HYDRAULIC DESIGN OF OPEN CHANNEL TRANSITIONS

Dissertation submitted in partial fulfilment
of the requirements for the Degree
of Master of Engineering.

In

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By

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ABSTRACT

The dissertation thus presented here, deals with the developments so far made in the design of open channel transitions. Greater portion of this work is concentrated on subcritical flow, as many irrigation and other channels usually flow under subcritical conditions. Supercritical flow is rarely permitted in open channel transitions as it would lead to larger losses and possibility of unstable conditions.

In the subcritical flow, much of the work has been devoted to divergent flow or expansion flow as that has more difficult approach than the converging or contracted flow. Also a converging flow may be considered to be irrotational and as such analysis of flow can be done with the help of a flow net. Some experimental work was carried out on two expansion transitions, one straight and the other hyperbolic, the latter being the one recommended by Sri A.C.Mitra, to verify the efficiency of the two in a flume with horizontal bed level. The results obtained are compared with those of of an abrupt transition, the experiment on which was also carried out.

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Symbols

Q = Discharge.

V = Mean Velocity.

A = Flow cross-sectional area normal to the direction of flow.

ℓ = Characteristic length, here considered equal to the hydraulic radius R of a conduit.

L = Length.

ν = Kinematic viscosity of water.

g = Gravitational acceleration.

F = Froude Number.

D = Depth of flow.

Z_b = Elevation of the bed.

α = Energy coefficient or Coriolis Constant.

β = Momentum Coefficient or Boussinesq Coefficient.

w = Unit weight of water.

W = Weight of water enclosed in a section.

q = Discharge per unit width.

B_o = Width of channel.

D_0 = Surface width.

D_m = Mean Depth.

H_T = Total head.

$P_1 - P_2$ = Resultant of pressures acting on the two sections.

F_f = Total external force of friction and resistance acting along the surface of constant between the water and the channel.

$\beta_1' + \beta_2'$ = Correction Coefficients at two sections.

$\bar{z}$ = Depth of the centroid of the water area.

h = Pressure head on the elementary area dA

C = Pressure head correction.

H_v = Velocity head.

ΔH_v = Change in velocity head.

y = Any depth of flow.

y_L = Lateral Distance.

W_s = Water surface elevation.

ΔW_s = Change in water surface elevation.

Contents

CHAPTER I

INTRODUCTION

A channel transition may be defined as a local change in cross section which produces a variation in flow from one uniform state to another; though uniformity of open-channel flow is never fully attained; since the resulting variation in velocity and depth depends, not only upon the form of the transition but also upon the residence of the channel itself. The flow of water in an open channel of variable section is such a complicated phenomenon that a really exact analysis by Mathematical methods is more an ideal than a possibility, and as such only cut and try methods are available for its solution. It is convenient to place a practical limit upon the extent of the non-uniformity, and in addition to sub-divide regions of appreciable uniformity into two flow categories. That in which resistance plays the primary role is known as gradually varied flow, and that in which acceleration plays the primary role is known as rapidly varied flow. Rapidly varied flow occurs in the immediate vicinity of every transition, and a gradually varied flow can occur for up or downstream of the transition.

Under normal design and installation conditions, practically all canals and flumes require some type of transitions structure to and from the waterway. The function of such a structure is to avoid excessive losses to eliminate cross waves and other turbulence and provide safety for the structure and water way. The theory of gradually varied flow may be used to determine the water surface profile in existing transitions with subcritical flow. The essence of such a design will be discussed in connection with the application of the energy and momentum principles.

There is a fundamental difference between the flow in the inlet and outlet transitions since in one case it is convergent and in the other case is divergent. Fluid motion in convergent flow is irrotational and hence it can be analysed by method of flow nets. But in divergent flow the phenomenon of boundary layer separation occurs and is accompanied by back flow along the boundary and eddies which consume much energy. The occurrence of separation increases both the resistance to motion and the rate of dissipation of mechanical energy through turbulence and viscous shear. The method of analysis by flow nets becomes inapplicable in the case of divergent flow owing to the uneven distribution of velocities. It will, therefore, be clear from above that the same types of transitions for both inlet and outlet will not be suitable.

All the experiments conducted were done in a rectangular flume with a horizontal bed, the conditions of flow being subcritical.

CHAPTER 2.

THEORY OF WATER FLOW INCLUDING BOUNDARY LAYER THEORY AND SEPARATION PHENOMENON.

2.1 As open-channel flow can be classified into many types and described in various ways due to change in flow, depth with respect to time and space, the following classification can be made:

a) Steady flow and Unsteady flow- having time as the criterion.

b) Uniform flow and Varied flow having space as the criterion.

2.2 Steady Flow and Unsteady Flow.

Steady flow can be attained in an open channel if the conditions of flow are constant or do not change during the time interval under consideration. Unsteady flow is that which changes with time.

In most open-channel problems it is very convenient to study flow behaviour under steady conditions, which make the solution less complicated. In steady flow conditions continuity must be maintained; for normal condition's of flow in a channel. This can be expressed as

$$Q = V \times A \qquad \text{Eq. 2.1}$$

The mean velocity is generally got by dividing the discharge by the cross - sectional area of flow.

In many problems met with in steady flow, the discharge is constant, throughout the reach of the channel under consideration i.e., the flow is continuous.

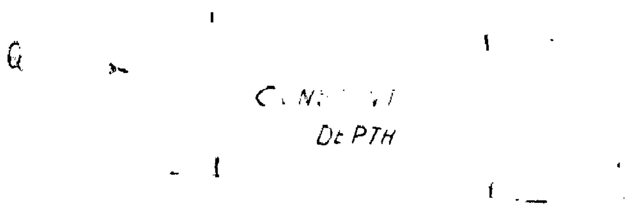
Using the equation below to illustrate-

$$Q = V_1A_1 = V_2A_2 \qquad \text{Eq. 2.2}$$

the subscripts denote different sections of the channel. The Eq. 22 is termed the continuity equation for a continuous steady flow. The Eq. 2.2 becomes invalid if the flow quantity changes and such a flow is termed spatially varied or discontinuous flow. Such a flow is found in roadside gutters and side channel spillways.

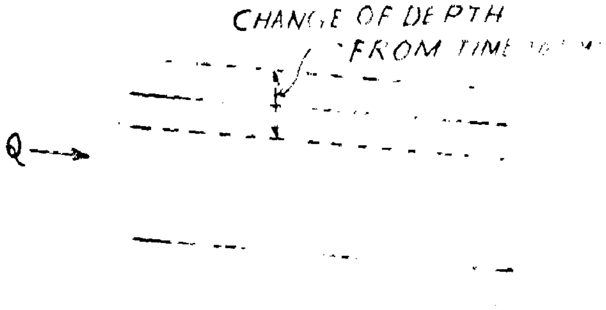
2.3 Uniform Flow and Varied Flow.

Considering now the uniform flow and varied flow with space as the criterion. An open channel flow is said to be uniform if the condition of flow is the same at every section of the channels but the uniform flow may be steady or unsteady, depending on whether or not the flow changes with time. The fundamental type of flow generally treated in open channel hydraulics is steady uniform flow, in here the depth of flow does not change during the time interval under consideration as in fig. 1 (a). An unsteady uniform flow when established



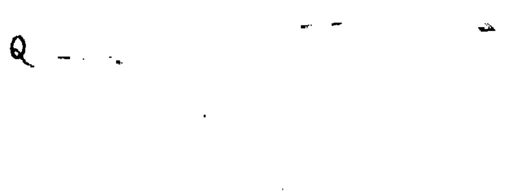
UN UNIFORM FLOW GENERALLY
FOUND IN LABORATORY CHANNELS

(a)



UNSTEADY UNIFORM FLOW
RARELY FOUND

(b)



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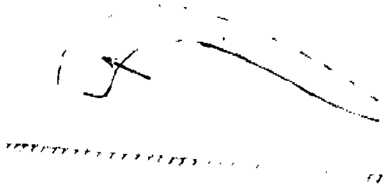


RAPIDLY VARIED FLOW
[illegible]

(d)

would require that the water surface fluctuates from time to time while remaining parallel to the channel bottom as in fig. I (b).

Varied flow occurs if the depth of flow changes along the length of the channel. Varied flow may be either steady or unsteady, but since unsteady uniform flow is very rare, the term 'Unsteady flow' will be used hereafter to refer to as unsteady varied flow.

As already mentioned, varied flow may be further classified as either rapidly or gradually varied. Rapidly varied flow occurs if the depth changes abruptly over a comparatively short distance, otherwise it is gradually varied flow as in fig. 1 (c). A rapidly varied flow as in fig. 1(d) is also known as a local phenomenon, examples being hydraulic jump and hydraulic drop.

2.4. State of Flow.

The behaviour of open channel flow is governed basically by the effects of viscosity and gravity relative to the inertia forces of the flow. Under certain circumstances the behaviour of flow may be affected by the surface tension of water, though it does not play a significant role in most open channel problems encountered in engineering.

The state flow may be laminar, turbulent or transitional depending on the effect of viscosity relative to inertia. The flow is said to be laminar, turbulent or transitional depending on the effect of viscosity relative to inertia. The flow is said to be laminar if the viscous forces are predominent or so strong relative to the inertia forces and thus viscosity plays a significant part in determining the flow behaviour. In laminar flow, the water particles appear to move the definite smooth, paths or stream lines, and infinitessimally thin layers of fluid seem to slide over adjacent layers.

A state of flow is turbulent if the viscous forces are weak relative to the inertia forces. At such a state the water particles move in irregular paths which are neither smooth nor fixed but which in the aggregate still present the forward motion of the entire stream.

The transitional state comes between laminar and turbulent states.

The effect of viscosity relative to inertia can be represented by the Reynold's number defined as

$$R = \frac{Vl}{\nu} \qquad \text{Eq. 2.3}$$

With the basic idea that a flow is considered laminar if the Reynold's number is small and turbulent if large, experiments conducted have shown that the flow in a pipe changes from laminar to turbulent in the range of Reynold's number between the critical value 2,000 and a value that may be as high as 10,000. In these experiments, the diameter of the pipe was taken as the characteristic length in defining the Reynold's number.

When the hydraulic radius is taken as the characteristic length, the corresponding range is from 500 to 2,500. Since the diameter of a pipe is four times its hydraulic radius. The effect of gravity upon the state of flow is represented by a ratio of inertia forces to gravity forces. The ratio is given by the Froude Number, defined as

$$F = \frac{V}{\sqrt{gL}} \qquad \text{Eq. 2.4}$$

Usually in open-channel flow the characteristic length is made equal to the hydraulic depth D, which is defined as the cross-sectional area of the water normal to the direction of flow in the channel divided by the width of the free surface. For a rectangular channel, this is equal to the depth of the flow section.

When the Froude Number is unity Equation (2.4) becomes

$$V = \sqrt{gD} \qquad \text{Eq. 2.5}$$

and the flow is said to be in a critical state. If F is less than unity, or $V < \sqrt{gD}$ the flow is subcritical. In this state, the role played by gravity forces is more pronounced, so the flow has a low velocity and is often described as tranquil and streaming. If F is greater than unity or $V > \sqrt{gD}$ the flow is super-critical, have therefore the inertia forces become dominant; so the flow has a high velocity. Such flows are usually said to be rapid, shooting and torrential.

2.5 Concepts of Boundary Layer and Separation Phenomenon.

When water enters a channel, the velocity distribution across the channel section, will vary with the distance over which the water travels in the channel owing to the presence of boundary roughness. If the flow is uniform and stable and if the channel is prismatic and of constant roughness, the velocity distribution will eventually reach a definite pattern.

Assuming the following to simplify the discussion.

1. The flow entering the channel is laminar and of uniform velocity distribution.
2. No restriction exists at the entrance that will cause abrupt disturbance of the water surface and the velocity distribution.
3. No restriction exists at the entrance that will

~~cause abrupt disturbance.~~

3. The depth of flow is indefinitely large, so the depth of flow can be considered constant as the water enters the channel.

In the Fig. 2 the effect on the velocity distribution due to boundary roughness indicated by the line ABC. Outside the surface represented by ABC, though not distinctive is known as the boundary layer and its thickness is designated by δ , and since the boundary layer is not distinctive its thickness has been defined arbitrarily in various ways. The definition which is generally given to it is that the thickness δ is the magnitude of the normal distance from the boundary surface at which the velocity V is equal to 99% of the limiting velocity V_o which the velocity - distribution curve in the boundary layer approaches asymptotically. The boundary - layer thickness tends to increase with distance in the direction of flow. In a channel of finite depth, the boundary layer ultimately extends over the full depth of the channel.

2.6 Separation Phenomenon.

Considering the fig. 3 below, it shows to a greatly enlarged radial scale the distribution of velocity in the neighbourhood of a boundary which curves in such a way as to produce a general increase in velocity (as shown by

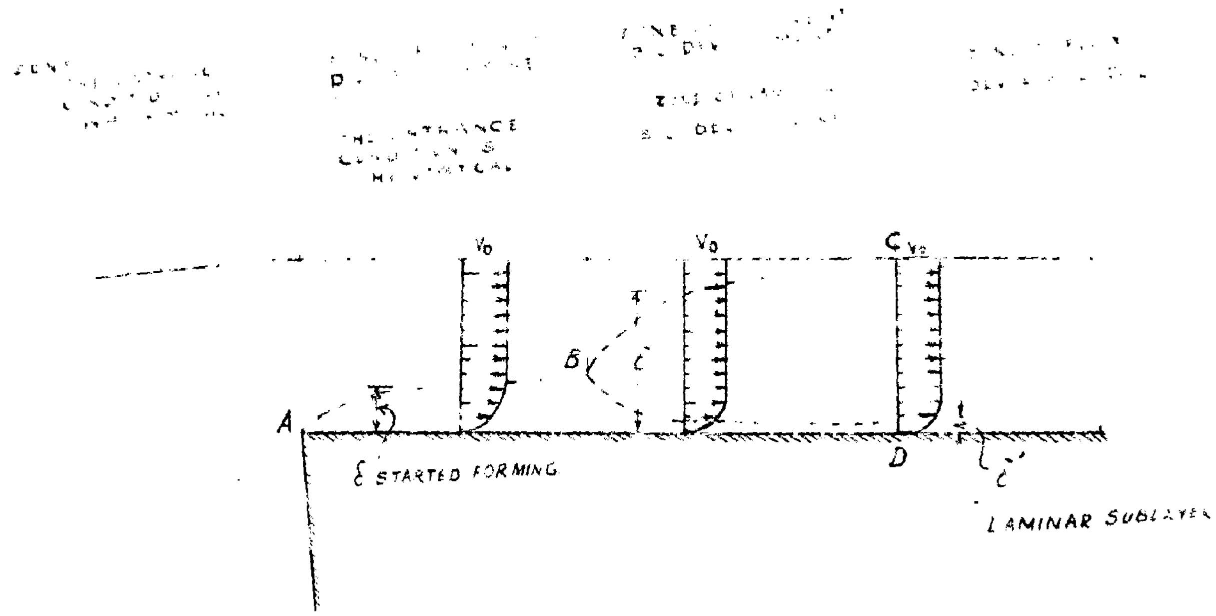
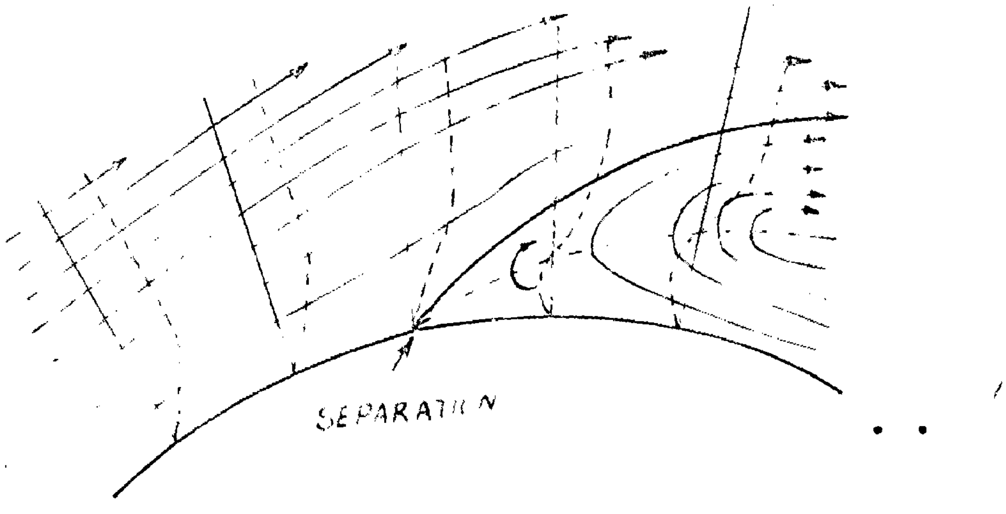


FIG- 2.



BOUNDARY-LAYER SEPARATION IN A ZONE OF DECELERATION.

FIG.

the converging stream lines, followed by a subsequent deceleration.

Due to increase in velocity the pressure decreases, as afore-mentioned the thickness of boundary layer increases with distance in the direction of flow, but in this case such a tendency is more or less counterbalanced by the convergence of the stream lines and the corresponding acceleration of the flow in general. Except for the thin layer of retarded fluid near the boundary, therefore, the velocity distribution of the flow as a whole may be obtained from the configuration of the flow net; which in turn permits the evaluation of the pressure distribution at all points of the flow and of the boundary as well.

The right half of figure 3 on the contrary, illustrates the behaviour of the flow as the boundary curves in such a way as to produce deceleration of the fluid-deceleration, at least as would be indicated by the diverging lines of the flow net. This is accompanied by an increase in pressures. In this case therefore the tendency of the boundary layer to expand with distance in the direction of flow is strengthened by the tendency of the stream lines to diverge from the boundary as the flow decelerates. As the velocity near the boundary is small, further increase in pressure requires a deceleration which is not physically possible hence a point called the point of separation where the flow has come to a stop.

The flow as a whole can continue beyond this zone only if a discontinuity at the boundary is produced, and such discontinuity involves separation of the flow from the boundary. As shown in the figure the stream line which abruptly leaves the boundary dividing the on coming flow from a region of reverse flow on the downstream side.

This phenomenon is of particular importance to design of expanding transitions.

2.7 Energy and Momentum Equations in respect of open channels.

The surface level of an open channel is variable, but the pressure at the surface is constant (atmospheric) and taking this as the datum for pressure, the potential head in straight flow is constant over a vertical section and is equal to $D + Z_b$ as shown in fig. 4.

Also $$H_T = D + Z_b + \frac{\alpha V^2}{2g} \qquad \text{Eq. 2.6}$$

where α is a constant accounting for variation in velocity, and varies from .98 to 1.02 ,this is the general equation for straight line flow, the elevation of the total energy line above the bed level as the datum line is

$$H_o = D + \frac{\alpha V^2}{2g} \qquad \text{Eq. 2.7}$$

$$H_o = D + \frac{V^2}{2g} \qquad \text{Eq. 2.8}$$

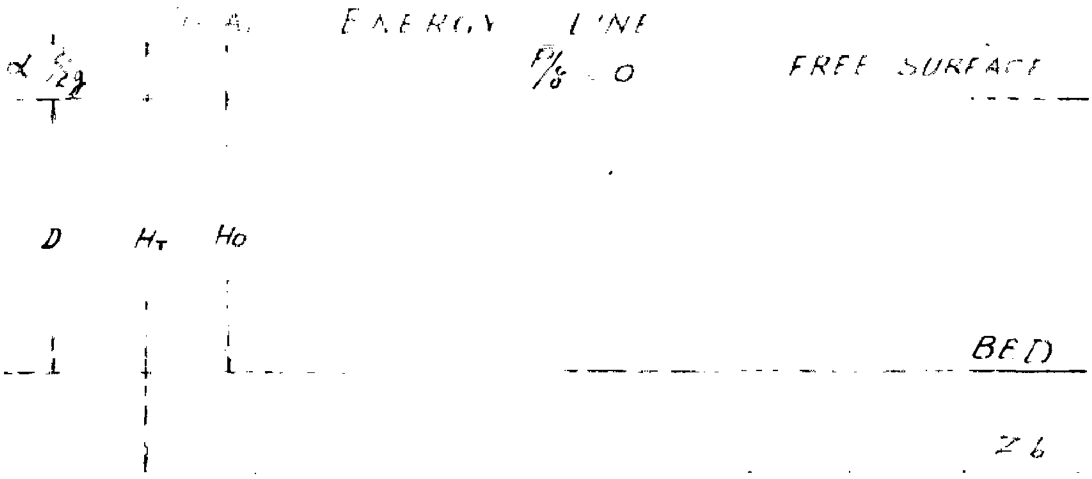
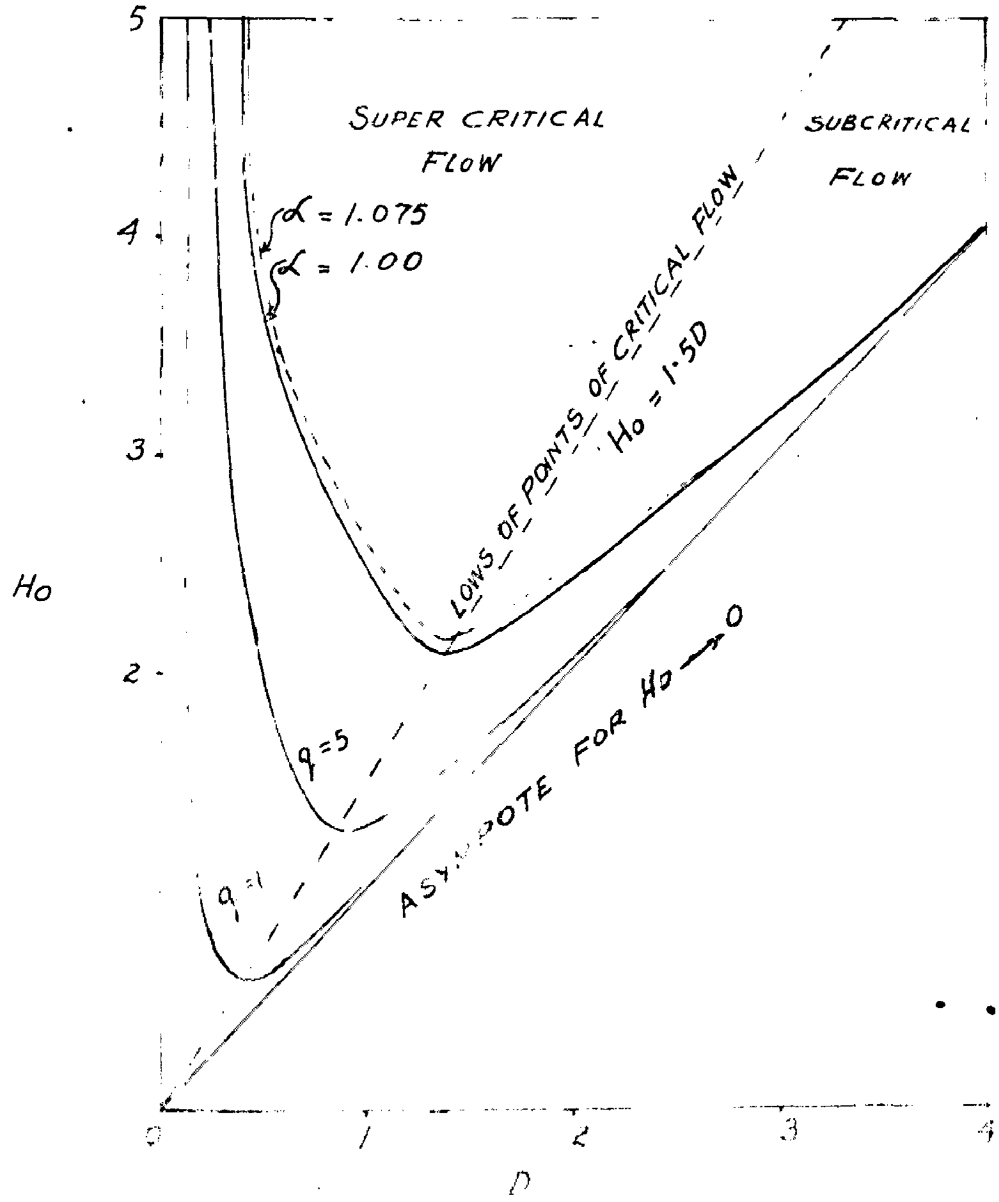


FIG 4.



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when $\alpha = 1$

From above H_0 is known as the energy of flow, designated by E_f, or specific energy. In a rectangular channel with level bed

$$V = \frac{q}{D} \qquad \text{Eq. 2.9}$$

Equation becomes

$$H_0 = D + \frac{\alpha q^2}{2gD^2} \qquad \text{Eq. 2.10}$$

and reduces

$$H_0 = D + \frac{q^2}{2gD^2} \qquad \text{Eq. 2.11}$$

The equation 2.11 when $\alpha = 1$ is plotted in fig. 5, with D as the abscissa and H_0 as the ordinate.

These curves for various discharge are useful in the design of flumed structures. It can be seen that for correct value of H_0 a given discharge q per unit width can have two alternate depths of flow and that a minimum H_0 exists for each value of q. The depth and velocity at which the minimum occurs are known as the critical depth and critical velocity. As have already been mentioned that the Froude number here is 1. The equation connecting the depth and velocity becomes

$$\alpha \frac{V^2}{gD} = 1 \qquad \text{Eq. 2.12}$$

Knowing that at critical flow

$$H_0 = \frac{3}{2} D \qquad \text{Eq. 2.13}$$

and substituting that $\frac{\alpha V^2}{gD} = 1$ in the equation

$$D = \left(\frac{\alpha q^2}{g}\right)^{1/3} = \frac{\alpha V^2}{g} \qquad \text{Eq. 2.14}$$

Therefore,

$$V^2 = \frac{gD}{\alpha} \qquad \text{Eq. 2.15}$$

$$V = \sqrt{\frac{gD}{\alpha}} \qquad \text{Eq. 2.16}$$

In the non-rectangular uniform channels. The corresponding equation are

$$H_0 = D + \frac{\alpha Q^2}{2gA^2} \qquad \text{Eq. 2.17}$$

$$H_0 = D + \frac{Q^2}{2gA^2} \qquad \text{Eq. 2.18}$$

Where the bed is the datum for H_0 and D.

Considering say $dA = B_s dD$

the critical velocity and the critical depth are given by

$$\frac{\alpha V^2}{g D_m} = 1$$

$$V = \sqrt{\frac{gA}{\alpha B_s}} = \sqrt{\frac{g D_m}{\alpha}} \quad . \; .$$

When the velocity is less and the depth greater than the critical the flow is said to be subcritical. Conversely, when the velocity is greater and the depth less than the critical the flow is called hyper or super critical.

A difference between sub-critical and super-critical flow is that in the former a disturbance or afflux affects the upstream conditions where as in the latter as such effect is produced unless the disturbance is sufficient to form a standing wave which advancing upstream changes to flow from supercritical to subcritical. The explanation lies in the fact that the critical velocity of flow is the same as the velocity of a gravity wave. Hence with velocity exceeding the critical, the gravity wave, produced by a disturbance can move only downstream.

Another difference between the subcritical and the super-critical flow is that in the former a contraction producing a converging flow whether on the sides or bed of the channel, results in a reduced depth of flow provided the depth is comparatively small, where as in the super critical flow it results in an increased depth. The effect of convergence due to a rising bed is shown in the figure 6. A contraction in width increases the discharge per unit width q, while the total energy head H_o remains unchanged. If any horizontal line is followed in figure 6 it will be seen that with measuring q, D decreases in the subcritical region and increases in the super critical. On the other

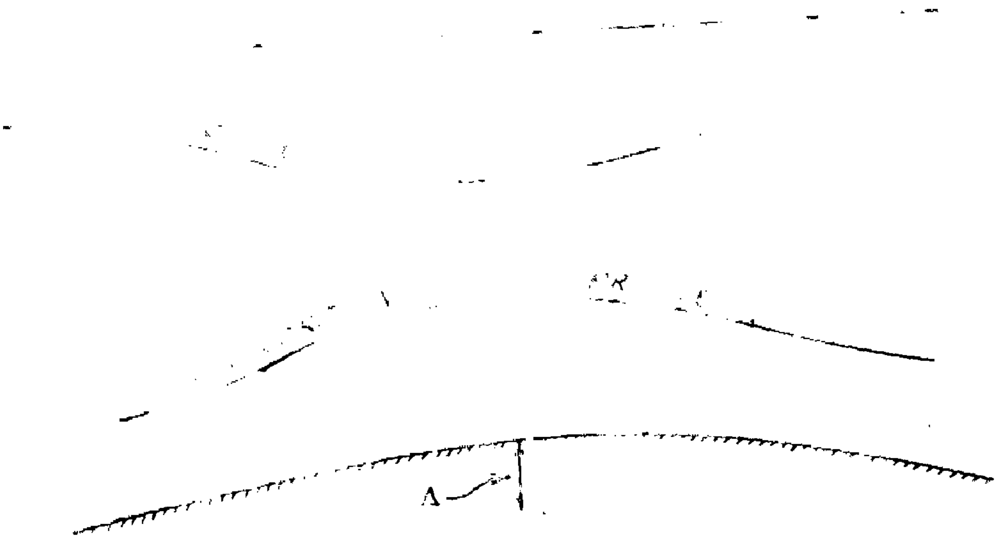

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FIG 6.

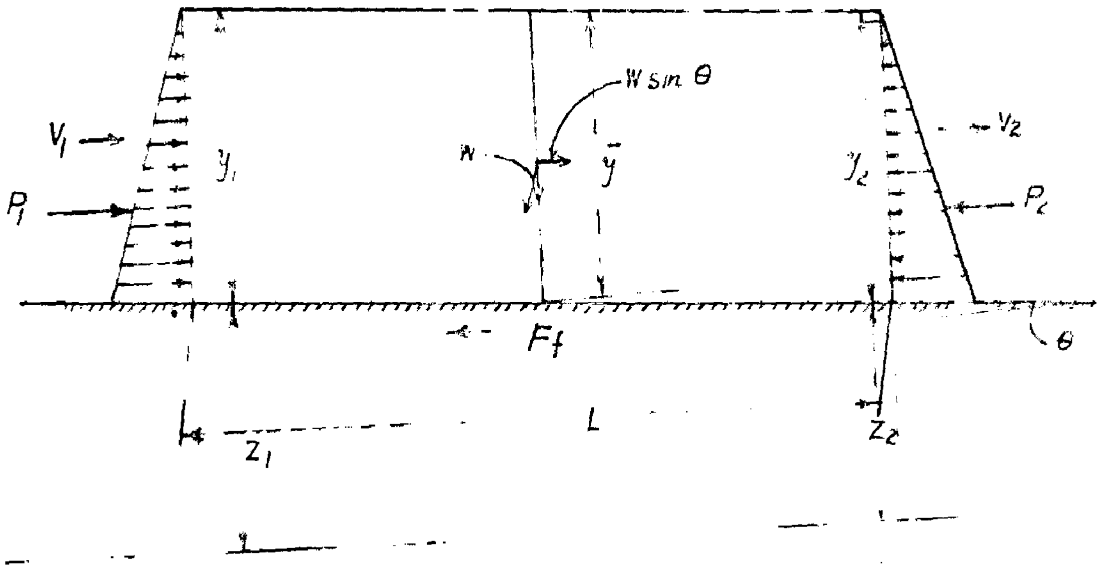

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FIG —

hand, or rise in bed decreases the total head H_o and q remains unchanged. If again any line of constant q is followed, it will be seen that the decreasing of H_o, D decreases in the subcritical region and increases in the super critical. It is converse in the case for expansion.

2.8 Criterion for a critical state of flow.

The critical state of flow has been defined as the condition for which the Froude Number is equal to unity. A more common function is that it is the state of flow at which the specific energy is a minimum for a given discharge. A theoretical criterion for critical flow may be developed from this equation already mentioned with $\alpha = 1$

$$H_o = D + \frac{Q^2}{2gA^2}$$

Differentiating with respect to D and noting that is constant

$$\frac{dH_o}{dD} = 1 - \frac{Q^2}{gA^3}\frac{dA}{dD} = 1 - \frac{V^2}{gA}\frac{dA}{dD} \qquad \text{Eq. 2.19}$$

The differential water area dA near the free surface is equal to TdD

$$\text{Now} \quad \frac{dA}{dD} = T$$

But $D = \frac{A}{T}$ being the hydraulic depth,

$$\frac{dH_o}{dD} = 1 - \frac{V^2 T}{gA} = 1 - \frac{V^2}{gD} \qquad \text{Eq. 2.20}$$

At the critical state of flow, the specific energy is minimum

i.e. $\frac{dH_0}{dD} = 0$

$$\frac{V^2}{2g} = \frac{D}{2} \qquad \text{Eq. } 2.21$$

This is the criterion for critical flow, which states that at the critical state of flow, the velocity head is equal to half the hydraulic depth. The above equation may be written as

$$\frac{V}{\sqrt{gD}} = F = 1 \qquad \text{Eq. } 2.22$$

which is Froude number and being equal to 1, shows the criticality of flow.

2.9 Momentum in open-channel flow.

As mentioned earlier owing to the non-uniform distribution of velocities over a channel section, the velocity head of an open channel flow is generally greater than the value computed according to $\frac{V^2}{2g}$

The non-uniform distribution of velocities also affects the computation of momentum in open channel flow. From principles of mechanics the momentum of the fluid passing through a channel section per unit time is expressed by $\frac{\beta \omega Q V}{g}$.

It has been generally found that the value of β for fairly straight prismatic channels varies approximately from 1.01 to 1.12. According to Newtons' Second Law of motion, the change of momentum per unit of time in the body of water in a flowing channel is equal to the resultant of all the external forces that are acting on the body.

Applying this principle to a channel of large slope, the following expression for the momentum change per unit time in the body of water enclosed between sections 1 and 2 of Fig. 7. may be written

$$\frac{Q\omega}{g}\left(\beta_2 V_2 - \beta_1 V_1\right) = P_1 - P_2 + \omega \sin\theta - Ff \qquad \text{Eq. 2.23}$$

The above equation is known as the momentum equation. For a parallel or gradually varied flow, the value of P_1 and P_2 in the momentum equation may be computed by assuming a hydrostatic distribution of pressure. For a curvilinear or rapidly varied flow, however, the pressure distribution is no longer hydrostatic; hence the values P_1 and P_2 cannot be so computed, but must be corrected for the curvature effect of the stream lines of the flow.

For simplicity P_1 and P_2 may be replaced respectively by $\beta_1' P_1$ and $\beta_2' P_2$ where β_1' and β_2' are the correction coefficients at the two sections, and they are referred to as pressure-distribution coefficients, but here since and are force, they can be called force coefficients. It can be shown that the force coefficient is expressed by

$$\beta' = \frac{1}{A\bar{z}}\int_0^A h\,dA = 1 + \frac{1}{A\bar{z}}\int_0^c c\,dA \qquad \text{Eq. } 2.24$$

It can easily be shown that β' is greater than 1.0 for concave flow, and less than 1.0 for convex flow and equal to 1.0 for parallel flow.

CHAPTER 3.

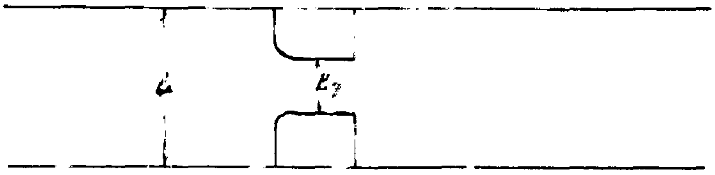
TRANSITIONS IN SUBCRITICAL FLOW.

3.1 E.W. Lane (7) in 1915-16, during his experiments to analyse the flow of water through contractions chose transition dimensions arbitrarily. In constructing the channel, the contraction was rounded by 5 ft. radius curves, tangential at the ends where the curves meet, the flume and abrupt contraction wall of the normal channels. The fluming was from 3' to 1' for a length of 2', after which he expanded it gradually with the parabolic equation of y = 0.0178 x as shown in Fig. 8 (b). The experimental results were studied in the light of the theories existing at that time on the flow of water through contractions, viz. these due to d' Aubuisson and Weisbach. Each of these theories defined a coefficient of discharge (coefficient to take account of losses due to friction, impact etc.), as shown in Fig. 8(a).

