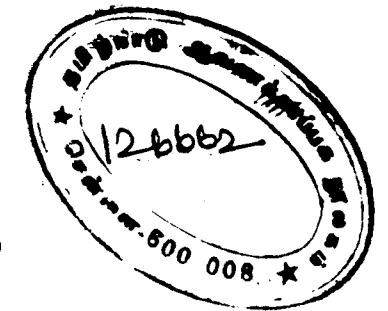
One of the Members of the U.S. Team Mr. Warrenleatham, testing the Soil Sample in Nallur Uppanar Area, Sirhali taluk, Thanjavur District. Mr. Gordon Manley is Looking on.



One of the Members of U.S. Team Mr. Manley, Construction Engineer, in the Laboratory at Chepauk.



One of the members of the U.S. Team Mr. Rinnan, Soil specialist, discussing in the Laboratory at Chepauk.



NEWS ROUND UP

34th Annual Session of Central Board of Irrigation and Power.

The 34th Annual Research Session of Central Board of Irrigation and Power was held at Simla from 2nd to 7th July 1964.

The session was inaugurated by Sri Ragbir Singh, Minister for Irrigation and Power, with Sri M. R. Chopra, Chairman, Central Water Power Commission in the chair. Dr. K. L. Rao in his address exhorted the engineers to rise up to the occasion to solve the food problem in the country. He said: "We produce 360 lakhs tons of rice and the shortage is hardly one per cent. Can we go on deputations to other countries with a fleet of officers, begging our neighbours, for just a lakhs of tons of rice?" The gap could be easily bridged by judicious operation of irrigation canals, utilisation of drainage waters for irrigation, agricultural pumping and protecting crops from floods.

A symposium on "Management of Irrigation water" followed the inaugural address. In the technical discussions, which covered four days, 30 papers were discussed on the following subjects:—

1. Hydraulics.
2. Construction Materials and Practices.
3. Soil Mechanics and Foundation Engineering.
4. Power Engineering.

The session concluded on the 9th July, Mr. Chopra in his concluding remarks stated

that the next session would be held at Ranchi, Bihar.

The Fifteenth Zonal Meeting of Basic and Fundamental Research—Central Board of Irrigation and Power.

The fifteenth zonal meeting of basic and fundamental Research, Central Board of Irrigation and Power, was held at Mysore from 26th to 28th October 1964. Technical discussions were held on Central Board of Irrigation and Power problems on Soil Mechanics, Concrete Technology and Hydraulic Research. There had been significant contribution in respect of research conducted on these three branches of Science.

The Madras P.W.D. Research Laboratories were the pioneers in research conducted on design of High Head gates.

Visit of Mr. Watts

Mr. George M. Watts, Assistant Chief Engineer, Development Division, Coastal Engineering Research Centre, U.S. Army corps Engineers visited Irrigation Research Division on 18th September 1964. He evinced keen interest in the models and appreciated the good work turned out in the Station. He said "The station had a very bright and exciting future."

Visit of the U.S.S.R. Delegation

The U.S.S.R. delegation consisting of the following members visited the Concrete and Soil Laboratory, Chepauk, on 20th November

1964 and the Irrigation Research Station, 27th Annual Session of Central Board of Irrigation and Power.

1. Sri M. P. Zapary, Hydrologist.
2. Sri A. A. Kadyavou, Engineering Irrigator.
3. Sri V. Sukharev, Irrigation Agronomist.
4. Kumari A. N. Kisseleva, Interpreter.

The delegates were greatly impressed by the Research work at these Stations. They were all praise for the high quality of research conducted by the experienced staff of these stations.

SPECIFICATIONS.

1. Describe and define materials so that all bidders base their proposals on a common understanding.
2. Convey the designer's meaning precisely to those who execute the work specified.
3. To Serve as a legal document with a minimum of ambiguous interpretation.

**தமிழ்நாடு ஆவணக்காப்பக நூலகம்,
எழும்பூர், சென்னை-600 008.**

தவணைத்தாள்.

சே எண் : 126662 ப. எண் :

ப. எண் :

[illegible]

S.F. 59A-6 - 1,00,000 - 30-1-84