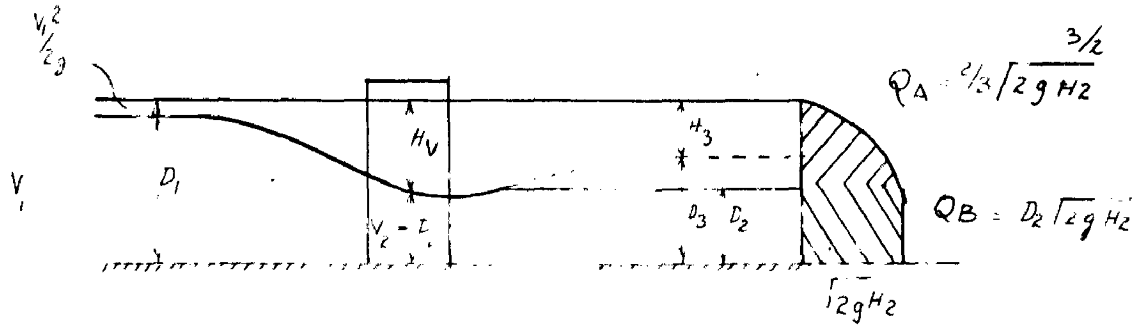
d' Aubuisson Coefficient $C_A = \dfrac{Q}{B_2 D_2 \sqrt{2g H_{v_2}}}$ Eq. 3.1

Weisbach Coefficient $C_w = \dfrac{Q}{B_2 \sqrt{2g} \left(\frac{2}{3} H_{v_2}^{3/2} + D_2 H_{v_2}^{1/2} \right)}$ Eq. 3.2

d' Aubuisson formula is an application of Bernoullis, and Weisbach considered the discharge to be made of Q_A, representing flow over a weir, and Q_B a flow through an



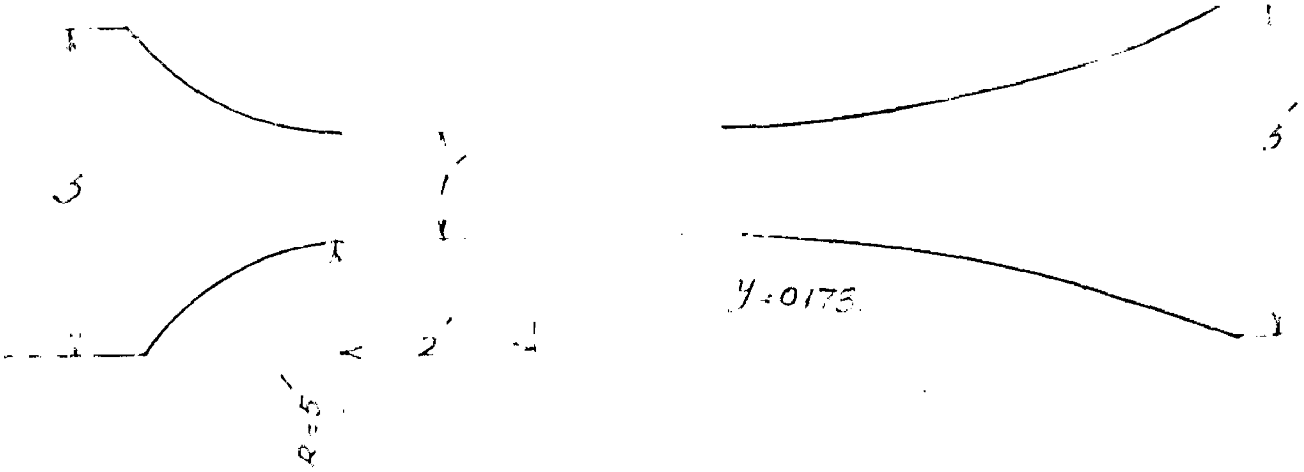
PLAN OF FLUME



FLOW PROFILE

FIG 8 (a)

SCALE 1" = 2'



PLAN OF FLUME

FIG. 8(b)

orifice of width B_2, and height D_2.

Lane found from the experiments, that d' Aubuisson coefficient can best be applied to contractions in which the water flows from one stage to another without a great deal of turbulence. The two areas of measurement must be the actual area of flow of the water, and the conditions at these points are such that the static pressure on each particle is equal to the weight of the water above it, upto the free water surface. These conditions are fulfilled by the flow through the prismatic section of the expanding flume and the short flume with rounded entrance, and also in the contractions with rounded entrance. To a certain extent it can be applied to the short flume with sharp corners, resulting in a lower value of the coefficient due to turbulence introduced by the sharp edges.

The conditions most favourable to Weisbach theory are those existing at the sharp edge contraction of narrow width, and on account of the surface drop which always takes place above a contraction, the condition assumed in the Weisbach theory will be only approximately attained.

He found out that the flow in the expanding flume follows very closely the theory of back water curves. For a frictionless channel, with level bottom and variable width, the following four back-water curves are possible.

Case 1- Width increases, water surface slopes upward; depth greater than the critical value.

Case 2- Width increases; water surface slopes downward; depth less than the critical depth.

Case 3- Width decreases; water surface slopes upward; depth less than the critical value.

Case 4- Width ~~in~~de?creases; water surface slopes downward; depth greater than the critical value.

3.2 Julian Hinds in 1928 (8) presented some ways of designing transitions as adopted by U.S.B.R. in the design of transition for flumes and syphons. He dealt with the above with reference to these principal facts.

1. Unimportant transitions, where velocities are low, may be designed arbitrarily, by adaptation from successful structures operating under similar conditions.

2. For important structures, especially those involving velocities in excess of 6 to 8 ft. per sec. careful detailed computations must be made.

3. Experimental data collected by U.S.B.R. are summarised showing efficiencies that may be expected with various types of construction. As far as subcritical flow is concerned he divided the subject into two parts namely,

1) Unimportant or less velocity structures, not involving

hydraulic jump or flow at critical depth.

ii) Important or high velocity structures, not involving the hydraulic jump or flow at critical depth.

1. Design of Secondary Structures not affected by critical flow.

Having found out that all structures for changing the shape or cross sectional area of a water conduit cause disturbances in flow, which might be objectionable in themselves or in the resulting losses in head, he thought it necessary to provide for a drop in the water surface, sufficient to produce the required increase in velocity head and to overcome friction and entrance losses; but at an outlet the water surface will rise theoretically a vertical distance equal to the reduction in velocity head; for the actual rise, usually referred to is recovery of head, is less than the theoretical because of losses due to friction and the outlet. No satisfactory theory for computing transition losses for a given structure has been proposed, but the magnitude of these losses can be determined by experiments only. A large variety of detailed design for transitions structures of secondary importance had been done by the U.S.B.R. Here the preparation of a new design for a structure of this class is usually accomplished by changing the details of a previous structure, known to be satisfactory to suit the new conditions of design. Some of the designs are shown in Ref. (8). On the design of important transition not involving critical flow it was found out that method of design outlined for simple structures is not adequate for important installations especially where velocities are relatively high, and under such conditions the detailed dimensions and the forms of the structure, through out its entire length becomes important. On investigation in the United States on Columner Tunnel, Treten canal outlet, it was found that this outlet was less efficient than the more abrupt outlet from the North Fork thus indicating that the assumption made that the energy loss can be minimised by reducing the angle of divergence of the transition is not universally applicable. Also having changed the designed dimensions of the Dear Coulch outlet to meet local conditions in the field, the result was that the outlet became so inefficient that as to require reconstruction. After considering the losses met with, the conclusion arrived at was that careful attention should be given to detail dimensions of the whole length of the transition. The computed water surface profile was found to be irregular in the case of known faulty structures, as a result a new criterion for design was adopted namely, that the computed water surface of the transition should be smooth continuous curve approximately tangential to the water surface curves in the channel above and below.

As there are no data available for the determination of the most suitable curve for a given set of conditions the irregularity of the water surface profile can be avoided by adopting or assuming a curve at the inlet such that the rate of change of acceleration is changed in a way such that the water surface profile becomes a smooth continuous curve.

The U.S.B.R. adopted a plan based on the principles illustrated in Fig. 9 for all important transitions.

1) Assumption of the dimensions of the structure, and by Bernoulli's theorem, the surface curve computed, the dimensions being subsequently changed and the curve recalculated until satisfactory results obtained 2) The determination of the water surface curve, and then computing the dimensions of the structure to conform. The review of experimental data is very necessary before attempting to prepare designs, especially that part which relates to the effect of curvature in flumes on the motion of the outlet structures; sketches of some designs are shown in Fig. 10.

3.3 (b) In the design of a flume inlet example of which is given in (8). The hydraulic properties of the canal and the flume may be assumed to be known, and the first go is to determine approximately the length of the transition. Adopting the U.S.B.R. rule, the length should be such that a straight line joining the flow line at the two ends of the transition will make an angle of about 12½° with the axis of the structure. The entrance loss is assumed to be $0.1\Delta H_v$, i.e., the change in velocity head is to be $1.1\Delta H_v$

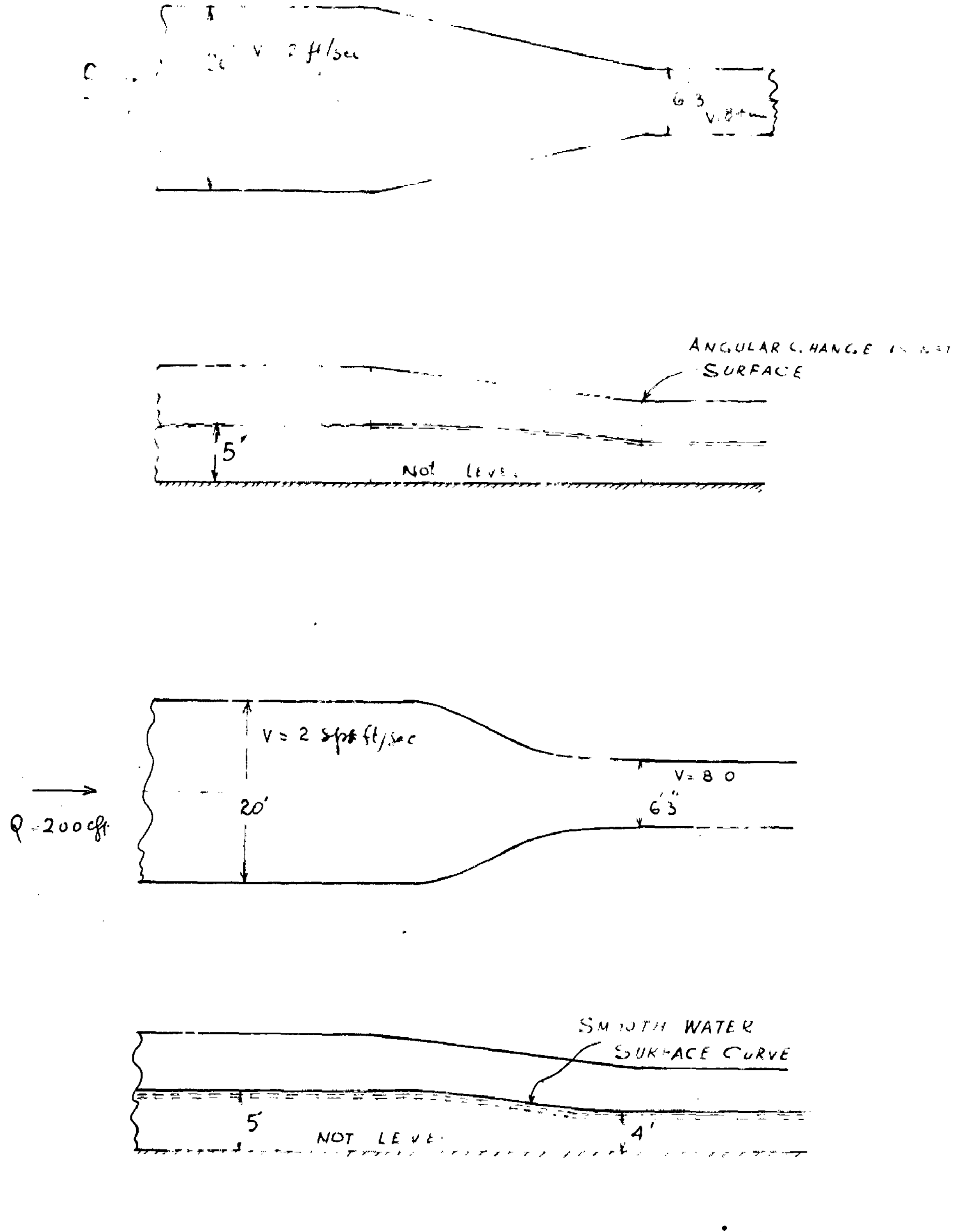
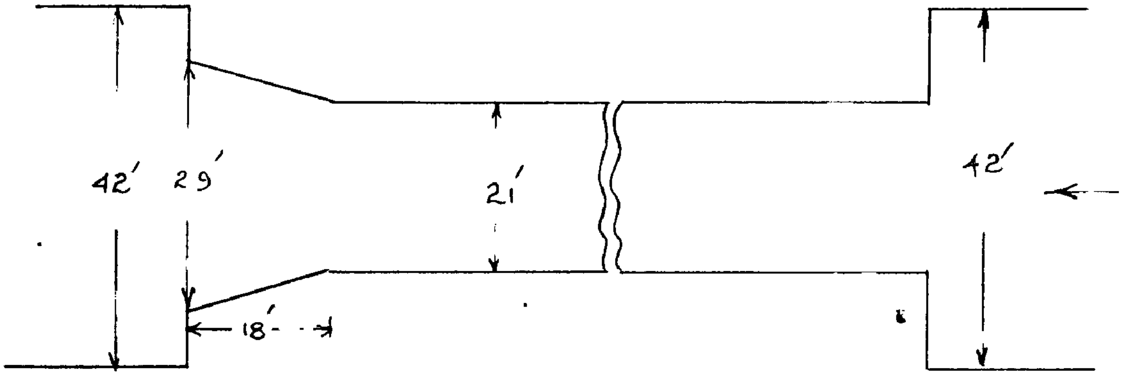
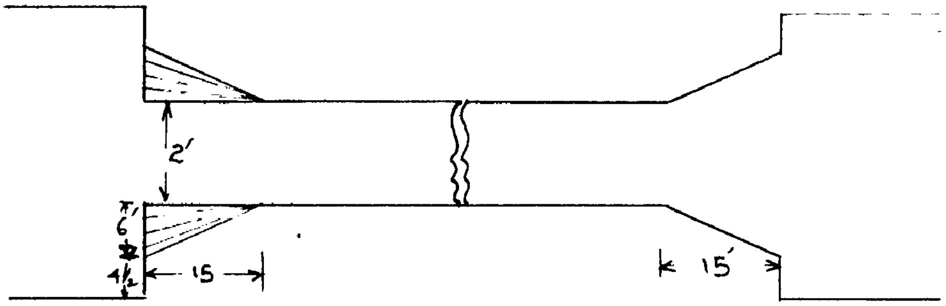

20'
V = 2 ft/sec
6.3
V. 8 ft
ANGULAR CHANGE IN WAT SURFACE
5'
Not level
V = 2 ft/sec
20'
Q = 200 cfs
V = 8.0
6.3'
SMOOTH WATER SURFACE CURVE
5'
NOT LEVEL
4'

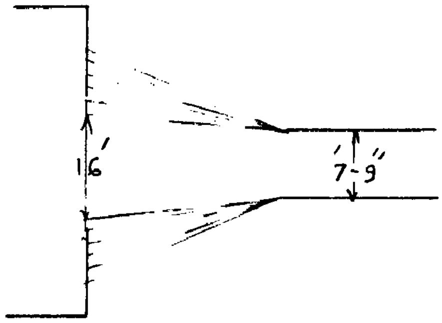
FIG. 9



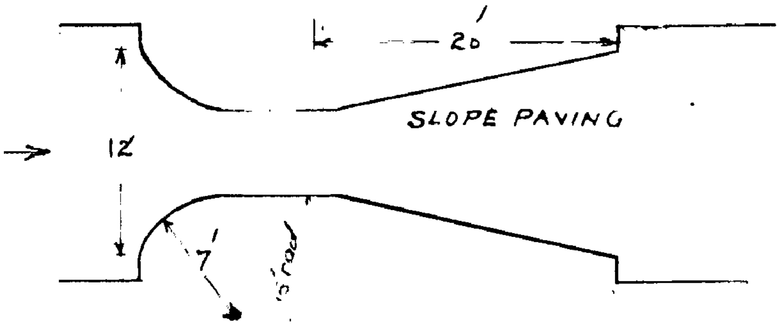
SIMPLE STRENGTH TRANSITION IN A RECTANGULAR FLUME



STRAIGHT VERTICAL WALL TRANSITION RECTANGULAR FLUME



STRAIGHT WARPED INLET OR OUTLET RECTANGULAR FLUME



CURVED VERTICAL WALL TRANSITION RECTANGULAR FLUME

FIG-10.

being the total drop and then adding the drop necessary to overcome friction. Knowing the water surface elevation in the canal at the upper end of the inlet, but temporarily neglecting friction, the water surface elevation of the flume at the lower end of the transition can be found, thereby determining the two ends of the water surface profiles. Not knowing the particular curve that fits the given conditions better than any other, any smooth curve tangent to the normal water surface in the canal and the flume may be used, either arbitrarily drawn or computed. One of the cases can be adopting two equal parabolas (neglecting friction), and they should be tangential to the water surface slope in both the canal and the flume.

Then choosing a number of sections, equally spaced for convenience, across the transition, the elevations of the water surface and the structural dimensions are computed. For the construction of the parabolas one has its vertex at the beginning of the transition, and the other at the end of the transition i.e. at the beginning of the flumed part. Half of the drop of the transition, is taken to be from one end to the middle of the transition. The drops to points between the beginning to the middle of the transition are proportioned to the squares of the distances from the beginning of the transition. Taking the beginning to the middle of the transition to be left, the drops for the right half of the surface curve are obtained by subtracting the corresponding drops for the left half from the total drop. Assuming that the conversion loss is distributed over the entire length of the transition in proportion to the change in velocity head, values of ΔH_v are obtained by dividing the computed values of ΔW_s by 1.10. The velocity head is obtained by adding ΔH_v to the velocity head at the beginning of the transition. The velocities corresponding to the assumed water surface are then determined and recorded. Friction loss being neglected, the area of the structure at that particular section can be computed.

3.3 (b) In the design of the flume outlet, the same procedure for the inlet is adopted here, but the only essential difference being that the conversion loss is subtracted from ΔH_v to obtain ΔW_s , where ΔW_s is the change in water surface elevations. The length of the outlet structure is determined on the same basis as the length of the inlet structure, but it is generally conceded that for equal efficiency an inlet may be shorter than an outlet, for the same velocity change. But inlets are often made the same length as outlets because -

1. Short transitions do not afford secure anchorage to the canal.
2. Using the same length makes the forms interchangeable.
3. Sharper warps are difficult to construct.
4. An average divergence of 12½ yields a structure of

pleasing and reasonable cost.

Also in the case of outlet an allowance of $0.2\Delta H_v$ is made for outlet losses. Where an allowance is made for a greater outlet less than is actually necessary, the destruction of the excess energy needs be considered. A recovery of head will actually occur at the outlet, whether provided for or not if the flume velocity is relatively low, the depth being greater than the critical depth.

Summarising the art of designing transition structures, as practised by the U.S.B.R., it can be said that:

1. Sufficient fall must be allowed at all inlet structures to accelerate the flow and to overcome frictional and entrance losses.

2. The theoretical recovery at an outlet structure is reduced by frictional and outlet losses.

3. At open channel outlets a small factor of safety may be obtained by setting the transition for less than its maximum recovering capacity, but erosion below the structure may be slightly increased.

4. Important structures, where velocities are high, must be carefully designed to conform to a smooth theoretical water surface. Sharp angles must be avoided.

5. No definite data as to the best form of water surface profile, best form of structure, or most efficient length of transition are available.

3.4 In 1940 Sri A.C.Mitra (unpublished note, Irrigation Department, Uttar Pradesh) came out with a new idea as to the design of the expansion portion of the transitions in which the water depth was constant. This design has been tried on various irrigation channels in Uttar Pradesh and has usually proved successful where F no was less than 0.6. In his analysis of the problem, he assumed the following.

1. There should be constant variation of velocity throughout the length of the transition.

2. There should be a constant depth of flow.

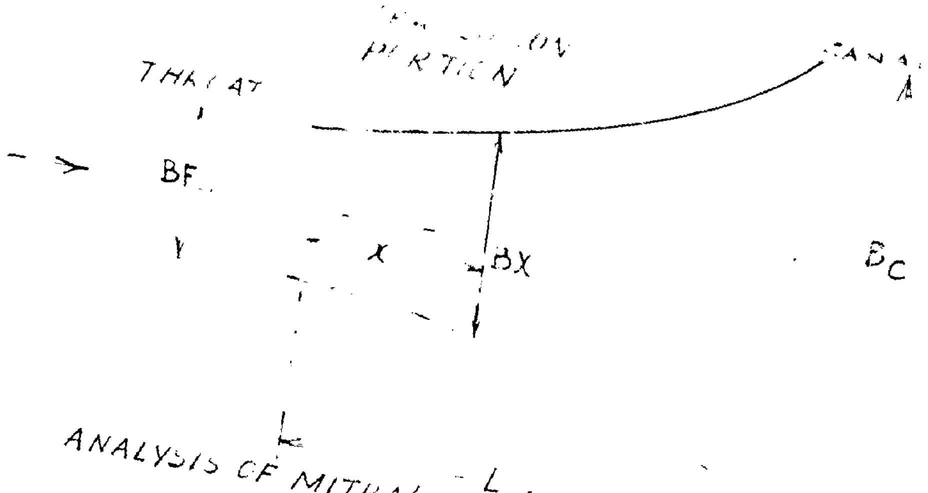
Refering to the figure 11

The length of the expansive transition must be fixed say by adopting U.S.B.R. Rule.

If V_f and V_c are the mean velocities at B_f and B_c and x is any axial distance from the throat where the velocity be V_x and the width is B_x. Then by the second assumption

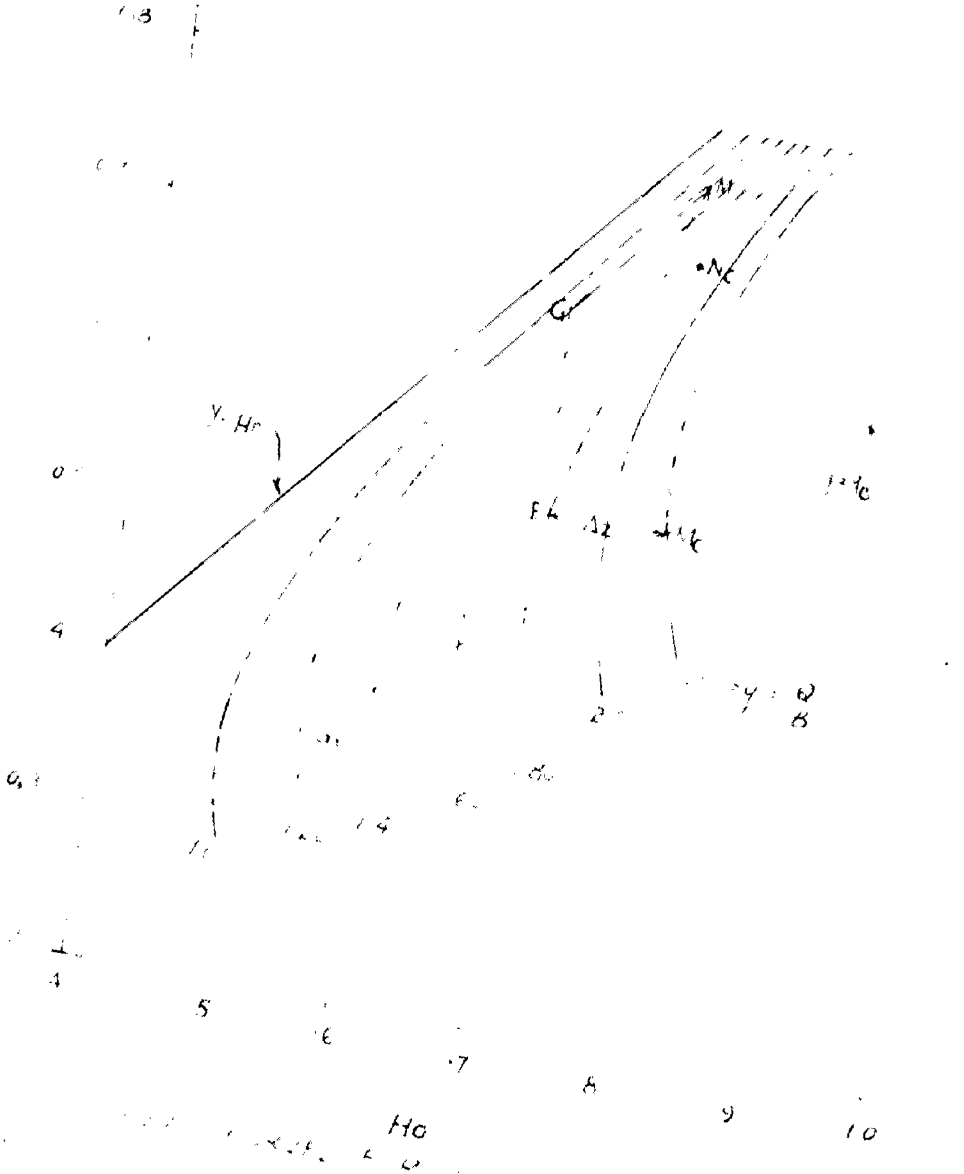
$$\frac{V_f - V_x}{x} = \frac{V_f - V_c}{L} \qquad \text{Eq. 3-3}$$

Then by the first assumption and by the continuity equation



ANALYSIS OF MITRA'S HYPERBOLIC EXPANSION

FIG. 11



$$B_f \times V_f = B_x \times V_x = B_c \times V_c = \frac{Q}{D} = K \qquad \text{Eq. 3-4}$$

$$\therefore \quad V_f = \frac{K}{B_f} , \quad V_x = \frac{K}{B_x} , \quad V_c = \frac{K}{B_c} \qquad \text{Eq. 3-5}$$

Now substituting the values in Equation it becomes

$$\frac{\frac{K}{B_f} - \frac{K}{B_x}}{x} = \frac{\frac{K}{B_f} - \frac{K}{B_c}}{L} \qquad \text{Eq. 3-6}$$

which reduces to

$$B_x = \frac{B_c . B_f . L}{L . B_c - x (B_c - B_f)} \qquad \text{Eq. 3-7}$$

As will be seen from Eq. 3-7 , the curve of the expanding side walls will be hyperbolic. As there is the possibility of the depth at the throat not being the same as in the normal section, he justified his assumption by depressing gradually the bed towards the throat to the extent $\left(\frac{V_f^2}{2g} - \frac{V_c^2}{2g} \right)$ and then sloping up towards the exit in the expansion transition.

3.5 Ippen 1950 (9) has broadly followed Hinds suggestions, in flume design. He considered a minimum of energy dissipation consistent with economy of construction to be the usual ~~gxidx~~ guiding principle of design. Under these conditions, the following assumptions were made.

1. The gradient of total head is constant over the transition length or the change in total head can be evaluated with sufficient accuracy from Mannings equations for uniform flow.

2. The velocity varies primarily as a function of distance and the factors α and β can either be considered unity or

be evaluated only for the end sections with changes apportioned gradually over the transition length.

3. Curvature effects are negligible, the pressure distribution is hydrostatic, and separation does not occur.

The specific energy curve was adopted as the tool for the analysis of such transitions, a series of these curves may be prepared for the given discharge and the various cross section involved in a particular design.

For example $H_o = y + \frac{Q^2}{2gA^2}$ can be plotted as shown in Fig. 12 . For a rectangular channel section in which the geometric change in channel shape is confined to charges in width, B, resulting in separate curves for each value of $\frac{Q}{B} = q$, being the discharge per unit length. Here A = By where as in general A is a more complex function of y depending upon the geometry of the local cross section.

The following information is usually available for the design of subcritical transitions.

(a) The geometric shape of upstream and downstream channels.

(b) The discharge with corresponding values of y and A for upstream and downstream channels.

(c) The elevation of the total-head line for the upstream section and with estimated adjustments, for all sections throughout the transition.

(d) The geometric type of transition as dictated by the economic importance of the structure, and hence the specific -head curve for any intermediate cross-section.

With the above information in hand, the arrangement of the proposed channel sections, can be found by one or the other of the following two methods.

1. A smooth surface profile is assumed between the end sections of the transition reach, thus fixing all intermediate velocity heads. This will in turn define a curve in the plot from the initial section to the final section since for each plotted curve corresponding to a geometric section. There is only one value of H_o and y. The value of y or H_o is read from the curve and plotted in the longitudinal section, and thus the elevation of the bottom is located.

2. A smooth bottom profile is assumed between the end sections. Thus all values of H_o along the transition are fixed. With these values of H_o the depth y can be determined from H_o:y plot for all stations along the transition for which such curves have been plotted.

In the first approach the resulting bottom curves, and in the second the resulting surface curves may not come out as continuous smooth lines. Both methods require re-adjustments, therefore, initial smooth lines are obtained by changing the longitudinal spacing for given geometric shapes of cross section by adjusting the geometry itself and finally by alternating, adjusting bottom and surface elevations. With the help of Fig. 12 the various existing possibilities for transition of decreasing channel cross-section may be discussed, for the rectangular channel form. A decrease in section or a channel contraction may in this case be reduced to two distinct geometric operations: a decrease in depth and a decrease in width. Starting with point M representing a given geometric and hydraulic situation a similar situation defined by point E, may be arrived out by the following methods.

The bottom may be continued at the original slope, thus keeping the specific energy H_o constant, while the channel width is decreased. The corresponding depth changes can be read from the intersections of the vertical time through M with successive curves for larger values of $\frac{Q}{B}$. After a desired width has been obtained at N , further changes in section may be produced by raising the channel bottom gradually while keeping the width constant. The differences in bottom elevation are simply deducted from the specific energy H_o or from its local value adjusted for loss due to resistance. The resulting values of the decreasing depth are read from the curve N to E. Consequently, following the path MNE will give smaller depth changes during a given reduction in width than can be obtained by proceeding from M first to G and then to E.

Successive changes of width and bottom elevation may naturally be combined into a single procedure represented by the dotted line connecting M and E, with simultaneous adjustments in width and bottom elevations. In general, the side contraction is best accomplished with concerned side walls while the depths are still larger. As such a line from M to E following path MNE rather than MGE will result in shorter contractions with smaller curvature effects. Contractions are easily designed, as long as both, points M and E are well in the subcritical range, since small surface slopes can be maintained. The more nearly point E approaches the critical depth, the steeper will be the surface curve.

The effect of boundary resistance is invariably to reduce the total head in the direction of flow, and it can be of the specific head. Thus, in subcritical contractions in general head losses are evidenced by slightly lower depths than these computed on the basis of negligible resistance. Such adjustment has been found satisfactory by measurement on good designs by the corps of Engineers and U.S.B.R.

On the other hand, for expansions, basic surface curves may be evaluated in reverse, the same procedure being applied with regard to geometry and specific head developments. However, there is one fundamental distinction in expansion flow which prevents as close agreement with the theory as that found for well designed contractions, and this is the difficulty of adequate recovery of Kinetic energy in expansions due to the different behaviour of the liquid near the Channel walls.

3.6 Ven Te Chow (10) has examined the flow conditions for sudden transitions in open channels. When the change in cross-sectional dimensions of a transition occurs in relatively short distance, it induces rapidly varied flow, and such transitions include sudden contractions and expansions vertically, horizontally or both. Considering the horizontal contraction as an example as in Fig. 13 .

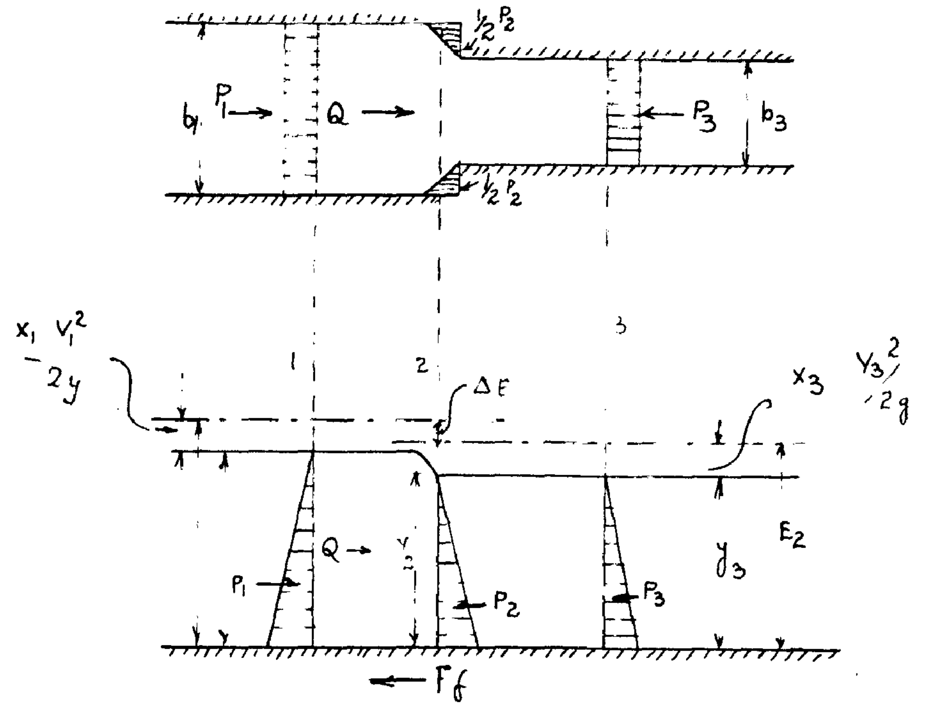
Applying the momentum equation to sections 1-1, 2-2, 3-3,

$$\frac{Q\omega}{g}(\beta_3 V_3 - \beta_1 V_1) = P_1 - P_2 - P_3 - F_f \qquad \text{Eq. 3-8}$$

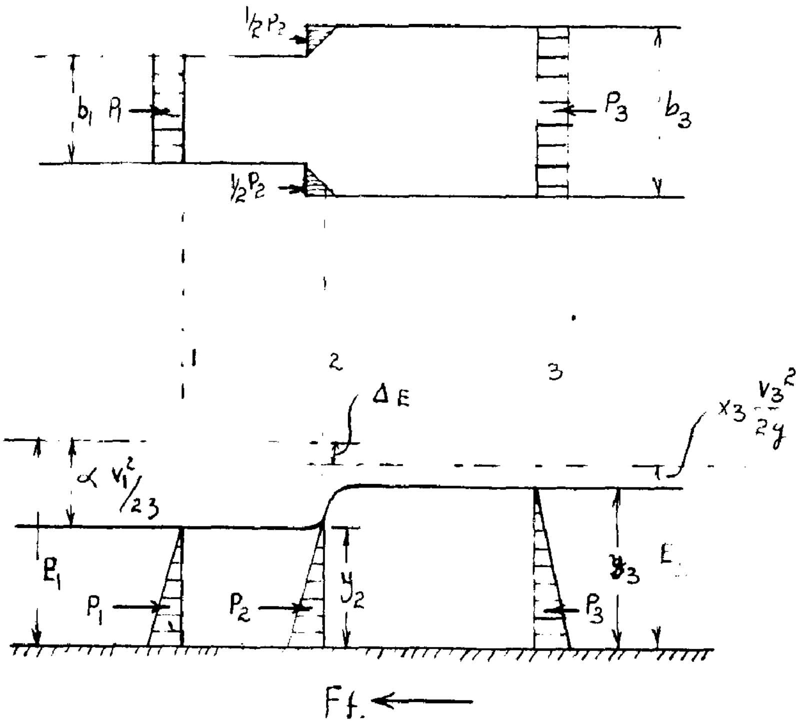
$$= \frac{1}{2}\omega b_1 y_1^2 - \frac{1}{2}\omega (b_1 - b_3) y_2^2 - \frac{1}{2}\omega b_3 y_3^2 - F_f \qquad \text{Eq. 3-9}$$

By the continuity equation

$$Q = v_1 b_1 y_1 = V_3 b_3 y_3 \qquad \text{Eq. 3-10}$$



SUDDEN CONTRACTION



SUDDEN EXPANSION.

FIG. 13.

For convenience in the theoretical discussion, the following may be assumed

$$F_f = 0 \, , \quad \beta_1 = \beta_3 = 1 \, , \quad y_2 = y_3$$

Under the above conditions the Equation 3(9) may be reduced to

$$F_1^2 = \frac{(y_3/y_1)\left[(y_3/y_1)^2 - 1\right]}{2\left[(y_3/y_1) - \left(\frac{1}{b_3/b_1}\right)\right]} \qquad \text{Eq. 3-11}$$

where $F_1^2 = \dfrac{V_1^2}{g y_1}$

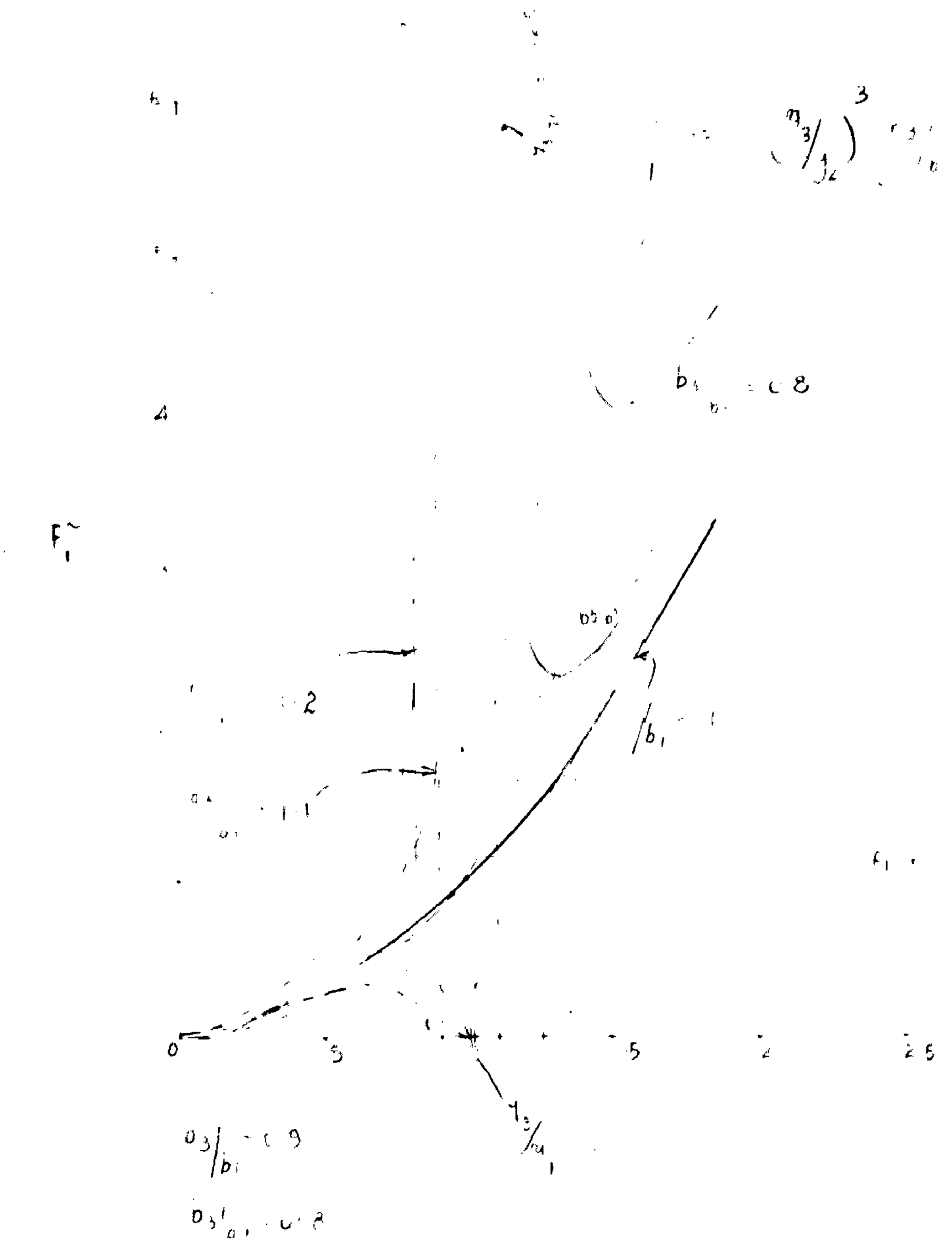
The above equation can be plotted using b_3/b_1 as a parameter. Having plotted the curves as shown in Fig. 14 the family of curves got are similar hyperbolas of a higher order, having the following characteristics.

1. The curves are considered only for positive values of F_1 and y_3/y_1 .

2. All the hyperbolas pass through the points ($F_1 = 0$, $y_3/y_1 = 0$), and ($F_1 = 0$, $y_3/y_1 = 1$) are asymtotic to the vertical line $y_3/y_1 = b_1/b_3$

3. In the special case of $b_3 = b_1$, indicating the hydraulic jump in a prismatic channel, represented by the curve.

$$F_1^2 = 0{\cdot}5\,(y_3/y_1)\left[(y_3/y_1) + 1\right] \qquad \text{Eq. 3-12}$$



PLOT OF F_1^2 AGAINST y_3/y_1 USING b_3/b_1 AS A PARAMETER

SUDDEN HORIZONTAL [illegible] ACTING
EXPANSION

FIG. 14.

4. The upstream flow is supercritical in the region above the horizontal line $F_1^2 = 1$ or $F = 1$ and subcritical below the line. The downstream flow is subcritical in the region below the curve

$$F_1^2 = (y_3/y_1)^3 (b_3/b_1)^2$$

and supercritical above it.

5. Theoretically speaking, certain parts of the curves represent flows that can not actually occur, because they necessitated an increase of energy or a negative energy loss, what is contradictory to the fact that the flow always losses energy in passing through a transition. The difference between the energies before and after the transition is,

$$\Delta E = y_1 + \frac{V_1^2}{2g} - y_3 - \frac{V_3^2}{2g} \qquad \text{Eq. 3-13}$$

Dividing through by y_1,

$$\frac{\Delta E}{y_1} = 1 + \frac{F_1^2}{2} - \left[y_3/y_1 + \frac{F_1^2}{2(y_3/y_1)^2(b_3/b_1)^2} \right] \qquad \text{Eq. 3-14}$$

By applying this equation to a certain part of the curve the flow can be found to be impossible if the computed value of ΔE is negative. It should be noted however that this discussion is intended to present a theoretical analysis of the phenomenon and to develop a classification of the flow through sudden transitions. In real problems the theoretically impossible flow may become actually possible because the [illegible] assumptions made in the above derivation may not be true under actual circumstances.

Under actual circumstances, β_1 and β_2 are not exactly equal to 1.0, and could lie between y_1 and y_3. Thus, the negative energy loss would become positive, and the theoretically impossible flow would become actually possible. The loss ΔE is a very small amount and can be readily changed to positive by a slight change in the items in the Eq. 3-14.

Similar analysis is applicable to sudden expansions as below.

Applying the momentum equation to sections (1-1), (2-2) and (3-3) of the Fig. 13 (b)

$$\frac{Q\omega}{g}\left(\beta_3 V_3 - \beta_1 V_1\right) = P_1 + P_2 - P_3 - F_f \qquad \text{Eq. 3-15}$$

Assuming $\beta_1 = \beta_3 = 1$, $F_f = 0$ and $y_2 = y_1$

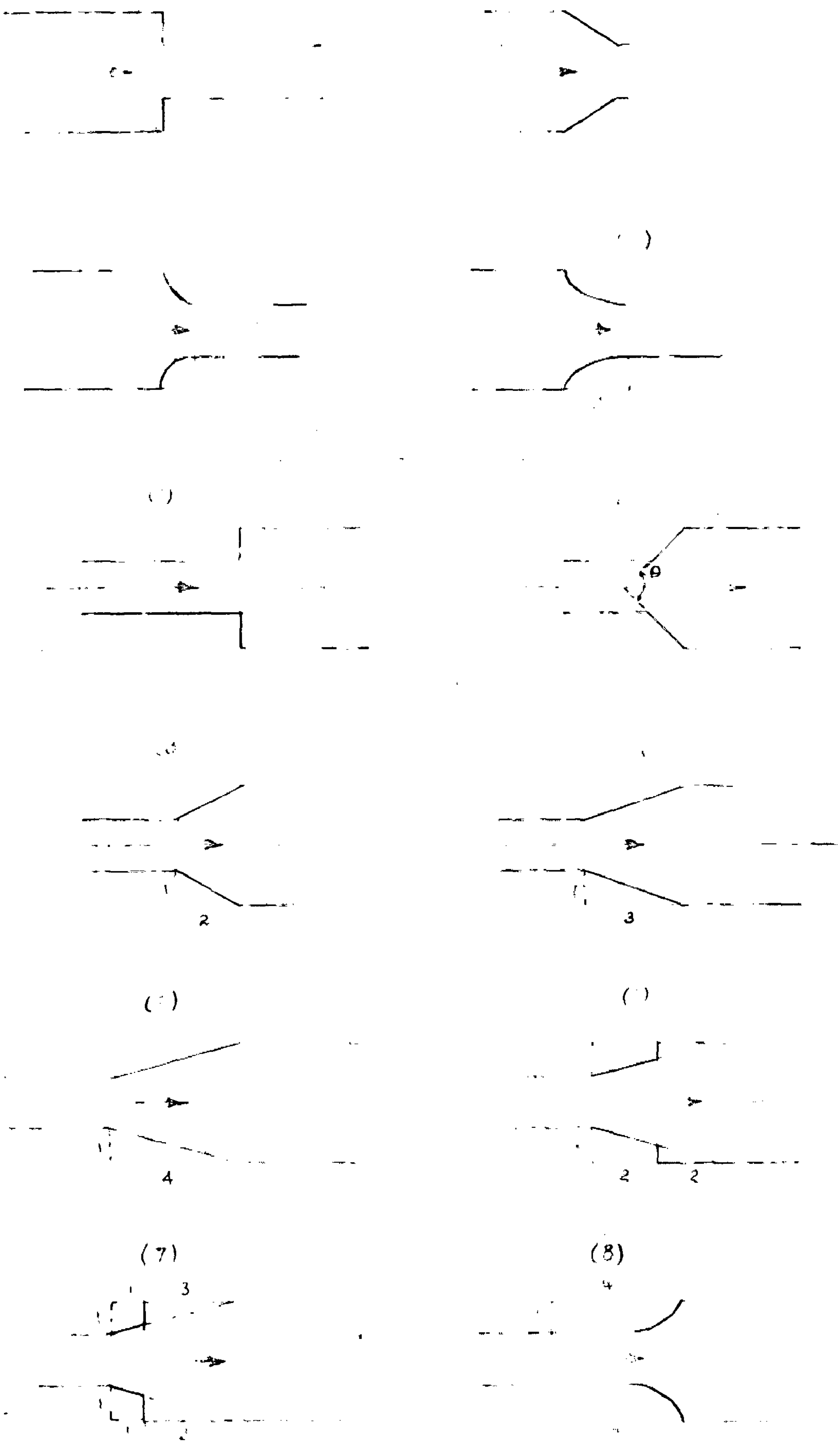
$$\frac{Q\omega}{g}\left(V_3 - V_1\right) = \frac{1}{2}\omega b_1 y_1^2 + \frac{1}{2}\omega(b_3 - b_1)y_2^2 - \frac{1}{2}\omega b_3 y_3^2 \qquad \text{Eq. 3-16}$$

By continuity $\quad Q = V_1 b_1 y_1 = V_3 b_3 y_3$

On substituting $\frac{V_1 b_1 y_1}{b_3 y_3}$ for V_3 in Eq. 3-16 and solving the above equation can be plotted as before using b_3/b_1 as a parameter. A family of hyperbolas similar to each other can be got, with the same characteristics as mentioned for the contraction.

3.7 Fornica (quoted by Ven Te Chow) thought of various designs for sudden transitions with a sub-critical flow through them, and he carried on some experiments on the designs shown in Fig. 15. The typical measured flow profiles and energy lines represent the specific energy $y + \alpha \frac{v^2}{2g}$. Near the section where the transition takes place, the velocity head can not be measured easily because of the turbulent conditions of flow; hence, the energy lines are simply extended. The vertical intercept between the extended upstream and downstream lines at the transition section represents the energy loss E. The asterisk shown for the sudden expansion indicates the position of the energy line assuming $\alpha = 1$.

Fig. 16 shows the computed values of the energy coefficient α at different sections of the channel for various design of transitions. The values are apparently very close to unity immediately after the sudden contractions, but are generally higher than unity after the sudden expansions. This indicates that the flow in a sudden expansion is irregularly diffused. The head losses for various designs of transitions at different discharges are shown Ref. 13 . It can be seen that in general the sudden contractions have higher losses than the sudden expansions. A process of conversion for potential to Kinetic energy is followed immediately by a process of recoversion from Kinetic to potential



SUDDEN TRANSITIONS

FIG-15.

MEASURED PROFILE AND ENERGY LINES THROUGH SUDDEN TRANSITIONS

[illegible] COEFFICIENTS NEAR SUDDEN TRANSITION energy. As a result much less energy than in a sudden expansion is recovered, as in a sudden contraction the flow is first contracted and then expanded. However, the energy loss in a sudden contraction of design I (Fig.15.) can be greatly reduced by modifying the sharp-edged corners of the entrance of the reduced channel, as in designs II to IV. The differences among these designs are evidently insignificant. The energy loss in a sudden expansion can be reduced by gradually enlarging the channel section or decreasing the angle of divergence (θ in Fig.15) but this advantage may be nullified by such modifications to those in designs 6 to 8. The length of the gradual enlargement of the expansion has a limit beyond which the gain in efficiency becomes insignificant. Adopting the energy loss in closed conduits to open channel being expressed by

$$E = \frac{KV_3^2}{2g} \qquad \text{Eq. 3-17}$$

and in a sudden expansion by

$$E = \frac{\epsilon(V_1 - V_3)^2}{2g} \qquad \text{Eq. 3-18}$$

Where K and are coefficients, and V_1 and V_3 are the upstream and downstream velocities, respectively, Fornica obtained the following average values of ϵ for sudden expansions.

Type or Design.	1	2	3	4	5	6	7	8
ϵ	0.82	0.87	0.68	0.41	0.27	0.29	0.45	0.44

According to the experimental data obtained by Fornica the values of K for sudden contractions seem to vary in a wide range, increasing with the discharge. The approximate median value of K for design I is 0.10 and for design II to IV 0.06.

3.8 In 1953 D.Doddiah divided transition structures into two groups for purposes of design.

1. Minor and low velocity structures and
2. Major and high velocity structures.
    a) Moderately big structures.
    b) Very big structures carrying several thousand cusecs.

The inlets and outlets for the first group of above look to be of secondary importance, and did not deserve much time for computations. The moderately big structures he adopted the U.S.B.R. method described previously.

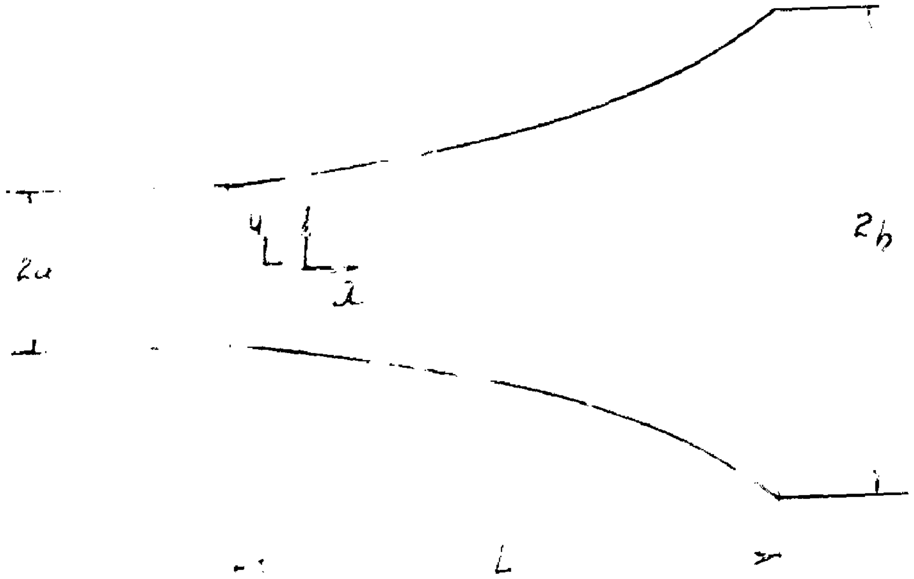
In the case of very big structures carrying several thousand cusecs, he considered the length of transition for the inlet to be too long to be economically feasible, if the angle formed by the line joining the two ends of water surface with the centre line joining the two ends of water surface with the centre line of structure taken to be about 12 30'. He suggested the length to be fixed such that the steepest angle that the water surface profile in plan makes with the centre line of the structure to lie between 26° and 30° for inlet transition and 21° - 30' and 27.0 for an outlet transition. Further, he proposed standard curves for the side walls which could be easily computed and which, according to him, would lead to almost as satisfactory conditions as those computed according to the method of Hinds.

3-9 In a recently published paper (12), Prof. R.S.Chaturvedi has examined Mitra's transition design theoretically as well as experimentally.

The analysis of his approach is thus given below. With reference to Fig. 17.

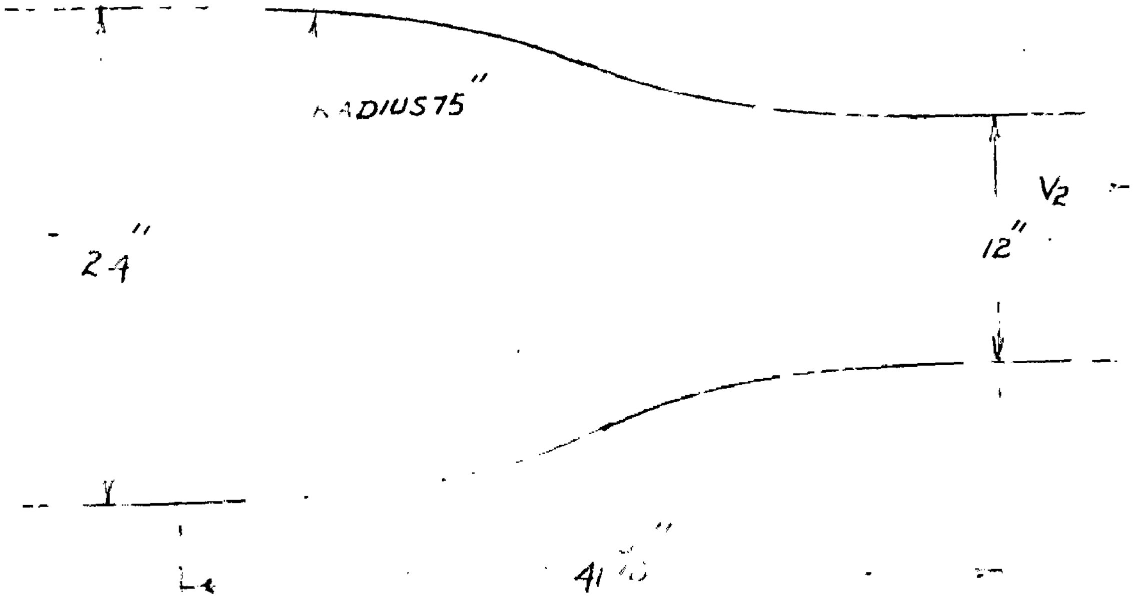
Let x - direction be along the axis of the flume
and y_L - direction be right angles to it in the plan at the bottom.

Adopting Mitra's second assumption that the depth should be constant, and by the continuity equation.



CHATURVEDI'S APPROACH

FIG - 17



CONTRACTION COMPOSED OF TWO CURVES.

FIG - 18.

is positive and increases directly with the divergence downstream. Mitra's curve could be obtained by the above procedure.

Now $\frac{dx}{dy_L}$ can be represented by $\frac{dx}{dt} \Big/ \frac{dy_L}{dt}$

But $\frac{dx}{dt} = \frac{c}{y_L}$, $\frac{dy_L}{dt} = c' y_L$

$$\therefore \quad \frac{dx}{dy_L} = \frac{c}{y_L} \Big/ c' y_L$$

$$= \frac{c}{c'} \times \frac{1}{y_L^2}$$

$$\therefore \quad dx = \frac{c}{c'} \cdot \frac{dy_L}{y_L^2}$$

If A is substituted for $\frac{c}{c'}$ and integrated we have

$$X = -\frac{A}{y_L} + B$$

Or $y_L(x - B) + A = 0$ Eq. 3-23

This is the equation of a hyperbola. Now evaluating the constants A and B the conditions are

$y_L = a$ when $x = 0$

$y_L = b$ when $x = L$

Hence $b.(L - B) + A = 0$

and $a.(0 - B) + A = 0$

which gives $B = \frac{L.b}{b - a}$

$$V_x \cdot y_L = \text{constant}$$ Eq. 3-19

$$\frac{dx}{dt} = V_x = \frac{c}{y_L}$$ Eq. 3-20

Assuming that

$$\frac{dV_x}{dx} = -C'$$ (negative because velocity is decreasing) Eq. 3-21

Hence $$\frac{dV_x}{dx} = \frac{d\left(\frac{c}{y_L}\right)}{dx} = \frac{d\left(\frac{c}{y_L}\right)}{dt} \Bigg/ \frac{dx}{dt} = -c'$$

Expanding the latter portion

$$\frac{d\left(\frac{c}{y_L}\right)}{dt} = -c'\frac{dx}{dt}$$

$$-\frac{c}{y_L^2}\frac{dy_L}{dt} = -c' \cdot \frac{dx}{dt}$$

$$\therefore \frac{dy_L}{dt} = +c' \cdot \frac{dx}{dt} \cdot \frac{y_L^2}{c}$$

✓ By equation 3-20

$$\frac{dy_L}{dt} = c' \cdot \frac{y_L^2}{c} \cdot \frac{c}{y_L} = c' y_L$$ Eq. 3-22

Hence $$V_{y_L} = \frac{dy_L}{dt} = c' y_L$$

$$\therefore \frac{dV_{y_L}}{dx} = \frac{d^2 y_L}{dt^2} = \frac{dy_L}{dt} \cdot \frac{d(V_{y_L})}{dy_L}$$

$$= c' y_L \cdot c'$$

$$= c'^2 y_L$$

which shows that acceleration along y_L or lateral acceleration

incomplete ? ?.

matter missing

and $$A = \frac{L \times a \times b}{b - a}$$

?

Substituting for A and B in Eq. 3-23 gives

$$y_L = \frac{L \cdot a \cdot b}{Lb - x(b - a)} \qquad \text{Eq. 3-24}$$

The above result if multiplied by 2 gives identically the same as Mitra's equation.

Taking into consideration the lateral acceleration and the influence of the lateral velocity, he thought that problem could be approached in a more general manner, by taking the lateral velocity as a more general function of y_L.

$$\frac{dy_L}{dt} = V_{y_L} = f(y_L) = Ky_L^n \qquad \text{Eq. 3-25}$$

Where "n" can be integral or fractional, positive or negative and K is any constant. Having obtained similar other curves their hydraulic properties can be studied on models. He carried on to derive and analyse some other forms of transition curves, by giving different values to n in the general equation

$$f(y_L) = \frac{dy_L}{dt} = Ky_L^n$$

He choses the values

$$-1, -\tfrac{1}{2}, 0, \tfrac{1}{2}, 1, \tfrac{3}{2}, 2$$

Now by the continuity equation

$$\frac{dx}{dt} = \frac{c}{y_L}$$

and the general equation already derived is

$$\frac{dy_L}{dt} = K y_L^n$$

But $$\frac{dx}{dy_L} = \frac{dx}{dt} \Big/ \frac{dy_L}{dt}$$

which gives $$\frac{dx}{dy_L} = \frac{c}{y_L} \cdot \frac{1}{K y_L^n}$$

$$\therefore \quad dx = \frac{c}{K} \cdot \frac{1}{y_L^{n+1}} \, dy_L \qquad \text{Eq. 3-26}$$

On solving the above differential equation 3-26 with $n \neq 0$

We have

$$x = \frac{c}{K} \cdot \frac{y_L^{-n}}{n} + A \qquad \text{Eq. 3-27}$$

or $$x = B \cdot \frac{y_L^{-n}}{n} + A$$

When n = 0

$$dx = \frac{C}{K} \frac{dy}{y_L}$$

$$x = B \log y_L + A \qquad \text{Eq- 3.28}$$

By giving different values of "n" in Eq. 2-46 different equation for curves can be got. The values of the constants of Eqs. 3-27 and Eq. 3-28 can be evaluated with the help of the following boundary conditions.

$y_L = 0$ when $x = 0$

$y_L = b$ when $x = L$

Table 1 gives the general equations of the seven curves derived. He worked out the lateral accelerations as follows.

Denoting a_{y_L} as lateral acceleration i.e. $\frac{d^2y_L}{dt^2}$

We have

$$a_{y_L} = \frac{d^2y_L}{dt^2} = \frac{dV_{y_L}}{dt} = V_{y_L} \cdot \frac{dV_{y_L}}{dy_L}$$

and with $V_{y_L} = K \cdot y_L^n$

$$a_{y_L} = V_{y_L}(nKy_L^{n-1}) = K^2ny_L^{2n-1} \quad \text{Eq. 3-29}$$

These accelerations are also included in Table 1.

From the table it can be seen that if n become -1, it yields a straight line transition with negative acceleration which may indicate a possible central jetting action or even one sided flow, with the exception of n = 0 gives positive accelerations which increase with the increase of the value of "n".

Considering the criterion for transition efficiency an element in the canal circuit, he compared the output energy at the transition exit to the energy entering the transition, but found out to be misleading as if the jetting takes place and there is no appreciable conversion of

Kinetic energy into potential energy; the efficiency criterion was, therefore, taken as the ratio of the recovery head to the total Kinetic energy.

$$\eta = \frac{\rho g (h_1 - h_0)}{\frac{\rho}{2} \int_{A_0} U_0^3 \, dA}$$

After conducting some experiments on models, he found that the best geometrical shape for sub-critical expansive transition is given by the formula

$$x \cdot = \frac{B}{y_L^{3/2}} + A \qquad \text{Eq. 3-30}$$

Eliminating the constants by using the same boundary conditions in Equation 2-49

$$x = \frac{L . b^{3/2}}{b^{3/2} - a^{3/2}} \left(1 - \frac{a^{3/2}}{y_L} \right) \qquad \text{Eq. 3-31}$$

DERIVED CURVES, THEIR EQUATIONS AND CHARACTERI[illegible] OF FLOW

S. NO.	EQUATION DERIVED CURVE.	EQUATIONS OF THE TRANSITION FOR THROAT = 2a CANAL WIDTH = 2b LENGTH = L	VALUE OF "n" IN $V_{y_L} = K y_L^n$	LATERAL VELOCITY $V_{y_L} = \frac{dy_L}{dt}$	LATERAL ACCEBRA $\alpha_{y_L} = \frac{d^2 y_L}{dt^2}$
1.	$x = By_L + A$	$x = \frac{L}{b-a} \cdot y_L - \frac{aL}{b-a}$	-1	$\frac{K}{y_L}$	$-\frac{K^2}{y_L^3}$
2	$x = By_L^{1/2} + A$	$x = \frac{L}{\sqrt{b}-\sqrt{a}}(\sqrt{y_L} - \sqrt{a})$	$-1/2$	$\frac{K}{y_L^{1/2}}$	$-1/2 \frac{K^2}{y^2}$
3.	$x = B\log y_L + A$	$x = \frac{(\log y - \log a)}{\log_L b - \log a}$	0	K	0
4	$x = \frac{B}{\sqrt{y_L}} + A$	$x = \frac{L\cdot\sqrt{b}}{\sqrt{b}-\sqrt{a}}(1 - \sqrt{a/y_L})$	$1/2$	$Ky_L^{1/2}$	$1/2 K^2$
5.	$x = B/y_L + A$	$x = \frac{Lb}{(b-a} - \frac{L\cdot a\cdot b}{(b-a)y_L}$	1	Ky_L	$K^2 y_L$
6.	$x = \frac{B}{y_L^{3/2}} + A$	$x = \frac{L\cdot b^{3/2}}{b^{3/2} - a^{3/2}}\left[1 - (a/y_L)^{3/2}\right]$	$3/2$	$Ky_L^{3/2}$	$3/2 K^2 y_L^2$
7.	$x = B/y_L^2 + A$	$x = \frac{Lb^2}{b^2 - a^2}\left[1 - \frac{a^2}{y_L^2}\right]$	2	Ky_L^2	$2K^2 y_L^3$

TABLE – 1

CHAPTER 4.

TRANSITIONS IN SUPERCRITICAL FLOW

4.1 In the case of supercritical flow in channels, small changes in the channel geometry cannot have appreciable influence on the flow upstream from the section at which such changes take place. Instead boundary changes cause a lot of surface disturbances of considerable magnitude which cross and recross as standing waves. It will be very difficult to have approximate constant velocity and depth. In the case of subcritical flow specific energy could be one of the means of designing, but over here with the appearance of standing wave there is not the likelihood of using the specific energy as a tool. In steep chutes, depth measured normal to the bottom can no longer represent the pressure head on the bottom. With high velocity heads, vertical curvature of flow may produce dynamic pressure on the bottom greatly in excess of the hydrostatic pressure. In long transitions an entrance will cause bulkling of the flow and such depths will have to be derived for water-air motion rather than for the water discharge alone.

4.2 The accent of design in the case of supercritical flow is shifted to the reduction or eventual elimination of the standing wave patterns which appear as a result of such flow. Here also cut and try method can be the sole tool of design, though being a little tedious. The aims of design can be said to be having a minimum surface disturbance.

The theoretical basis of design of contraction thought of by Ippen and Dawson (13) is supplied by the principles of non-uniform flow applied successively to short channel sections. The variation velocity and depth induced by the converging boundaries are assumed to occur only along the channel axis. Basic surface curves identical for all longitudinal sections are derived in first approximation by assuming that the total head remains constant, and surface curves are obtained by considering the friction losses in the direction of flow. The velocities and depths at any station or section are assumed to be unaffected by the curvature of the lateral boundaries and hence, constant in transverse sections. The primary aim of economical design being minimum of energy loss because of standing wave occurrence. Velocities in supercritical flow will vary in magnitude and direction in a systematic friction in transverse sections, and surface elevations will be constant. The effect of a transition is not confined to the immediate vicinity of the structure as in subcritical flow, but may affect the flow conditions downstream from the transition for very long distances.

4.3 In Lehigh University and Massachusettes Institute of Technology Ippen and Dawson attacked the flow through contraction, and it was a cut and try method. A curve was chosen as one that might be typical of a contraction, designed from conventional knowledge of open channel flow. The converging arcs in the upstream part and the reverse arcs in the downstream part of the contraction were chosen to be equal and with a 16° central angle each. The nature of the curves gave the radius of curvature of 75" and the length shown. The channel contraction was 2 ft. wide and its upper end to 1 ft. wide in longer reach of 30' below the contraction. Flows with Froude numbers of 2 to 8 were used for the experiment. It was found that there was no disturbance at the centre in almost all the cases. Positive lines tending to converge originate along the first or concave part of the wall and the diverging or negative lines emanate from the inverse part of the contraction.

4-4 A straight wall contraction was considered also. The aim of rational design for supercritical flow must be oriented first towards lower standing waves. Secondly, it can be towards reduction or possible removal of standing waves patterns in the channel section downstream of the contraction. It was found that the total deflection angle Θ determine the wave height, regardless of the degree of curvature of the side wall. It was found logical, therefore, to decrease this angle to a minimum. The minimum angle Θ is lead by connecting the upstream and downstream tangent points by straight lines possibly rounding the corners slightly for the sake of appearance. The contraction composed of circular arcs, 16^o central angle, and for a straight wall 8^o deflection angle. The experiments conducted on the above showed that in supercritical flow straight-wall contraction are better than curved wall contractions from the stand point of maximum wave height and compared on the basis of equal length. Also for given reductions in channel width correct deflecting angles may be found, which will result in minimum disturbances in the downstream channel.

4-5 It was found out theoretically that (1) The straight-wall contractions are always better than the curved wall contractions from the stand point of maximum wave height and compared on the basis of equal length, (2) For given reductions in channel width, correct deflection angle θ may be found which results in minimum disturbances in the downstream channel.

4-6 Hunter, Rouse, B.V.Bhoota, En-Yu-Hsu (6) in the design of expansion channels under three sequent heading for their design.

1. Surface configuration at abrupt expansion.
2. Efficient curvature of expanding boundaries.
3. Elimination of disturbances at the end of transitions.

As in other problems of steady flow in an open channel of non-uniform cross section, the variation in velocity and depth through a channel expansion will depend on the geometry of the channel boundaries, the rate of flow and the final properties. Under boundary geometry must be considered the form of the channel walls, the slope and form of the flow and the surface roughness of the floor and walls. In supercritical flow since the Froude number is greater than 1 , the problem of design and evaluation becomes one of gravity wave analysis in this expansive flow, since each increment of the boundry deflection may be considered to generate an incremental surface wave which crosses the flow at an angle depending upon the Froude number and the boundary form; only through determination of the cumulative effect of all such waves may the depth and velocity at each and every point be predicted. Sloping walls were to be avoided in non-uniform high velocity flow because of their tendency to exagerate surface disturbances. The method of design thought of could be of cut and try method, and this can be done graphically.

Only two factors in actuality tend to limit the graphical method in its use for the complete design of well proportioned expansion.

Firstly the assumption of boundary geometry to determine the best form of transition involves tedious process of try and error. Secondly, if the hydraulic jump is to form at the end of the expansion, the method of specific energy offers no clue as to the inherent stability of the phenomenon. As a matter of fact the formation of hydraulic jump in a transition may lead to an asymmetric pattern of flow within the transition which is wholly unpredictable.

4-7 Experiments were carried out at the Hydraulic Research of the State University of Iowa on abrupt expansive transition gave the clue as to the nature of the suitable boundary curve. Though there are rules pertaining to expansion transitions that the angle of divergence in supercritical flow can be $\Theta = \tan^{-1}\frac{1}{6}$ to $\theta = \tan^{-1}\frac{1}{9}$ regardless of the depth and the velocity of flow. In the case of the abrupt expansion transition, the boundary equation eventually found to be most satisfactory was of the form

$$\frac{y_L}{b_1} = \frac{1}{2}\left(\frac{x}{b_1 F_1}\right)^{3/2} + \frac{1}{2}$$

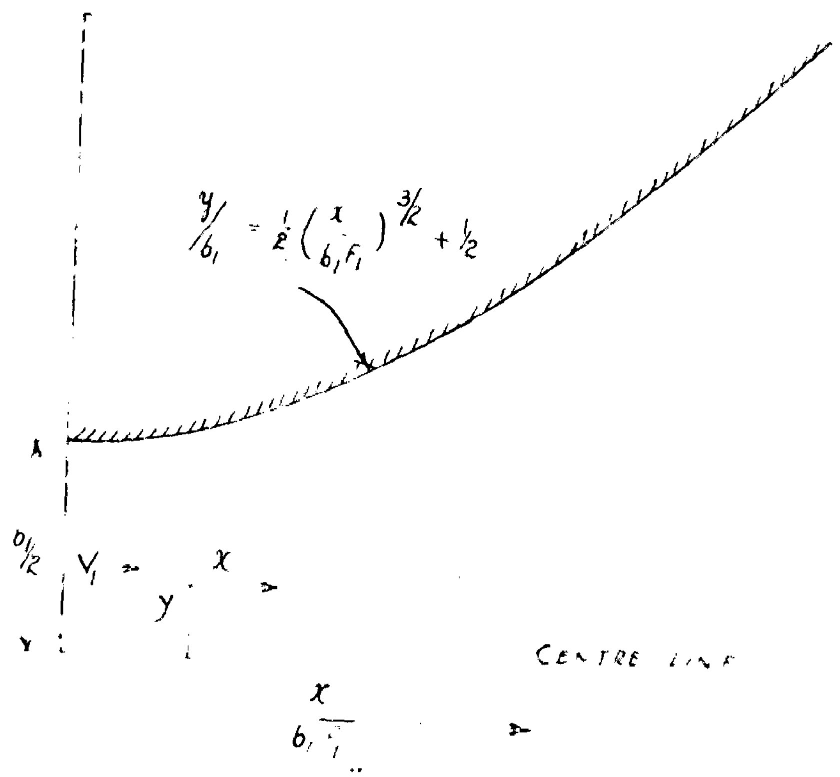
where

b_1 = width of the entrance.

y_L = lateral distance from longitudinal axis.

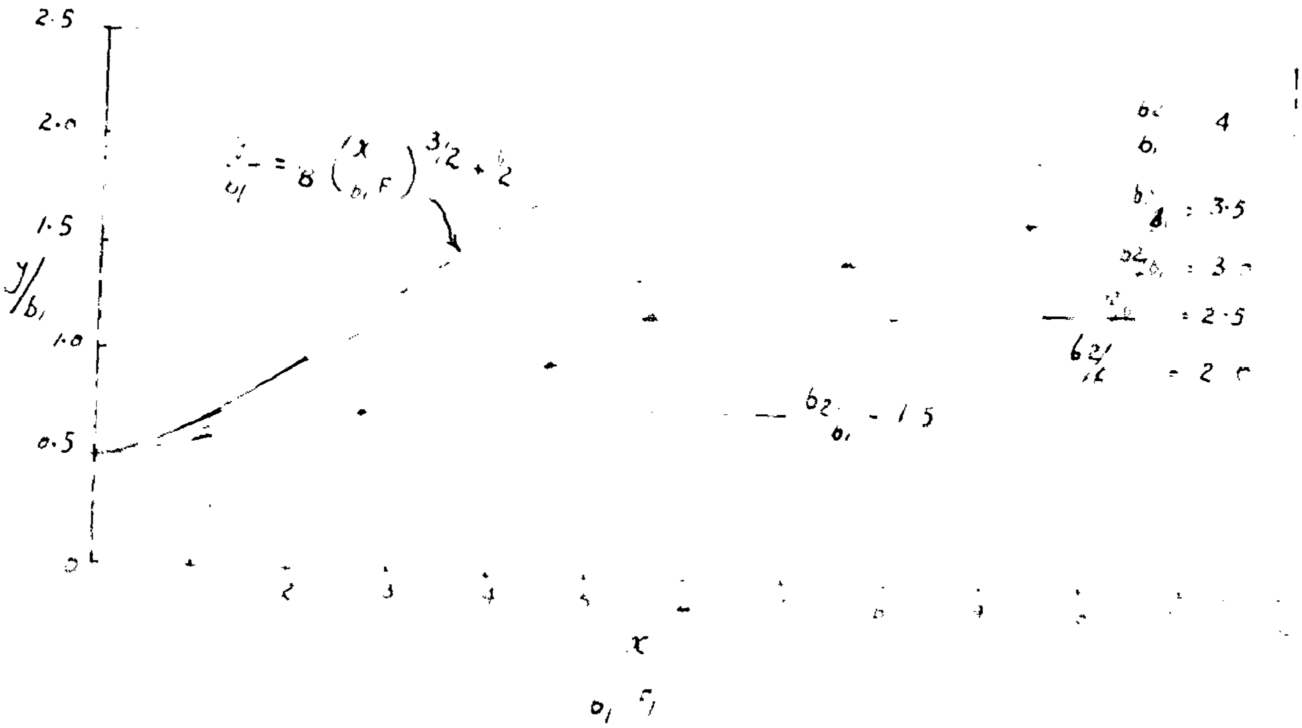
F_1 = Froude number at the exit.

x = Longitudinal distance from entrance.



HALF SECTION [illegible] AN [illegible] EXPANSION IN TRANSITION ACCORDING TO THE EQUATION ABOVE.

FIG. 19.



BOUNDARY CURVE WITH FLUMING RATIOS

FIG-20.

By interpolation from a series of solutions for various Froude numbers and expansion ratios by the method of characteristics, a generalised series of boundary curves for successively greater expansion ratios were obtained and presented in fig. 20. These curves are to be regarded merely as guides in the preliminary design for the following reasons.

1. Each of the curves represents the average form of several somewhat different curves for different Froude numbers.

2. The outlet depths as much as from 20% to 40% are in excess of that indicated by the simple wave theory even though the flow was essentially uniform at the exit.

A drop at the beginning of the parallel section at the end of the expansion when provided may stabilize the jump formation, though the change should depend on the Froude number. The actual magnitude of the drop is to be determined by the momentum equation.

CHAPTER 5.

EXPERIMENTS AND RESULTS.

The experiments were carried out in the laboratory in an 80' long a cement-plastered open channel 3' wide x 2' high. The bed of the channel was horizontal. The up-stream portion of the channel had a glass wall which stretched to a distance of 8', and by means of this the reverse flow downstream of the abrupt transition could be accurately detected.

The discharge was measured by means of a sharp crested weir, placed in another channel running parallel to the experimental channel, but at a lower elevation.

The weir was located sufficiently downstream from the tail end of the experimental channel to even out disturbances. However, there was some unavoidable fluctuation in discharge during the experiments as an independent feed line was not available for the work.

The levels of the water surfaces and the bottom or bed of the channel were measured with point gauges with a least count of .001 ft that were mounted on two rods, joined together by wooden blocks that could slide on the cement mortared top surfaces of the walls.

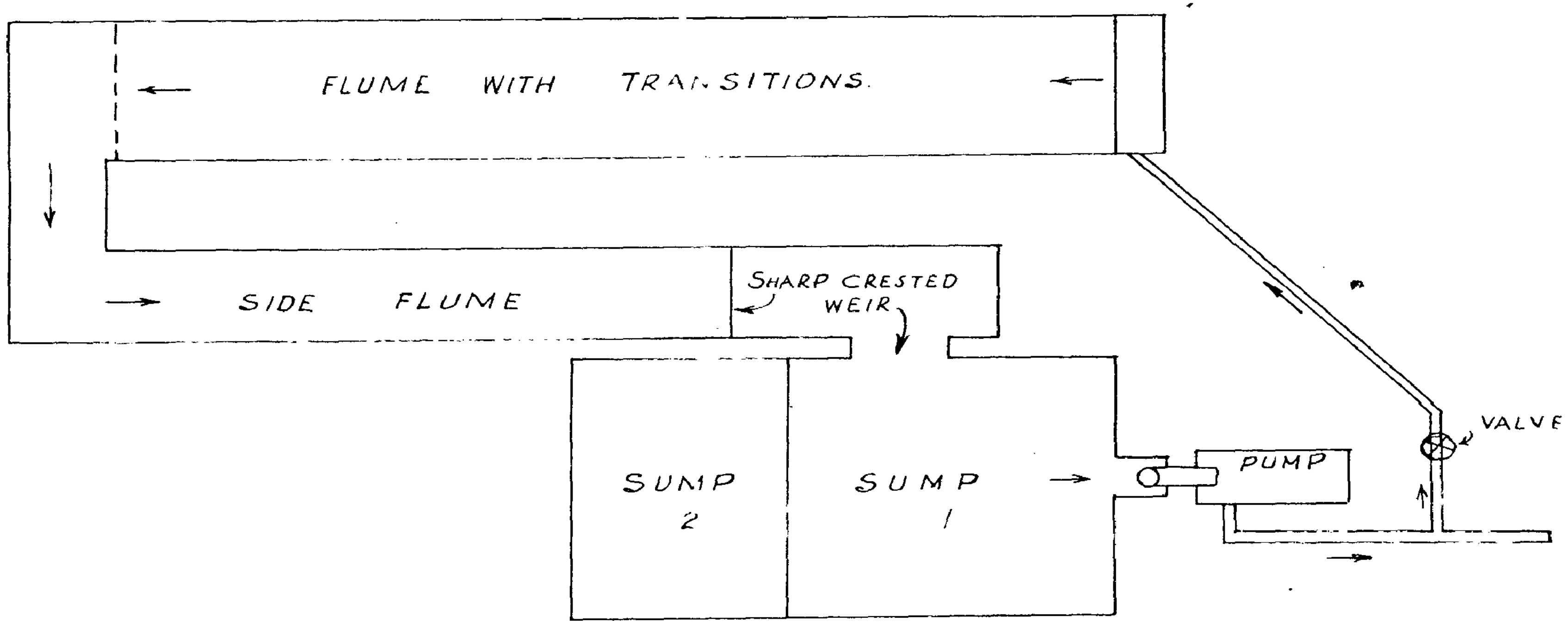
To plot the stream lines in the abrupt section (down stream of the throat), a grid of 6" intervals was put on the top of the wall, covering a distance of six feet downstream of the abrupt transition; and a woollen thread with one end tied to a glass rod was made to float downstream. The stream lines were plotted after the flow had been stabilised, though it was still liable to sway from one side of the channel to the other.

The velocity measurements were taken with a Pitot tube. They were taken at different depths, and traversely with 6" spacing and by means of plotting the iso-vels at different section the average velocities at those particular sections were got. Velocities were taken at the upstream and downstream of the throat, also at the contracted and expanded portions of the transitions.

The boundary pressures were read on a board on which open manometer tubes were fitted.

To consider the scouring effect of these transitions, a sand bed of width 3' and length 3' was made at the end of each of the transition. In the case of the abrupt transition, the bed was at a distance of 5' from the end of the throat. The scour patterns were drawn for each run. The scour readings were taken with a point gauge with a small plate piece attached to the end of the pointer

LINE DIAGRAM OF EXPERIMENTAL SET UP



FIG— 21.

in order that the pointer might rest on the bed without it penetrating deep into the sand bed.

The calibration of the measuring weir was done with the help of Rehbock formula.

$$K = 3.25 + \frac{1}{60H-.56} + 0.428 \frac{H}{P}$$

where

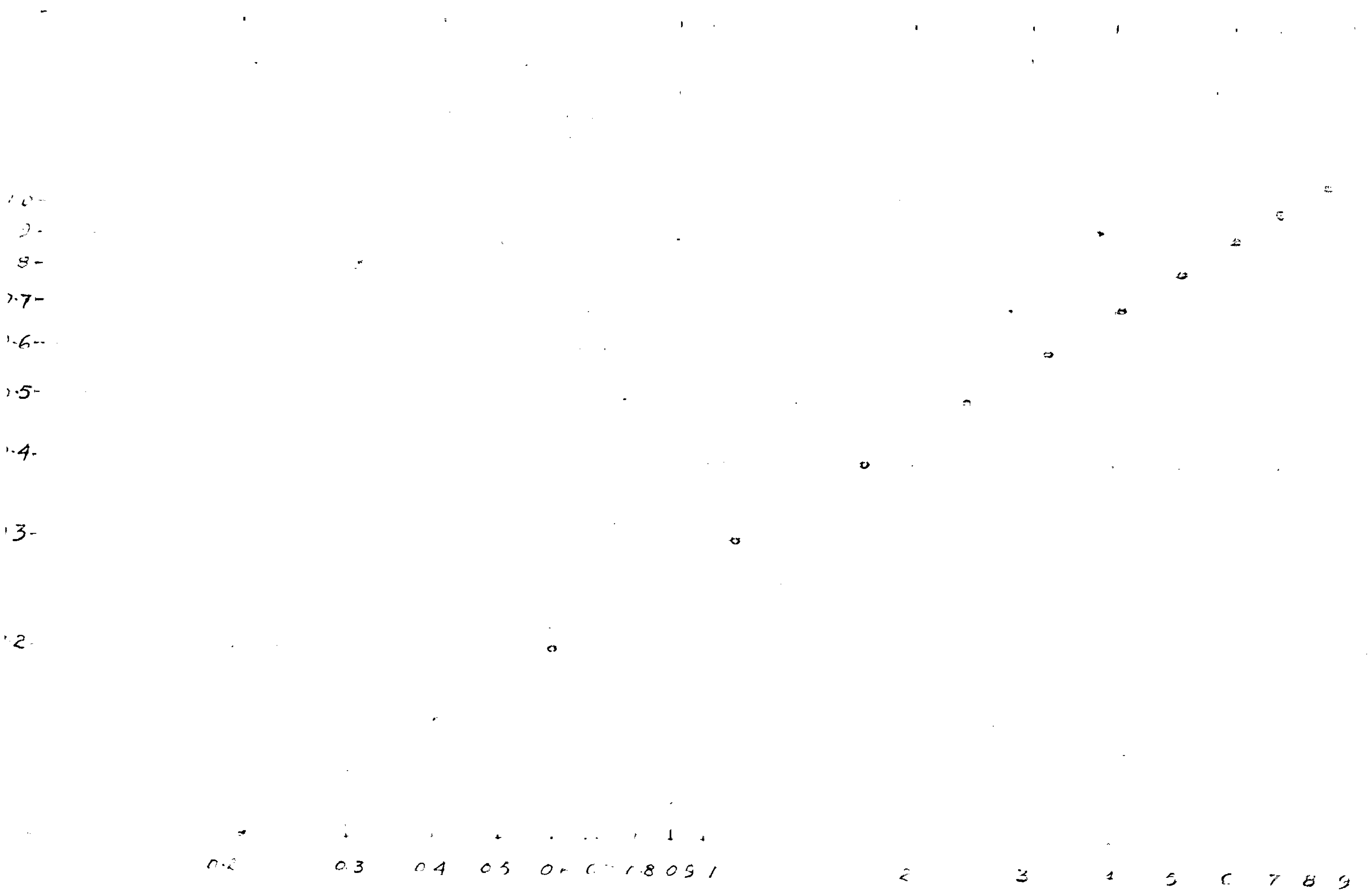
L is the crest length,

P is the height of the crest and,

H is the depth of the nappe.

The plot was done on a double logarithmic (2 cycles by 2 cycles) paper, with Q as absisca and H the ordinate as shown in Fig.22. The discharge was calculated with the sharp crested formula

The readings were all recorded and are attached in the appendix given at the end of this chapter. In order to compute the losses in the transitions, the internal velocities at each section must be calculated. To calculate the actual velocities for each section, iso-vels were drawn for each section from the u/s of the throat where the contraction starts to the d/s where the expansion ends.


1.0
.9
.8
.7
.6
.5
.4
.3
.2
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
2
3
4
5
6
7
8
9

The various sections of the transitions where readings were taken are shown in fig. 24 . The scale used for plotting the iso-vels in the expansion region is double that at the contracted portion as the study is more on expansion. In order to work out the energy content at each section and hence the loss of energy through the transition, it was necessary to determine the actual velocity energy of the flow at the sections. This was done by plotting iso-vels from the velocity measurements during the experiments. The summation of the products of the region or area between two isovels and the mean of the isovel values, divided by the total sectional area gave the actual mean velocity. The total energy line is got by the addition of the velocity head, and the depth to the bottom elevation. Three discharges, 1.0 cusecs, 0.7 cusecs and 0.35 cusecs were used with each of the three transitions studied. The various energy lines corresponding to various discharges are shown in figs. 25 a,b,c.

On plotting the energy lines, the author found that the abrupt transition gave very high losses for the discharges of 0.7 cusec and 1 cusec. The straight expansion (U.S.B.R.) as compared to the Hyperbolic expansion Mitra gave a little higher loss. The losses for each of the above transitions were not found to be appreciably different for the lowest discharge of 0.35 cusecs given in table below

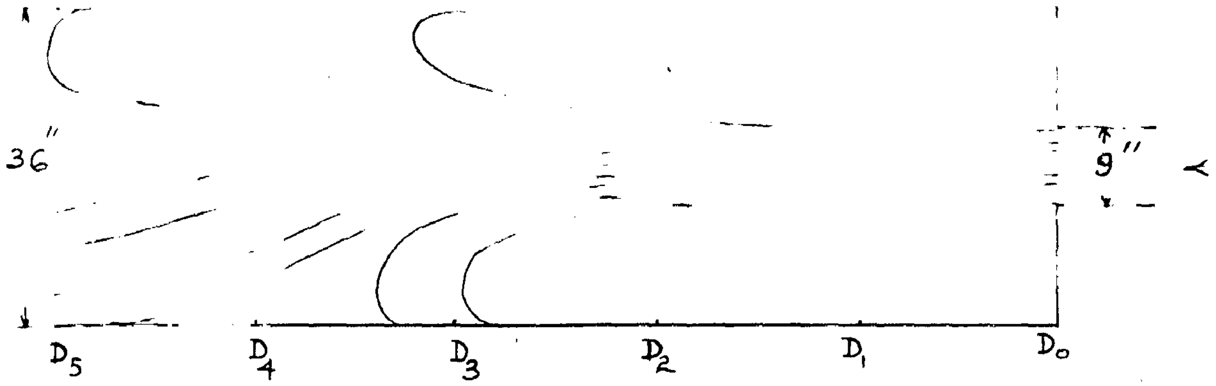
Discharge cusecs	Energy loss in ft. of water.		
	Abrupt	Straight (U.S.B.R)	Hyperbolic
0.35	.0406	.0476	.0399
0.70	.0721	.0366	.0249
1.00	.0717	.0352	.0264

It therefore appears that for very low discharges the design of transition is not so important and any of the above can be used. On the other hand for the discharge 0.7 and 1.0 cusec, there is appreciable difference in the losses.

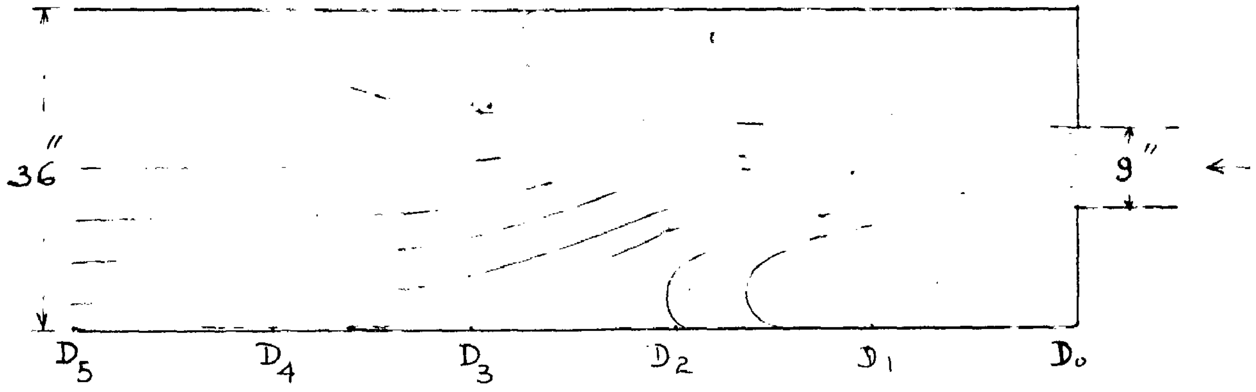
As expected, there were large areas of back flow for the abrupt transition. These can be seen in Fig. 23. In the case of the straight (U.S.B.R.) and Hyperbolic (Mitra's) transition the back flows were not prominent. For the range of discharges in the present experiments the hyperbolic transition gave no sign of separation. The Froude's number calculated at the throats of the various transitions varied from .527 to .91; thus indicating that the flows were well within the subcritical range.

As already mentioned, the study on the effect of the expansion transition on the bed material was also studied. The particle size used for the lower discharge of 0.35 cusec was 0.18 m.m. and for discharges of 0.7 and 1 cusec 1.06 mm. The scour patterns for the various transitions (expansion) for each discharge are shown in fig. 26 . The flow in the Abrupt transition (expansion) was non-symmetrical and so the scour is seen to be much concentrated on the side to which the flow lines sway. The reason for using particles of smaller diameter for the 0.35 cusec is that the coarse particle of large diameter could not be moved by the tractive force of the 0.35 cusec discharge.

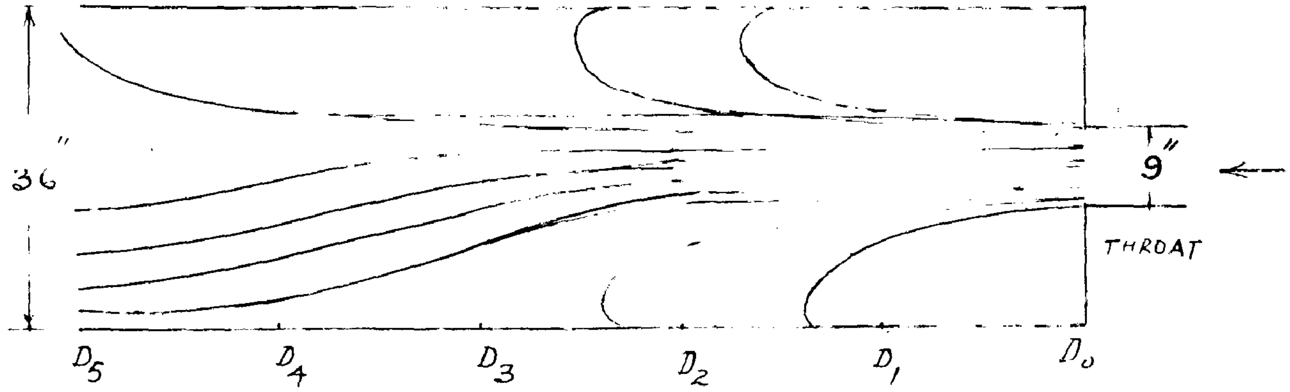
A study of the scour patterns reveals that the hyperbolic expansion has the best performance in this respect as well.



0.7 cusecs,



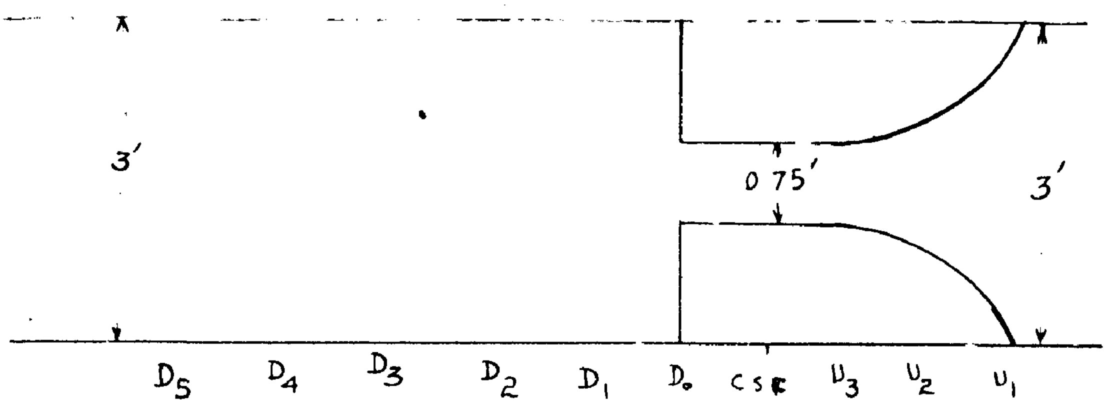
·35 cusecs



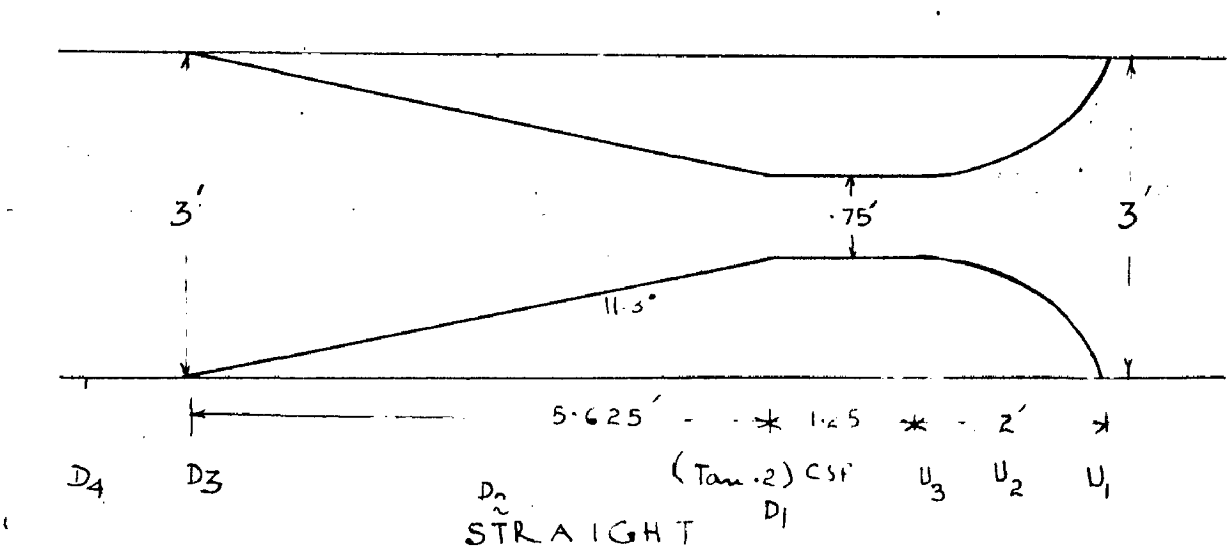
susecs

SURFACE FLOW PROFILES FOR ABRUPT EXPANSION.

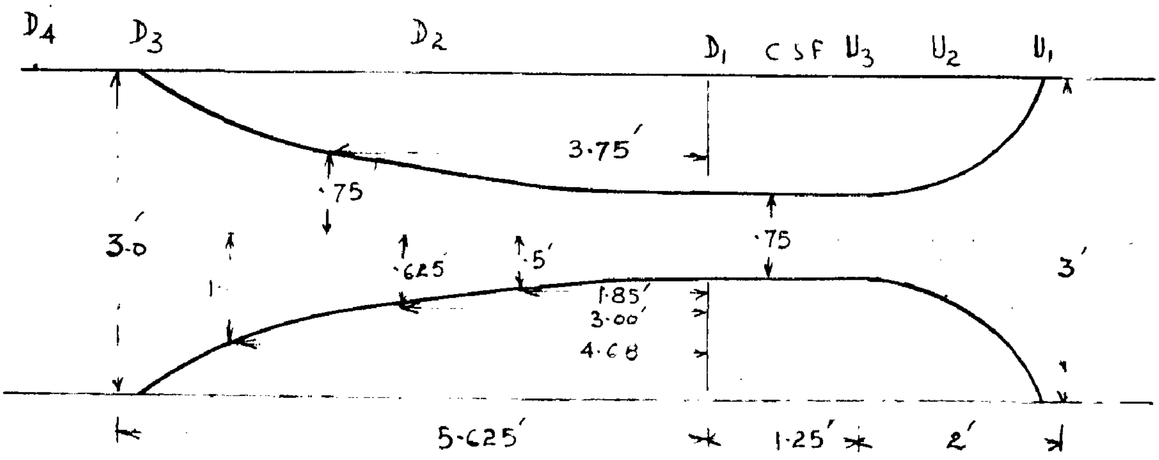
FIG. - 23.



ABRUPT



STRAIGHT

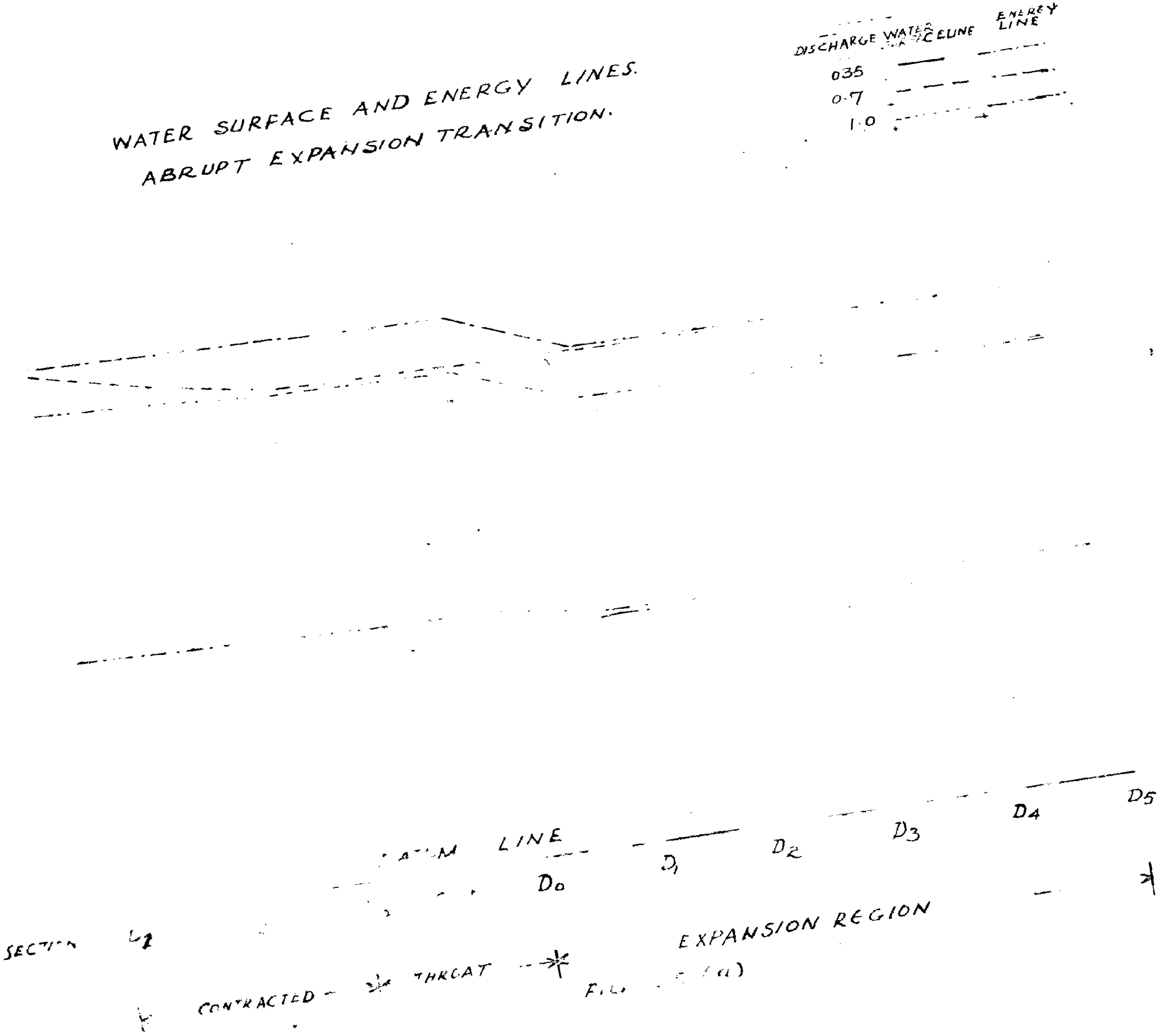


HYPERBOLIC

LINE DIAGRAM OF TRANSITIONS.

FIG—24.

WATER SURFACE AND ENERGY LINES.
ABRUPT EXPANSION TRANSITION.



F... (a)

WATER SURFACE AND ENERGY LINES
STRAIGHT (U.S.B.R) EXPANSION TRANSITION.

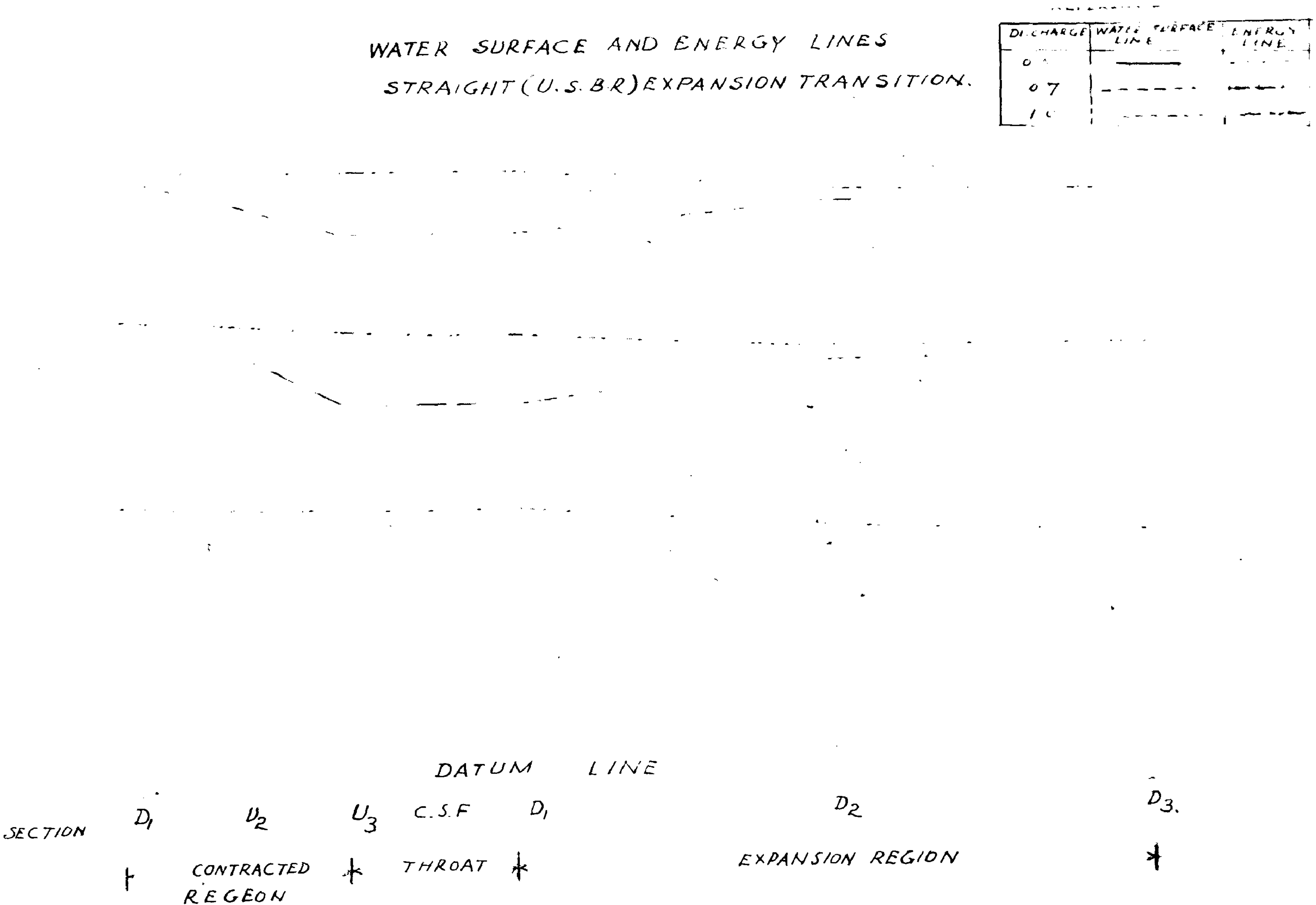


FIG. 25 (b)

WATER SURFACE AND ENERGY LINES
HYPERBOLIC EXPANSION TANSION

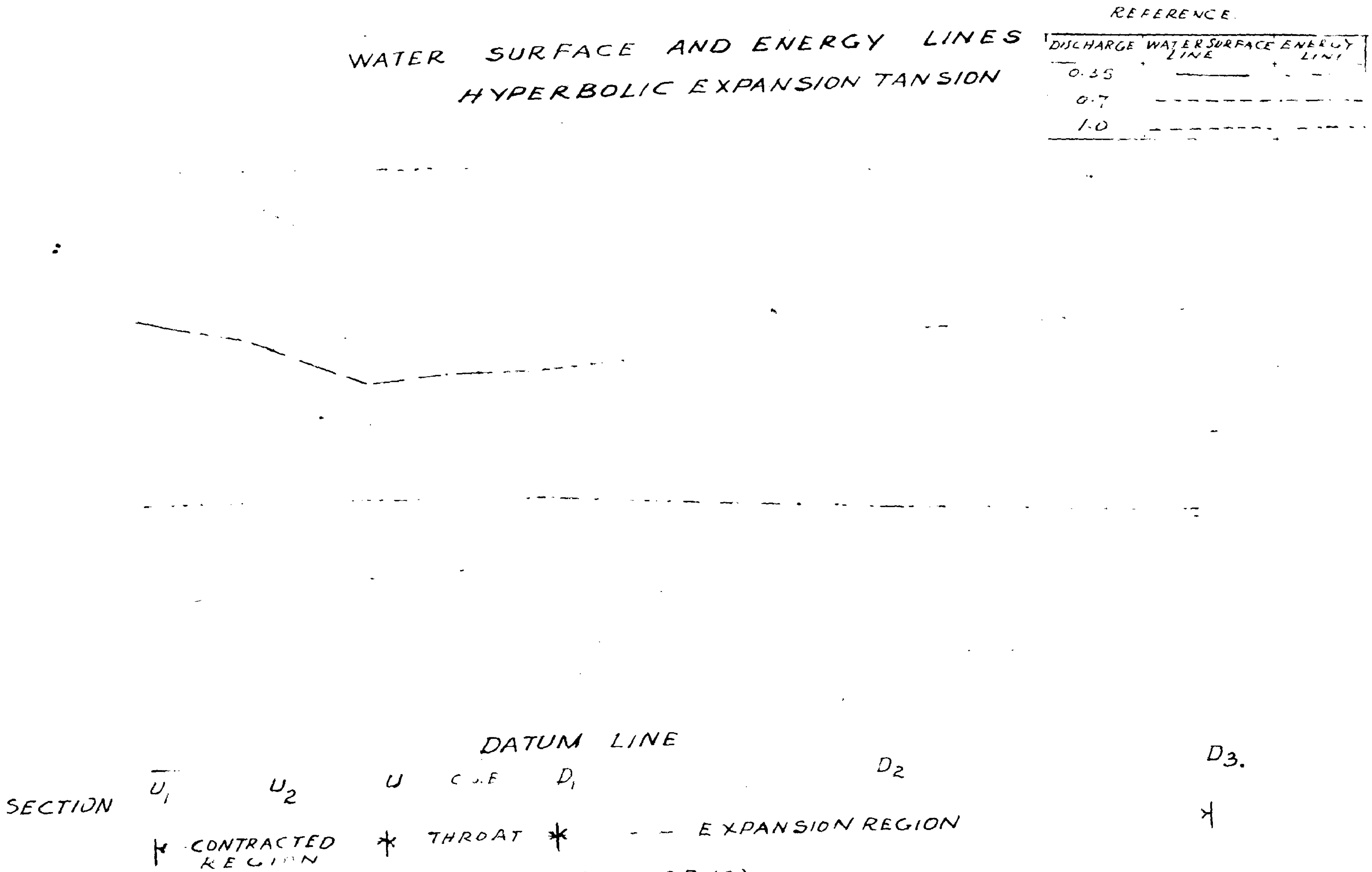
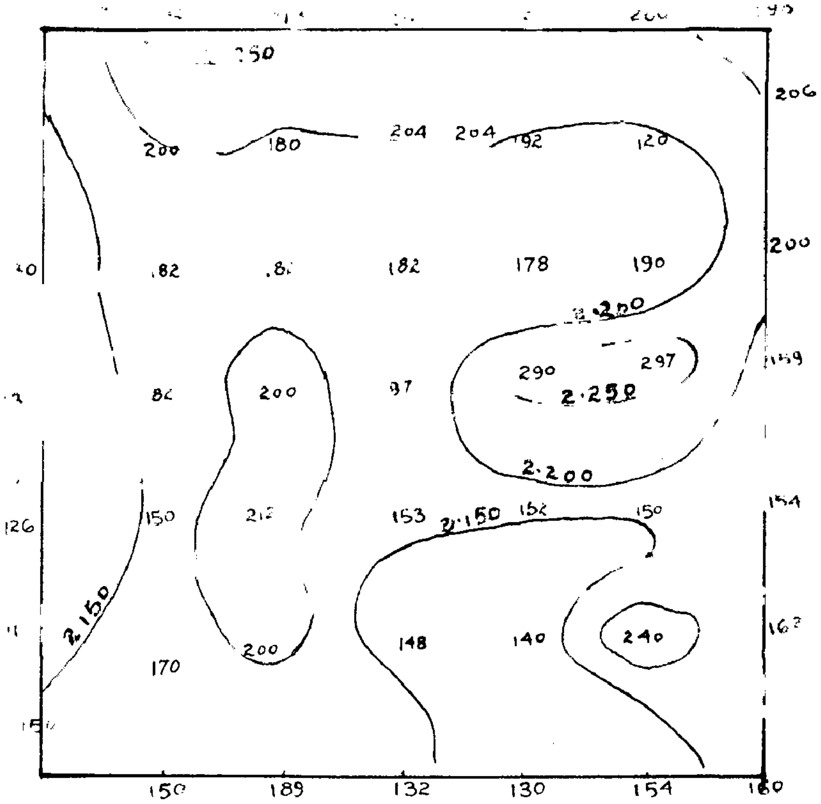


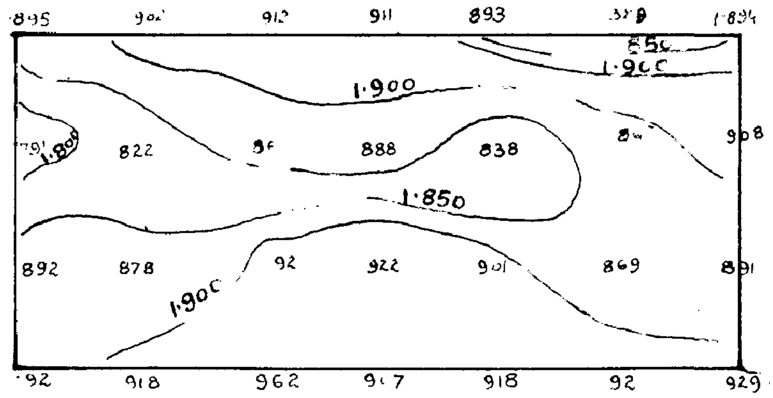
FIG-25 (C)

SCOUR PATTERN
FOR
DISCHARGE OF
0·35 CUSEC

HYPERBOLIC



ABRUPT



STRAIGHT
(U.S.E.R)

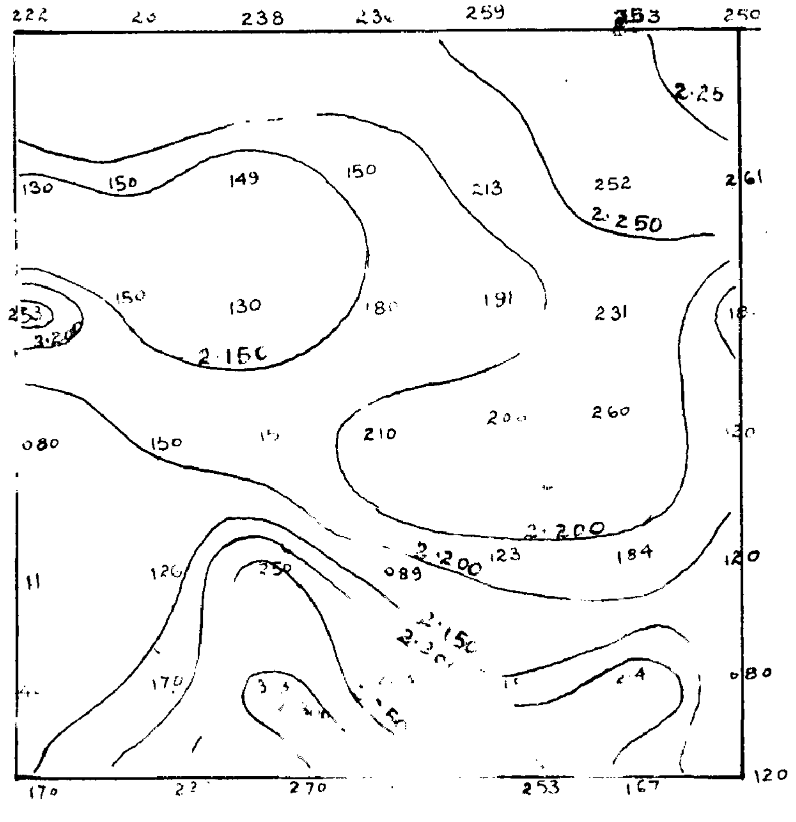
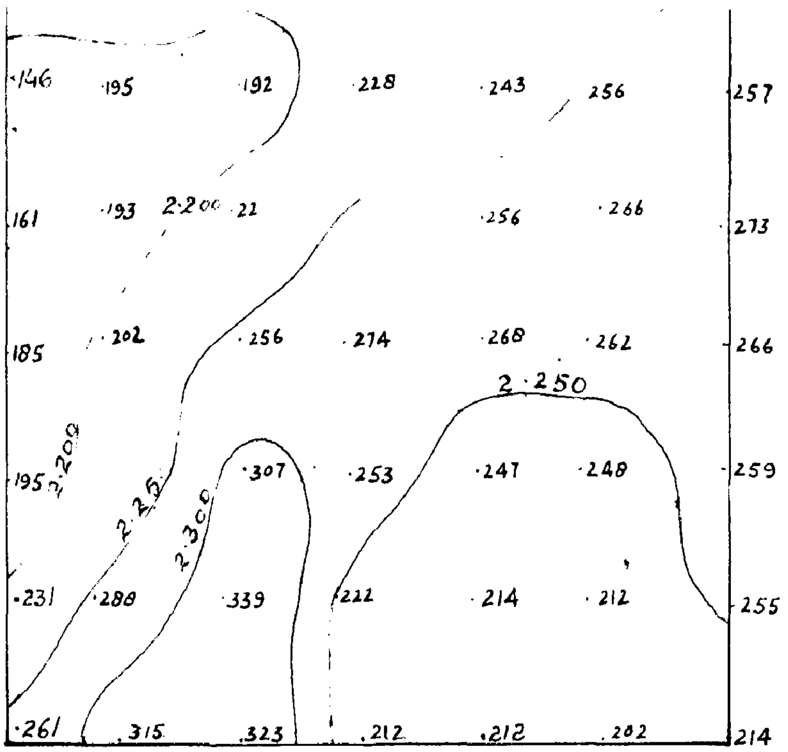


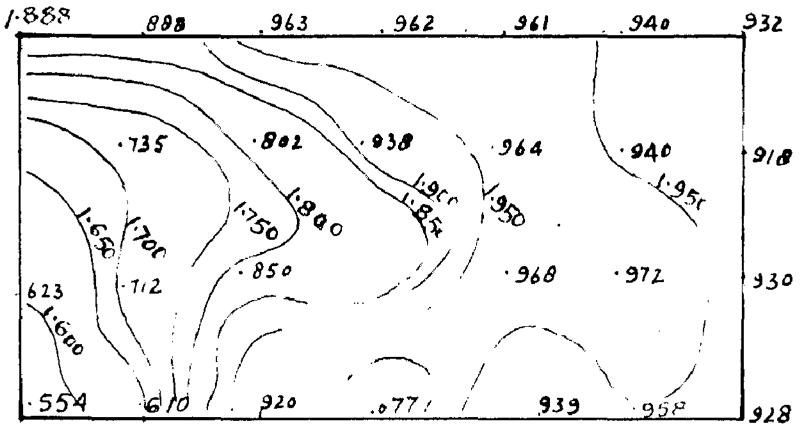
FIG-26a

SCOUR PATTERN
FOR
DISCHARGE OF
0.7CUSECS.

HYPERBOLIC



ABRUPT



STRAIGHT

S B. R)

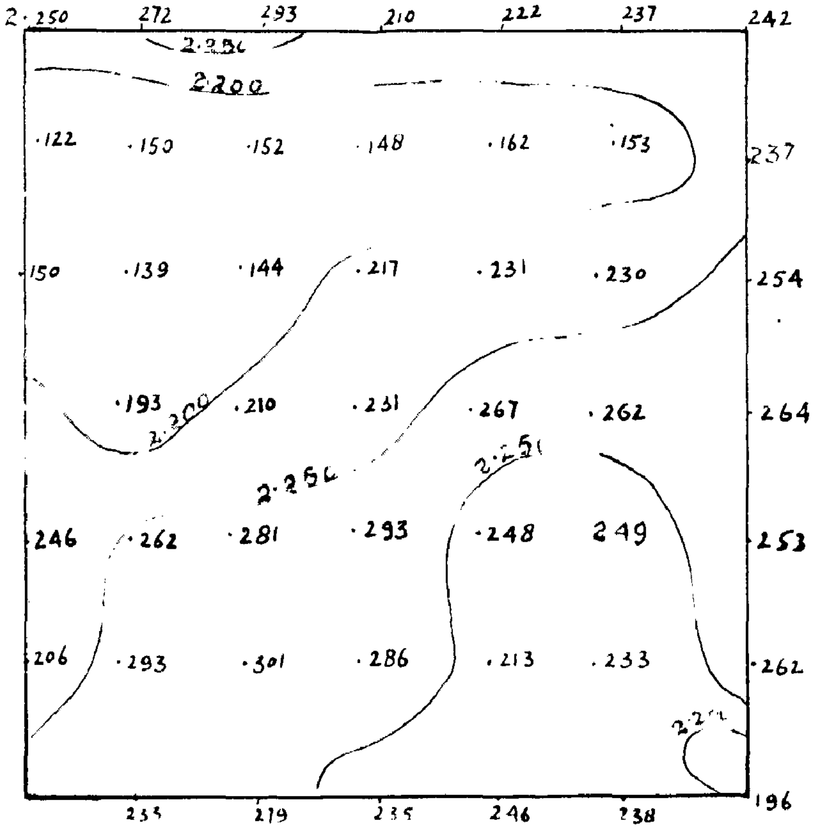
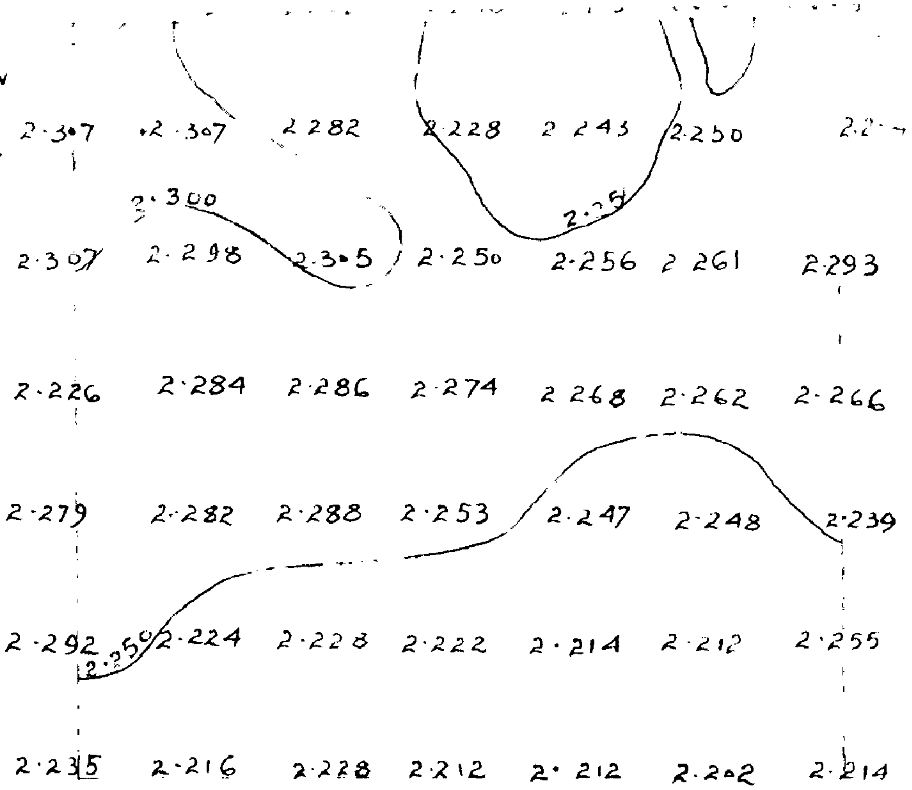
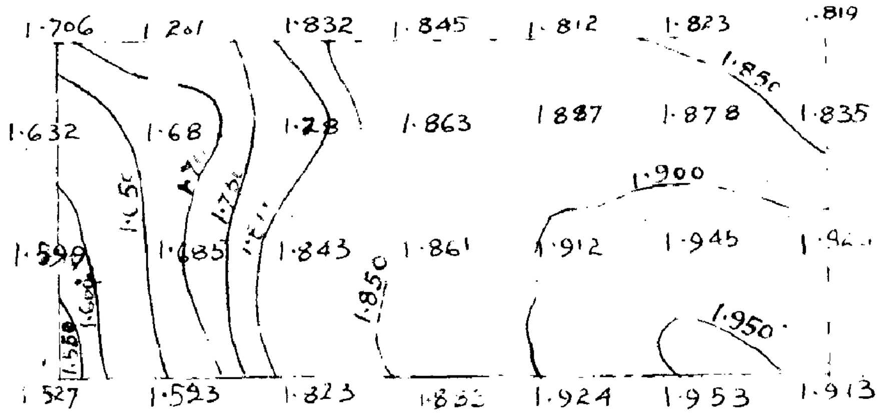


FIG_26

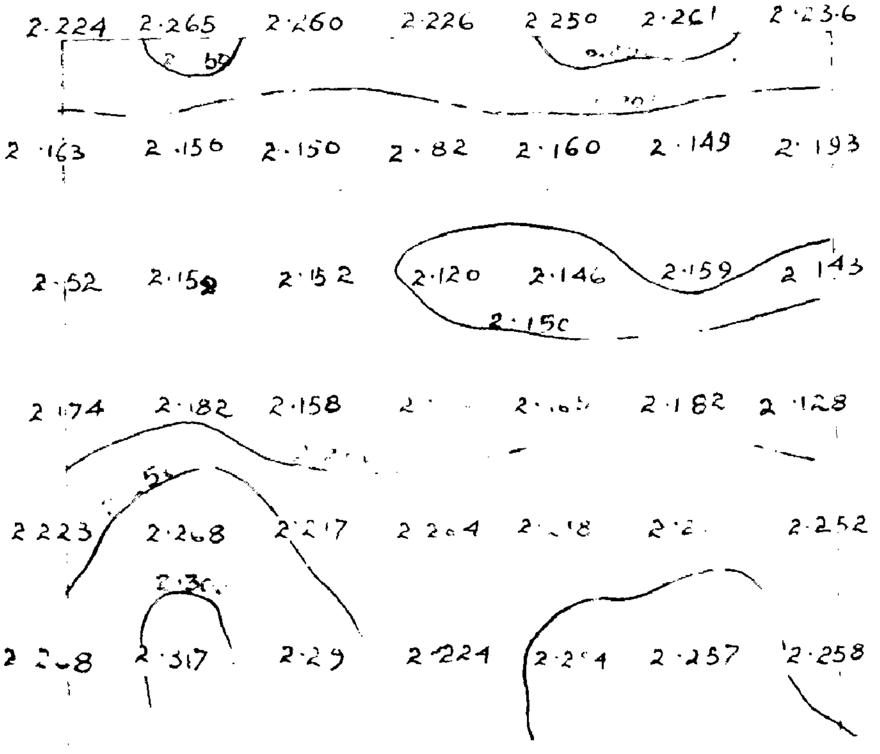
SCOUR PATTERN
FOR
DISCHARGE OF
1 CUSEC



HYPERBOLIC



ABRUPT



STRAIGHT
(U.S.B.R)

Tabulation of Results.

Q = .35 cusec.

Transition Type	Section	Computed Vel	Computed V.Head	Depth	Mean Vel.	Specific Energy	Bottom Elev.	Energy line.
Abrupt	U_1	.46	.0027	.3408	.342	.3435	1.9358	2.2793
	U_2	1.07	.0178	.321	.875	.3388	1.9336	2.2782
	U_3	1.66	.0429	.293	1.6	.3359	1.941	2.2769
	C.S.F	1.73	.0465	.272	1.72	.3185	1.958	2.2765
	D_o	1.665	.043	.287	1.62	.33	1.946	2.2760
	D_1	.457	.0032	.3078	.379	.3110	1.9412	2.2522
	D_2	.461	.0033	.2995	.39	.3028	1.947	2.2498
	D_3	.441	.003	.3061	.381	.2920	1.9482	2.2473
	D_4	.508	.004	.281	.415	.2850	1.959	2.2440
	D_5	.47	.0034	.2925	.399	.2959	1.9428	2.2387
Straight	U_1	.65	.0065	.287	0.406	.2925	2.279	2.5725
	U_2	1.22	.023	.271	1.03	.2940	2.276	2.5706
	U_3	1.85	.0534	.254	1.84	.3074	2.261	2.5684
	C.S.F.	1.88	.055	.261	1.79	.316	2.252	2.568
	D_1	1.89	.0563	.257	1.81	.3133	2.254	2.5673
	D_2	1.09	.0185	.280	0.77	.2985	2.239	2.5375
	D_3	.759	.0089	.255	0.458	.2639	2.261	2.5249

Tabulation of Results

Q = .35 cusec.

Transition Type	Section	Computed		Depth	Mean Vel.	Specific Energy	Bottom Elev.	Energy Line.
		Vel.	V.Head					
Hyperbolic	U_1	.78	.0095	.267	.44	.2765	2.291	2.5675
	U_2	1.32	.027	.244	1.15	.271	2.295	2.566
	U_3	2.4	.899	.19	2.46	.2799	2.285	2.5649
	C.S.F.	2.34	.085	.206	2.26	.2866	2.278	2.5646
	D_1	2.31	.0828	.21	2.22	.2928	2.271	2.5638
	D_2	1.34	.0279	.243	1.13	.2709	2.271	2.5419
	D_3	.902	.0126	.206	.568	.2186	2.309	2.5276

Tabulationof Results.

Q =.7 Cusec.

Transition Types.	Section.	Computed Vel.	V.Head	Depth	Mean Vel.	Speciifc Energy	Bottom Elev.	Energy Elevation
Straight	U_1	.710	.0072	.513	.45	.5208	2.279	2.7898
	U_2	1.35	.0234	.488	1.15	.5164	2.276	2.7824
	U_3	2.23	.0774	.445	2.14	.5284	2.261	.7794
	C.S.F	2.16	.0725	.457	2.04	.5355	2.252	.7775
	D_1	2.18	.0738	.452	2.07	.5318	2.254	.7758
	D_2	1.18	.0218	.496	.87	.5170	2.239	2.7568
	D_3	.68	.0072	.485	.482	.4922	2.261	.27532
Hyperbolic	U_1	.746	.0086	.489	.48	.4976	2.291	2.7886
	U_2	1.39	.03	.462	1.21	.492	2.295	2.787
	U_3	2.21	.076	.424	2.2	.500	2.285	2.785
	C.S.F.	2.14	.0695	.436	2.14	.5055	2.278	2.7835
	D_1	2.17	.0713	.441	2.11	.5123	2.271	2.7833
	D_2	1.36	.0287	.468	1.19	.4967	2.271	2.7677
	D_3	.75	.0087	.446	.525	.4547	2.309	2.7637

Tabulation of Results.

Q = .7 Cusec.

Transition Type	Section	Computed.		Depth	Mean Vel.	Specific Energy	Bottom Ele.	Energy Elevation.
		Vel	V.Head					
	U_1	.672	.009	.6704	.348	.6794	1.9358	2.6152
	U_2	1.88	.055	.624	.9	.679	1.9335	2.6125
	U_3	2.2	.0753	.588	1.59	.6633	1.941	2.6043
	c.s.f.	2.24	.078	.567	1.64	.645	1.958	2.603
	D_0	2.23	.0773	.573	1.64	.651	1.946	2.5963
Abrupt	D_1	.615	.0073	.5978	.391	.6051	1.9412	2.5463
	D_2	.655	.0068	.5921	3.95	.5981	1.947	2.5459
	D_3	.647	.0065	.5904	.396	.5969	1.9482	2.5451
	D_4	.595	.0055	.580	.403	.5865	1.959	2.5445
	D_5	.584	.0052	.5951	.392	.6003	1.9428	2.5431

Tabulation of Results

Q = 1 Cusec.

Transition Type.	Section	Computed Vel.	Computed V.Head	Depth	Mean Vel.	Specific Energy	Bottom Elevation.	Energy Elev.
Abrupt	U_1	.525	.0043	.736	.452	.7403	1.9358	2.677
	U_2	1.575	.0385	.703	1.14	.7415	1.9335	2.6750
	U_3	2.06	.0658	.667	2.01	.7328	1.941	2.6738
	C.S.F.	2.057	.0655	.650	2.05	.7155	1.958	2.0738
	D_0	2.06	.666	.661	2.02	.727	1.946	2.673
	D_1	.624	.006	.6652	.5	.6712	1.9412	2.6124
	D_2	.623	.006	.659	.498	.665	1.947	2.612
	D_3	.642	.0064	.657	.507	.6634	1.9482	2.6116
	D_4	.678	.00715	.645	.518	.65212	1.959	2.61112
	D_5	.648	.0065	.656	.508	.6625	1.9428	2.6053
Straight	U_1	.709	.0078	,678	.492	.6968	2.279	2.9648
	U_2	1.53	.0364	.648	1.23	.6844	2.276	2.904
	U_3	2.16	.073	.625	2.13	.6985	2.261	2.9595
	C.S.F.	2.2	.0752	.632	2.11	.7072	2.252	2.9592
	D_1	2.18	.0734	.627	2.13	.7048	2.254	2.9588
	D_2	1.135	.02	.675	.912	.695	2.239	2.934
	D_3	.785	.0096	.659	.506	.6686	2.261	2.296

Tabulation of Results

Q = 1 Cusec.

Transition Type.	Section	Computed		Depth	Mean Vel.	Specific energy	Bottom Elevation	Energy Elev.
		Velo.	V.Head					
Hyperbolic	U_1	.707	.0078	.653	.51	.6608	2.291	".(£*
	U_2	1.46	.033	.622	1.28	.6550	2.29	2.95
	U_3	2.54	.101	.561	2.37	.662	2.285	2.947
	C.S.F.	2.55	.102	.564	2.365	.666	2.278	2.944
	D_1	2.59	.1039	.569	.285	.6727	2.271	2.9437
	D_2	1.48	.034	.62	1.29	.6548	2.271	2.9258
	D_3	.78	.0094	.607	.549	.6164	2.309	2.9254

CONCLUSIONS

In the preceding chapters, a review of the more notable literature on transition design has been given and a few experiments done by the author have been described. From this study the following conclusions may be drawn.

The criteria determining the suitability of a transition design for a given fluming ratio are two-fold. Firstly it should cause minimum loss of head and secondly it should be economical. The second criterion requires that normal flow should be restored in as small a length of the transition as possible. No theoretical solution to indicate the best design for a given set of conditions is yet available.

Hinds utilised the usual energy relationship for designing sub-critical transitions. His method, however, involves a number of uncertainties and assumptions. More important of these are that the energy line is straight, the water surface line (or alternatively curvature of the side walls) must be assumed, the coefficient of Kinetic energy is unknown and has to be assumed as 1 though it may depart considerably from unity in the expansion transition. Further, the implicit assumptions that separation of flow from the boundary does not occur and that surface curvature may be neglected in applying the energy equation, are justifiable in very mild transitions only.

Subsequent workers continudd to follow the approach of Hinds, as inspite of its limitations no better approach seemed to be available. Ven Te Chow has applied the momentum equation to sudden expansions and contractions in open channels to obtain a correlation between Froude's no., flaring ratios, and ratios of water depths in the normal section and the flumed section.

A different approach was initiated by Prof A.C.Mitra which resulted in hyperbolic transitions. This approach was based on the basic assumptions that the water depth through the transition remained constant and that the acceleration in the direction of flow was also to be maintained constant. In those transitions where the required water depth in the flumed section and the normal section is the same, the first assumption can be approximated, more or less closely, by adjusting the bed. No theoretical explanation has been provided for the second.

Prof.N.S. Chaturvedi's recent study brings out that in hyperbolic transitions, the lateral acceleration increases directly with the distance in the direction of flow. His experiments indicate that hyperbolic transitions are superior to those of Hinds, In the field, a number of transitions ~~with~~ those based on Hinds' method as well as hyperbolic, have given satisfactory service. An accurate comparative study in the field would be invaluable in evaluating the relative performance ~~if~~ of these two types. For transitions, where the width as well as the water depth are to be changed, Hinds is still the only available method.

The author's experiments concerned three expansions with a fluming ratio of 4— (i) abrupt, (ii) straight line with a splay of 1 in 5 and (iii) hyperbolic with the same length as the straight line transition.

Out of these the hyperbolic transition gave the best results as shown in Chapter 5.

The proportion of energy loss to difference of velocity head in the three cases was as below.

	Q = 0·35 Cusecs.	Q = 0·7 Cusecs.	Q = 1·0 Cusecs.	~~Av.~~ Average
(i) Abrupt	.94	.745	1.12	.935
(ii) Straight line	.895	.339	.46	.568
(iii) Hyperbolic	.515	.314	.194	.34

For supercritical flow, the position of transition design is still more uncertain. The most reliable work yet available seems to be that of Rouse and Bhoota (14) who give the following equation for supercritical expansions on basis of experiments

$$\frac{y_L}{b_1} = \frac{1}{2}\left(\frac{x}{b_1 F_1}\right)^{3/2} + \frac{1}{2}$$

where

y_L = lateral distance from longitudinal axis.

b_1 = width of throat.

x = longitudinal direction.

F_1 = Froude Number at inlet.

References.

1. Central Board of Irrigation and Power on "Fluming" Publication No.6. New Delhi, 1957.

2. Madras Irrigation Research - Basic Studies on Fluming of Canals Publication No.6. Item 16, 1949.

3. Davis- Applied Hydraulics. McGraw Hill

4. Albertson M.L, Dai Y.B. Jensen R.A. and Hunter Rouse. Diffusion of Submerged jets. Transactions A.S.C.E. Vol. 115, 1950 Pp. 639-694.

5. Sharma K.R. Irrigation Engineering. India Printers. Pages 284-287.

6. Hunter Rouse- Engineering Hydraulics. John Wiley & Sons.

7. Lane E.W. Experiments on the Flow of Water through Contractions in an open channel. Transactions A.S.C.E. Vol. 83 1919-1920 Page 1149-1208

8. Hinds Julian- The hydraulic design of flume and syphon Transactions. Transactions of A.S.C.E. Vol. 92 1928. Page 1423 - 1085.

9. Ippen A.T. Channel Transition and Controls Hunter Rouse Engineering Hydraulics.

10. Ven Te Chow- Open Channel Hydraulics- McGraw Hill, 1959. Page 461-474.

11. Doddiah. D. Design of Open Transition with special reference to inverted syphon's Research Publication No.8, Mysore Irrigation Research Station.

12. Chaturvedi R.S- Expansive Sub-critical Flow in Open Channel Transition. Roorkee University Research Journal. November, 1960.

13. Ippen A.T. and Dawson J.H.- Design of Channel Contractions. Proceedings A.S.C.E.Vol. 75 No.9 1949 Pages 1348 - 1368.

14. Rouse Hunter, and Bhoota B.V. Design of Channel Expansion. Proceeding A.S.C.E. Vol. 75 No.9 Page 1369-1385.

15. Iyer K.H- Fluming in Irrigation Works. Irrigation and Power Journal Vol. 9 April, 1952. Pages 233-245.

16. F.H.Bucket- Question of Expansion from Pressure Considerations. Punjab Engineering Congress, Lahore. Paper No.125.

17. Report of the Punjab Irrigation Research Institute- Experiment on transitions April, 1939.

APPENDIX

Readings and Iso- vels.

Transition Abrupt

Discharge 0.35 Cusec.

Sec-tion		Point Gauge Reading			Velocity Head							Dist. from bottom.
		W.S.	B.L.	W.D.	1"	6"	12"	18"	24"	30"	35"	
L	L	2.293	1.938	.355	.0025	.0025	.005	.005	.005	.0025	.0025	1"
	C	2.276	1.939	.337	.0025	.0025	.005	.005	.005	.0025	.0025	3"
U_1	R	2.283	1.953	.330								
	L	2.269	1.943	.326			L	C	R			
	C	2.269	1.955	.314			.0175	.0175	.02			1"
U_2	R	2.269	1.945	.324			.02	.02	.025			3"
	L	2.244	1.964	.280			.04	.045	.045			1"
	C	2.245	1.942	.303			.04	.045	.05			3"
U_3	R	2.245	1.948	.297								
	L	2.231	1.962	.269			.045	.045	.05			1"
	C	2.229	1.951	.278			.045	.05	.055			3"
VSF	R	2.231	1.962	.269								
	L	2.232	1.943	.289			.04	.045	.045			1"
	C	2.234	1.944	.290			.045	.05	.05			3"
D_o	R	2.232	1.951	.281								
	L	2.249	1.942	.307								
	C	2.264	1.942	.312	-.0025	-.0025	.02	.045	.0025	-.0025		1"
D_1	R	2.252	1.947	.305	-.0025	-.0025	.025	.045	.0025	-.0025		3"

Transition Abrupt

Discharge 0.35 Cusec.

Sec-tion		W.S.	B.L.	W.D.	1"	6"	12"	18"	24"	30"	35"	Distance from bottom
D2	L	2.240	1.946	.294								
	C	2.243	1.942	.301	-.0075	-.0025	0035	.035	.005	-.0025	-.0025	1"
	R	2.245	1.941	.304	-.005	-.0025	.0375	.0425	.0075	-.0025	-.0025	3"
D3	L	2.253	1.944	.309								
	C	2.252	1.947	.305	-.005	.015	.0375	.015	00	-.0025	-.005	1"
	R	2.251	1.946	.305	-.005	.015	.0275	.025	00	-.0025	-.005	3"
D4	L	2.238	1.954	.284								
	C	2.232	1.955	.277	-.015	0.025	.03	.005	00	-.0025	-.005	1"
	R	2.235	1.952	.283	.015	.03	.035	.0075	00	-.0025	-.006	3"
D5	L	2.228	1.941	.287								
	C	2.236	1.937	.299	.02	.025	.025	.0025	-.0025	-.0025	-.005	1"
	R	2.235	1.943	.292	.025	.03	.025	.005	-.0025	-.005	-.005	3"

Transition Straight

Discharge 0.35 Cusec.

Sec-tion		Point Gauge Reading W.S.	B.L.	W.D.	1"	6"	12"	18"	24"	30"	Velocity Head. 35"	Depth from bottom
U$_1$	L	2.565	2.258	.307	.005	.005	.01	.015	.015	.005	.005	1"
	C	2.568	2.277	.291	.005	.005	.01	.015	.015	.005	.005	3.0"
	R	2.564	2.302	.262								
							L	C	R			1"
U2	L	2.546	2.267	.279			.D15	.025	.025			3"
	C	2.547	2.276	.271			.02	.03	.025			
	R	2.564	2.284	.263								
U3	L	2.516	2.256	.2[illegible]			.05	.055	.055			1"
	C	2.513	2.266	.247			.055	.06	.055			3"
	R	2.514	2.260	.254								
C.S.F.	L	2.510	2.250	.260			.05	.06	.055			1"
	C	2.516	2.249	.267			.055	.06	.060			3"
	R	2.513	2.257	.256								
D1	L	2.512	2.256	.256			.05	.055	.055			1"
	C	2.512	2.252	.260			.05	.055	.05			3"
	R	2.509	2.255	.254								

Transition Straight

Discharge 0.35 Cusec.

		Point Gauge Reading.			Velocity Head.							
Section		W.S.	B.L.	W.D.	1"	6"	12"	18"	24"	30"	35"	Depth from bottom.
	L	2.519	2.238	.281			.015	.03	.025			1"
D2	C	2.517	2.231	.286			.02	.015	.025			3"
	R	2.519	2.247	.272								
	L	2.518	2.262	.256	-.005	-.005	.005	.015	.015	.02	.01	1"
D3	C	2.515	2.26	.255	-.005	-.005	.005	.015	.02	.025	.01	3"
	R	2.515	2.262	.253								
1' D/s	L	2.514	2.26	.254	-.005	-.005	.005	.01	.01	.01	.005	1"
D3	C	2.520	2.258	.262	-.005	-.005	.005	.01	.015	.015	.01	3"
	R	2.517	2.264	.253								

Transition Hyperbolic Expansion

Discharge 0.35 Cusec.

Section		Point Gauge	Reading					Velocity Head				Distance from bottom.
		U.S.	B.L.	W.D. 1"		6"	12"	18"	24"	30"	35"	
U 1	L	2.555	2.29	.265	.005	.005	.01	.02	.01	.005	.005	1"
	C	2.557	2.294	.263	.005	.005	.01	.02	.015	.005	.005	2"
	R	2.561	2.288	.273								
							L	C	R			
U 2	L	2.538	2.301	.237			.02	.0325	.01			1"
	C	2.538	2.295	.243			.025	.0325	.01			2"
	R	2.541	2.288	.253								
U 3	L	2.472	2.285	0.187			.085	.09	.08			1"
	C	2.473	2.284	0.189			.09	.095	.09			2"
	R	2.480	2.286	0.194								
C.S.F	L	2.482	2.275	0.203			.09	.095	.08			1"
	C	2.479	2.276	0.203			.09	.1	.085			2"
	R	2.491	2.278	0.213								
D 1	L	2.480	2.272	0.208			.08	.1	.085			1"
	C	2.480	2.269	0.211			.09	.105	.09			2"
	R	2.484	2.270	0.211								
D_2	L	2.516	2.273	0.243			.04	.065	.025			1"
	C	2.517	2.268	0.243			.035	.05	.02			2"
	R	2.510	2.273	0.237								

Transition Hyperbolic Expansion.

Discharge 0.35 Cusec.

Section		Point Gauge Reading.			Velocity Head.							Distance from bottom.
		W.S.	B.L.	W.D.	1"	6"	12"	18"	24"	30"	35"	
	D	2.524	2.316	0.208.	.01	.02	.03	.02	-.0005	-.0025	-.0025	1"
D_3	C	2.519	2.31	0.209	.01	.025	.03	.025	-.005	.005	.005	2"
	R	2.507	2.302	0.205								
1'	L	2.510	2.307	0.203	.0025	.02	.025	.01	0.00	-.0025	.0025	1"
D/s	C	2.52	2.31	0.21	.005	.025	.03	.01	00	-.0025	.0025	2"
D_3	R	2.52	2.32	0.19								

Transition Abrupt Expansion

Discharge 0.7 Cusec.

Section		Point Gauge Reading.			Velocity Heads.							Distance from bottom.
		W.S.	B.L.	W.D.	1"	6"	12"	18"	24"	30"	35"	
U$_1$	L	2.610	1.938	.672	.005	.01	.015	.015	.01	.01	-.01	1"
	C	2.609	1.939	.670	.01	.01	.015	.015	.01	.01	.01	6"
	R	2.622	1.953	.669								
U$_2$	L	2.571	1.943	.628			L	C	R			
	C	2.572	1.955	.617			.05	.05	.055			1"
	R	2.573	1.945	.628			.055	.06	.06			6"
U$_3$	L	2.539	1.964	.575								
	C	2.538	1.942	.598			.08	.08	.085			1"
	R	2.539	1.948	.591			.075	.08	.085			6"
C.S.F	L	2.525	1.962	.563			.075	.08	.08			1"
	C	2.525	1.951	.574			.075	.08	.08			4"
	R	2.526	1.962	.564			.08	.08	.085			6"
D$_o$	L	2.520	1.943	.577			.07	.075	.075			1"
	C	2.519	1.944	.575			.075	.075	.08			4"
	R	2.519	1.951	.568			.075	.085	.085			6"
D$_1$	L	2.541	1.942	.599	-.005	.005	.05	.07	.005	00	0.00	1"
	C	2.545	1.942	.603	-.01	.005	.045	.07	.005	00	0.00	4"
	R	2.539	1.947	.592	-.01	.005	.04	.07	.01	00	0.00	6"

Transition Abrupt Expansion

Discharge 0.7 Cusec.

Point Gauge Reading.					Velocity Head.							Depth from surface.
Section.		U.S.	D.L.	U.D.	1"	6"	12"	18"	24"	30"	33"	
D_2	L	2.530	1.943	.584	-.01	.015	.07	.035	.005	0	-.01	1"
	C	2.540	1.942	.595	-.015	.01	.07	.045	.01	0	-.005	4"
	R	2.537	1.941	.596	-.01	.02	.07	.035	.01	0	-.005	6"
D_3	L	2.530	1.944	.586	-.015	.015	.04	.025	.005	-.01	-.005	1"
	C	2.523	1.947	.581	-.01	.03	.035	.035	.01	-.005	-.01	4"
	R	2.540	1.946	.603	-.01	.025	.06	.03	.01	-.005	-.01	6"
D_4	L	2.538	1.954	.584	.015	.035	.03	.01	.005	-.005	-.01	1"
	C	2.530	1.955	.575	.03	.045	.04	.01	.01	-.005	-.01	4"
	R	2.533	1.952	.581	.03	.04	.05	.01	.01	-.005	-.005	6"
D_5	L	2.537	1.941	.595	.03	.035	.035	.01	.005	-.005	-.005	1"
	C	2.534	1.937	.597	.055	.045	.035	.01	.005	-.01	-.01	4"
	R	2.535	1.943	.592	.045	.055	.04	.015	.005	-.01	-.005	6"

Transition Straight Expansion.

Discharge 0.7 Cusec.

Section		W.S.	B.L.	W.D.	1"	6"	12"	18"	24"	30"	35"	Depth from bottom.
U_1	L	2.771	2.258	.513	.005	.005	.0075	.015	.015	.005	.005	1"
	C	2.780	2.277	.503	.005	.0075	.01	.015	.015	.006	.005	5"
	R	2.795	2.302	.493								
							L	C	R			
U_2	L	2.75	2.267	.483			.03	.03	.03			1"
	C	2.759	2.276	.483			.035	.035	.035			5"
	R	2.743	2.284	.469								
U_3	L	2.693	2.256	.437			.08	.085	.08			1"
	C	2.698	2.266	.432			.08	.085	.08			5"
	R	2.695	2.260	.435								
C.S.F	L	2.70	2.25	.45			.07	.075	.075			1"
	C	2.698	2.249	.449			.075	.095	.075			3"
	R	2.695	2.257	.442			.07	.085	.08			5"
D_1	L	2.705	2.256	.449			.08	.085	.08			1"
	C	2.700	2.252	.438			.08	.095	.085			3"
	R	2.695	2.255	.440			.085	.085	.085			5"

Transition Straight Expansion,

Discharge 0.7 Cusec.

Section		W.S.	B.L.	W.D.	1"	6"	12"	18"	24"	30"	35"	Depth from bottom.
D$_2$	L	2.733	2.238	.495			-.005	.025	.025			1"
	C	2.743	2.231	.502			-.005	.035	.025			3"
	R	2.739	2.247	.492			-.005	.045	.025			5"
D$_3$	L	2.742	2.262	.482	-.0025	-.0025	-.005	.01	.02	.03	.015	1"
	C	2.744	2.26	.484	-.005	-.005	.005	.01	.035	.04	.025	3"
	R	2.752	2.262	.490	-.005	-.005	.015	.03	.025	.04	.025	5"
1' D/s D$_3$	L	2.739	2.26	.479	-.0025	-.0025	.005	.015	.025	.02	.02	1"
	C	2.742	2.258	.484	-.005	-.005	.01	.02	.03	.02	.025	3"
	R	2.749	2.264	.485	-.005	-.005	.01	.025	.035	.025	.025	5"

Transition Hyperbolic Expansion

Discharge 0.7 Cusec.

Point Section		Gauge Reading W.S.	B.L.	W.D.	1"	6"	12"	18"	24"	30"	35"	Depth from bottom.
U$_1$	L	2.776	2.29	.486	.005	.005	.01	.015	.01	.005	-.005	1"
	C	2.784	2.294	.490	.005	.0075	.015	.015	.01	.005	-.005	5"
	R	2.779	2.288	.491								
							L	C	R			
U$_2$	L.	2.759	2.301	.458			.035	.03	.03			1"
	C.	2.758	2.295	.463			.04	.035	.035			5"
	R	2.754	2.288	.466								
U$_3$	L	2.711	2.285	.425			.075	.08	.07			1"
	c	2.716	2.284	.432			.08	.085	.075			5"
	R	2.700	2.286	.414								
C.S.F	L	2.708	2.279	.429			.075	.075	.065			1"
	C	2.712	2.276	.436			.075	.075	.06			3"
	R	2.720	2.278	.442			.065	.065	.05			5"
D$_1$	L	2.706	2.272	.434			.07	.075	.075			1"
	C	2.712	2.269	.443			.08	.075	.06			3"
	R	2.719	2.273	.446			.07	.065	.05			5"

Transition Hyperbolic Expansion,

Discharge 0.7 Cusec.

Point Section		Gauge Reading.			Velocity Head.							Depth from bottom.
		W.S.	B.L.	W.D.	1"	6"	12"	18"	24"	30"	35"	
D_2	L	2.747	2.273	.472			.02	.04	.005			1"
	C	2.732	2.268	.464			.035	.04	.01			3"
	R	2.740	2.273	.467			.03	.035	.01			5"
D_3	L	2.769	2.316	.453	.01	.03	.025	.025	.01	00	-.0025	1"
	C	2.758	2.31	.448	.0025	.03	.03	.03	.015	00	-.0025	3"
	R	2.740	2.302	.438	.005	.035	.03	.03	.01	00	-.0025	5"
1" D/s D_3	L	2.740	2.307	.433	.005	.03	.03	.02	.0025	-.0025	00	1"
	C	2.755	2.31	.445	00	.03	.03	.02	.0025	-.0025	00	3"
	R	2.753	2.33	.433	00	.025	.025	.015	.0025	-.0025	00	5"

Transition Abrupt Expansion

Discharge 1 Cusec.

Section		Point Gauge Reading.			Velocity Head							Depth from bottom.
		W. S.	B.L.	W.D.	1"	6"	12"	18"	24"	30"	35"	
U_1	L	2.676	1.938	.738	.0025	.005	.01	.01	.01	.0055	.0025	1"
	C	2.681	1.939	.742	.0025	.005	.01	.01	.01	.005	.0025	7.5"
	R	2.682	1.953	.729								
U_2	L	2.651	1.943	.708			L	C	R			
	C	2.650	1.955	.695			.03	.035	.035			1"
	R	2.651	1.945	.706			.03	.04	.04			7.5"
U_3	L	2.616	1.964	.652			.06	.07	.065			1"
	C	2.622	1.942	.680			.065	.07	.065			4.0"
	R	2.616	1.948	.668			.06	.065	.06			7.5"
C.S.F	L	2.604	1.962	.642			.06	.07	.06			1"
	C	2.606	1.951	.655			.065	.07	.065			4"
	R	2.606	1.962	.644			.065	.07	.06			7.5"
D_o	L	2.596	1.943	.653			.06	.07	.06			1"
	C	2.595	1.944	.651			.055	.07	.06			4"
	R	2.596	1.951	.645			.06	.065	.055			7.5"

Transition -Abrupt Expansion

Discharge 1 Cusec.

Section		Point Gauge Reading.			Velocity Head.							Depth from bottom.
		W.S.	B.L.	W.D.	1"	6"	12"	18"	24"	30"	35"	
D_1	L	2.606	1.942	.664	-.005	.005	.015	.07	.01	00	-.005	1"
	C	2.614	1.942	.672	-.0025	.005	.01	.08	.005	-005	-.005	4"
	R	2.607	1.947	.660	-.005	.01	.015	.08	.005	00	00	7.5"
D_2	L	2.602	1.946	.656	-.005	.015	.045	.045	.005	00	-.005	1"
	C	2.602	1.942	.660	-.01	.005	.05	.06	.005	00	-.005	4"
	R	2.601	1.941	.660	-.01	.01	.05	.06	.005	00	-.005	7.5"
D_3	L	2.606	1.944	.662	-.005	.005	.04	.025	.005	-.005	-.005	1"
	C	2.605	1.947	.658	-.005	.025	.04	.025	-.0025	-.005	-.000	4"
	R	2.597	1.946	.651	-.005	.02	.04	.028	-.005	-.005	.00	7.5"
D_4	L	2.607	1.954	.653	.015	.02	.03	.025	-.005	-.005	.005	1"
	C	2.594	1.955	.639	.015	.025	.03	.025	-.005	-.005	.005	4"
	R	2.595	1.952	.643	.015	.04	.04	.025	-.005	.01	.005	7.5"
D_5	L	2.602	1.941	.661	.015	.02	.025	.015	-.005	-.005	-.005	1"
	C	2.595	1.937	.658	.025	.04	.030	.015	-.005	-.005	-.005	4"
	R	2.593	1.943	.650	.025	.04	.035	.02	-.005	-.005	-.005	7.5"

Transition - Straight Expansion

Discharge 1.00 Cusec.

Section.	Point Gauge Reading.			Velocity Heads.							Distance from bottom.
	W.S	B.L.	W.D.	1"	6"	12"	18"	24"	30"	35"	
L	2.952	2.258	0.694	.0025	.005	.01	.015	.01	.005	.005	1"
C	2.953	2.277	0.676	.0025	.005	.0075	.015	.01	.005	.005	6"
R	2.967	2.302	0.665			L	C	R			
L	2.917	2.267	0.650			.035	.04	.035			1"
C	2.927	2.276	0.651			.04	.04	.035			6"
R	2.927	2.284	0.643								
L	2.885	2.256	0.629			.095	0.1	.09			1"
C	2.883	2.266	0.617			.095	0.105	.09			6"
R	2.890	2.260	0.630								
L	2.881	2.250	0.631			.08	.085	.085			1"
C	2.892	2.249	0.643			.085	.09	.085			4"
R	2.878	2.257	0.621			.09	.095	.09			6"
L	2.876	2.256	0.620			.08	.0[illegible]	.08			1"
C	2.886	2.252	0.634			.085	.095	.08			4"
R	2.883	2.255	0.628			.085	0.1	.085			6"

Discharge 1.00 Cusec.

Section		Point Gauge Reading			Velocity Heads.							Distance from bottom.
		W. S.	B.L.	W. D.	1"	6"	12"	18"	24"	30"	35"	
D$_2$	D	2.908	2.238	0.671			.02	.01	00			1"
	C	2.916	2.231	0.685			.02	.015	00			4"
	R	2.917	2.247	0.670			.025	.03	00			6"
D$_3$	L	2.916	2.262	0.654	.02	.025	.02	.01	.005	.005	-.005	1"
	C	2.920	2.26	0.660	.01	.02	.03	.015	.005	.005	-.005	4"
	R	2.925	2.262	0.663	.015	.02	.04	.02	.015	.005	-.005	6"
1' D/S D3	L	2.908	2.26	0.648	.01	.02	.015	.01	.005	.005	-.005	
	C	2.910	2.258	0.652	.015	.025	.03	.015	.005	.00	-.005	
	R	2.924	2.264	0.660	.015	.03	.03	.02	.005	.00	-.005	

Transition - Hyperbolic Expansion

Discharge 1.00 Cusec.

Section		Point Gauge Reading.			Velocity Head.							Distance from bottom.
		W.S.	B.L.	W.D.	1"	6"	12"	18"	24"	30"	35"	
U_1	L	2.944	2.29	0.654	0.00	.01	.015	.01	.015	.005	0.00	1"
	C	2.942	2.94	0.648	0.00	.01	.015	.015	.015	.005	0.00	6"
	R	2.945	2.88	0.657			L	C	R			
U_2	L	2.919	2.301	0.618			.035	.04	.035			1"
	C	2.918	2.295	0.628			.035	.04	.035			6"
	R	2.909	2.288	0.624								
U_3	L	2.845	2.285	0.56			.105	.105	.11			1"
	C	2.840	2.284	.0556			.105	.105	.11			6"
	R	2.854	2.286	0.568								
CSF	L	2.834	2.279	0.555			.095	.1	.09			1"
	C	2.847	2.276	0.571			.1	.105	.1			4"
	R	2.845	2.278	0.567			.115	.11	.1			6"
D_1	L	2.834	2.272	0.562			.09	.095	.075			1"
	C	2.843	2.269	0.574			.09	.0985	.075			4"
	R	2.845	2.273	0.572			.095	.095	.085			6"

Transition - Hyperbolic Expansion

Discharge 1.00 Cusec.

Section		Point Gauge Reading.										
		W.S.	B.L.	W.D.	1"	6"	12"	18"	24"	30"	35"	Distance from bottom
D_2	L	2.903	2.273	0.63			.015	.025	.015			1"
	C	2.889	2.268	0.621			.015	.04	.025			4"
	R	2.392	2.273	0.619			.02	.065	.03			6"
D_3	L	2.928	2.316	0.612	0.00	.0025	.025	.025	.015	.015	.005	1"
	C	2.915	2.31	0.605	0.00	.005	.035	.03	.025	.03	.005	4"
	R	2.305	2.302	0.603	0.00	.01	.04	.04	.03	.02	.005	6"
1' D/s D_3	L	2.884	2.307	0.577	0.005	.015	.025	.0275	.005	.00	.00	1"
	C	2.90	2.31	0.597	0.005	.02	.035	.04	.04	.0025	.00	4"
	R	2.917	2.33	0.587	0.0025	.035	.04	.045	.045	.0025	.00	6"

Scour Pattern Readings.

for 0.35 cusec.

		0	6"	12"	18"	24"	30"	36"
Abrupt	0	1.895	1.902	1.912	1.911	1.893	1.889	1.894
	6"	1.791	1.822	1.861	1.888	1.838	1.865	1.908
	12"	1.892	1.878	1.92	1.922	1.901	1.869	1.891
	16"	1.92	1.918	1.962	1.907	1.918	1.92	1.929
Straight	0	2.222	2.230	2.238	2.236	2.259	2.253	2.200
	6"	2.130	2.150	2.149	2.150	2.213	2.252	2.261
	12"	2.253	2.150	2.130	2.180	2.191	2.231	2.132
	18"	2.080	2.150	2.151	2.210	2.206	2.266	2.120
	24"	2.110	2.126	2.250	2.080	2.193	2.184	2.120
	30"	2.140	2.170	2.313	2.226	2.150	2.214	2.080
	36"	2.171	2.222	2.270	2.227	2.253	2.167	2.120
Hyperbolic	0	2.197	2.292	2.278	2.246	2.230	2.200	2.190
	6"	2.120	2.200	2.180	2.204	2.192	2.190	2.206
	12"	2.130	2.188	2.182	2.182	2.178	2.190	2.200
	18"	2.130	2.182	2.200	2.197	2.290	2.297	2.159
	24"	2.126	2.150	2.212	2.153	2.152	2.150	2.154
	30"	2.110	2.190	2.200	2.148	2.140	2.240	2.162
	36"	2.150	2.150	2.189	2.182	2.130	2.134	2.160

Scour Pattern Readings.

for 0.7 Cusec.

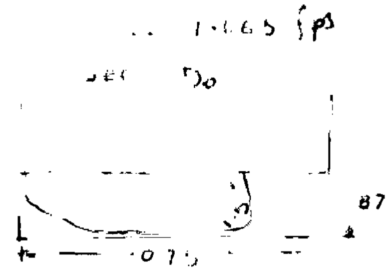
		0	6"	12"	18"	24"	30"	36"
Abrupt	0	1.888	1.888	1.963	1.962	1.961	1.940	1.932
	6"	1.662	1.735	1.802	1.933	1.964	1.940	1.918
	12"	1.623	1.712	1.850	1.847	1.968	1.972	1.930
	18"	1.554	1.610	1.920	2.007	1.939	1.958	1.928
Straight	0	2.250	2.273	2.293	2.210	2.222	2.237	2.242
	6"	2.122	2.156	2.152	2.148	2.162	2.153	2.222
	12"	2.150	2.139	2.144	2.217	2.231	2.230	2.254
	18"	2.219	2.153	2.210	2.231	2.267	2.262	2.264
	24"	2.246	2.262	2.281	2.293	2.248	2.233	2.253
	30"	2.206	2.293	2.301	2.286	2.213	2.233	2.262
	36"	2.258	2.233	2.279	2.235	2.246	2.238	2.296
Hyperbolic	0	2.242	2.295	2.284	2.248	2.23	2.314	2.279
	6"	2.146	2.195	2.192	2.228	2.243	2.256	2.254
	12"	1.161	2.193	2.22	2.255	2.256	2.266	2.273
	18"	2.185	2.202	2.256	2.274	2.268	2.262	2.266
	24"	2.195	2.23	2.307	2.253	2.247	2.248	2.263
	30"	2.231	2.288	2.339	2.222	2.214	2.212	2.255
	36"	2.261	2.315	2.323	2.212	2.312	2.202	2.214

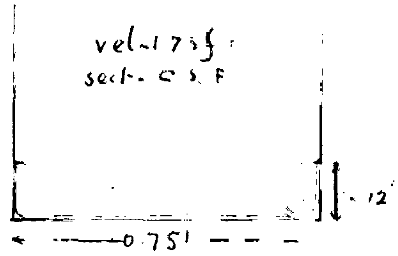
Scour Pattern Readings

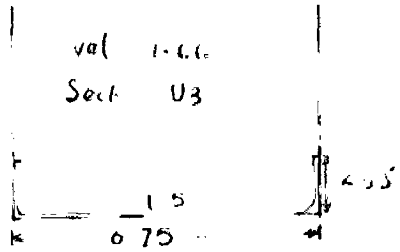
for 1 cusec

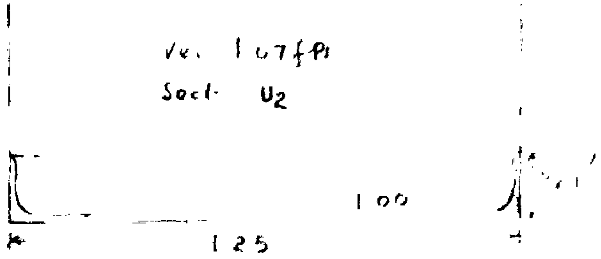
		0	6"	12"	18"	24"	30"	36"
Abrupt	0	1.706	1.701	1.832	1.845	1.812	1.823	1.819
	6"	1.632	1.683	1.78	1.868	1.887	1.878	1.825
	12"	1.599	1.685	1.843	1.868	1.912	1.945	1.921
	18"	1.527	1.593	1.823	1.883	1.924	1.953	1.918
Stra-ight	0	2.224	2.265	2.2	2.226	2.250	2.261	2.236
	6"	2.163	2.150	2.150	2.182	2.160	2.164	2.173
	12"	2.152	2.159	2.152	2.170	2.146	2.159	2.143
	18"	2.174	2.182	2.158	2.166	2.165	2.182	2.178
	24"	2.223	2.268	2.217	2.204	2.218	2.230	2.202
	30"	2.268	2.317	2.294	2.224	2.256	2.257	2.228
	36"	2.254	2.344	2.279	2.238	2.288	2.294	2.273
Hyper-bolic	0	2.346	2.294	2.282	2.248	2.28	2.314	2.279
	6"	2.307	2.307	2.282	2.228	2.243	2.256	2.254
	12"	2.307	2.298	2.305	2.255	2.256	2.266	2.273
	18"	2.296	2.284	2.286	2.274	2.268	2.262	2.266
	24"	2.279	2.282	2.288	2.253	2.247	2.248	2.259
	30"	2.262	2.224	2.228	2.222	2.214	2.212	2.255
	36"	2.235	2.216	2.228	2.212	2.212	2.202	2.214

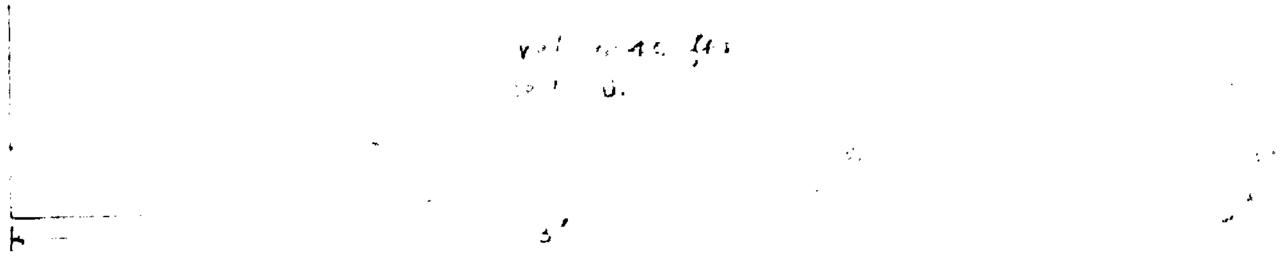
150-VELS [illegible]











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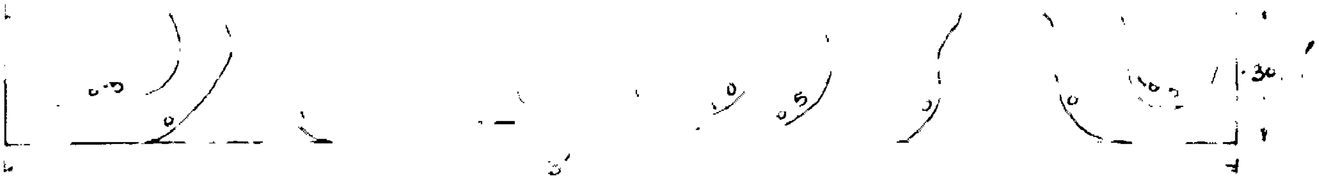
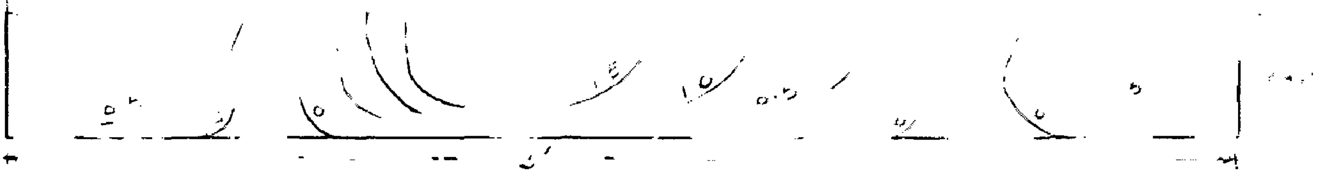


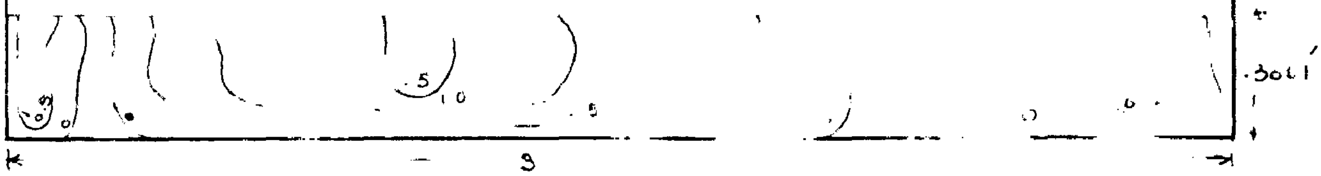
[illegible] 441 ft

[illegible] D$_2$



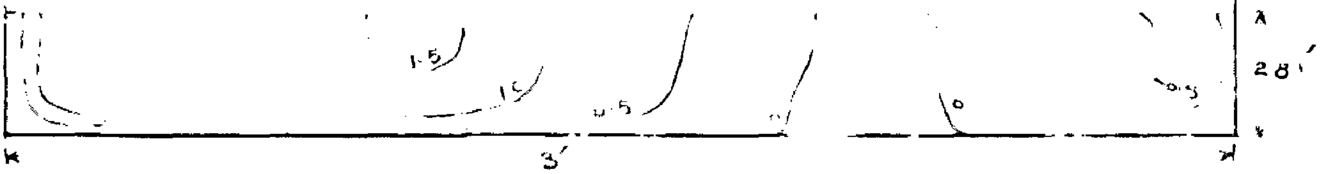
Ve [illegible] 0 441 ft

Se [illegible] D$_3$



V [illegible] ft

[illegible] D$_4$

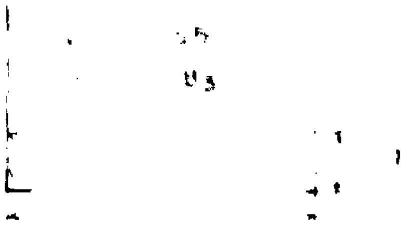


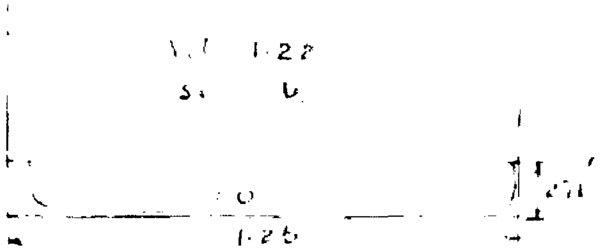
V [illegible]

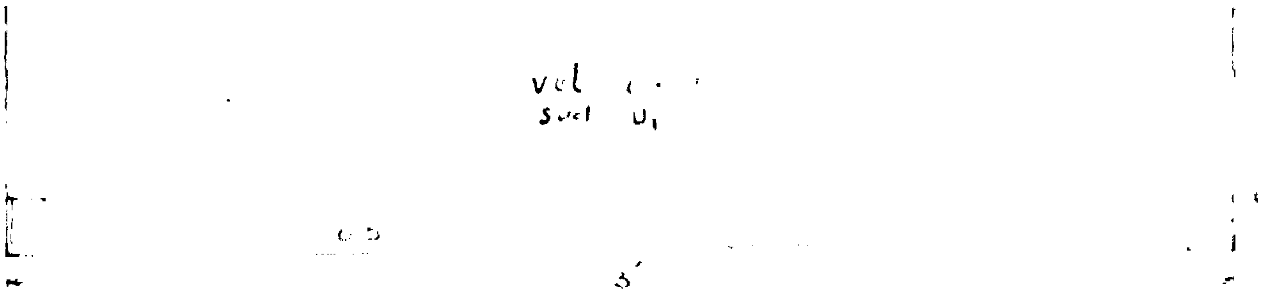
[illegible]

[illegible] Ver. 1″ = 0.5′

[illegible] 1″ [illegible]′

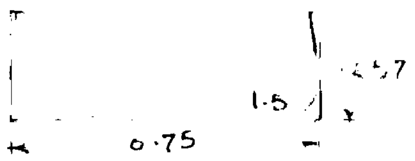




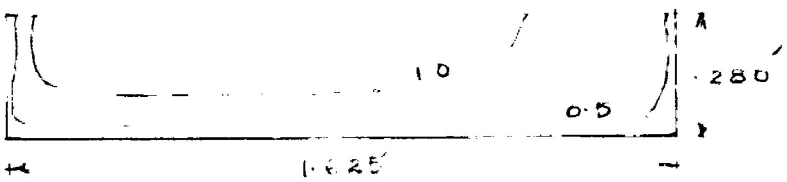


SCALE V 1″ . 10′

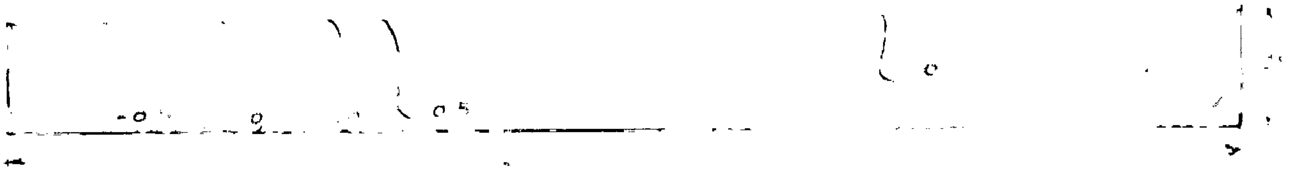
Vel . 89 fps
sect - D1



Vel = 1.07 fps
sect - D2

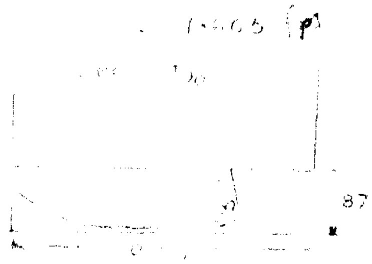


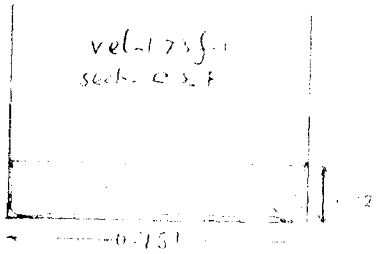
Vel . .759 fps
sect D3

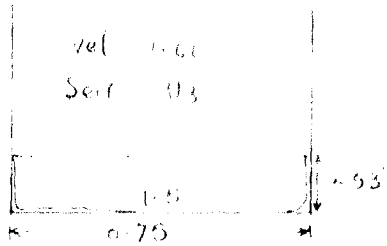


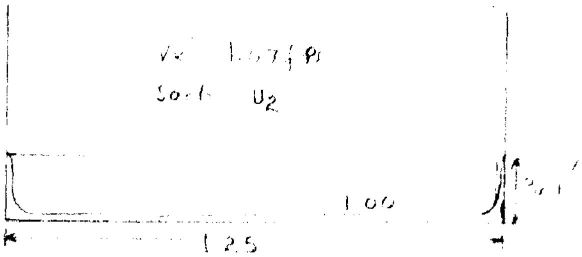
[illegible] . 74
[illegible] 4

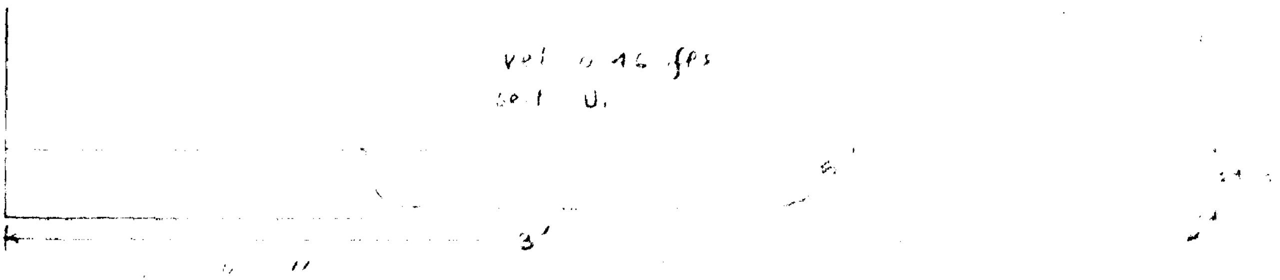
50-VELS 75



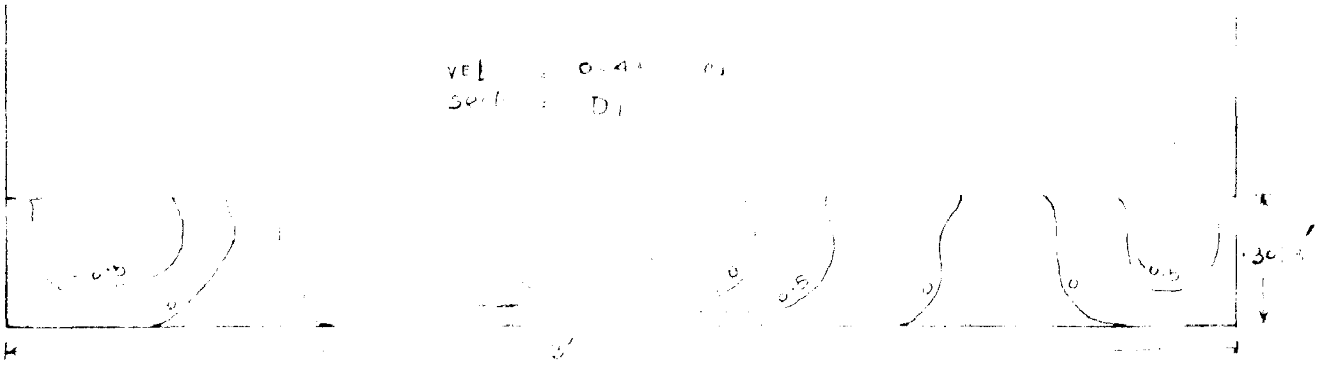


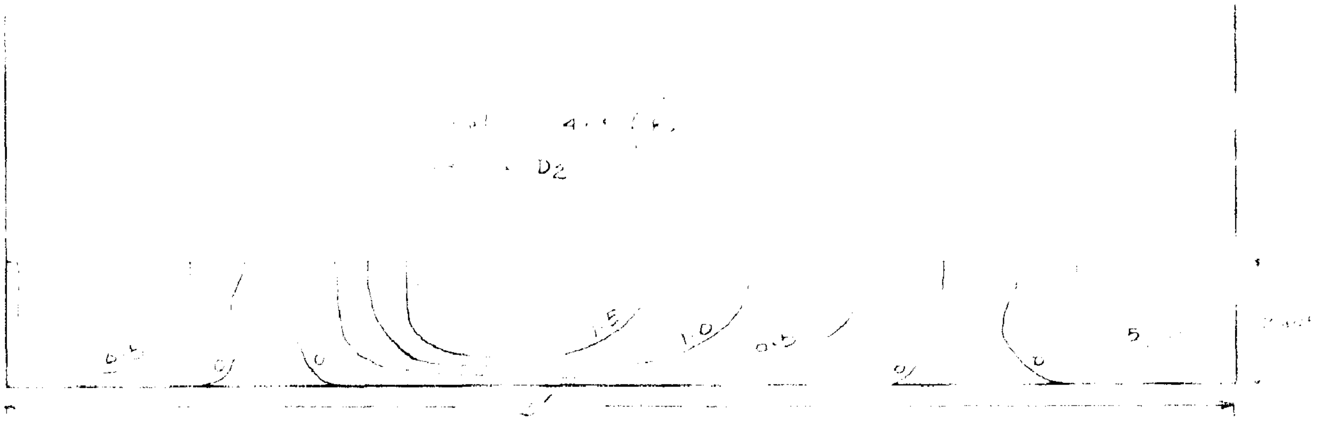


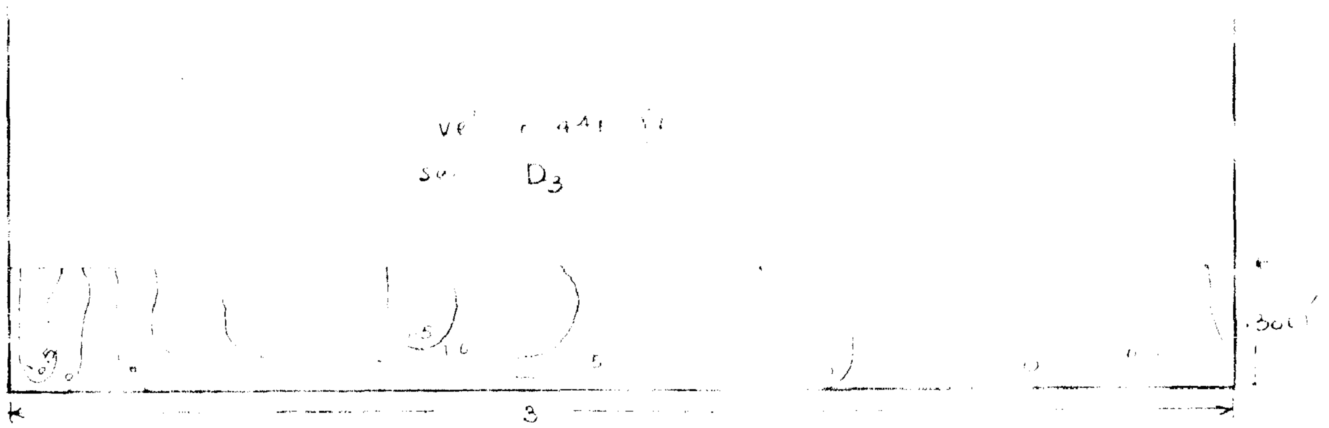


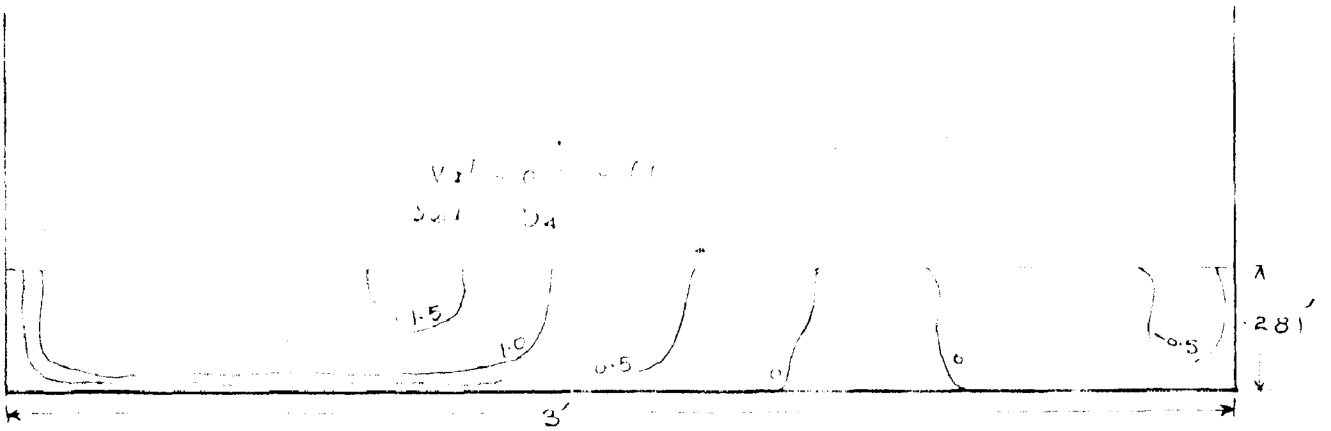


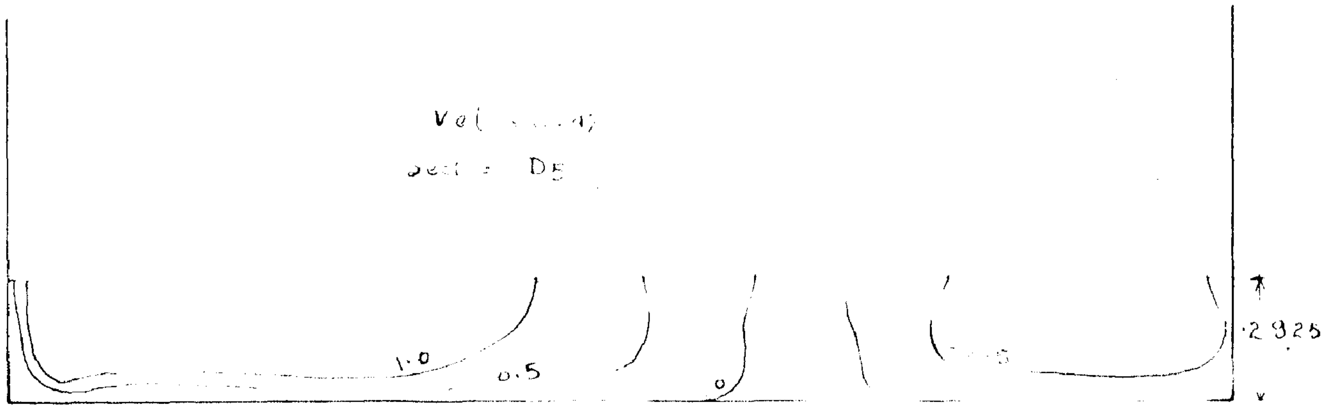
SCALE V 1" 1'
H 1" 0.5



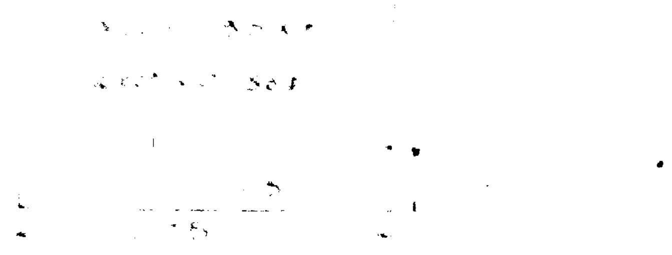


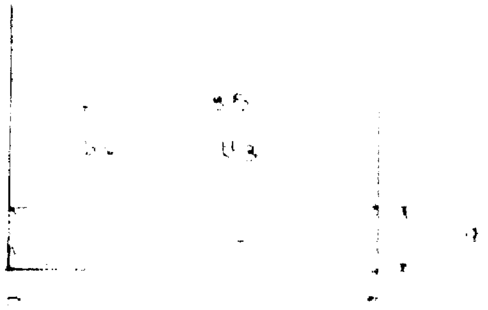


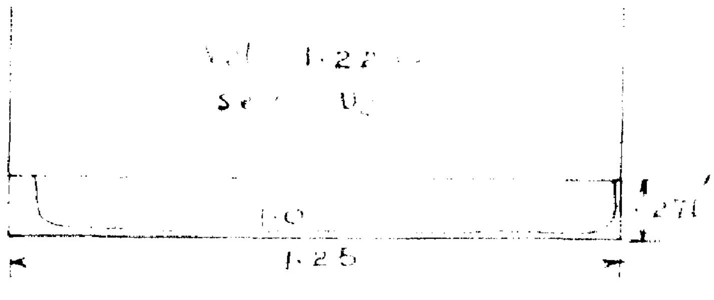


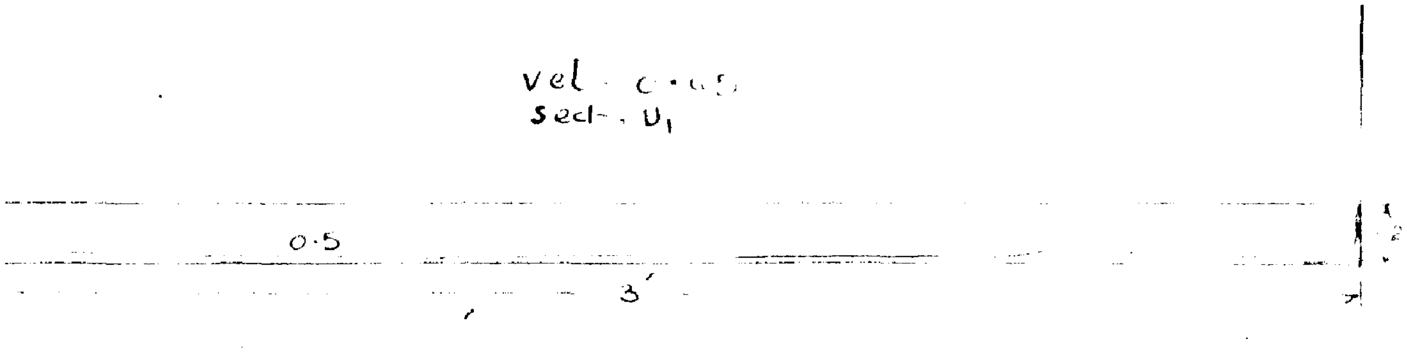


SCALE Ver.— 1" = 0.5'
1" = 5'

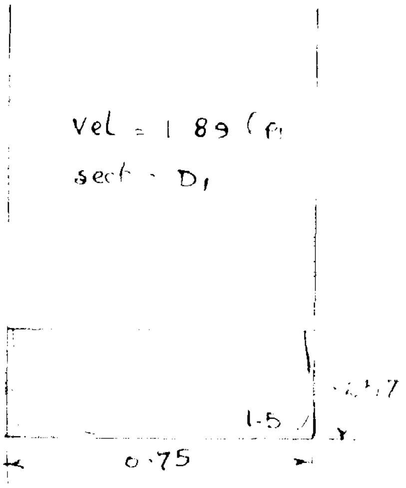


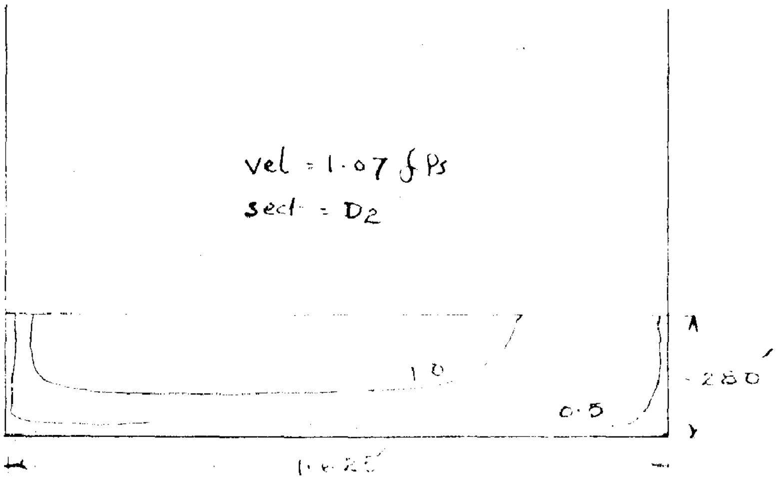


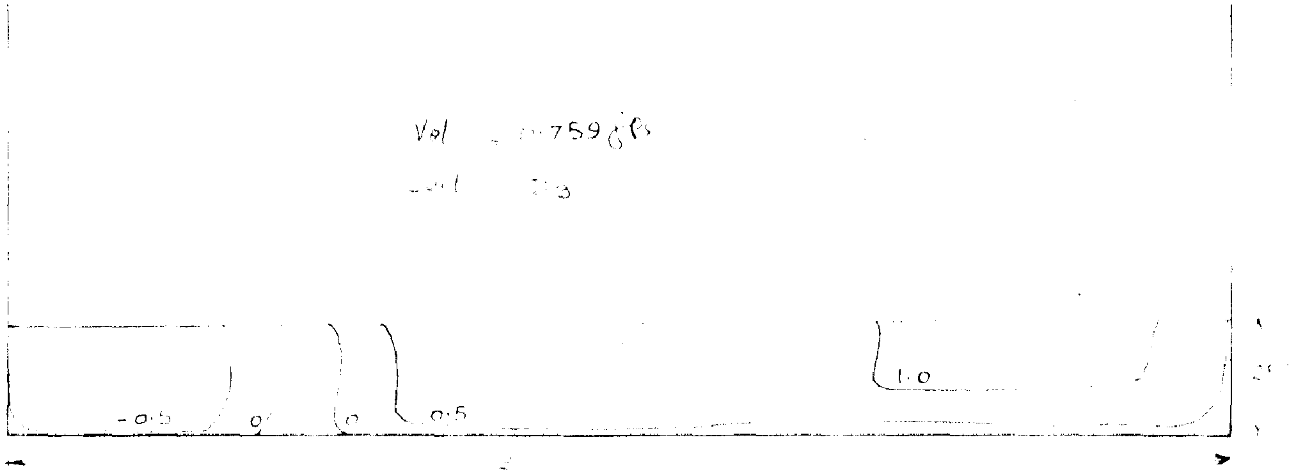


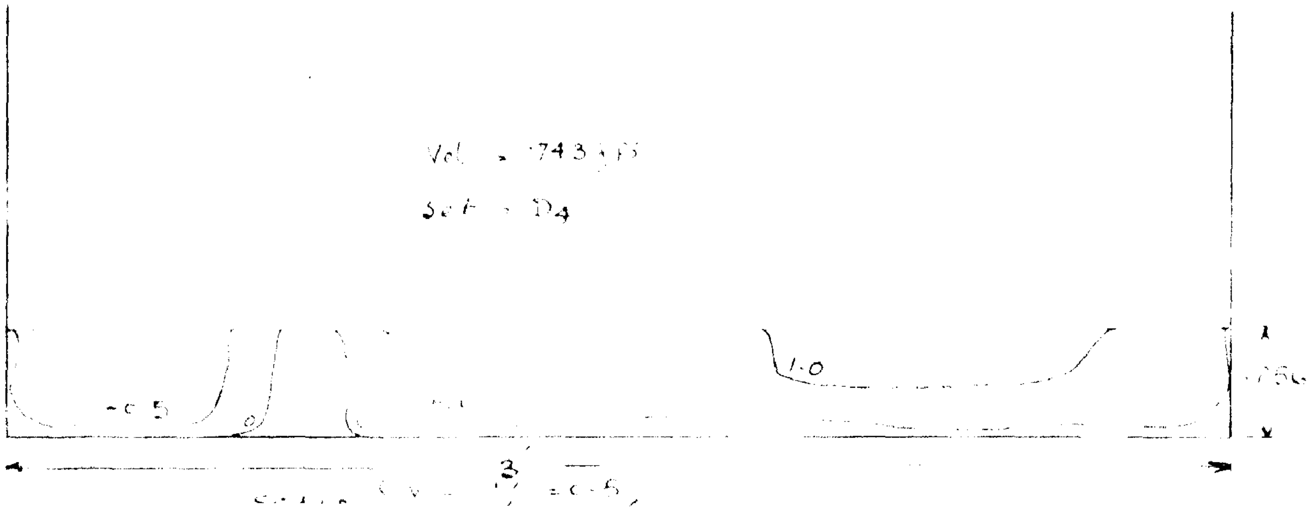


SCALE V- 1" = 1·0'

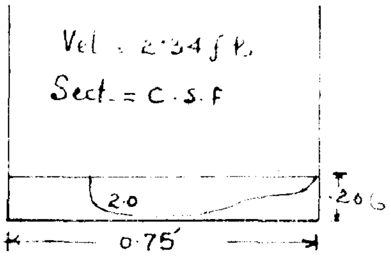

Vel = 1 89 fps
sect = D1
·257
1·5
0·75

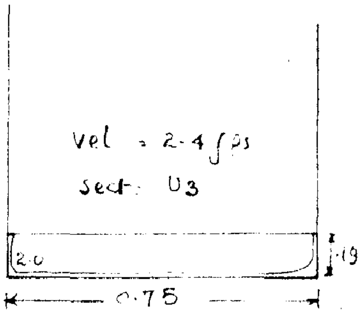

vel = 1·07 fps
sect = D2
1·0
·280′
0·5
1·625

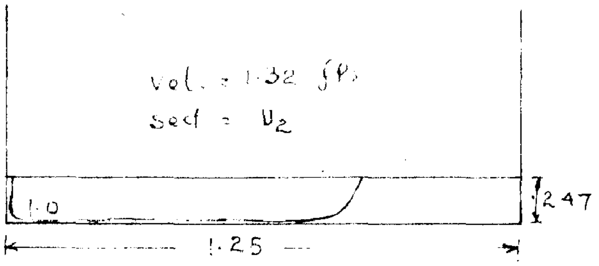

Vel = 0·759 fps
sect = D3
1·0
-0·5
0
0
0·5

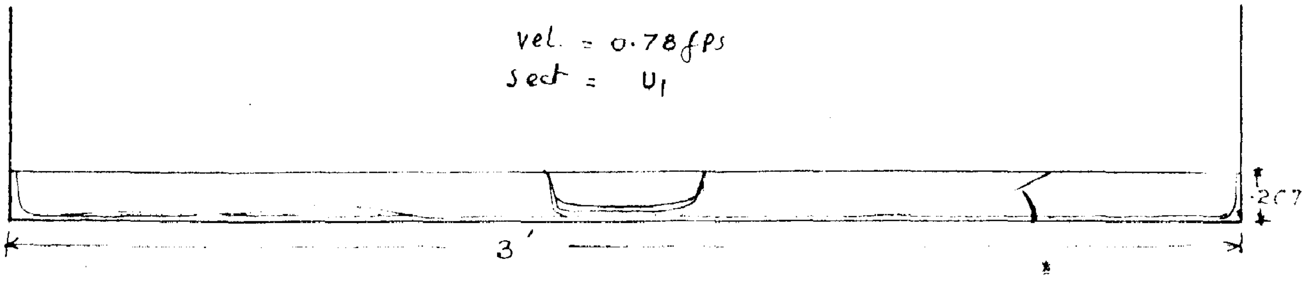

Vel = ·743 fps
sect = D4
1·0
·756
-0·5
0
3′

TRANSITION - HYPERBOLIC.



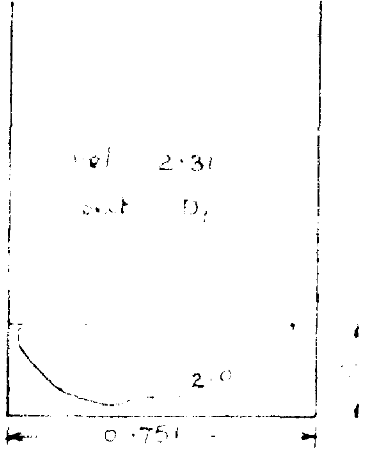


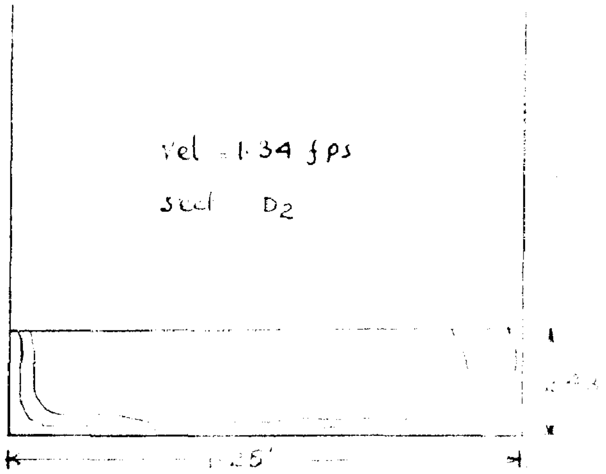


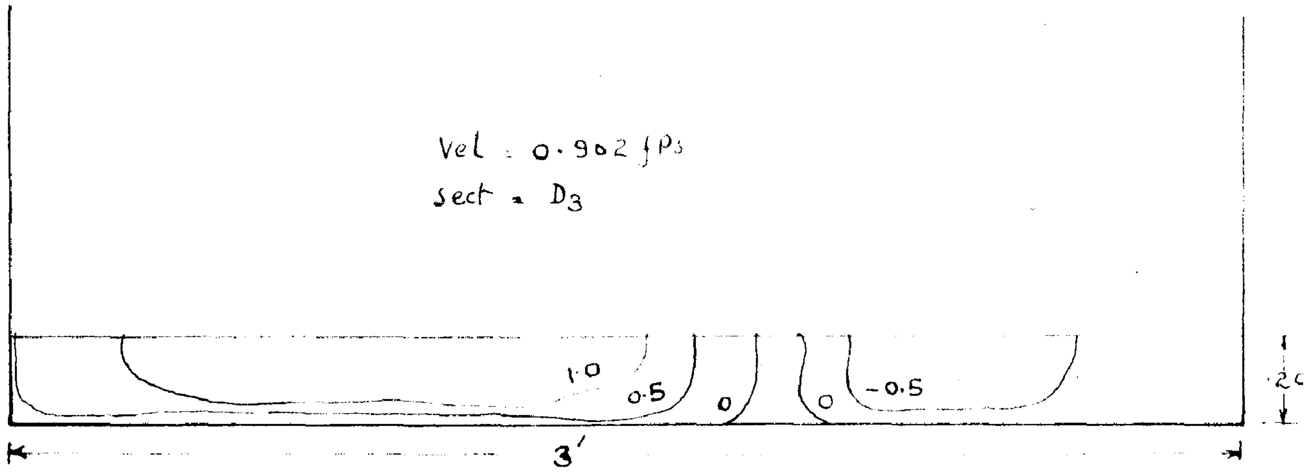


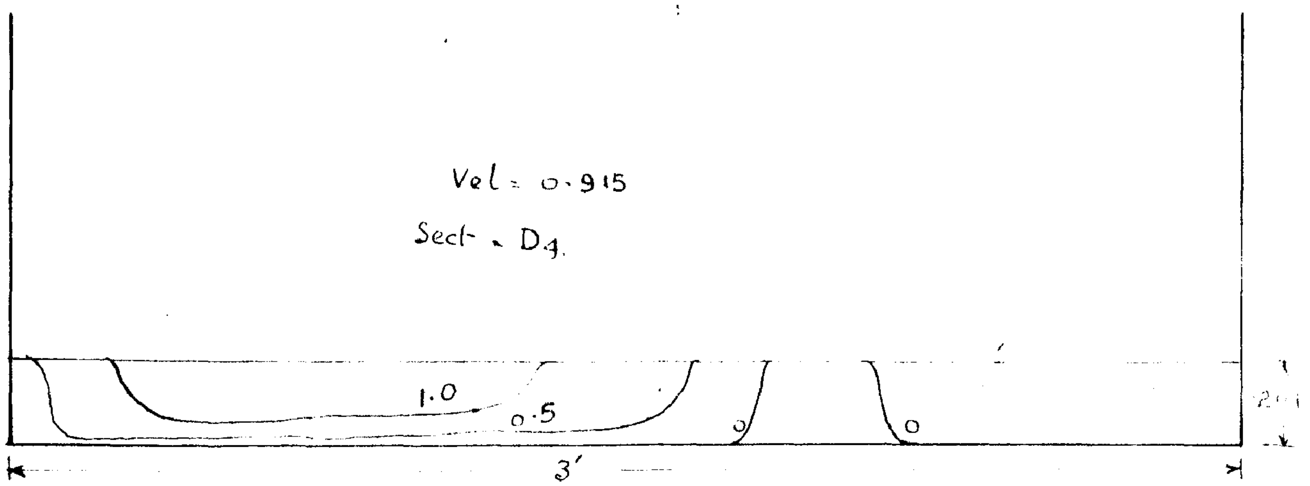
SCALE VET. - 1″ = 1′
1″ = 0.5′

TRANSITION : HYPERBOLIC



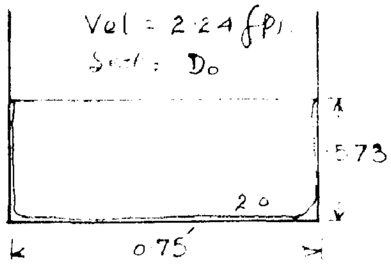


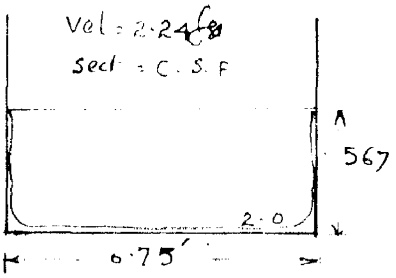


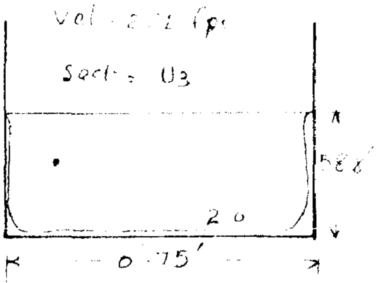


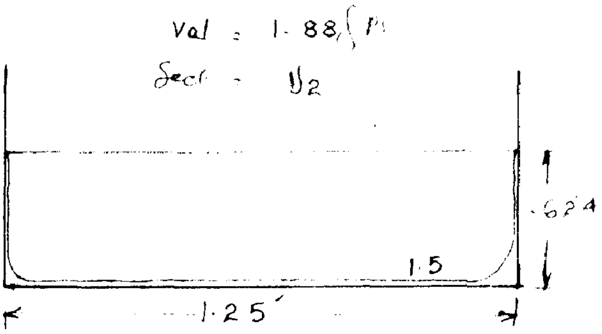
SCALE V1 = 1" = 0·5'

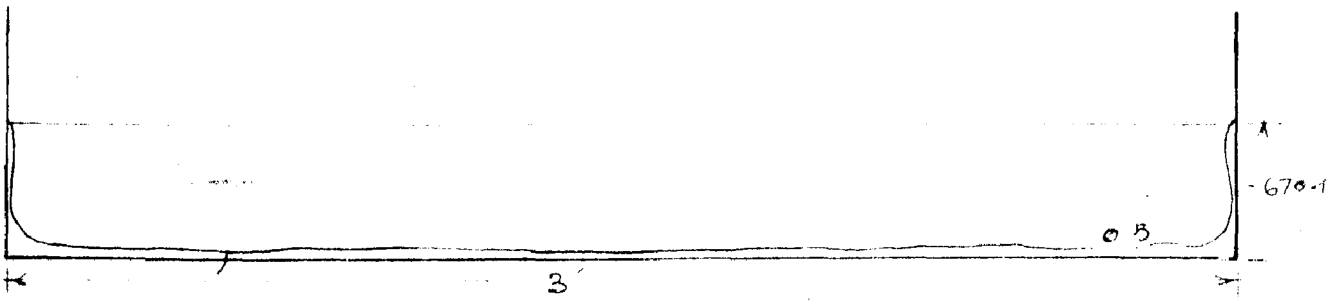
TRANSITION ABRUPT.









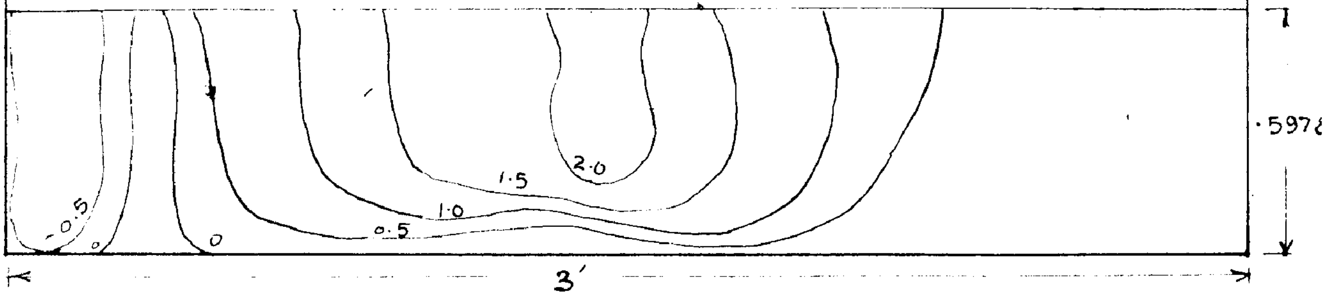


SCALE V - 1" = 1'

[illegible]

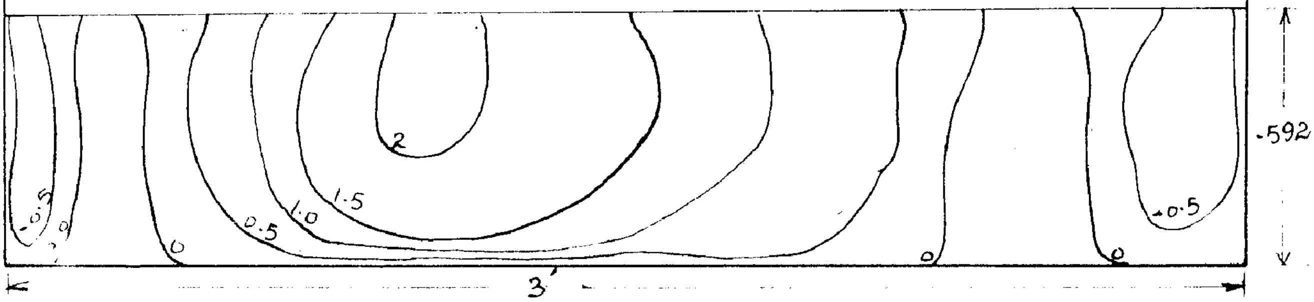
Vel = 0·615 fps

Sect = D1



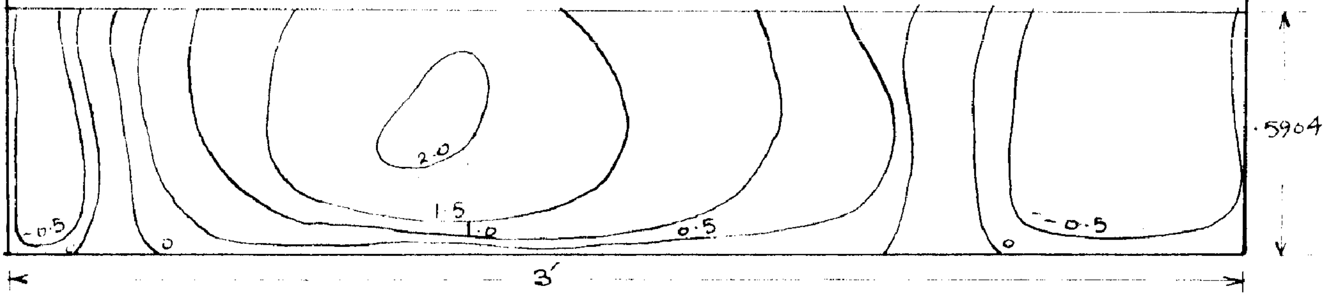
Vel = 0·655 fps.

Sect = D2



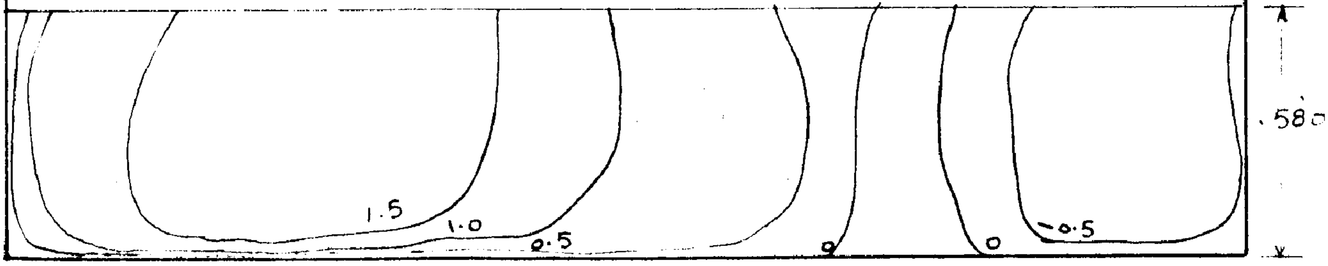
Vel = ·647 fps.

Sect = D3



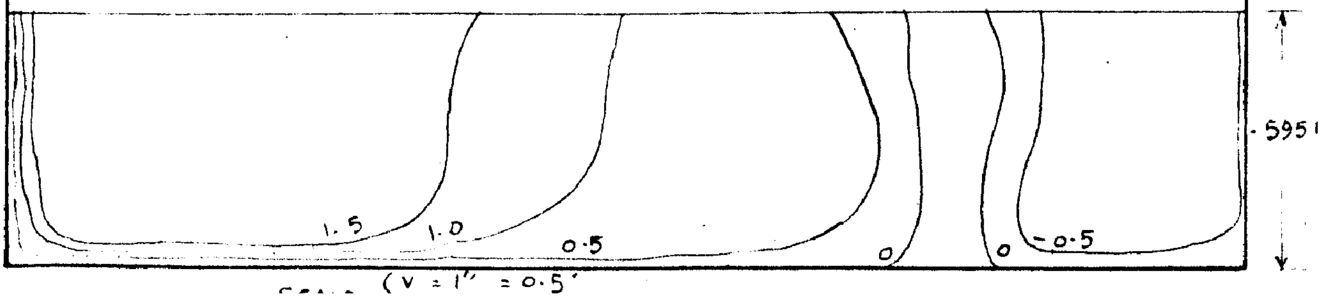
Vel = ·595

Sect = D4



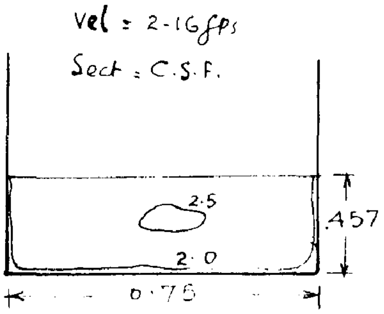
Vel = ·584

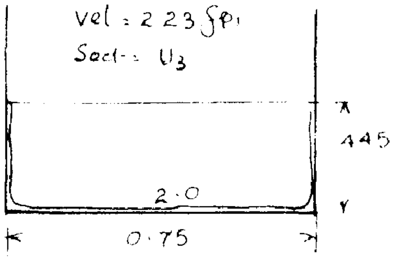
Sect = D5

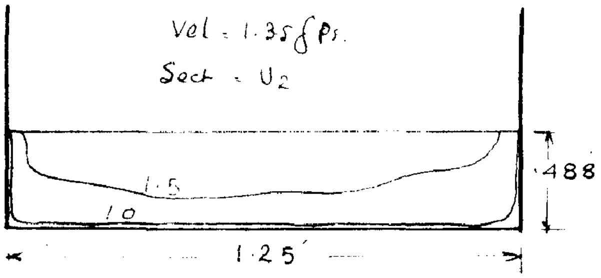


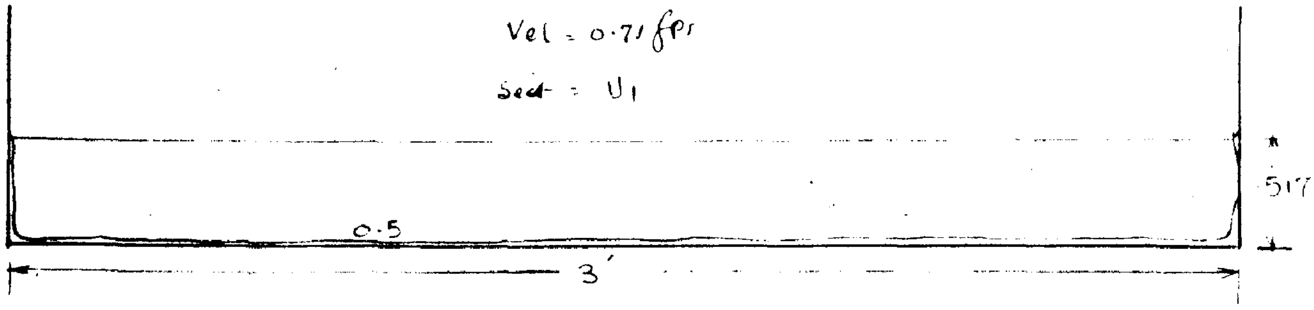
(V = 1'' = 0·5'

DISCHARGE 0·7 Cusec.
TRANSITION STRAIGHT.





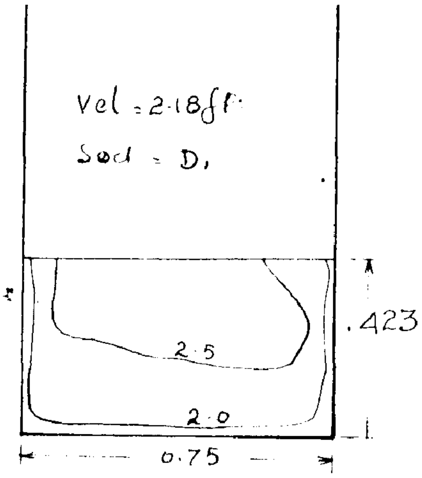


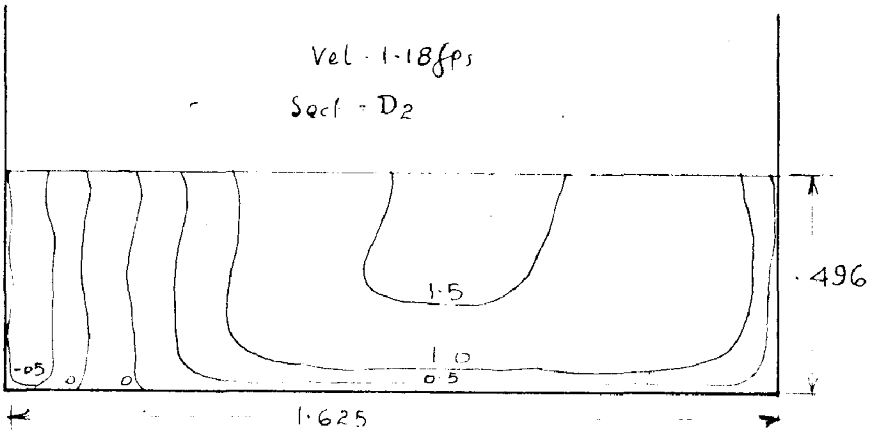


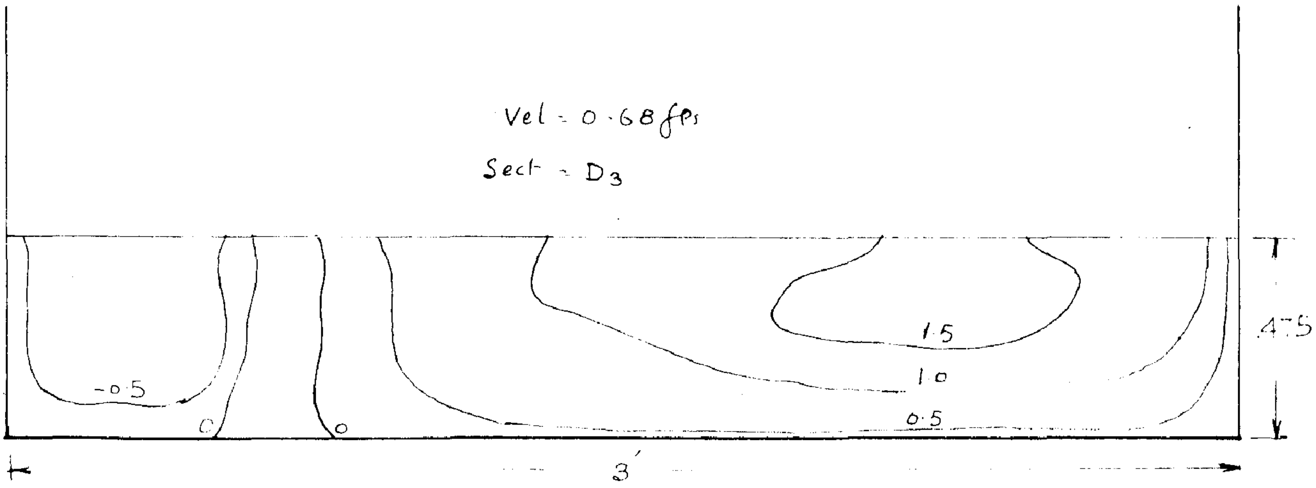
SCALE V - 1" = 1'
H - 1" = 0·5

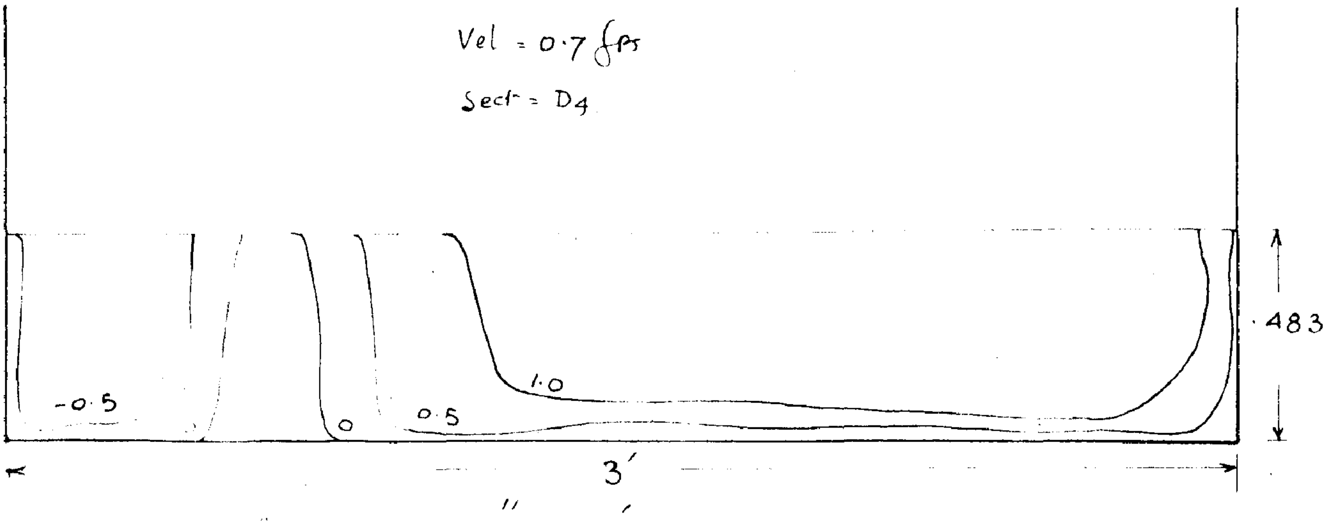
DISCHARGE 0·7 CUSEC

TRANSITION STRAIGHT.





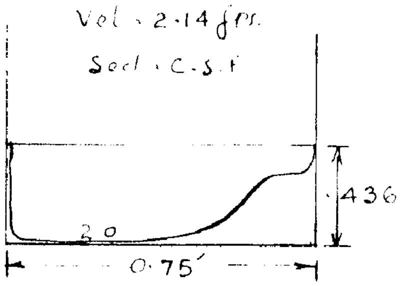


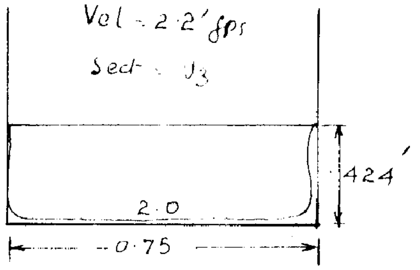


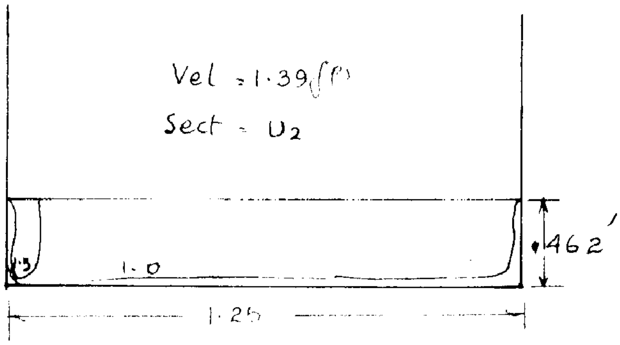
SCALE V - 1" = 0·5'
H - 1" = 0·5'

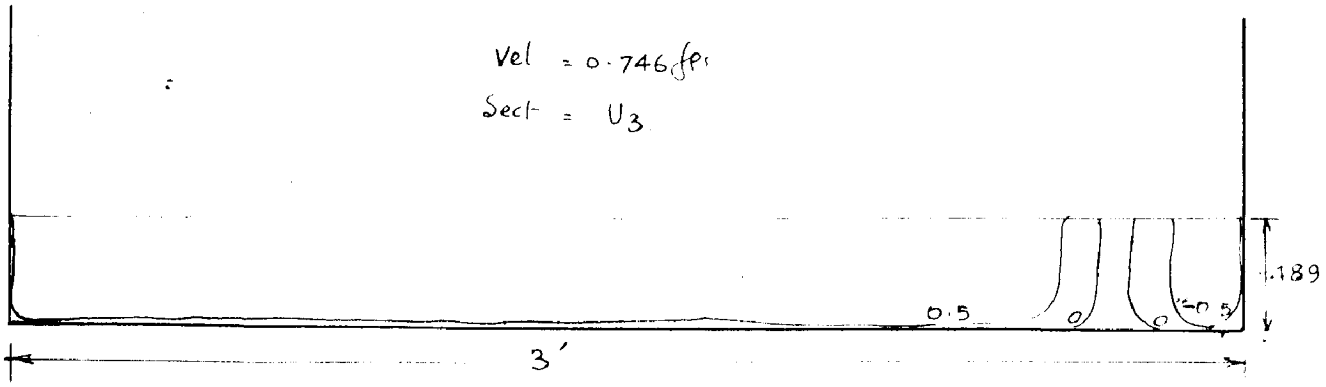
DISCHARGE = 0.7 CUSEC

TRANSITION HYPERBOLIC





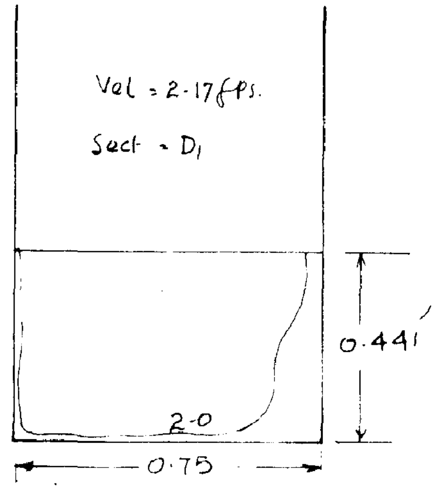


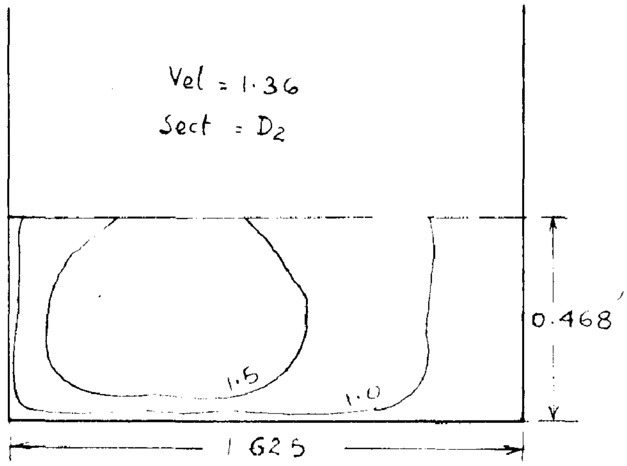


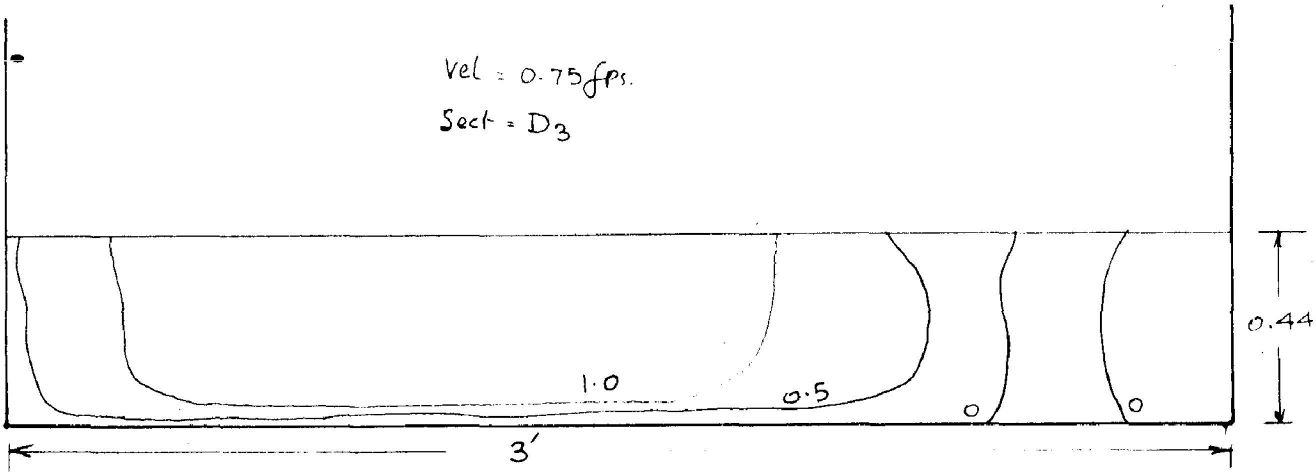
SCALE V - 1″ = 1′

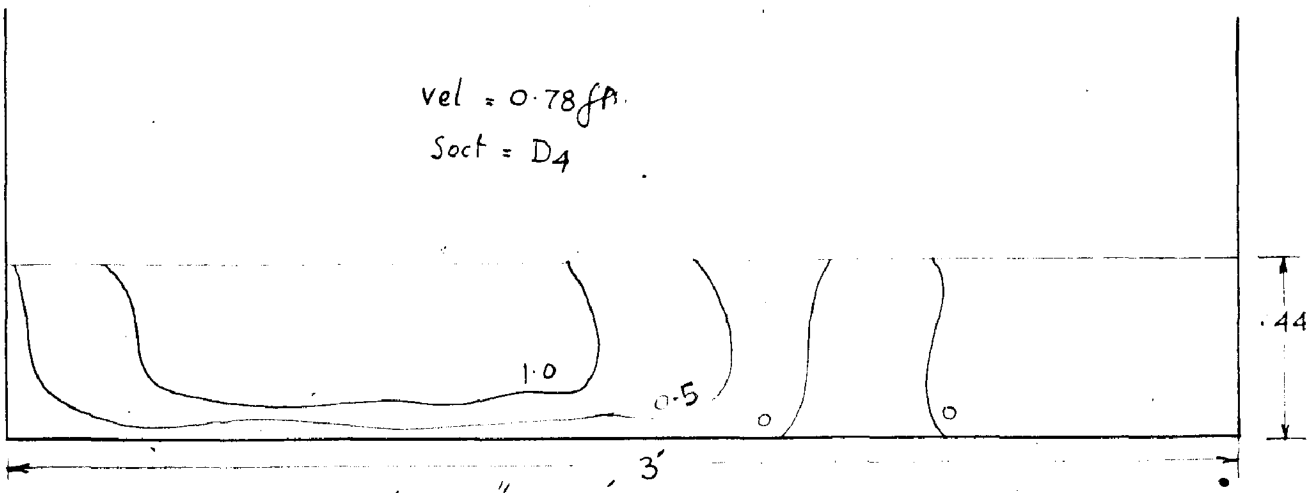
H - 1″ = 0.5′

TRANSITION HYPERBOLIC



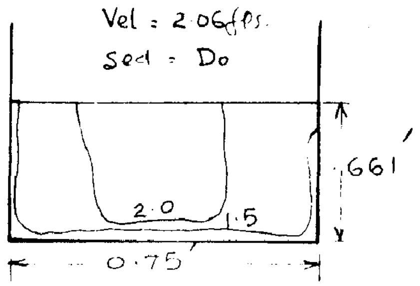


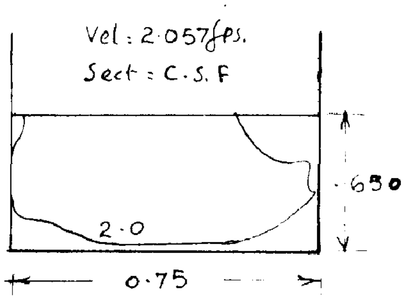


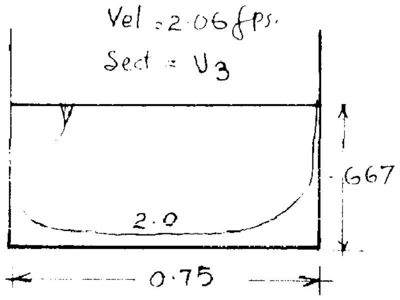


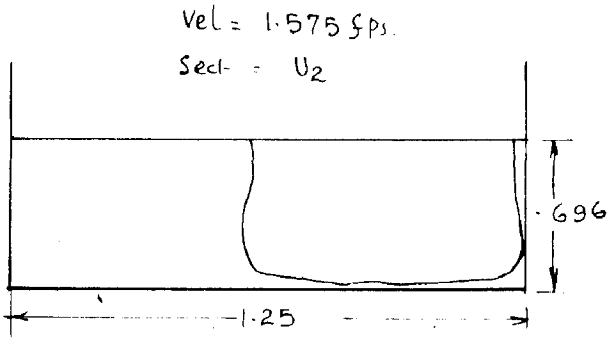
V - 1" = 0·5'

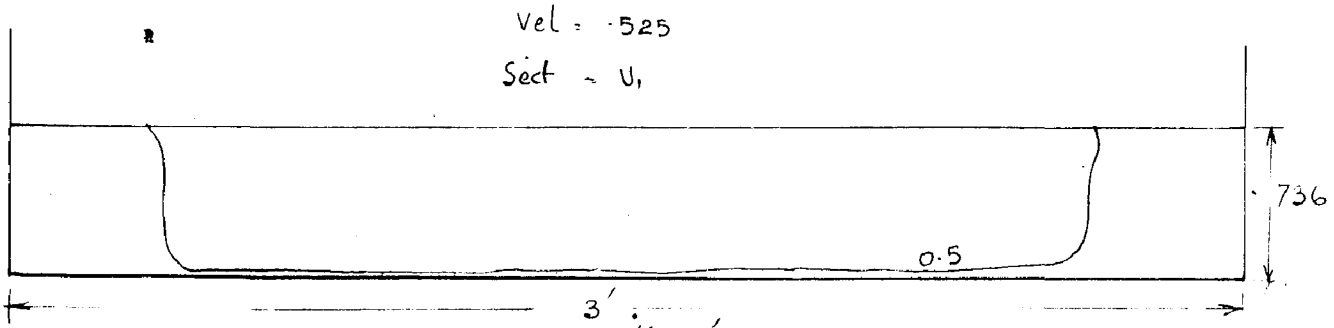
TRANSITION ABRUPT





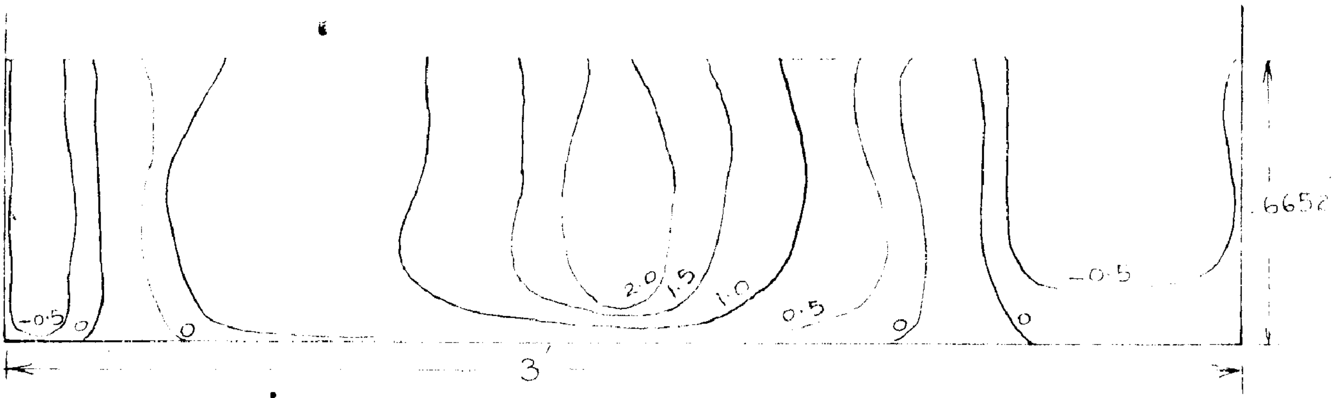






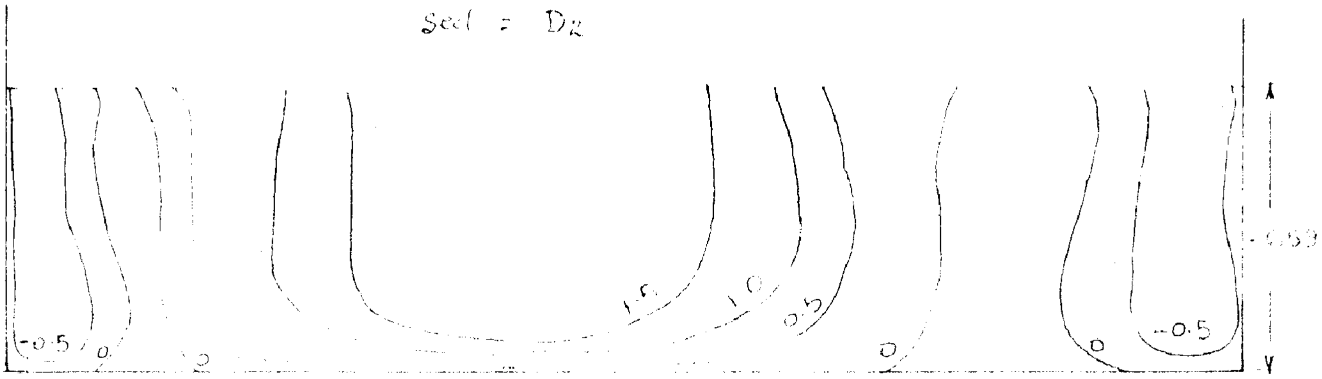
SCALE V - 1'' = 1'
'' '

Sect - D1



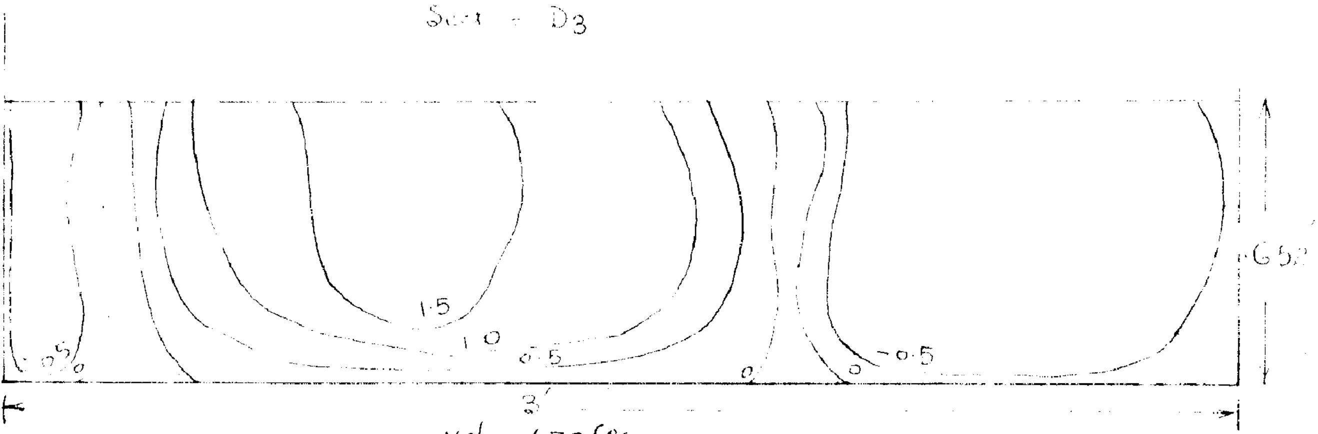
Vel = ·623 fps.

Sect = D2



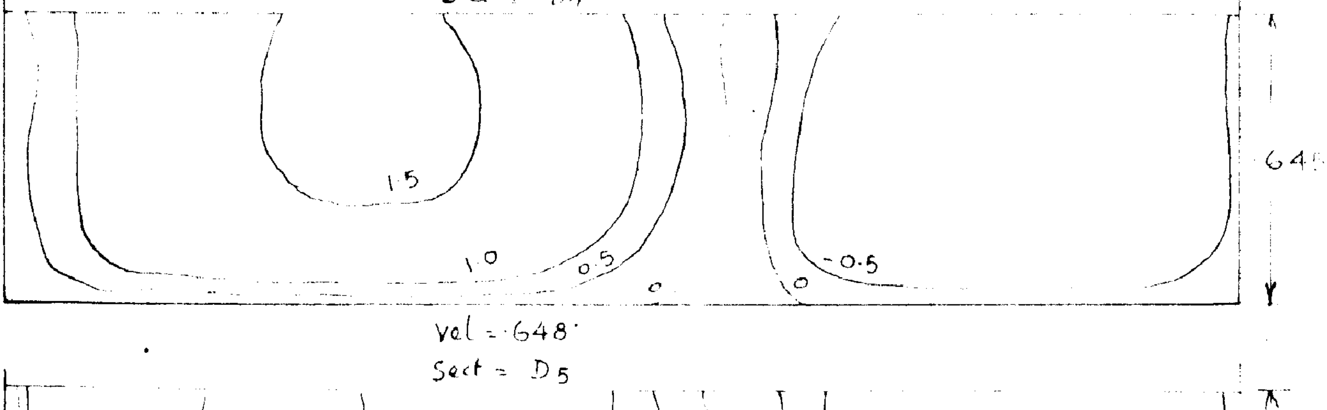
Vel = ·642 fps

Sect = D3



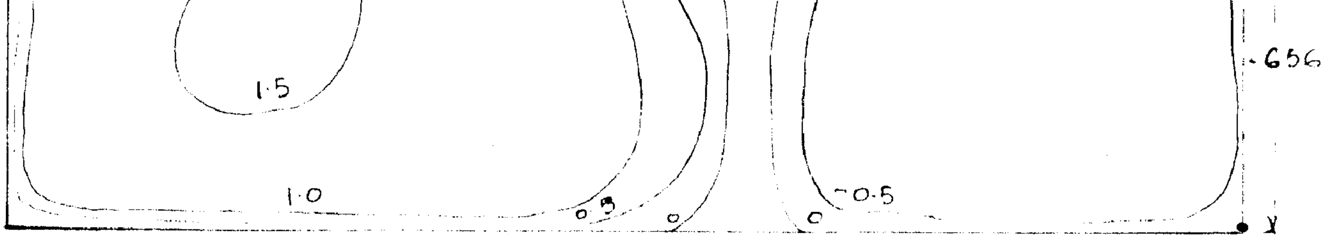
Vel = ·678 fps.

Sect = D4



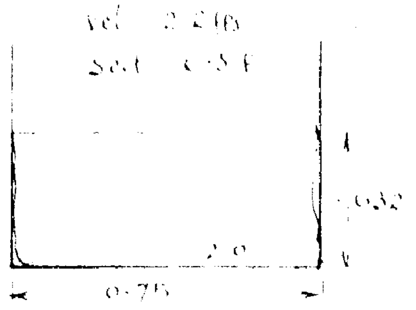
Vel = ·648·

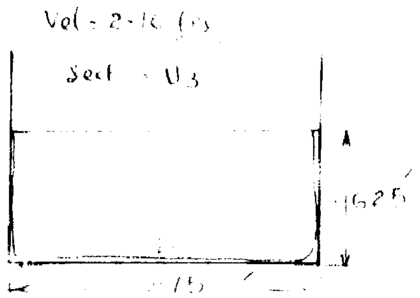
Sect = D5

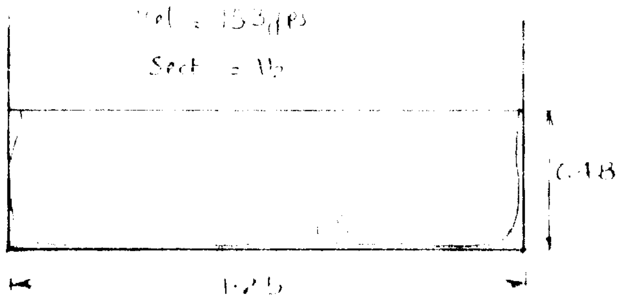


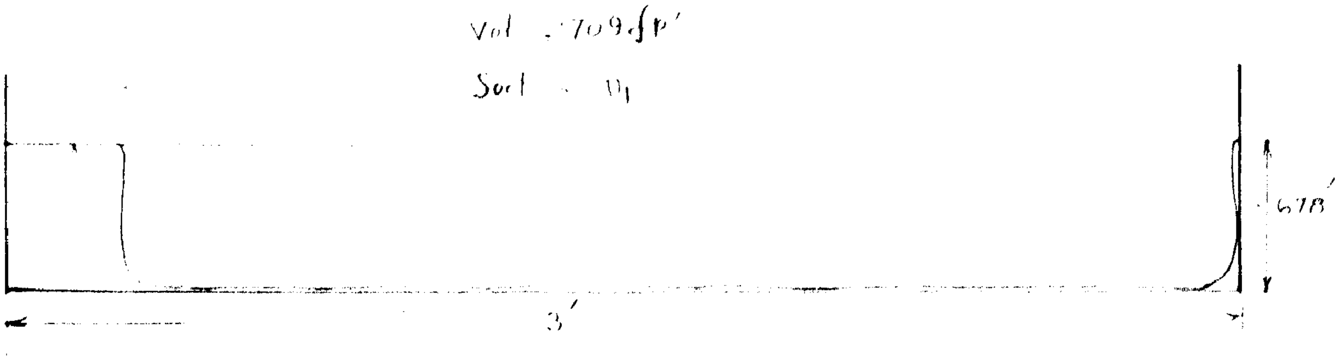
SCALE Y - 1" = 0.5

DISCHARGE FOCUSED
TRANSITION STRAIGHT



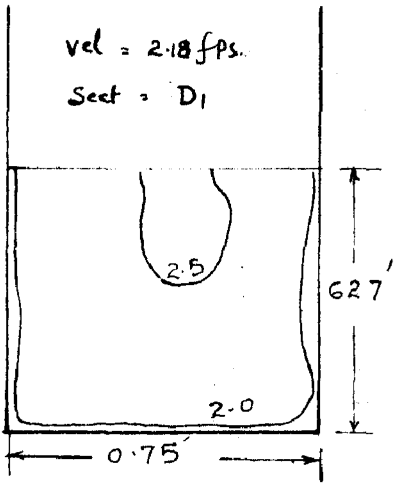




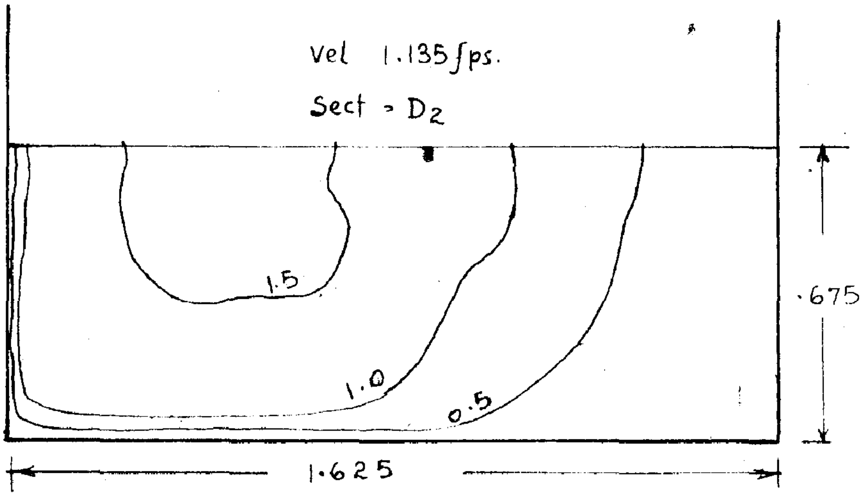


SCALE V 1″ = 1′
H 1″ = 0.5′

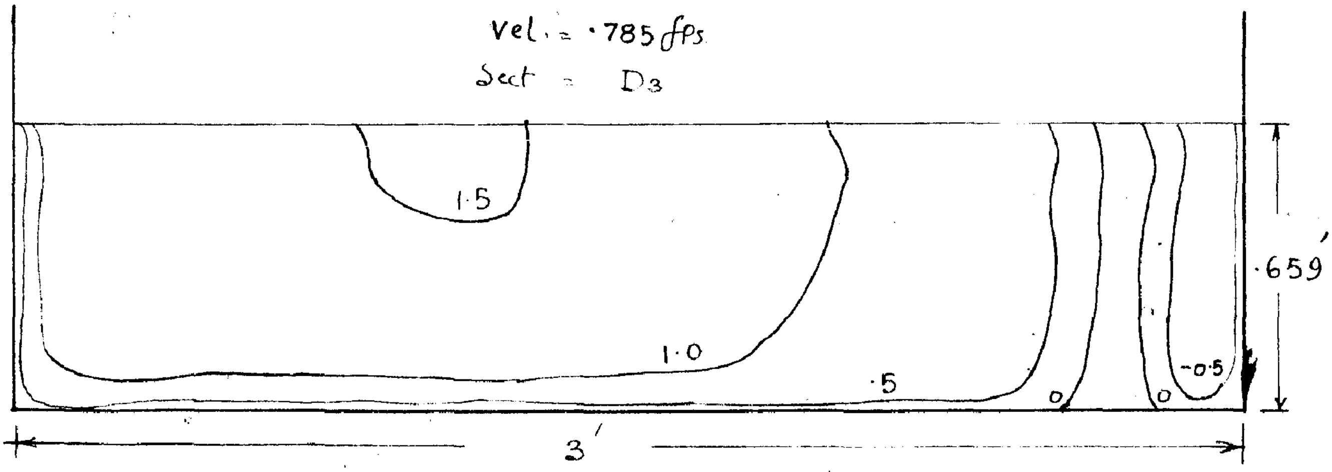
DISCHARGE [illegible] CUSEC.
TRANSITION STRAIGHT



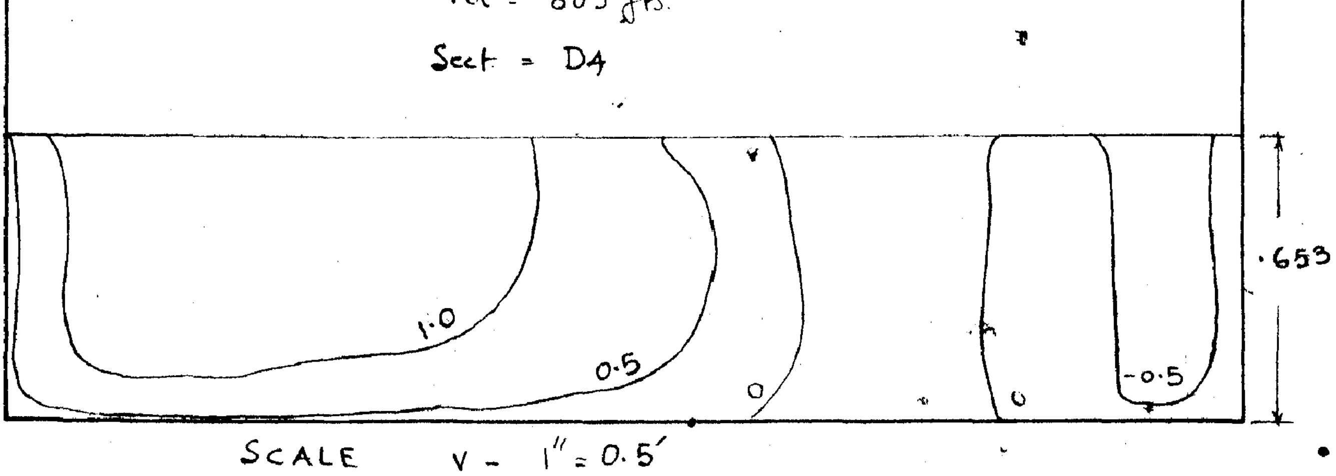
Vel = 2·18 fps.
Sect = D1



Vel 1.135 fps.
Sect = D2



Vel. = ·785 fps
Sect = D3



Vel = ·805 fps.
Sect = D4

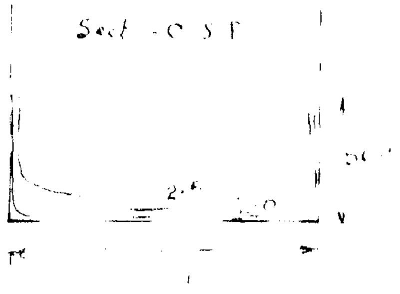
SCALE V - 1″ = 0.5′

DISCHARGE - 1.0 Cusec.

TRANSITION - HYPERBOLIC.

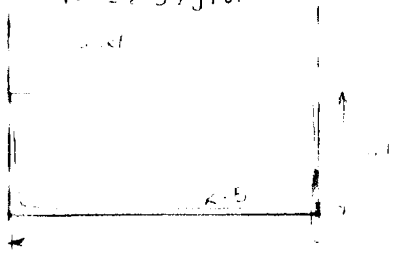
Vel - 2.55 fps.

Sect - C S F



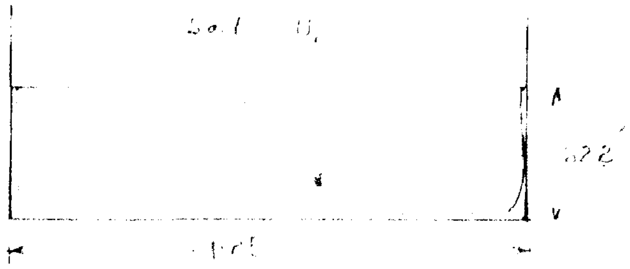
Vel - 2.54 fps.

Sect



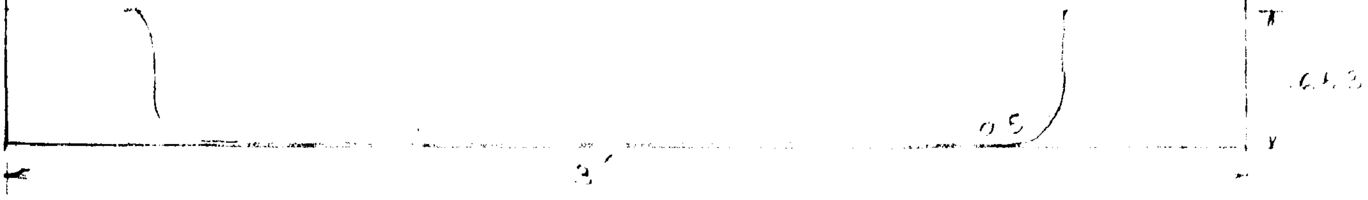
1.46 fps

Sect D1



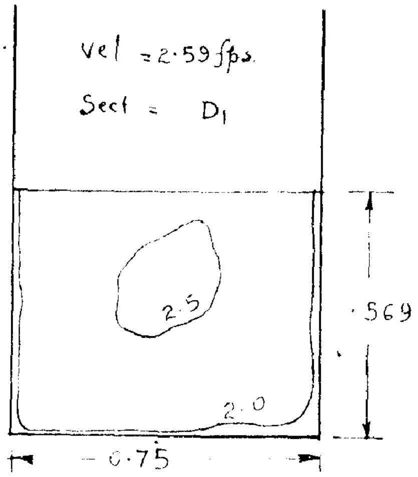
Vel .707

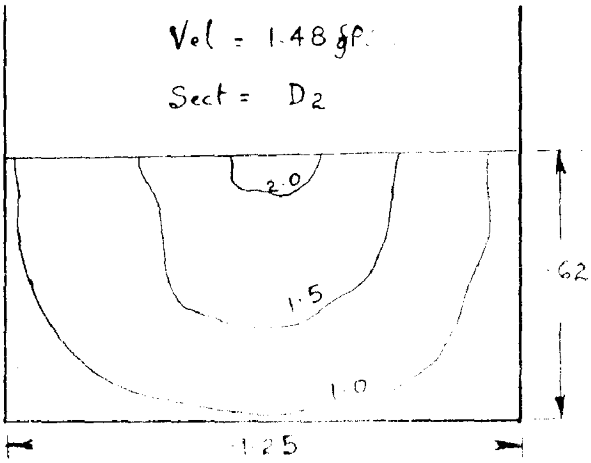
Sect D1

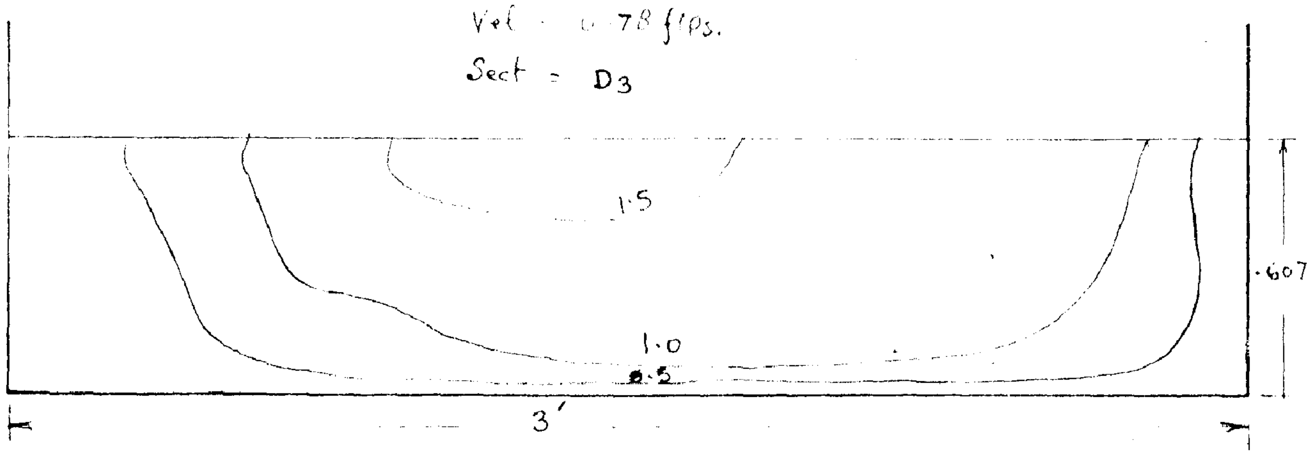


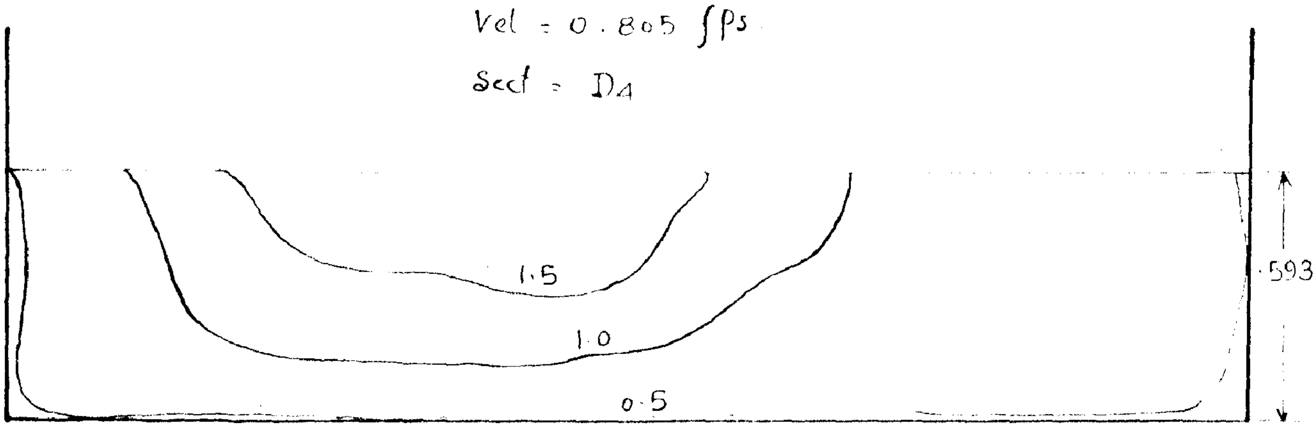
SCALE V - 1" 1"

H - 1" 0.5'









SCALE V - 1″ = 0·5′
H - 1″ = 0·5′